GEOTECHNICAL INVESTIGATION

PROPOSED JUNIOR LIFEGUARD FACILITY
50 MAIN STREET
NEWPORT BEACH, CALIFORNIA

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

JEFF KATZ ARCHITECTURE
NEWPORT BEACH, CALIFORNIA

PROJECT NO. W1033-88-01

SEPTEMBER 5, 2019
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Project No. W1033-88-01
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Mr. Jeff Katz
Jeff Katz Architecture
6353 Del Cerro Boulevard
San Diego, California 92120

Subject: GEOTECHNICAL INVESTIGATION
PROPOSED JUNIOR LIFEGUARD FACILITY
50 MAIN STREET, NEWPORT BEACH, CALIFORNIA

Dear Mr. Katz:

In accordance with your authorization of our proposal dated July 9, 2019, we have prepared this geotechnical investigation report for the proposed junior lifeguard facility to be located within Parking Lot A at the subject site. The accompanying report presents the findings of our study, and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our investigation, it is our opinion that the project can be developed as proposed provided the recommendations in 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.

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed junior lifeguard facility located within Parking Lot A at the subject site (Vicinity Map, Figure 1). The purpose of this investigation was to evaluate the subsurface soil and geologic conditions underlying the area of proposed construction and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of proposed 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 August 5, 2019 by excavating two 8-inch diameter borings to depths of approximately 20½ feet and 50½ feet below the existing ground surface using a truck-mounted mud-rotary drilling machine. The approximate locations of the exploratory borings are depicted on the Site Plan (see Figure 2). A detailed discussion of the field investigation, including boring logs, is presented in Appendix A.

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

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

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

2. SITE CONDITIONS & PROJECT DESCRIPTION

The subject site is located at 50 Main Street in the City of Newport Beach, California. The existing parking lot (Parking Lot A) is bounded by the Newport Balboa Bike Trail and residential structures to the north, by a grass field park to the east, by the beach and ocean to the south, and by Balboa Pier to the west. The area of the proposed construction is currently an asphalt paved parking lot. Surface water drainage at the site appears to be by sheet flow along the ground surface to area drains and the city streets. Vegetation onsite consists of grass and trees.

Information concerning the proposed project was furnished by the client. It is our understanding that the proposed development will consist of a new 4,000 square-foot Junior Lifeguard Facility, as well as miscellaneous paving and utility improvements. We assume that the proposed structure will be single-story. Due to the preliminary nature of the project, formal plans depicting the proposed development are not available for inclusion in this report. The existing site conditions are depicted on the Site Plan (see Figure 2).

Based on the preliminary nature of the design at this time, wall and column loads were not available. It is anticipated that column loads for the proposed structure will be up to 100 kips, and wall loads will be up to 2 kips per linear foot.

Once the design phase proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Geocon should be contacted to determine the necessity for review and possible revision of this report.

3. GEOLOGIC SETTING

The subject site is located on Balboa Peninsula, a narrow strip of land at the southern edge of the Orange County Coastal Plain, bound by Newport Harbor to the north and the Pacific Ocean to the south. The Coastal Plain is a relatively flat-lying alluviated surface with an average slope of less than 20 feet per mile. The lowland surface is bounded by hills and mountains on the north and east and by the Pacific Ocean to the south and southwest (Department of Water Resources, 1967). Prominent structural features within the Orange County Coastal Plain include the central lowland plain, the northwest trending line of low hills and mesas near the coast underlain by the Newport-Inglewood Fault Zone (Newport Mesa, Huntington Beach Mesa, Bolsa Chica Mesa, and Landing Hill), and the San Joaquin Hills to the southeast (Department of Water Resources, 1967).

4. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the site is underlain by artificial fill and Holocene age beach deposits that are in turn underlain by Pleistocene age marine deposits (CDMG, 1981; CGS, 2012). Detailed stratigraphic profiles of the materials encountered at the site are provided on the boring logs in Appendix A.

4.1 Artificial Fill

Artificial fill was encountered in our field explorations to a maximum depth of 1½ feet below existing ground surface. The artificial fill generally consists of light brown poorly graded sand with some shell fragments. The artificial fill is characterized as moist and medium dense. The fill is likely the result of past grading or construction activities at the site. Deeper fill may exist between excavations and in other portions of the site that were not directly explored.

4.2 Beach Deposits

The artificial fill is underlain by Holocene age unconsolidated beach deposits consisting of light brown fine- to medium-grained sand. The beach deposits extend to depths of approximately 9½ to 11 feet beneath the existing ground surface and are characterized as loose to medium dense and moist to wet.

4.3 Old Marine Deposits

Pleistocene age marine deposits were encountered beneath the younger beach deposits and consist primarily of light brown to brown, gray to olive gray, or olive brown poorly-graded sand and silty sand with varying amounts of shell fragments. The marine deposits are primarily moist to wet and medium dense to very dense.

5. GROUNDWATER

Review of the Seismic Hazard Zone Report for the Newport Beach Quadrangle (California Division of Mines and Geology [CDMG], 1997a) indicates that the historically highest groundwater level in the area is less than 10 feet beneath the ground surface. Groundwater information presented in this document is generated from data collected in the early 1900's to the late 1990s. Based on current groundwater basin management practices, it is unlikely that groundwater levels will ever exceed the historic high levels.

Groundwater was encountered in borings B1 and B2 at depths of 7 and 6 feet below the existing ground surface, respectively. Given the proximity of the site to the coastline, the depth to groundwater is likely influenced by tidal fluctuations. Based on these considerations, groundwater may be encountered during construction. Also, it is not uncommon for groundwater levels to vary seasonally or for groundwater seepage conditions to develop where none previously existed, especially in impermeable fine-grained soils which are heavily irrigated or after seasonal rainfall. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the immediate site vicinity. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 7.14).

6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

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

The site is not within a state-designated Alquist-Priolo Earthquake Fault Zone (CGS, 2019a and 2019b;) for surface fault rupture hazards. No active or potentially active faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. However, the site is located in the seismically active Southern California region, and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults. The faults in the vicinity of the site are shown in Figure 3, Regional Fault Map.

The closest surface trace of an active fault to the site is the Newport-Inglewood Fault Zone located approximately 0.6 mile to the south-southwest (Ziony and Jones, 1989). Other nearby active faults are the Palos Verdes Fault Zone (offshore segment), the Whittier Fault, and the Elsinore Fault located approximately 12.5 miles southwest, 22.5 miles north-northeast, and 23.5 miles northeast of the site, respectively (Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 54 miles northeast of the site (Ziony and Jones, 1989).

Several buried thrust faults, commonly referred to as blind thrusts, underlie the Los Angeles Basin and the Orange County 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. These thrust faults and others in the greater Los Angeles area are not exposed at the surface and do not present a potential surface fault rupture hazard at the site; however, these deep thrust faults are considered active features capable of generating future earthquakes that could result in moderate to significant ground shaking at the site.

6.2 Seismicity

As with all of Southern California, the site has experienced historic earthquakes from various regional faults. The seismicity of the region surrounding the site was formulated based on research of an electronic database of earthquake data. The epicenters of recorded earthquakes with magnitudes equal to or greater than 5.0 in the site vicinity are depicted on Figure 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	46	NE
Long Beach	March 10, 1933	6.4	4	WNW
Tehachapi	July 21, 1952	7.5	116	NW
San Fernando	February 9, 1971	6.6	63	NW
Whittier Narrows	October 1, 1987	5.9	33	NNW
Sierra Madre	June 28, 1991	5.8	46	N
Landers	June 28, 1992	7.3	93	ENE
Big Bear	June 28, 1992	6.4	74	ENE
Northridge	January 17, 1994	6.7	56	NW
Hector Mine	October 16, 1999	7.1	116	NE
Ridgecrest	July 5, 2019	7.1	150	N

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 below are for the risk-targeted maximum considered earthquake (MCE_R).

2019 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2019 CBC Reference
Site Class	D	Section 1613.2.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.397g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.496g	Figure 1613.2.1(2)
Site Coefficient, F_A	1.0	Table 1613.2.3(1)
Site Coefficient, F_V	1.804*	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.397g	Section 1613.2.3 (Eqn 16-36)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	0.894g*	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.931g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.596g*	Section 1613.2.4 (Eqn 16-39)

Note:
*Per Section 11.4.8 of ASCE/SEI 7-16, a ground motion hazard analysis shall be performed for projects for Site Class "E" sites with Ss greater than or equal to 1.0g, and for Site Class "D" and "E" sites with S1 greater than 0.2g. Section 11.4.8 also provides exceptions which indicates that the ground motion hazard analysis may be waived provided the exceptions are followed. Using the code based values presented in the table above, in lieu of a performing a ground motion hazard analysis, requires the exceptions outlined in ASCE 7-16 Section 11.4.8 be followed.

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	0.613g	Figure 22-7
Site Coefficient, F_{PGA}	1.1	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.674g	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 6.66 magnitude event occurring at a hypocentral distance of 7.65 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 6.55 magnitude occurring at a hypocentral distance of 17.41 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 Newport Beach Quadrangle (1997b) indicates that the site is located in an area designated as having a potential for liquefaction. In addition, the City of Newport Beach (2006) indicates that the site is located within an area identified as having a potential for liquefaction.

Liquefaction analysis of the soils underlying the site was performed using an updated version of the spreadsheet template LIQ2_30.WQ1 developed by Thomas F. Blake (1996). This program utilizes the 1996 NCEER method of analysis. This semi-empirical method is based on a correlation between values of Standard Penetration Test (SPT) resistance and field performance data.

The liquefaction analysis was performed for a Design Earthquake level by using a high groundwater table of 5 feet below the ground surface, a magnitude 6.68 earthquake, and a peak horizontal acceleration of 0.490g ($\frac{2}{3}PGA_M$). The enclosed liquefaction analysis, included herein for boring B1, indicates that the alluvial soils below the historic high groundwater level could be susceptible to approximately 1.1 inches of total settlement during Design Earthquake ground motion (see enclosed calculation sheets, Figures 5 and 6).

It is our understanding that the intent of the Building Code is to maintain "Life Safety" during Maximum Considered Earthquake level events. Therefore, additional analysis was performed to evaluate the potential for liquefaction during a MCE event. The structural engineer should evaluate the proposed structure for the anticipated MCE liquefaction induced settlements and verify that anticipated deformations would not cause the foundation system to lose the ability to support the gravity loads and/or cause collapse of the structure.

The liquefaction analysis was also performed for the Maximum Considered Earthquake level by using a high groundwater table of 5 feet below the ground surface, a magnitude 6.78 earthquake, and a peak horizontal acceleration of 0.734g (PGA_M). The enclosed liquefaction analysis, included herein for boring B1, indicates that the alluvial soils below the historic high groundwater level could be susceptible to approximately 1.1 inches of total settlement during Maximum Considered Earthquake ground motion (see enclosed calculation sheets, Figures 7 and 8).

6.5 Lateral Spreading

Lateral spread occurs as a result of liquefaction induced lateral ground movement and typically occurs due to the presence of liquefiable soils over a gently sloping ground surface or sloping geologic contact. For the purposes of this report, we have assumed that the marine terrace deposits underlying the potentially liquefiable soils may be sloping away from the site at a gradient of 0.5 percent.

Analysis of the potential for lateral spread was performed using the method proposed by Zhang et. al. (2004) to evaluate the potential for lateral spread and the resulting lateral displacements. The analyses of lateral spread were performed by assuming a high groundwater table of 5 feet below the surface, a magnitude 6.67 earthquake, a peak horizontal acceleration of 0.734g (PGA_M), and a ground slope of 0.5 percent. Based on the results of the analyses, it is anticipated that lateral displacements of 1.5 feet could occur at the ground surface (see enclosed calculation sheet, Figure 8).

The foundation design recommendations presented in this report are intended to minimize the effects of lateral spread on the proposed improvements.

6.6 Slope Stability

The topography at the site is relatively level and the site is not located within an area identified as having a potential for slope instability (CDMG, 1997b; City of Newport Beach, 2006). There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Therefore, the potential for slope stability hazards to adversely affect the proposed development is considered low.

6.7 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. Based on a review of the City of Newport Beach (2006) and the Orange County Safety Element (2004), the site is not located within a potential inundation area for an earthquake-induced dam failure. Therefore, the probability of earthquake-induced flooding is considered very low.

6.8 Tsunamis, Seiches, and Flooding

The site is located approximately 250 feet from the Pacific Ocean. According to the City of Newport Beach General Plan (2006) and the State of California (CGS, 2009), the site is located within a tsunami inundation hazard zone. Therefore, there is a potential for tsunamis to adversely impact the site.

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

The site is within an area of minimal flooding (Zone X) as defined by the Federal Emergency Management Agency (FEMA, 2019, City of Newport Beach, 2006).

6.9 Oil Fields & Methane Potential

Based on a review of the California Division of Oil, Gas and Geothermal Resources (DOGGR) Well Finder Website (DOGGR, 2019), the site is not located within the limits of an oilfield and oil or gas wells are not located in the immediate site vicinity. However, due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the location map and undocumented wells could be encountered during construction. Any wells encountered during construction will need to be properly abandoned in accordance with the current requirements of the DOGGR.

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

6.10 Subsidence

Subsidence occurs when a large portion of land is displaced vertically, usually due to the withdrawal of groundwater, oil, or natural gas. Soils that are particularly subject to subsidence include those with high silt or clay content. The site is not located within an area of known ground subsidence (Orange County, 2004). No large-scale extraction of groundwater, gas, oil, or geothermal energy is occurring or planned at the site or in the general site vicinity. There appears to be little or no potential for ground subsidence due to withdrawal of fluids or gases at the site.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

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

7.1.2 Up to 1½ feet of existing artificial fill was encountered during the site investigation. The existing fill encountered is believed to be the result of past grading and construction activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. It is our opinion that the existing fill, in its present condition, is not suitable for direct support of proposed foundations or slabs. The existing fill and site soils are suitable for re-use as engineered fill provided the recommendations in the Grading section of this report are followed (see Section 7.4).

7.1.3 The enclosed liquefaction settlement analyses indicates that the site soils could be susceptible to approximately 1.1 inches of total settlement as a result of the Design Earthquake peak ground acceleration ($\frac{2}{3}PGA_M$). Differential settlement at the foundation level is anticipated to be less than 0.7 inches over a distance of 30 feet. Furthermore, the analyses indicate that lateral displacements of 1.5 feet could affect the site. The foundation design recommendations presented herein are intended to minimize the effects of settlement on proposed improvements.

7.1.4 Potentially liquefiable soils were encountered between 5 and 11 feet below the ground surface. Below this depth, the in situ soils are relatively dense and not considered susceptible to liquefaction. These materials are not considered suitable for direct support of the proposed structure. The potentially liquefiable soils must be excavated and replaced, improved, or penetrated through by foundation excavations.

7.1.5 Based on our conversations with the design team, it is recommended that ground improvement consisting of Rapid Impact Compaction (RIC) be performed. Where feasible, it is recommended that the RIC extend laterally a minimum distance of 20 feet beyond the building footprint area. The Client should be aware that RIC is designed and performed by a specialty geotechnical contractor. Recommendations for the design of Rapid Impact Compaction are provided in Section 7.5.

7.1.6 Subsequent to performing RIC, the proposed building may be supported on reinforced concrete mat foundation deriving support in the improved soils. Recommendations for the design of a mat foundation are provided in Section 7.6.

7.1.7 Improvements which are not supported on improved soils, such as walkways, paving, and utilities, may still be subject to seismic and/or static settlement. The client should consider the flexibility of the products and pavements being installed. Utilities traversing through existing site soil should use flexible connections in order to minimize the damage to underground installations caused by potential soil movements.

7.1.8 It should be noted that implementation of the recommendations presented herein is not intended to completely prevent damage to the structure during the occurrence of strong ground shaking as a result of nearby earthquakes. It is intended that the structure be designed in such a way that the amount of damage incurred as a result of strong ground shaking be minimized.

7.1.9 Groundwater was encountered a depths of 6 to 7 feet below existing ground surface. Given the proximity of the site to the coastline, the depth to groundwater is likely also influenced by tidal fluctuations. Furthermore, it is our understanding that future sea level rise is possible and future water levels should be considered for design. Based on these considerations, groundwater may be encountered during construction activities.

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

7.1.11 Foundations for small outlying structures, such as block walls up to 6 feet high, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and proper compaction cannot be performed or is undesirable, foundations may derive support directly in the undisturbed old marine deposits found at or below a depth of 18 inches below existing ground surface, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved in writing by a Geocon representative.

7.1.12 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 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 7.10).

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

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

7.2 Soil and Excavation Characteristics

7.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Caving should be anticipated in unshored excavations, especially where saturated and granular soils are encountered. The contractor should be aware that casing will likely be required during foundation construction and formwork may be required to prevent caving of shallow foundation excavations.

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

7.2.3 All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping and shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 7.13).

7.2.4 The upper 5 feet of existing site soils encountered during this investigation are considered to have a "very low" expansive potential (EI = 0) and are classified as "non-expansive" in accordance with the 2016 California Building Code (CBC) Section 1803.5.3 (see Figure B11). The recommendations presented herein assume that proposed foundations and slabs will derive support in these materials.

7.3 Minimum Resistivity, pH, and Water-Soluble Sulfate

7.3.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on representative samples of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the soils are considered "moderately corrosive" with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B13) and should be considered for design of underground structures.

7.3.2 Laboratory tests were performed on representative samples of the site materials to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B13) and indicate that the on-site materials possess a sulfate exposure class of "S0" to concrete structures as defined by 2016 CBC Section 1904 and ACI 318-14 Table 19.3.1.1. However, concrete structures extending below a depth of 5 feet could be subject to seawater exposure and aggressive sulfate attack. ACI 318 requires a minimum of Type II cement or Type I plus a pozzolan to resist the moderate sulfate attack from seawater (ACI 318-14 Table 19.3.1.1).

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

7.4 Grading

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

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

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

7.4.4 It is recommended that the proposed structure be supported on a reinforced concrete mat foundation deriving support in the alluvial soils which have been improved by Rapid Impact Compaction (RIC). Where feasible, it is recommended that the RIC extend laterally a minimum distance of 20 feet beyond the building footprint area. Recommendations for the design of Rapid Impact Compaction are provided in Section 7.5.

7.4.5 All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

7.4.6 All fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to optimum moisture content, and properly compacted to a minimum of 90 percent of the maximum dry density per ASTM D 1557 (latest edition).

7.4.7. Where new paving is to be placed, it is recommended that all existing fill 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 optimum moisture content, and compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 7.10).

7.4.8 Foundations for small outlying structures, such as block walls up to 6 feet high, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and proper compaction cannot be performed or is undesirable, foundations may derive support directly in the undisturbed old marine deposits found at or below a depth of 18 inches, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved in writing by a Geocon representative.

7.4.9 Although not anticipated for this project, all imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than 6 inches in diameter shall not be used in the fill. Import soils used as structural fill should have an expansion index less than 20 and corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B13).

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

7.4.11 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.

7.5 Ground Improvement – Rapid Impact Compaction

7.5.1 Due to the potential for seismically-induced settlements, it is recommended that soil improvement consisting of Rapid Impact Compaction (RIC) be performed. Subsequent to the performance of RIC, the proposed structure may be supported on a reinforced concrete mat foundation deriving support in the improved soils.

7.5.2 Ground improvement through RIC uses a hydraulic hammer to repeatedly strike the ground surface to achieve densification. RIC is most effective when being used to treat granular soils up to 20 feet below the surface. As compaction and densification is achieved, additional fill may be required to maintain the desired elevation. Additionally, since RIC uses dynamic compaction, the vibrations and noise produced by RIC must be tolerable to the site and adjacent properties.

7.5.3 The pattern and depth of the ground improvements may vary depending upon the purposes of mitigation and stratigraphic conditions. The contractor should design the RIC based on the settlement and bearing pressure criteria stated herein. The contractor should evaluate the post-ground improvement static and dynamic settlements within the remediation zone and provide this information to the project structural engineer for consideration in the design of the structures.

7.5.4 The RIC ground improvement should extend at least 20 feet laterally outside the edge of planned building structure, where feasible.

7.5.5 RIC design should be based on settlement criteria of a maximum combined static and seismic differential settlement of 1 inch over a distance of 40 feet with an allowable bearing pressure of 3,500 pounds per square foot (psf).

7.5.6 The RIC design package should be submitted to Geocon West, Inc. for review at least two weeks prior to mobilization for construction. Within the design package, the specialty contractor should outline a performance and load testing program to verify the effectiveness of the ground improvement and to confirm the bearing capacity of the improved soils. During the load testing, a representative of Geocon should be present to observe the RIC and testing. The information obtained from the load testing should be used to modify the depth necessary to achieve design capacities, as well as develop installation criteria that can be used during construction.

7.5.7 Common testing methods include a plate-load test or geophysical test methods. Where plate-load testing is performed, the load test should be performed to a capacity of 1.5 times the design load. As a minimum, we recommended at least two load tests be performed. Where geophysical test methods are performed, an initial baseline test should be performed prior to the start of ground improvement. Once the baseline measurements are established, the threshold for achieving the desired bearing pressure and settlement will be established.

7.6 Mat Foundation Design

7.6.1 Subsequent to performing ground improvement, the proposed structure may be supported on a reinforced concrete mat foundation deriving support in the improved soil. A mat foundation system is more capable of minimizing the effects of differential settlement and has sufficient rigidity to allow the structure to behave more uniformly. However, re-leveling of the mat foundation could be necessary following strong ground shaking through the use of mud jacking or other similar techniques if differential settlement occurs.

7.6.2 Based on information provided by a specialty design-build contractor, an average allowable soil bearing pressure of 3,500 psf is anticipated subsequent to performance of the RIC ground improvement. The design bearing pressure should be confirmed by the RIC contractor and through load testing (see Section 7.5).

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

7.6.4 It is recommended that a modulus of subgrade reaction of 150 pounds per cubic inch (pci) be utilized for the design of the mat foundation bearing in improved soils. This value is a unit value for use with a 1-foot square footing. The modulus should be reduced in accordance with the following equation when used with larger foundations:

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

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

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

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

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

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

7.7 Miscellaneous Foundations

7.7.1 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures, which will not be structurally supported by the proposed building, may be supported on conventional foundations deriving support on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed or is undesirable, such as adjacent to property lines, foundations may derive support in the undisturbed old marine deposits found at or below a depth of 18 inches, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials.

7.7.2 If the soils exposed in the excavation bottom are 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. Miscellaneous foundations may be designed for a bearing value of 1,500 psf, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 12 inches into the recommended bearing material. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

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

7.8 Lateral Design

7.8.1 Resistance to lateral loading may be provided by friction acting at the base of foundations, slabs and by passive earth pressure. An allowable coefficient of friction of 0.4 may be used with the dead load forces in the newly placed engineered fill and competent beach deposits or undisturbed old marine deposits.

7.8.2 Passive earth pressure for the sides of foundations and slabs poured against newly placed engineered fill or competent beach deposits above the groundwater table may be computed as an equivalent fluid having a density of 280 pcf with a maximum earth pressure of 2,800 psf. Passive earth pressure for the sides of foundations poured against undisturbed old marine deposits below the groundwater table may be computed as an equivalent fluid having a density of 140 pcf with a maximum earth pressure of 1,400 psf (values have been reduced for buoyancy). When combining passive and friction for lateral resistance, the passive component should be reduced by one-third. A one-third increase in the passive value may be used for wind or seismic loads.

7.9 Exterior Concrete Slabs-on-Grade

7.9.1 Exterior slabs, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Prior to construction of slabs, the upper 12 inches of subgrade should be moistened to optimum moisture content and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Crack control joints should be spaced at intervals not greater than 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.

7.9.2 The moisture content of the slab subgrade should be maintained and sprinkled as necessary to maintain a moist condition as would be expected in any concrete placement.

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

7.10 Preliminary Pavement Recommendations

7.10.1 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 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 twelve inches of paving subgrade should be scarified, moisture conditioned to optimum moisture content, and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

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

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

PRELIMINARY PAVEMENT DESIGN SECTIONS

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

7.10.4 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).

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

7.10.6 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.

7.11 Retaining Wall Design

7.11.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 5 feet. In the event that walls significantly higher than 5 feet are planned, Geocon should be contacted for additional recommendations.

7.11.2 Retaining walls with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure) of 30 pcf.

7.11.3 Restrained walls are those that are not allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure) of 57 pcf.

7.11.4 The wall pressures provided above assume that the proposed retaining walls will support relatively undisturbed sand dune deposits or engineered fill derived from onsite soils. If import soil will be used to backfill proposed retaining walls, revised earth pressures may be required to account for the geotechnical properties of the import soil used as engineered fill. This should be evaluated once the use of import soil is established. All imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site.

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

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

7.11.7 Retaining wall foundations may be supported on conventional foundations deriving support in newly placed engineered fill.

7.11.8 Continuous footings may be designed for an allowable bearing capacity of 1,500 pounds per square foot (psf), and should be a minimum of 12 inches in width and 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material.

7.11.9 Isolated spread foundations may be designed for an allowable bearing capacity of 2,000 psf, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material.

7.11.10 The soil bearing pressure above may be increased by 200 psf and 500 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing pressure of 2,500 psf. The allowable bearing pressure may be increased by one-third for transient loads due to wind or seismic forces.

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

7.11.12 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.

7.11.13 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.

7.12 Retaining Wall Drainage

7.12.1 Retaining walls should be provided with a drainage system extended at least two-thirds the height of the wall. At the base of the drain system, a subdrain covered with a minimum of 12 inches of gravel should be installed, and a compacted fill blanket or other seal placed at the surface (see Figure 9). 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.

7.12.2 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 10). 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.

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

7.12.4 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.

7.13 Temporary Excavations

7.13.1 Excavations up to 5 feet in height may be required during construction operations. The excavations are expected to expose artificial fill and beach deposits, which may be subject to excessive caving. Vertical excavations up to five feet in height may be attempted where not surcharged by adjacent traffic or structures; however, the contractor should be prepared for caving sands in open excavations.

7.13.2 Vertical excavations greater than five feet or where surcharged by existing structures will require sloping or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments could be sloped back at a uniform 1:1 slope gradient or flatter, up to a maximum height of 6 feet. A uniform slope does not have a vertical portion.

7.13.3 If excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures such as slot-cutting or shoring may be necessary in order to maintain lateral support of offsite improvements. Recommendations for alterative temporary excavation measures can be provided under separate cover, if needed.

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

7.14 Surface Drainage

7.14.1 Proper surface drainage is critical to the future performance of the project. Uncontrolled infiltration of irrigation excess and storm runoff into the soils can adversely affect the performance of the planned improvements. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the original designed engineering properties. Proper drainage should be maintained at all times.

7.14.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2016 CBC 1804.4 or other applicable standards. Positive site drainage should be provided away from structures, pavement, and the tops of slopes to swales or other controlled drainage structures. Pavement areas should be fine graded such that water is not allowed to pond.

7.14.3 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.

7.15 Plan Review

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

LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

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

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

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

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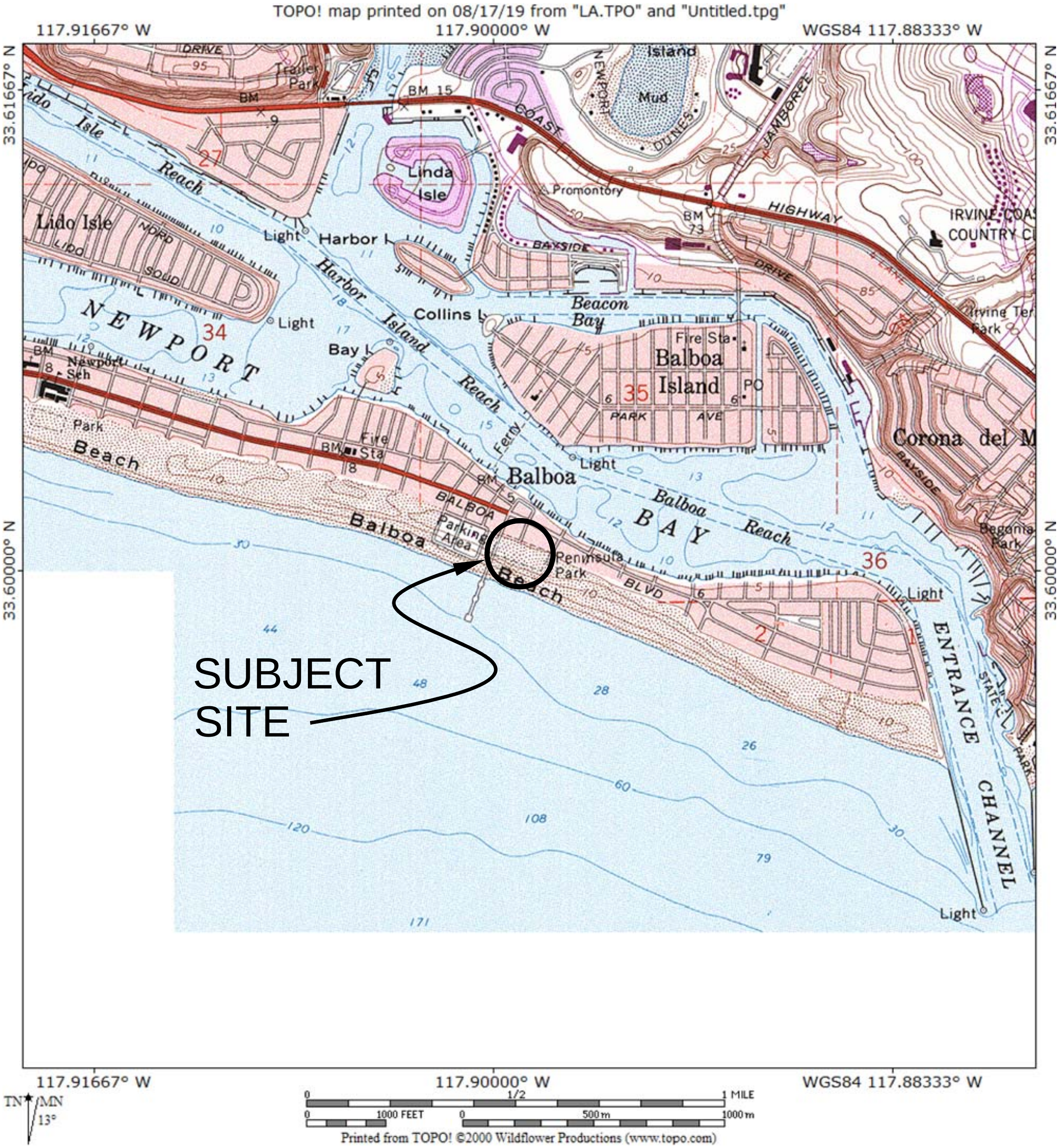

REFERENCE:U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, NEWPORT BEACH, CA QUADRANGLE

GEOCON WEST, INC.

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DRAFTED BY: RA	CHECKED BY: SFK

VICINITY MAP

50 MAIN STREET
NEWPORT BEACH, CALIFORNIA

OCT. 2020	PROJECT NO. W1033-88-01	FIG. 1

LEGEND

B2 Approximate Location of Boring

Approximate Location of Proposed Structure

0 30' 60'

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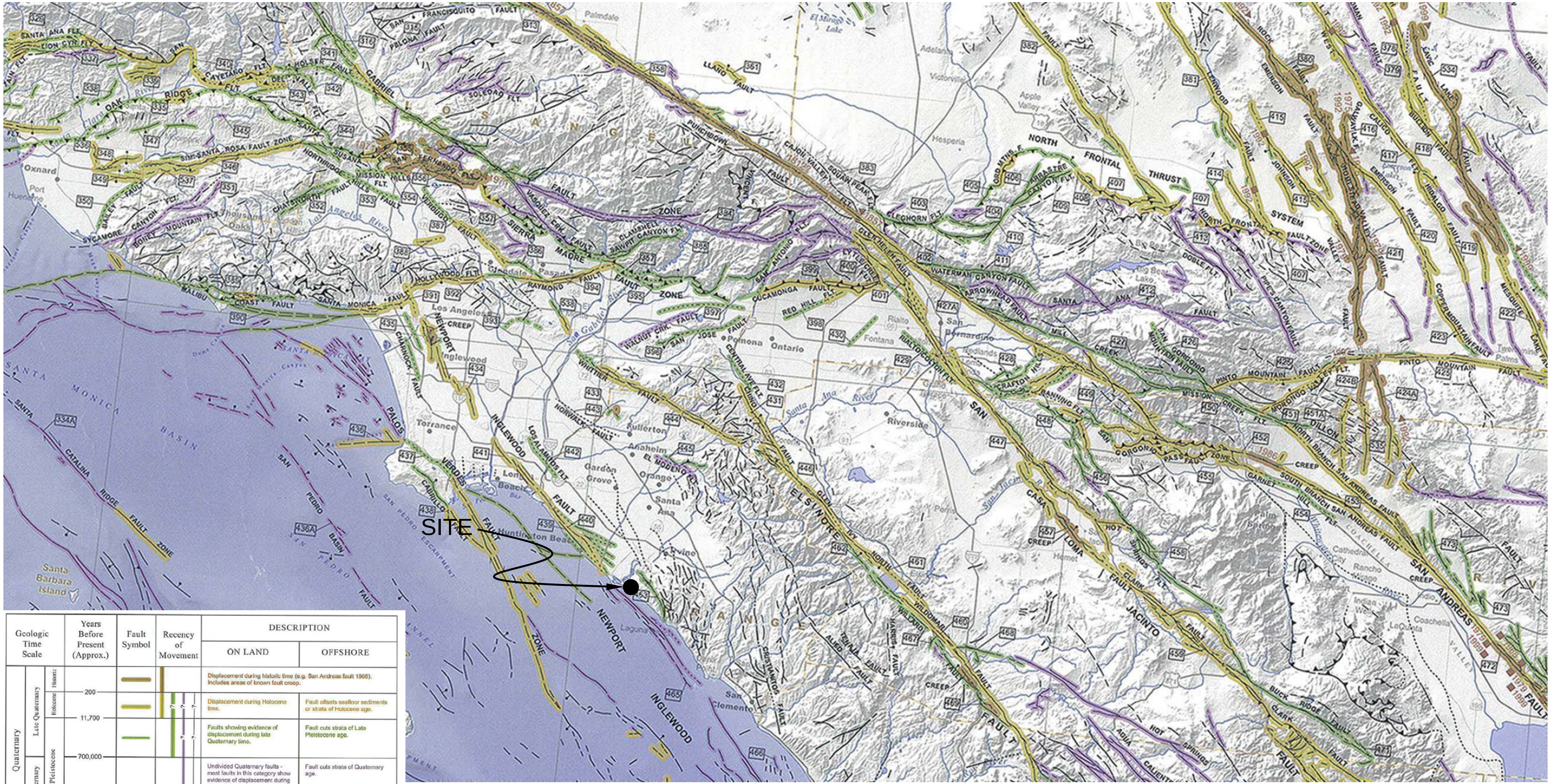
SITE PLAN

50 MAIN STREET
NEWPORT BEACH, CALIFORNIA

OCT. 2020	PROJECT NO. W1033-88-01	FIG. 2

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



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION: ON LAND	DESCRIPTION: OFFSHORE
Quaternary	Late Quaternary	Historic				Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	200			Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	11,700			Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		700,000			Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary			1,600,000*			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.
			4.5 billion (Age of Earth)				

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

N

0 12 24 Miles

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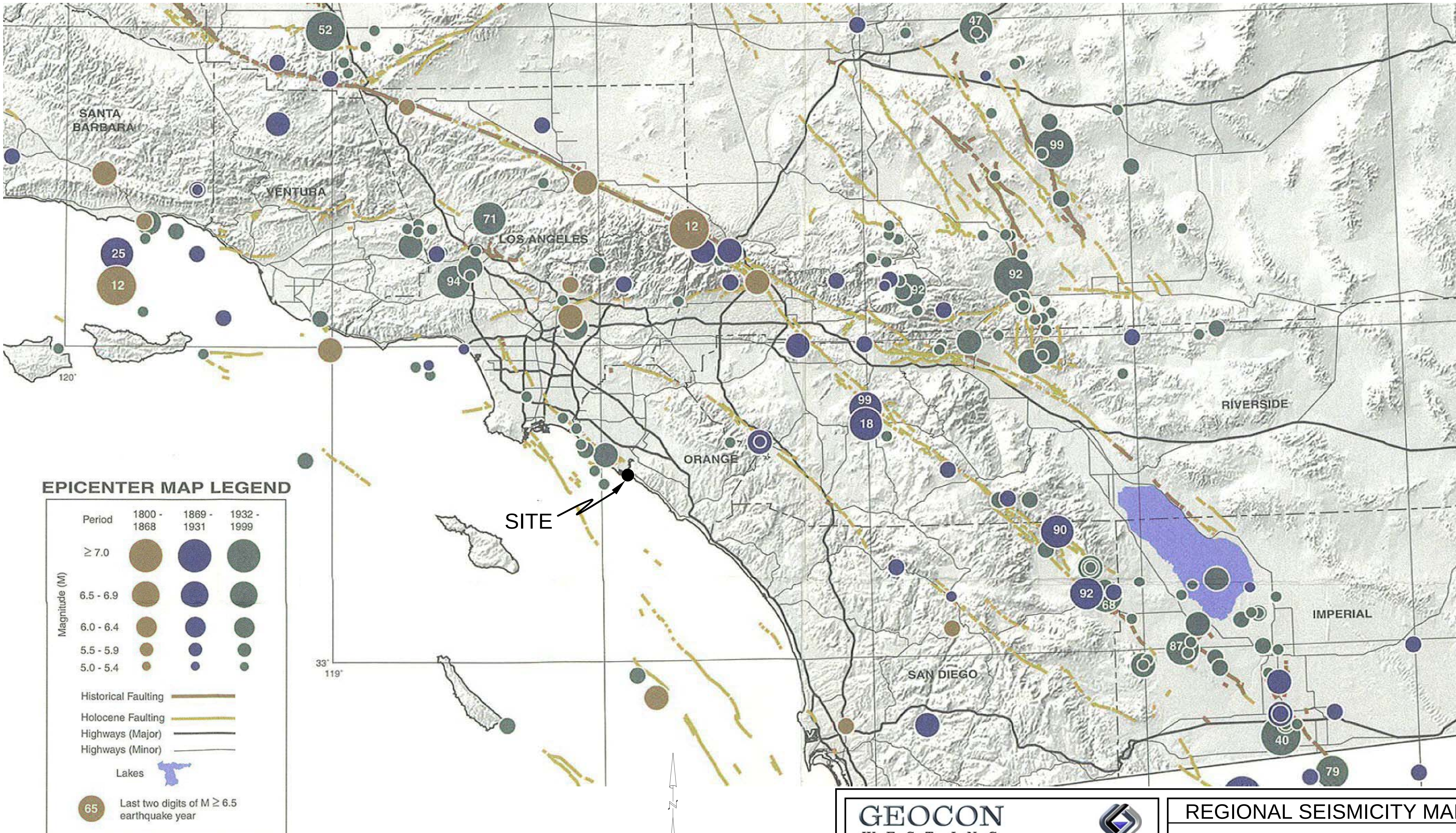
REGIONAL FAULT MAP

50 MAIN STREET
NEWPORT BEACH, CALIFORNIA

OCT. 2020 | PROJECT NO. W1033-88-01 | FIG. 3



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

GEOCON WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: RA	CHECKED BY: SFK

REGIONAL SEISMICITY MAP

50 MAIN STREET
NEWPORT BEACH, CALIFORNIA

OCT. 2020	PROJECT NO. W1033-88-01	FIG. 4

GEOCON

Client : 50 Main St, Newport Beach
File No. : W1033-88-01
Boring : 1

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
DESIGN EARTHQUAKE

NCEER (1996) METHOD

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.68
Peak Horiz. Acceleration PGA_M (g):	0.734
2/3 PGA_M (g):	0.490
Calculated Mag.Wtg.Factor:	0.747
Historic High Groundwater:	5.0
Groundwater Depth During Exploration:	7.0

By Thomas F. Blake (1994-1996)

ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.25
Rod Len.Corr.(CR)(0-no or 1-yes):	1.0
Bore Dia. Corr. (CB):	1.00
Sampler Corr. (CS):	1.20
Use Ksigma (0 or 1):	1.0

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf):	62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	111.1	0	11.0	1.0	1	2	80	1.700	21.0	111.1	0.230	0.998	0.237	--
2.0	111.1	0	11.0	2.0	1	2	78	1.700	21.0	111.1	0.230	0.993	0.236	--
3.0	111.1	0	11.0	3.0	1	2	76	1.700	21.0	111.1	0.230	0.989	0.235	--
4.0	111.1	0	11.0	4.0	1	2	75	1.700	21.0	111.1	0.230	0.984	0.234	--
5.0	111.1	1	11.0	5.0	1	2	73	1.700	21.0	48.7	0.230	0.979	0.248	0.93
6.0	111.1	1	11.0	6.0	1	2	73	1.700	21.0	48.7	0.230	0.975	0.274	0.84
7.0	124.2	1	11.0	7.0	1	2	72	1.700	21.0	61.8	0.230	0.970	0.294	0.78
8.0	124.2	1	11.0	8.0	1	2	71	1.658	20.5	61.8	0.224	0.966	0.309	0.73
9.0	124.2	1	11.0	9.0	1	2	70	1.595	19.7	61.8	0.215	0.961	0.321	0.67
10.0	124.2	1	15.0	10.0	1	4	81	1.538	26.0	61.8	0.302	0.957	0.330	0.92
11.0	124.2	1	15.0	10.0	1	4	81	1.487	25.1	61.8	0.287	0.952	0.338	0.85
12.0	134.4	1	28.0	15.0	1		104	1.437	48.7	72.0	Infin.	0.947	0.344	Non-Liq.
13.0	134.4	1	28.0	15.0	1		104	1.388	47.0	72.0	Infin.	0.943	0.348	Non-Liq.
14.0	134.4	1	28.0	15.0	1		104	1.344	45.5	72.0	Infin.	0.938	0.351	Non-Liq.
15.0	134.4	1	28.0	15.0	1		104	1.304	44.2	72.0	Infin.	0.934	0.354	Non-Liq.
16.0	134.4	1	28.0	15.0	1		104	1.268	42.9	72.0	Infin.	0.929	0.356	Non-Liq.
17.0	134.4	1	37.0	17.5	1		116	1.234	58.6	72.0	Infin.	0.925	0.357	Non-Liq.
18.0	134.4	1	37.0	17.5	1		116	1.203	57.1	72.0	Infin.	0.920	0.358	Non-Liq.
19.0	134.4	1	37.0	17.5	1		116	1.174	55.7	72.0	Infin.	0.915	0.359	Non-Liq.
20.0	130.9	1	37.0	17.5	1		116	1.147	54.5	68.5	Infin.	0.911	0.360	Non-Liq.
21.0	130.9	1	37.0	17.5	1		116	1.123	53.3	68.5	Infin.	0.906	0.361	Non-Liq.
22.0	130.9	1	37.0	22.5	1		110	1.101	56.7	68.5	Infin.	0.902	0.361	Non-Liq.
23.0	130.9	1	37.0	22.5	1		110	1.080	55.6	68.5	Infin.	0.897	0.361	Non-Liq.
24.0	130.9	1	37.0	22.5	1		110	1.060	54.6	68.5	Infin.	0.893	0.361	Non-Liq.
25.0	125.2	1	37.0	22.5	1		110	1.041	53.6	62.8	Infin.	0.888	0.361	Non-Liq.
26.0	125.2	1	37.0	22.5	1		110	1.025	52.8	62.8	Infin.	0.883	0.361	Non-Liq.
27.0	125.2	1	37.0	27.5	1		106	1.009	54.8	62.8	Infin.	0.879	0.361	Non-Liq.
28.0	125.2	1	37.0	27.5	1		106	0.994	54.0	62.8	Infin.	0.874	0.361	Non-Liq.
29.0	125.2	1	37.0	27.5	1		106	0.979	53.2	62.8	Infin.	0.870	0.361	Non-Liq.
30.0	126.0	1	37.0	27.5	1		106	0.966	52.5	63.6	Infin.	0.865	0.360	Non-Liq.
31.0	126.0	1	37.0	27.5	1		106	0.952	51.7	63.6	Infin.	0.861	0.360	Non-Liq.
32.0	126.0	1	39.0	32.5	1		105	0.939	54.9	63.6	Infin.	0.856	0.359	Non-Liq.
33.5	126.0	1	39.0	32.5	1		105	0.924	54.0	63.6	Infin.	0.850	0.358	Non-Liq.
34.0	122.6	1	40.0	37.5	1		102	0.918	55.1	60.2	Infin.	0.846	0.357	Non-Liq.
35.0	122.6	1	40.0	37.5	1		102	0.904	54.3	60.2	Infin.	0.842	0.357	Non-Liq.
36.0	122.6	1	40.0	37.5	1		102	0.894	53.6	60.2	Infin.	0.838	0.356	Non-Liq.
37.0	122.6	1	40.0	37.5	1		102	0.884	53.0	60.2	Infin.	0.833	0.356	Non-Liq.
38.0	122.6	1	40.0	37.5	1		102	0.874	52.4	60.2	Infin.	0.829	0.355	Non-Liq.
39.0	122.6	1	40.0	37.5	1		102	0.865	51.9	60.2	Infin.	0.824	0.354	Non-Liq.
40.0	127.3	1	40.0	37.5	1		102	0.855	51.3	64.9	Infin.	0.819	0.353	Non-Liq.
41.0	127.3	1	40.0	37.5	1		102	0.845	50.7	64.9	Infin.	0.815	0.352	Non-Liq.
42.0	127.3	1	47.0	42.5	1		107	0.836	59.0	64.9	Infin.	0.810	0.350	Non-Liq.
43.0	127.3	1	47.0	42.5	1		107	0.827	58.3	64.9	Infin.	0.806	0.349	Non-Liq.
44.0	127.3	1	47.0	42.5	1		107	0.819	57.7	64.9	Infin.	0.801	0.347	Non-Liq.
45.0	125.2	1	47.0	42.5	1		107	0.810	57.1	62.8	Infin.	0.797	0.346	Non-Liq.
46.0	125.2	1	47.0	42.5	1		107	0.802	56.6	62.8	Infin.	0.792	0.345	Non-Liq.
47.0	125.2	1	57.0	47.5	1		115	0.795	68.0	62.8	Infin.	0.787	0.343	Non-Liq.
48.0	125.2	1	57.0	47.5	1		115	0.787	67.3	62.8	Infin.	0.783	0.342	Non-Liq.
49.0	125.2	1	57.0	47.5	1		115	0.780	66.7	62.8	Infin.	0.778	0.340	Non-Liq.
50.5	127.8	1	57.0	47.5	1		115	0.771	65.9	65.4	Infin.	0.773	0.339	Non-Liq.

GEOCON

Client : 50 Main St, Newport Beach
File No. : W1033-88-01
Boring : 1

LIQUEFACTION SETTLEMENT ANALYSIS
DESIGN EARTHQUAKE

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.68
PGAM (g):	0.734
2/3 PGAM (g):	0.49
Calculated Mag.Wtg.Factor:	0.747
Historic High Groundwater:	5.0
Groundwater @ Exploration:	7.0

DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS O (TSF)	EFFECT STRESS O' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS (N1)60	Tav/σ'o	LIQUEFACTION SAFETY FACTOR	Volumetric Strain [e15} (%)	EQ. SETTLE. Pe (in.)
1.0	11	111.11	0.028	0.028	80	21	0.318	--	0.00	0.00
2.0	11	111.11	0.083	0.083	78	21	0.318	--	0.00	0.00
3.0	11	111.11	0.139	0.139	76	21	0.318	--	0.00	0.00
4.0	11	111.11	0.194	0.194	75	21	0.318	--	0.00	0.00
5.0	11	111.11	0.250	0.234	73	21	0.339	0.93	1.30	0.16
6.0	11	111.11	0.306	0.259	73	21	0.376	0.84	1.40	0.17
7.0	11	124.1643	0.364	0.286	72	21	0.405	0.78	1.40	0.17
8.0	11	124.1643	0.426	0.317	71	21	0.428	0.73	1.40	0.17
9.0	11	124.1643	0.489	0.348	70	20	0.447	0.67	1.60	0.19
10.0	15	124.1643	0.551	0.379	81	26	0.462	0.92	1.10	0.13
11.0	15	124.1643	0.613	0.410	81	25	0.476	0.85	1.10	0.13
12.0	28	134.4156	0.677	0.443	104	49	0.486	Non-Liq.	0.00	0.00
13.0	28	134.4156	0.745	0.479	104	47	0.494	Non-Liq.	0.00	0.00
14.0	28	134.4156	0.812	0.515	104	46	0.501	Non-Liq.	0.00	0.00
15.0	28	134.4156	0.879	0.551	104	44	0.507	Non-Liq.	0.00	0.00
16.0	28	134.4156	0.946	0.587	104	43	0.513	Non-Liq.	0.00	0.00
17.0	37	134.4156	1.013	0.623	116	59	0.517	Non-Liq.	0.00	0.00
18.0	37	134.4156	1.081	0.659	116	57	0.521	Non-Liq.	0.00	0.00
19.0	37	134.4156	1.148	0.695	116	56	0.525	Non-Liq.	0.00	0.00
20.0	37	130.9091	1.214	0.731	116	54	0.529	Non-Liq.	0.00	0.00
21.0	37	130.9091	1.280	0.765	116	53	0.532	Non-Liq.	0.00	0.00
22.0	37	130.9091	1.345	0.799	110	57	0.536	Non-Liq.	0.00	0.00
23.0	37	130.9091	1.410	0.833	110	56	0.539	Non-Liq.	0.00	0.00
24.0	37	130.9091	1.476	0.868	110	55	0.541	Non-Liq.	0.00	0.00
25.0	37	125.1816	1.540	0.900	110	54	0.544	Non-Liq.	0.00	0.00
26.0	37	125.1816	1.603	0.932	110	53	0.547	Non-Liq.	0.00	0.00
27.0	37	125.1816	1.665	0.963	106	55	0.550	Non-Liq.	0.00	0.00
28.0	37	125.1816	1.728	0.995	106	54	0.553	Non-Liq.	0.00	0.00
29.0	37	125.1816	1.790	1.026	106	53	0.555	Non-Liq.	0.00	0.00
30.0	37	126.0126	1.853	1.058	106	52	0.558	Non-Liq.	0.00	0.00
31.0	37	126.0126	1.916	1.089	106	52	0.560	Non-Liq.	0.00	0.00
32.0	39	126.0126	1.979	1.121	105	55	0.562	Non-Liq.	0.00	0.00
33.5	39	126.0126	2.058	1.161	105	54	0.564	Non-Liq.	0.00	0.00
34.0	40	122.5588	2.089	1.176	102	55	0.565	Non-Liq.	0.00	0.00
35.0	40	122.5588	2.166	1.214	102	54	0.568	Non-Liq.	0.00	0.00
36.0	40	122.5588	2.227	1.244	102	54	0.570	Non-Liq.	0.00	0.00
37.0	40	122.5588	2.288	1.274	102	53	0.571	Non-Liq.	0.00	0.00
38.0	40	122.5588	2.349	1.304	102	52	0.573	Non-Liq.	0.00	0.00
39.0	40	122.5588	2.411	1.334	102	52	0.575	Non-Liq.	0.00	0.00
40.0	40	127.3266	2.473	1.366	102	51	0.576	Non-Liq.	0.00	0.00
41.0	40	127.3266	2.537	1.398	102	51	0.577	Non-Liq.	0.00	0.00
42.0	47	127.3266	2.600	1.430	107	59	0.579	Non-Liq.	0.00	0.00
43.0	47	127.3266	2.664	1.463	107	58	0.580	Non-Liq.	0.00	0.00
44.0	47	127.3266	2.728	1.495	107	58	0.580	Non-Liq.	0.00	0.00
45.0	47	125.154	2.791	1.527	107	57	0.582	Non-Liq.	0.00	0.00
46.0	47	125.154	2.854	1.559	107	57	0.583	Non-Liq.	0.00	0.00
47.0	57	125.154	2.916	1.590	115	68	0.584	Non-Liq.	0.00	0.00
48.0	57	125.154	2.979	1.621	115	67	0.585	Non-Liq.	0.00	0.00
49.0	57	125.154	3.041	1.653	115	67	0.586	Non-Liq.	0.00	0.00
50.5	57	127.8424	3.120	1.693	115	66	0.587	Non-Liq.	0.00	0.00

TOTAL SETTLEMENT = 1.1 INCHES

Figure 6

Client : 50 Main St, Newport Beach
File No. : W1033-88-01
Boring : 1

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
MAXIMUM CONSIDERED EARTHQUAKE

NCEER (1996) METHOD

By Thomas F. Blake (1994-1996)

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LIQUEFACTION CALCULATIONS:

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1.0	111.1	0	11.0	1.0	1	2	80	1.700	21.0	111.1	0.230	0.998	0.369	--
2.0	111.1	0	11.0	2.0	1	2	78	1.700	21.0	111.1	0.230	0.993	0.368	--
3.0	111.1	0	11.0	3.0	1	2	76	1.700	21.0	111.1	0.230	0.989	0.366	--
4.0	111.1	0	11.0	4.0	1	2	75	1.700	21.0	111.1	0.230	0.984	0.364	--
5.0	111.1	1	11.0	5.0	1	2	73	1.700	21.0	48.7	0.230	0.979	0.387	0.59
6.0	111.1	1	11.0	6.0	1	2	73	1.700	21.0	48.7	0.230	0.975	0.426	0.54
7.0	124.2	1	11.0	7.0	1	2	72	1.700	21.0	61.8	0.230	0.970	0.457	0.50
8.0	124.2	1	11.0	8.0	1	2	71	1.658	20.5	61.8	0.224	0.966	0.481	0.47
9.0	124.2	1	11.0	9.0	1	2	70	1.595	19.7	61.8	0.215	0.961	0.499	0.43
10.0	124.2	1	15.0	10.0	1	4	81	1.538	26.0	61.8	0.302	0.957	0.515	0.59
11.0	124.2	1	15.0	10.0	1	4	81	1.487	25.1	61.8	0.287	0.952	0.527	0.54
12.0	134.4	1	28.0	15.0	1		104	1.437	48.7	72.0	Infin.	0.947	0.536	Non-Liq.
13.0	134.4	1	28.0	15.0	1		104	1.388	47.0	72.0	Infin.	0.943	0.542	Non-Liq.
14.0	134.4	1	28.0	15.0	1		104	1.344	45.5	72.0	Infin.	0.938	0.547	Non-Liq.
15.0	134.4	1	28.0	15.0	1		104	1.304	44.2	72.0	Infin.	0.934	0.551	Non-Liq.
16.0	134.4	1	28.0	15.0	1		104	1.268	42.9	72.0	Infin.	0.929	0.554	Non-Liq.
17.0	134.4	1	37.0	17.5	1		116	1.234	58.6	72.0	Infin.	0.925	0.557	Non-Liq.
18.0	134.4	1	37.0	17.5	1		116	1.203	57.1	72.0	Infin.	0.920	0.558	Non-Liq.
19.0	134.4	1	37.0	17.5	1		116	1.174	55.7	72.0	Infin.	0.915	0.559	Non-Liq.
20.0	130.9	1	37.0	17.5	1		116	1.147	54.5	68.5	Infin.	0.911	0.561	Non-Liq.
21.0	130.9	1	37.0	17.5	1		116	1.123	53.3	68.5	Infin.	0.906	0.561	Non-Liq.
22.0	130.9	1	37.0	22.5	1		110	1.101	56.7	68.5	Infin.	0.902	0.562	Non-Liq.
23.0	130.9	1	37.0	22.5	1		110	1.080	55.6	68.5	Infin.	0.897	0.562	Non-Liq.
24.0	130.9	1	37.0	22.5	1		110	1.060	54.6	68.5	Infin.	0.893	0.562	Non-Liq.
25.0	125.2	1	37.0	22.5	1		110	1.041	53.6	62.8	Infin.	0.888	0.562	Non-Liq.
26.0	125.2	1	37.0	22.5	1		110	1.025	52.8	62.8	Infin.	0.883	0.563	Non-Liq.
27.0	125.2	1	37.0	27.5	1		106	1.009	54.8	62.8	Infin.	0.879	0.563	Non-Liq.
28.0	125.2	1	37.0	27.5	1		106	0.994	54.0	62.8	Infin.	0.874	0.562	Non-Liq.
29.0	125.2	1	37.0	27.5	1		106	0.979	53.2	62.8	Infin.	0.870	0.562	Non-Liq.
30.0	126.0	1	37.0	27.5	1		106	0.966	52.5	63.6	Infin.	0.865	0.561	Non-Liq.
31.0	126.0	1	37.0	27.5	1		106	0.952	51.7	63.6	Infin.	0.861	0.560	Non-Liq.
32.0	126.0	1	39.0	32.5	1		105	0.939	54.9	63.6	Infin.	0.856	0.559	Non-Liq.
33.5	126.0	1	39.0	32.5	1		105	0.924	54.0	63.6	Infin.	0.850	0.558	Non-Liq.
34.0	122.6	1	40.0	37.5	1		102	0.918	55.1	60.2	Infin.	0.846	0.556	Non-Liq.
35.0	122.6	1	40.0	37.5	1		102	0.904	54.3	60.2	Infin.	0.842	0.556	Non-Liq.
36.0	122.6	1	40.0	37.5	1		102	0.894	53.6	60.2	Infin.	0.838	0.555	Non-Liq.
37.0	122.6	1	40.0	37.5	1		102	0.884	53.0	60.2	Infin.	0.833	0.554	Non-Liq.
38.0	122.6	1	40.0	37.5	1		102	0.874	52.4	60.2	Infin.	0.829	0.553	Non-Liq.
39.0	122.6	1	40.0	37.5	1		102	0.865	51.9	60.2	Infin.	0.824	0.551	Non-Liq.
40.0	127.3	1	40.0	37.5	1		102	0.855	51.3	64.9	Infin.	0.819	0.549	Non-Liq.
41.0	127.3	1	40.0	37.5	1		102	0.845	50.7	64.9	Infin.	0.815	0.547	Non-Liq.
42.0	127.3	1	47.0	42.5	1		107	0.836	59.0	64.9	Infin.	0.810	0.545	Non-Liq.
43.0	127.3	1	47.0	42.5	1		107	0.827	58.3	64.9	Infin.	0.806	0.543	Non-Liq.
44.0	127.3	1	47.0	42.5	1		107	0.819	57.7	64.9	Infin.	0.801	0.541	Non-Liq.
45.0	125.2	1	47.0	42.5	1		107	0.810	57.1	62.8	Infin.	0.797	0.539	Non-Liq.
46.0	125.2	1	47.0	42.5	1		107	0.802	56.6	62.8	Infin.	0.792	0.537	Non-Liq.
47.0	125.2	1	57.0	47.5	1		115	0.795	68.0	62.8	Infin.	0.787	0.535	Non-Liq.
48.0	125.2	1	57.0	47.5	1		115	0.787	67.3	62.8	Infin.	0.783	0.532	Non-Liq.
49.0	125.2	1	57.0	47.5	1		115	0.780	66.7	62.8	Infin.	0.778	0.530	Non-Liq.
50.5	127.8	1	57.0	47.5	1		115	0.771	65.9	65.4	Infin.	0.773	0.527	Non-Liq.

GEOCON

Client : 50 Main St, Newport Beach
File No. : W1033-88-01
Boring : 1

LIQUEFACTION SETTLEMENT ANALYSIS
MAXIMUM CONSIDERED EARTHQUAKE

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

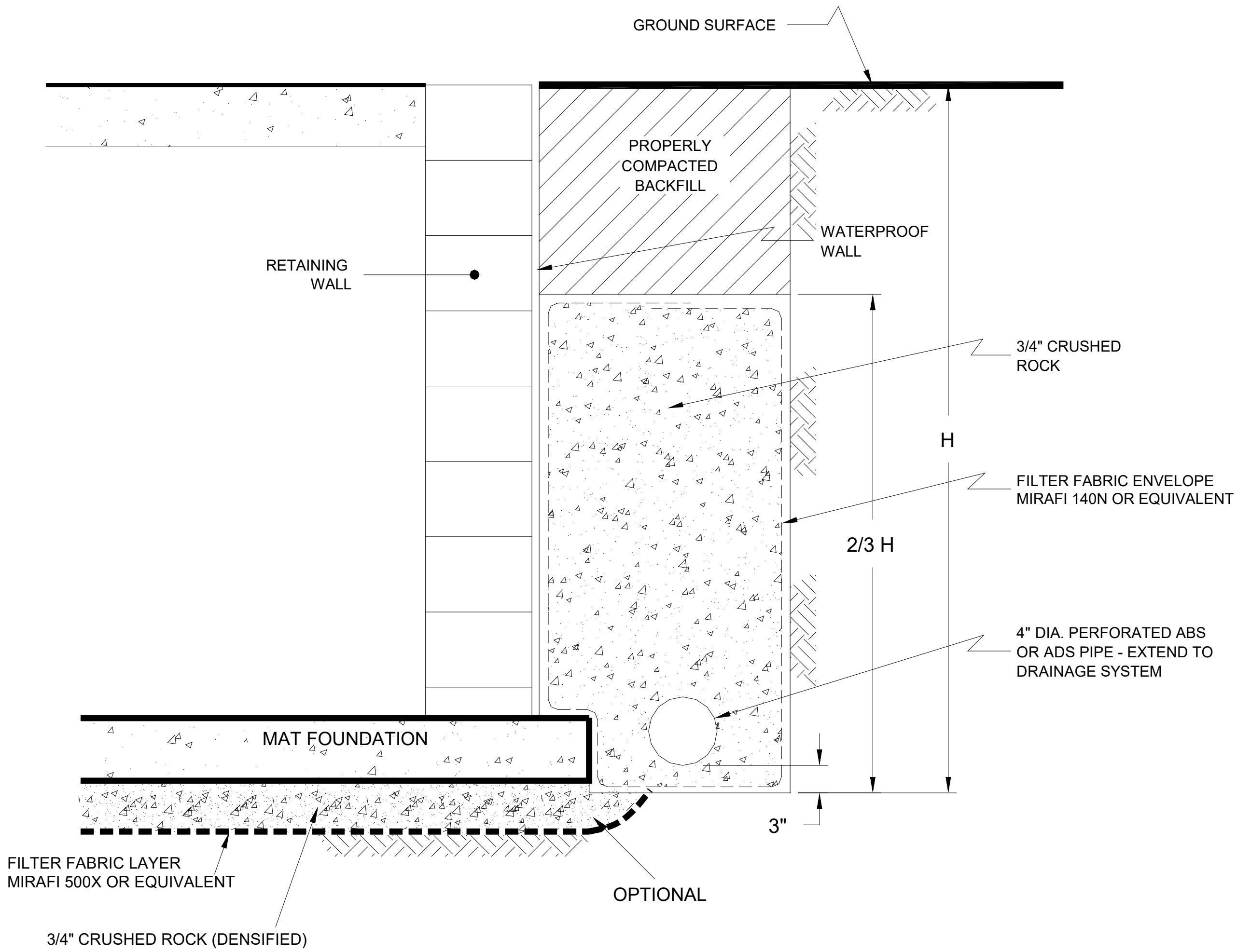
NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.67
PGA_M (g):	0.734
Calculated Mag.Wtg.Factor:	0.744
Historic High Groundwater:	5.0
Groundwater @ Exploration:	7.0

Slope, S	0.5
Height of Sloping Surface Below Ground Surface:	0
Distance to Face:	0

DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS O (TSF)	EFFECT STRESS O' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS (N1)60	Tav/σ'_o	LIQUEFACTION SAFETY FACTOR	Volumetric Strain [e_{15}} (%)	EQ. SETTLE. Pe (in.)	LAT. DISPLACE. LD (ft)
1	11	111.11	0.028	0.028	80	21	0.477	--	0.00	0.00	--
2	11	111.11	0.083	0.083	78	21	0.477	--	0.00	0.00	--
3	11	111.11	0.139	0.139	76	21	0.477	--	0.00	0.00	--
4	11	111.11	0.194	0.194	75	21	0.477	--	0.00	0.00	--
5	11	111.11	0.250	0.234	73	21	0.509	0.62	1.40	0.17	0.11
6	11	111.11	0.306	0.259	73	21	0.563	0.56	1.40	0.17	0.19
7	11	124.1643	0.364	0.286	72	21	0.607	0.52	1.40	0.17	0.31
8	11	124.1643	0.426	0.317	71	21	0.641	0.49	1.40	0.17	0.35
9	11	124.1643	0.489	0.348	70	20	0.670	0.45	1.60	0.19	0.35
10	15	124.1643	0.551	0.379	81	26	0.693	0.61	1.10	0.13	0.06
11	15	124.1643	0.613	0.410	81	25	0.713	0.57	1.10	0.13	0.08
12	28	134.4156	0.677	0.443	104	49	0.729	Non-Liq.	0.00	0.00	--
13	28	134.4156	0.745	0.479	104	47	0.741	Non-Liq.	0.00	0.00	--
14	28	134.4156	0.812	0.515	104	46	0.751	Non-Liq.	0.00	0.00	--
15	28	134.4156	0.879	0.551	104	44	0.761	Non-Liq.	0.00	0.00	--
16	28	134.4156	0.946	0.587	104	43	0.769	Non-Liq.	0.00	0.00	--
17	37	134.4156	1.013	0.623	116	59	0.776	Non-Liq.	0.00	0.00	--
18	37	134.4156	1.081	0.659	116	57	0.782	Non-Liq.	0.00	0.00	--
19	37	134.4156	1.148	0.695	116	56	0.787	Non-Liq.	0.00	0.00	--
20	37	130.9091	1.214	0.731	116	54	0.793	Non-Liq.	0.00	0.00	--
21	37	130.9091	1.280	0.765	116	53	0.798	Non-Liq.	0.00	0.00	--
22	37	130.9091	1.345	0.799	110	57	0.803	Non-Liq.	0.00	0.00	--
23	37	130.9091	1.410	0.833	110	56	0.808	Non-Liq.	0.00	0.00	--
24	37	130.9091	1.476	0.868	110	55	0.812	Non-Liq.	0.00	0.00	--
25	37	125.1816	1.540	0.900	110	54	0.816	Non-Liq.	0.00	0.00	--
26	37	125.1816	1.603	0.932	110	53	0.821	Non-Liq.	0.00	0.00	--
27	37	125.1816	1.665	0.963	106	55	0.825	Non-Liq.	0.00	0.00	--
28	37	125.1816	1.728	0.995	106	54	0.829	Non-Liq.	0.00	0.00	--
29	37	125.1816	1.790	1.026	106	53	0.833	Non-Liq.	0.00	0.00	--
30	37	126.0126	1.853	1.058	106	52	0.836	Non-Liq.	0.00	0.00	--
31	37	126.0126	1.916	1.089	106	52	0.839	Non-Liq.	0.00	0.00	--
32	39	126.0126	1.979	1.121	105	55	0.842	Non-Liq.	0.00	0.00	--
33	39	126.0126	2.042	1.153	105	54	0.845	Non-Liq.	0.00	0.00	--
34	40	122.5588	2.104	1.184	102	55	0.848	Non-Liq.	0.00	0.00	--
35	40	122.5588	2.166	1.214	102	54	0.851	Non-Liq.	0.00	0.00	--
36	40	122.5588	2.227	1.244	102	54	0.854	Non-Liq.	0.00	0.00	--
37	40	122.5588	2.288	1.274	102	53	0.857	Non-Liq.	0.00	0.00	--
38	40	122.5588	2.349	1.304	102	52	0.859	Non-Liq.	0.00	0.00	--
39	40	122.5588	2.411	1.334	102	52	0.862	Non-Liq.	0.00	0.00	--
40	40	127.3266	2.473	1.366	102	51	0.864	Non-Liq.	0.00	0.00	--
41	40	127.3266	2.537	1.398	102	51	0.866	Non-Liq.	0.00	0.00	--
42	47	127.3266	2.600	1.430	107	59	0.867	Non-Liq.	0.00	0.00	--
43	47	127.3266	2.664	1.463	107	58	0.869	Non-Liq.	0.00	0.00	--
44	47	127.3266	2.728	1.495	107	58	0.870	Non-Liq.	0.00	0.00	--
45	47	125.154	2.791	1.527	107	57	0.872	Non-Liq.	0.00	0.00	--
46	47	125.154	2.854	1.559	107	57	0.873	Non-Liq.	0.00	0.00	--
47	57	125.154	2.916	1.590	115	68	0.875	Non-Liq.	0.00	0.00	--
48	57	125.154	2.979	1.621	115	67	0.876	Non-Liq.	0.00	0.00	--
49	57	125.154	3.041	1.653	115	67	0.878	Non-Liq.	0.00	0.00	--
50	57	127.8424	3.104	1.685	115	66	0.879	Non-Liq.	0.00	0.00	--

TOTAL SETTLEMENT =	1.1 INCHES
TOTAL LATERAL DISPLACEMENT=	1.5 FEET



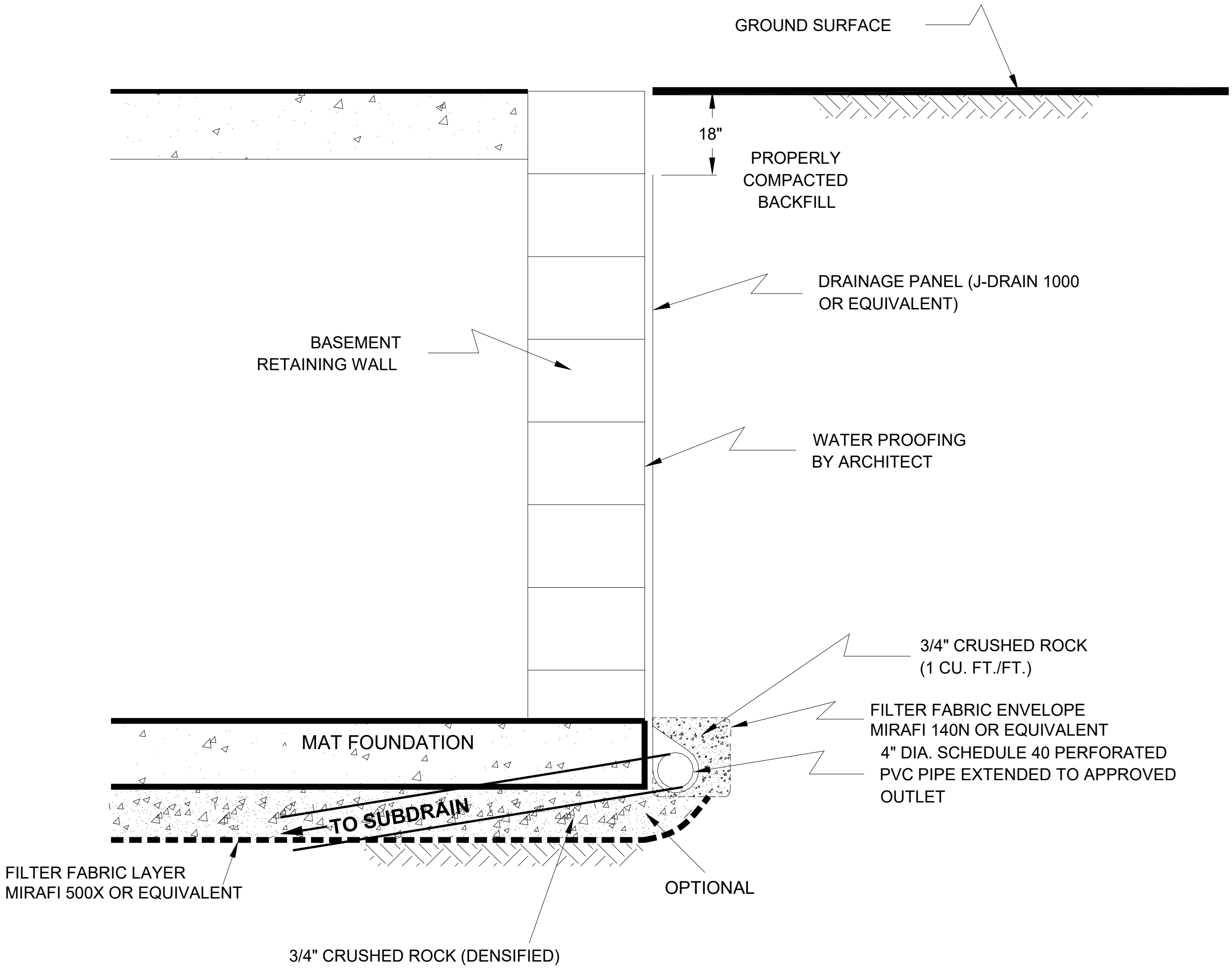
NO SCALE

GEOCON
W E S T, I N C.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

RETAINING WALL DRAIN DETAIL

50 MAIN STREET
NEWPORT BEACH, CALIFORNIA

DRAFTED BY: PZ	CHECKED BY: JTA	OCT. 2020	PROJECT NO. W1033-88-01	FIG. 9



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

NO SCALE

GEOCON
WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

RETAINING WALL DRAIN DETAIL

50 MAIN STREET
NEWPORT BEACH, CALIFORNIA

DRAFTED BY: PZ	CHECKED BY: JTA	OCT. 2020	PROJECT NO. W1033-88-01	FIG. 10

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was initially explored on August 5, 2019 by drilling two 8-inch diameter borings using a truck-mounted mud-rotary drilling machine. The borings were drilled to depths of 20½ and 50½ feet below the existing ground surface. Representative and relatively undisturbed samples were obtained by driving a 3 inch, O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch high by 2³/₈-inch diameter brass sampler rings to facilitate soil removal and testing. Bulk samples were also obtained. Standard Penetration Tests were performed in boring B1.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). The logs of the borings are presented on Figures A1 and A2. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The logs also include our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the boring logs were revised based on subsequent laboratory testing. The locations of the borings are shown on Figure 2.

PROJECT NO. W1033-88-01

BORING 1

ELEV. (MSL.) -- DATE COMPLETED 08/05/2019

EQUIPMENT Mud Rotary BY: JS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**3" AC / 9" Base**			
					ARTIFICIAL FILL Sand, poorly graded, medium dense, moist, light brown, fine-grained.			
2					**BEACH DEPOSITS**			
	B1@2.5'				Sand, poorly graded, medium dense, moist, light brown, fine-grained.	20	102.5	8.4
4								
	B1@5'			SP	- with brown mottles	11		14.9
6			▼					
8	B1@7.5'				- loose, brown, fine-grained with some medium-grained	12	102.7	20.9
10	B1@10'				- medium dense, moist to wet, fine- to medium-grained, trace coarse-grained	15		21.5
					OLD MARINE DEPOSITS Sand, medium dense, wet, brown, fine-grained.			
12	B1@12.5'					47	112.2	19.8
14								
	B1@15'				- fine-grained with some medium-grained	28		23.0
16					- dense			
18	B1@17.5'			SP		37		23.5
20	B1@20'				- very dense	94	108.1	21.1
22					- moist, dense, olive brown, fine-grained			
	B1@22.5'					37		27.0
24								
	B1@25'				- very dense	91	103.2	21.3
26					- dense, moist to wet			
28	B1@27.5'					37		26.2

W1033-88-01 BORING LOGS.GPJ

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

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

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

GEOCON

PROJECT NO. W1033-88-01

BORING 1

ELEV. (MSL.) -- DATE COMPLETED 08/05/2019

EQUIPMENT Mud Rotary BY: JS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B1@30'			SP	- moist, olive gray, some silt	88	102.2	23.3
32	B1@32.5'					39		26.6
34				SM	Silty Sand, dense, moist, gray, fine-grained.			
36	B1@35'					64	96.2	27.4
38	B1@37.5'					40		25.2
40	B1@40'				- very dense	95	102.6	24.1
42					- dense, trace shell fragments			
	B1@42.5'					47		23.0
44								
46	B1@45'				- fine-grained, decrease in shell fragments	50 (5")	102.0	22.7
48	B1@47.5'				- trace medium-grained	57		25.7
50	B1@50'					50 (5")	103.6	23.4

Total depth of boring: 50.5 feet.
Fill to 1.5 feet.
Groundwater encountered at 7 feet.
Backfilled with cement bentonite grout.
Surface restored.

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

W1033-88-01 BORING LOGS.GPJ

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

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

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

PROJECT NO. W1033-88-01

BORING 2

ELEV. (MSL.) -- DATE COMPLETED 08/05/2019

EQUIPMENT Mud Rotary BY: JS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**5" AC / 9" Base**			
					ARTIFICIAL FILL Sand, poorly graded, medium dense, moist, light brown, fine-grained with some medium-grained, some shell fragments.			
2.5	B2@2.5'			SP	**BEACH DEPOSITS** Sand, poorly graded, medium dense, moist, light brown, fine- to medium-grained, trace shell fragments.	29	102.4	6.5
5	B2@5'		▼		- loose, moist to wet, olive brown, fine-grained, some fine gravel	11	89.7	22.2
7.5	B2@7.5'				- medium dense, wet, fine-grained with some medium-grained	27	103.8	18.9
10	B2@10'			SP	**OLD MARINE DEPOSITS** Sand, dense, fine- to medium-grained, olive brown, trace fine shell fragments.	57	106.8	19.5
12.5	B2@12.5'				- increase in shell fragments, trace coarse-grained sand	68	108.5	18.9
15	B2@15'					62	107.8	18.8
20	B2@20				- moist, decrease in coarse-grained sand and shell fragments	71	100.0	25.3

Total depth of boring: 20.5 feet.
Fill to 1.5 feet.
Groundwater encountered at 6 feet.
Backfilled with cement bentonite grout.
Surface restored.

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

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

W1033-88-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

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

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

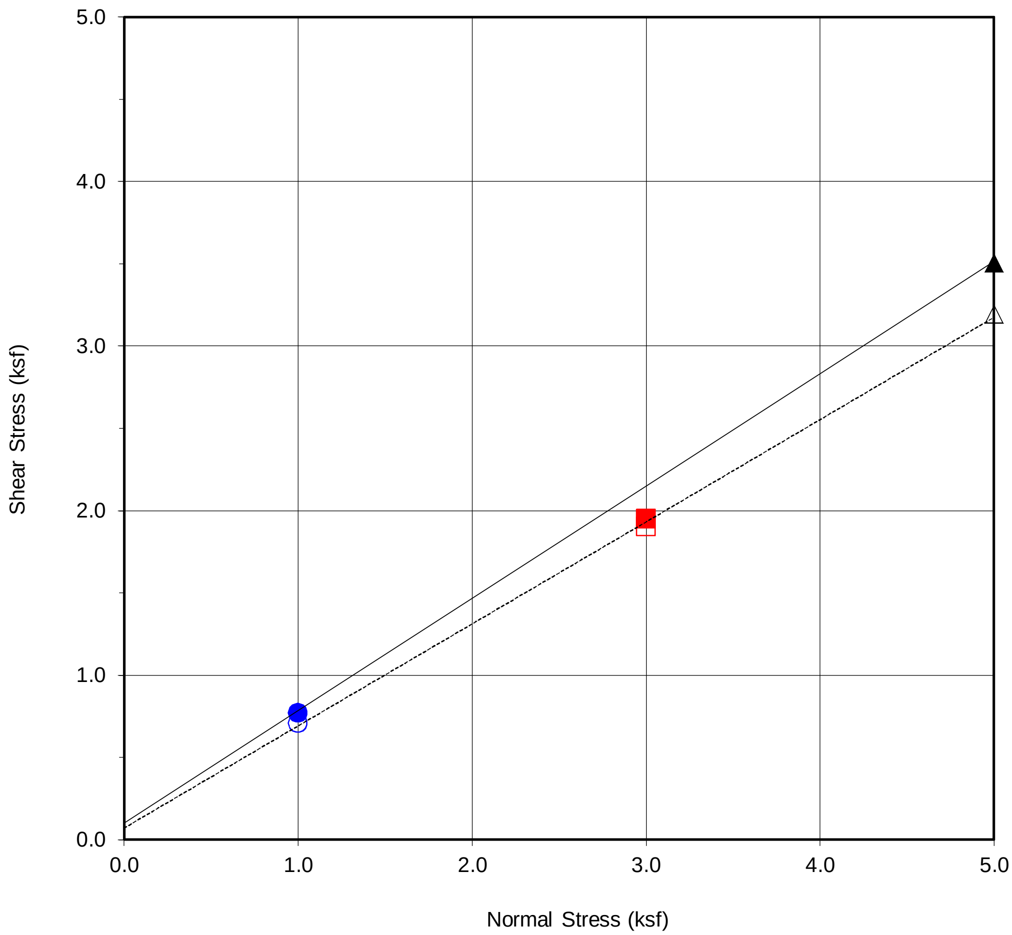
GEOCON

APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the "American Society for Testing and Materials (ASTM)", or other suggested procedures. Selected samples were tested for direct shear strength, consolidation and expansion characteristics, moisture density relationships, grain-size, corrosivity, in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B13. The in-place dry density and moisture content of the samples tested are presented in the boring logs, Appendix A.



Boring No.	**B-2**
Sample No.	**B2@0-5**
Depth (ft)	**0-5**
Sample Type:	Ring

Soil Identification:		
Light Brown Sand (SP)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	100	34.3
Ultimate	71	31.8

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	0.77	1.95	3.50
Shear Stress @ End of Test (ksf)	0.71	1.90	3.19
Deformation Rate (in./min.)	0.005R	0.005R	0.005R
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	13.4	12.9	10.2
Initial Dry Density (pcf)	89.6	90.0	92.2
Initial Degree of Saturation (%)	41.0	40.1	33.1
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	22.7	22.9	22.1

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

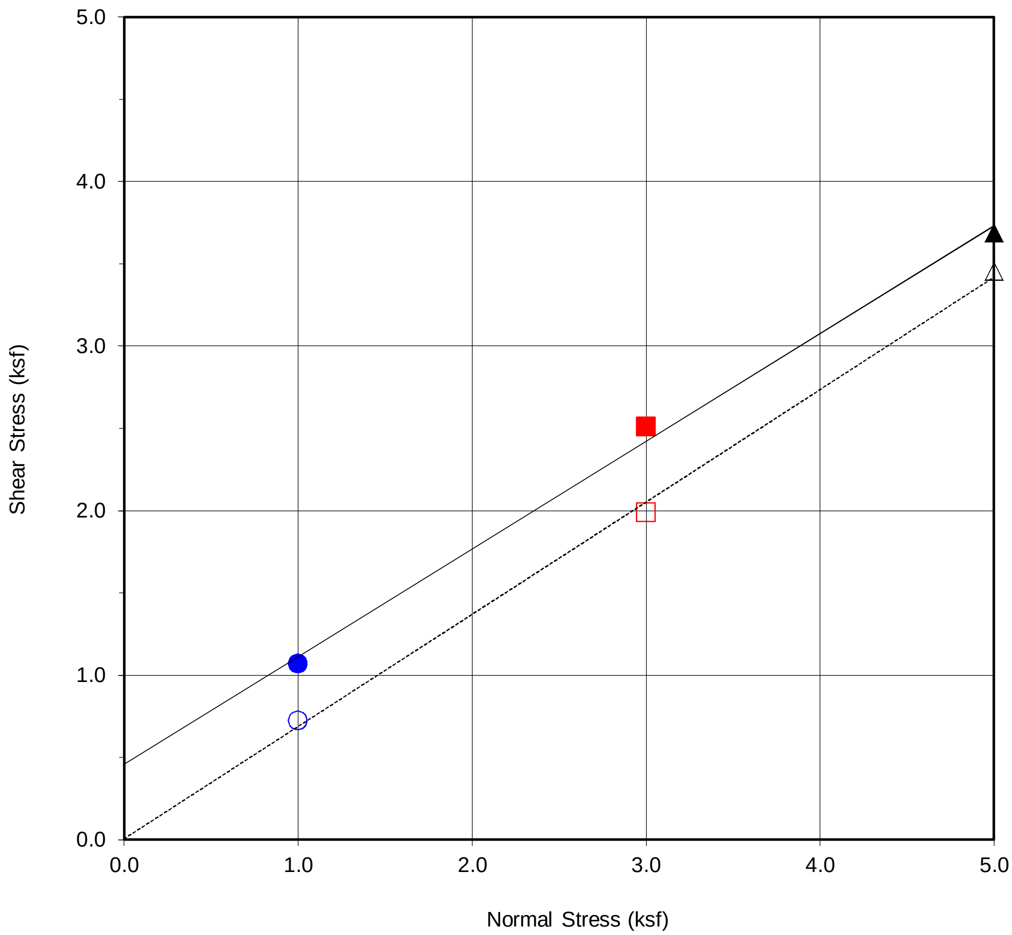
Checked by: PZ

Project No.: W1033-88-01

50 Main Street
Newport Beach, CA

Oct. 2020

Figure B1



Boring No.	B-2
Sample No.	B2@2.5
Depth (ft)	2.5
Sample Type:	Ring

Soil Identification:		
Light Brown Sand (SP)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	457	33.2
Ultimate	5	34.3

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.07	■ 2.51	▲ 3.69
Shear Stress @ End of Test (ksf)	○ 0.72	□ 1.99	△ 3.45
Deformation Rate (in./min.)	0.01	0.01	0.01
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	15.1	9.0	10.0
Initial Dry Density (pcf)	97.6	98.2	101.1
Initial Degree of Saturation (%)	56.0	33.8	40.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	23.0	21.3	20.9

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

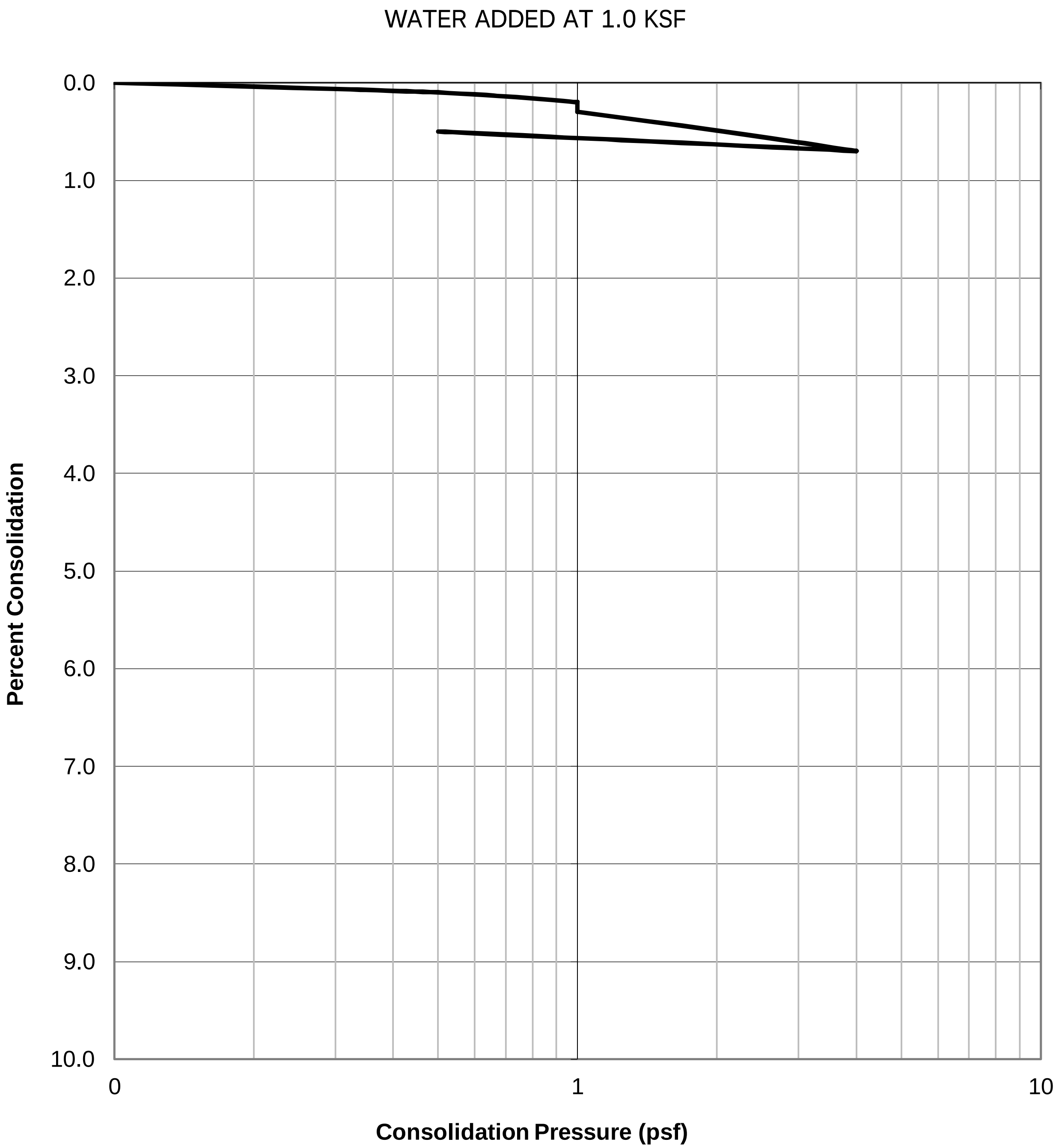
Checked by: PZ

Project No.: W1033-88-01

50 Main Street
Newport Beach, CA

Oct. 2020

Figure B2



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@2.5	Light Brown Sand (SP)	99.1	15.1	19.7

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

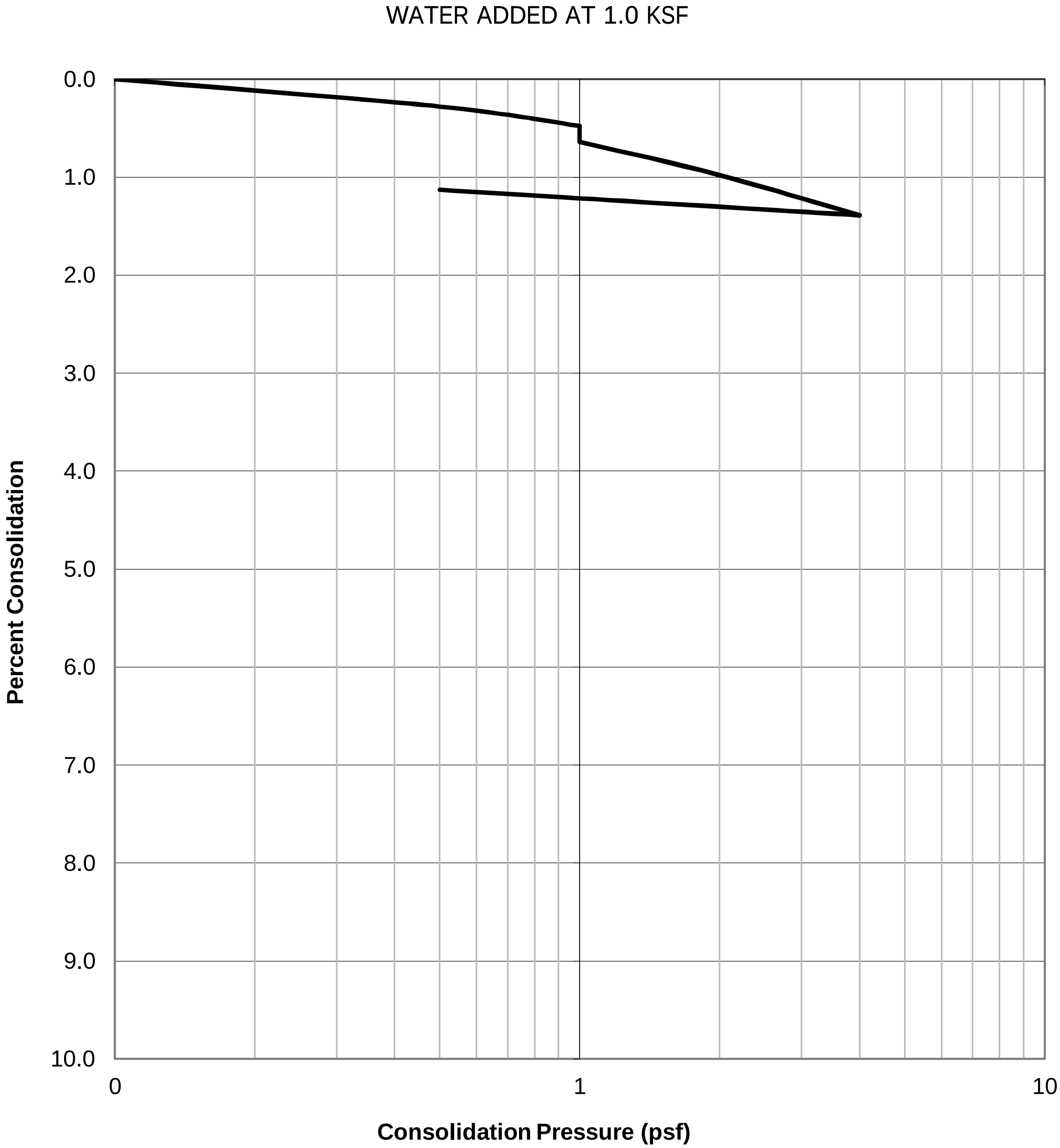
Checked by: PZ

Project No.: W1033-88-01

50 Main Street
Newport Beach, CA

Oct. 2020

Figure B3



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@5	Olive Brown Sand (SP)	98.2	24.9	24.1

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

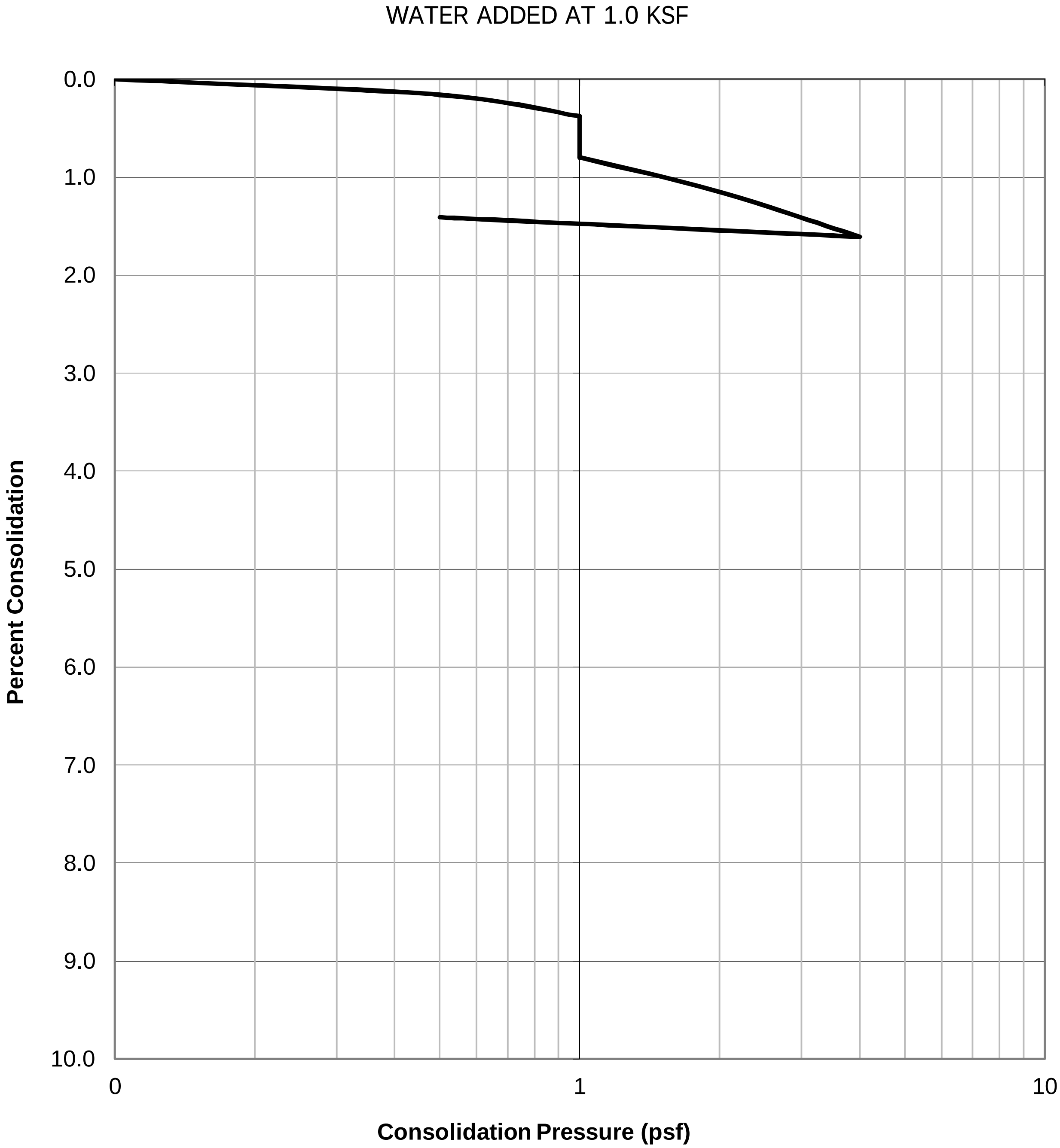
Checked by: PZ

Project No.: W1033-88-01

50 Main Street
Newport Beach, CA

Oct. 2020

Figure B4



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@7.5	Brown Sand (SP)	99.9	18.7	20.8

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

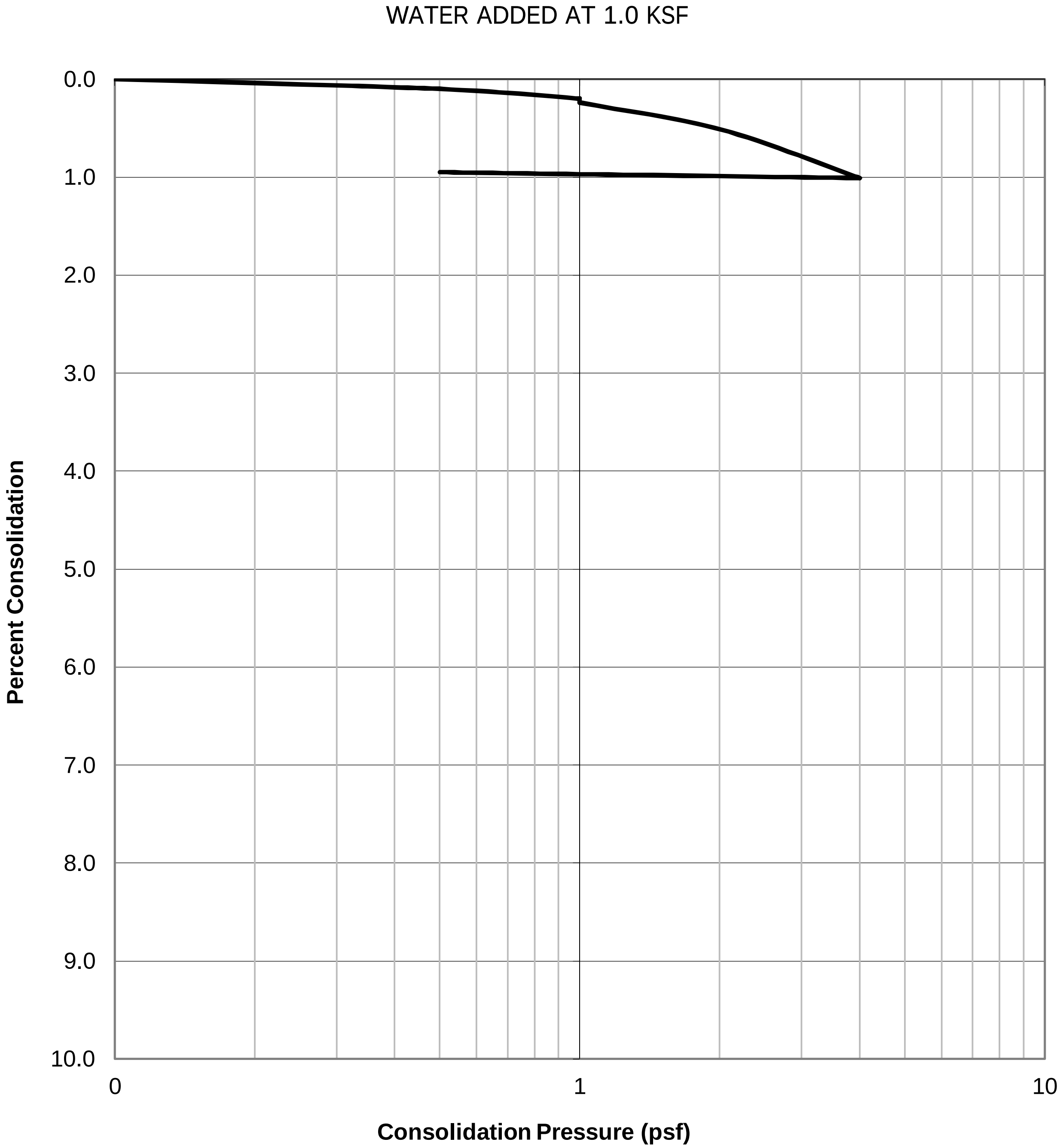
Checked by: PZ

Project No.: W1033-88-01

50 Main Street
Newport Beach, CA

Oct. 2020

Figure B5



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@10	Olive Brown Sand (SP)	103.5	20.0	20.8

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: PZ

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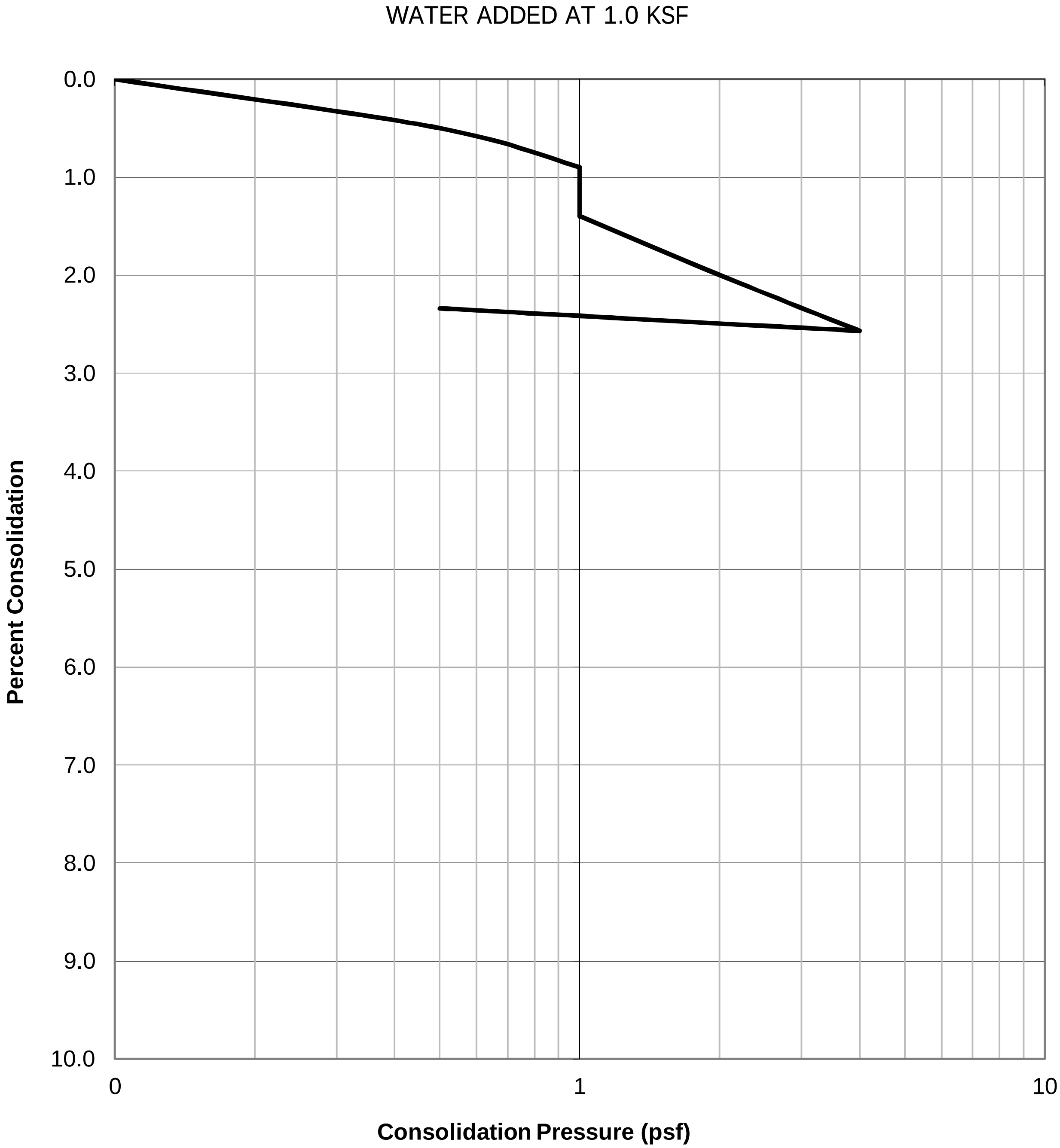
Figure B6



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@12.5	Brown Sand (SP)	107.9	20.7	21.6

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CONSOLIDATION TEST RESULTS

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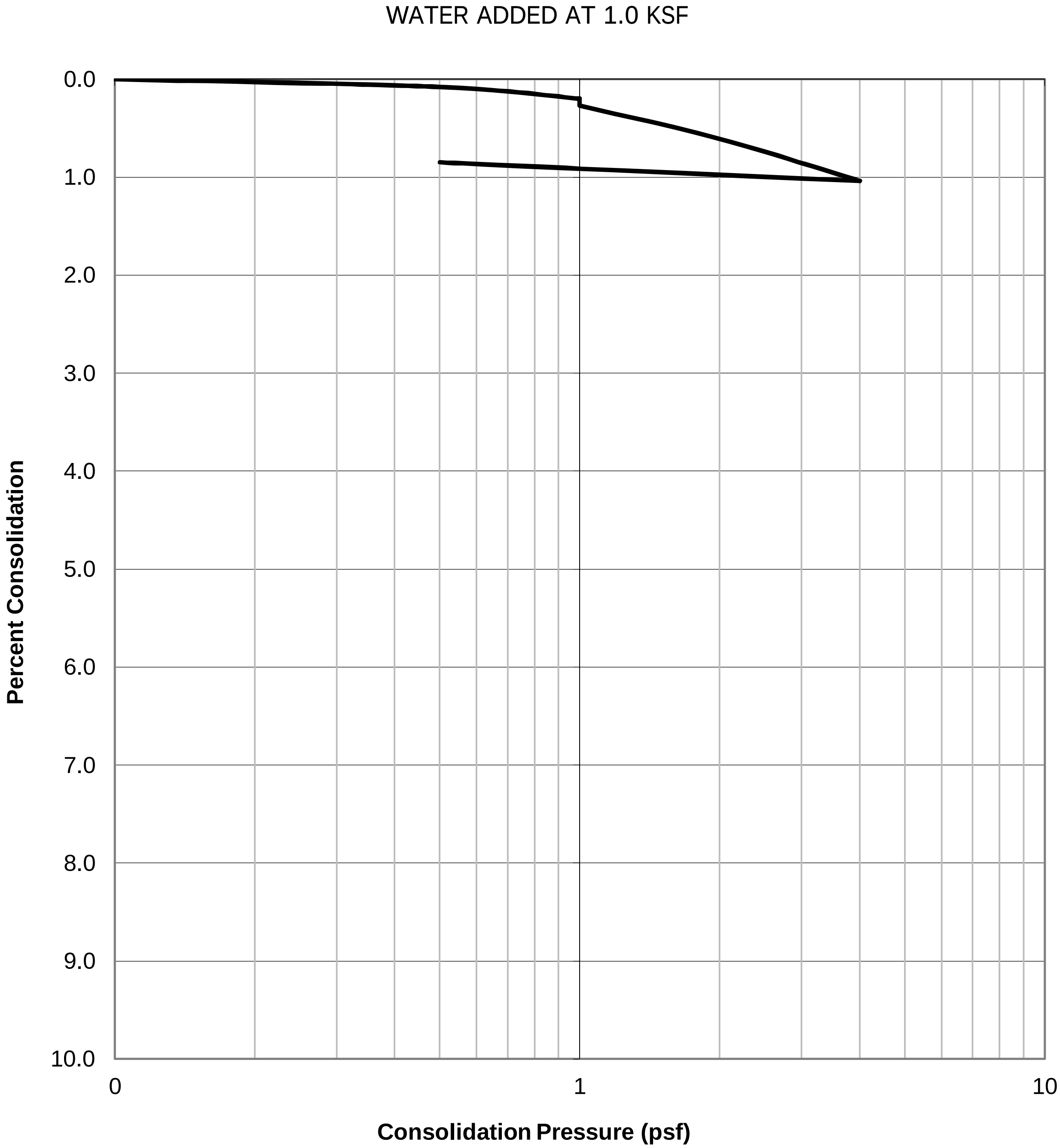
Figure B7



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@15	Olive Brown Sand (SP)	101.9	22.5	23.3

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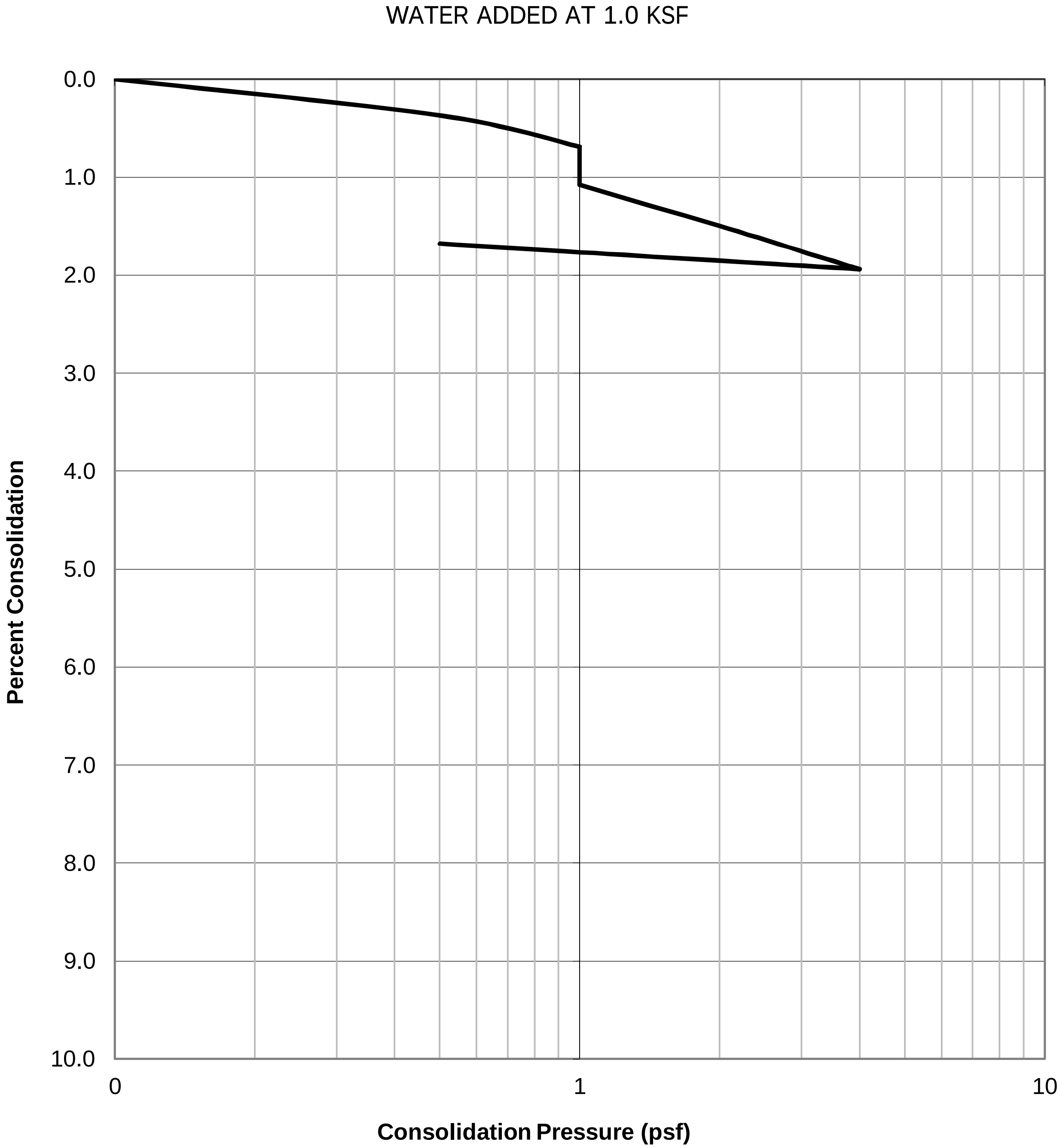
Figure B8



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@20	Olive Brown Sand (SP)	95.8	27.1	28.2

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CONSOLIDATION TEST RESULTS

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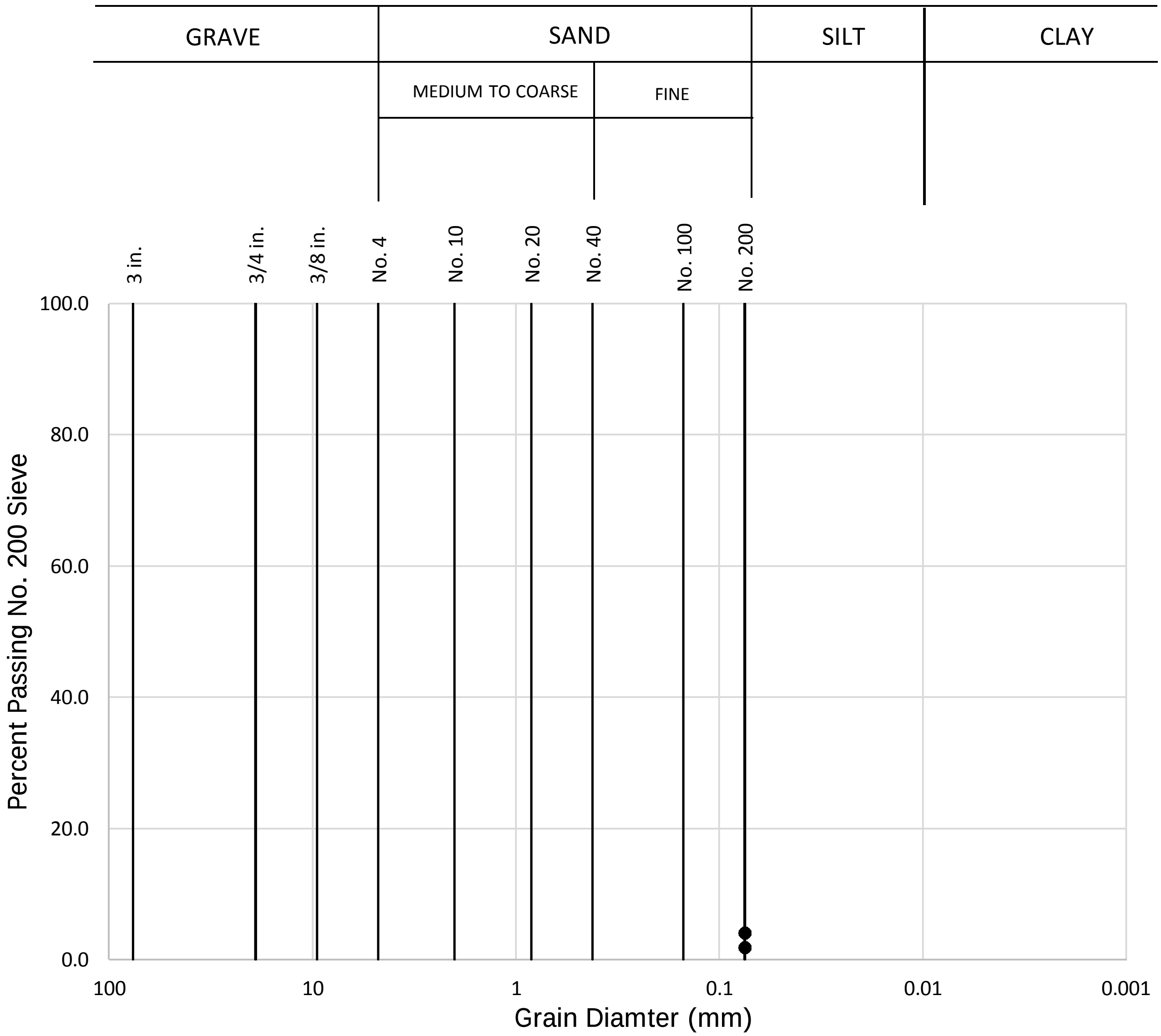
Figure B9



Sample No.	Percent Passing No. 200 Sieve
B-1 @ 5'	1.8
B-1 @ 10'	4.0

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GRAIN SIZE ANALYSIS

ASTM D-1140

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Figure B10

B2@0-5

MOLDED SPECIMEN		BEFORE TEST	AFTER TEST
Specimen Diameter	(in.)	4.0	4.0
Specimen Height	(in.)	1.0	1.0
Wt. Comp. Soil + Mold	(gm)	541.0	569.7
Wt. of Mold	(gm)	171.4	171.4
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	677.4	569.7
Dry Wt. of Soil + Cont.	(gm)	662.7	351.5
Wt. of Container	(gm)	377.4	171.4
Moisture Content	(%)	5.2	13.3
Wet Density	(pcf)	111.5	120.0
Dry Density	(pcf)	106.0	105.9
Void Ratio		0.6	0.6
Total Porosity		0.4	0.4
Pore Volume	(cc)	76.8	75.3
Degree of Saturation	(%) [S_{meas}]	24.0	62.3

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
8/13/2019	10:00	1.0	0	0.4051
8/13/2019	10:10	1.0	10	0.4053
Add Distilled Water to the Specimen				
8/14/2019	10:00	1.0	1430	0.3977
8/14/2019	11:00	1.0	1490	0.3977

Expansion Index (EI meas) =	-7.6
Expansion Index (Report) =	**0**

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: 2016 California Building Code, Section 1803.5.3

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

GEOCON

EXPANSION INDEX TEST RESULTS

ASTM D 4829

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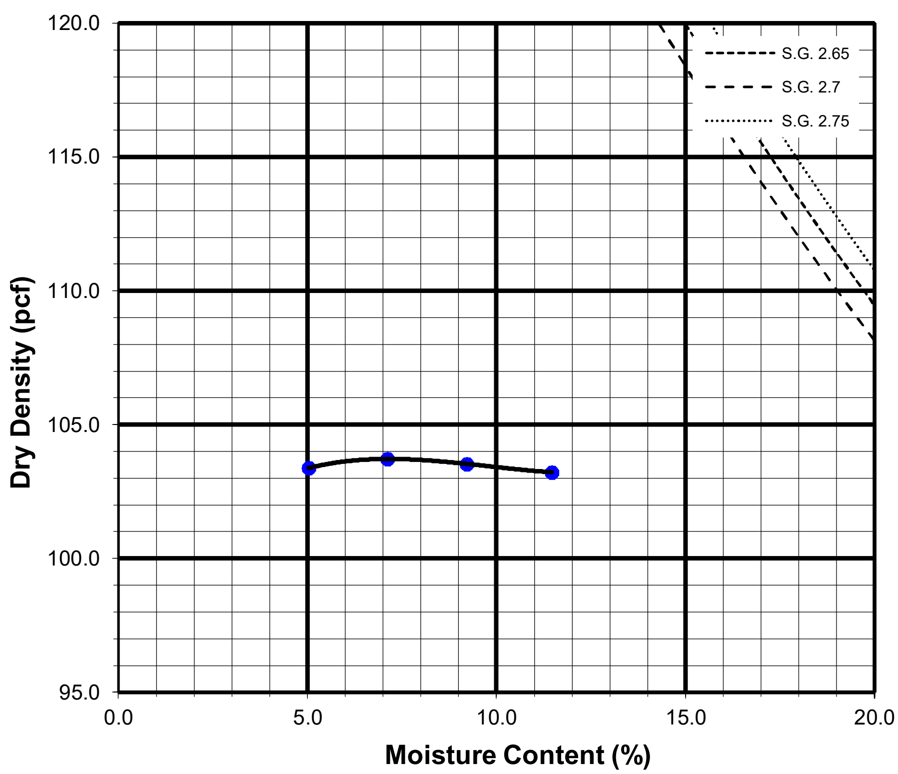
Figure B11

Sample No:

B2@0-5	Light Brown Sand (SP)

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	5936	5974	6004	6034		
Weight of Mold	(g)	4296	4296	4296	4296		
Net Weight of Soil	(g)	1640	1678	1708	1738		
Wet Weight of Soil + Cont.	(g)	1914.3	1938.4	2053.0	2097.1		
Dry Weight of Soil + Cont.	(g)	1840.8	1834.9	1911.7	1920.1		
Weight of Container	(g)	378.5	378.7	378.7	377.1		
Moisture Content	(%)	5.0	7.1	9.2	11.5		
Wet Density	(pcf)	108.6	111.1	113.1	115.1		
Dry Density	(pcf)	103.4	103.7	103.5	103.2		

Maximum Dry Density (pcf)	**104.4**	**Optimum Moisture Content (%)**	**8.2**



Preparation Method: A

GEOCON	**MODIFIED COMPACTION TEST OF SOILS** ASTM D-1557	Project No.: W1033-88-01
		50 Main Street Newport Beach, CA
	Checked by: PZ	Oct. 2020 Figure B12

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

Sample No.	pH	Resistivity (ohm centimeters)
B2 @ 0-5	8.9	7500 (Moderately Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B2 @ 0-5	0.108

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

Sample No.	Water Soluble Sulfate (% SQ_4)	Sulfate Exposure*
B2 @ 0-5	0.000	S0

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CORROSIVITY TEST RESULTS

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