Appendix D
Geotechnical Evaluation

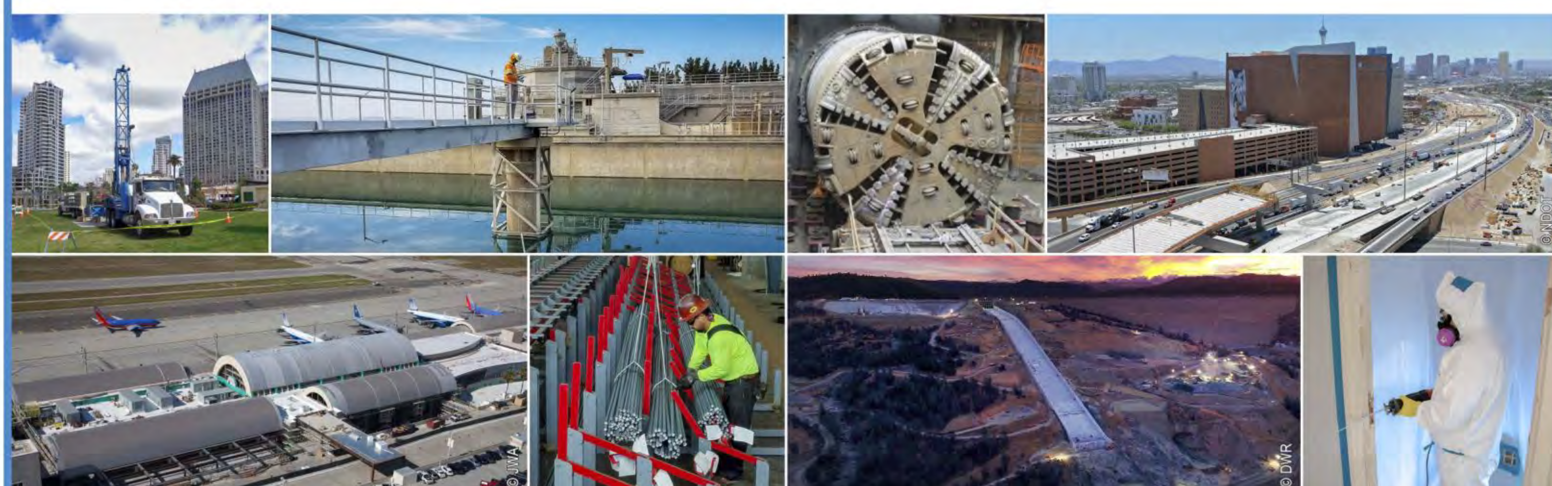
Geotechnical Evaluation

New U-Haul Facility
14206 Van Ness Avenue
Gardena, California

AMERCO Real Estate Company/U-Haul International

2727 North Central Avenue, Suite 5N | Phoenix, Arizona 85004

December 15, 2021 | Project No. 211798001



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Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

Geotechnical Evaluation
New U-Haul Facility
14206 Van Ness Avenue
Gardena, California

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December 15, 2021 | Project No. 211798001

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1 INTRODUCTION

In accordance with your request, we have performed a geotechnical evaluation for the proposed new U-Haul Facility to be located at 14206 Van Ness Avenue in Gardena, California. The approximate location of the site is depicted on Figure 1.

The purpose of our study was to evaluate the subsurface conditions and to provide design and construction recommendations regarding geotechnical aspects of the proposed project. Our services also included reviewing previous documents that were prepared for the property. This report presents the findings from our background review and subsurface exploration, results of our laboratory testing, conclusions regarding the subsurface conditions at the site, and geotechnical recommendations for design and construction of this project.

2 EXECUTIVE SUMMARY

A geotechnical evaluation has been conducted for the proposed new U-Haul facility to be constructed at 14206 Van Ness Avenue in Gardena, California. Three borings were drilled to depths ranging from approximately 26.5 to 51.5 feet below the ground surface and one Cone Penetration Test (CPT) sounding was performed to a depth of approximately 100.4 feet below the ground surface. Based on the information obtained from our background review, subsurface exploration, and laboratory testing, it is our opinion that the proposed improvements are feasible from a geotechnical standpoint, provided that the recommendations presented in this report are incorporated into the design and construction of the project. Summarized below are our key conclusions and recommendations.

- The site is generally underlain by artificial fill and alluvial soils. The fill materials generally consisted of moist, firm to very stiff, lean clay with sand, and medium dense, clayey sand and silty sand. The alluvium generally consisted of moist to wet, firm to hard, lean clay with sand, and loose to very dense, sandy silt, clayey sand, and silty sand.

- Excavation of the on-site soils should be feasible with earthmoving equipment in good working order. The on-site soils should be considered as Type C soils in accordance with Occupational Safety and Health Administration (OSHA) regulations. Slope excavations or temporary shoring should be provided in accordance with OSHA regulations. The granular soils encountered at the site have little cohesion and will be subject to caving.

- The near-surface site soils (consisting predominantly of lean clay) are not suitable for re-use as bearing material for structural footings or backfill for retaining walls. These soils should be overexcavated and either treated with lime and recompacted in place as engineered fill or replaced with imported structure backfill materials consisting of clean, non-expansive granular soil that satisfies the Greenbook criteria for structure backfill (Greenbook Section 217-3).

- Groundwater was measured at approximately 24.9 feet below existing grades during drilling. The depth to historic high groundwater at the site is mapped between 20 and 30 feet below the ground surface (California Geological Survey [CGS], 1998b). Groundwater is not expected to impact the design and construction of the improvements.

- The proposed building can be supported with spread and continuous footings and/or mat foundations.

- The depth of the overexcavation beneath the building areas should extend approximately 6 feet below the existing ground surface, or to a depth of approximately 3 feet below the bottoms of the proposed footings, whichever is deeper, and the overexcavated soils should be either treated with lime and recompacted in place as engineered fill or replaced with imported structure backfill material compacted to 90 percent relative compaction per ASTM International (ASTM) test method D 1557.

- The lateral limits of overexcavation for the building area should extend to approximately 9 feet beyond the building perimeter, or to a distance equal to the depth of overexcavation, whichever is greater.

- Spread footings should be at least 24 inches wide and extend 24 inches or more below the adjacent finished grade.

- Spread footings may be designed using an allowable bearing capacity of 3,000 pounds per square foot (psf).

- Mat foundations, if used, should be designed using a net allowable bearing capacity of 3,000 psf bearing on compacted fill underlain by competent native soils.

- Cast-in-drilled-hole (CIDH) piers used for light poles should be designed using an allowable side friction value of 80 psf under static compression and an allowable resistance of 50 psf for uplift starting at a depth of 2 foot below the ground surface.

- The lateral capacity of the CIDH piers may be evaluated using a lateral bearing resistance of 300 psf per foot of depth, up to a value of 3,000 psf.

- A design modulus of subgrade reaction of 150 kips per cubic feet (kcf) can be used for the compacted subgrade soils in evaluating deflections.

- Rammed Aggregate Piers (RAPs; Geopiers or equivalent) can be used to improve the shear strength of foundation soils, reduce the settlement of the proposed structures, and provide additional drainage for subsurface layers. However, due to the relatively shallow depths of overexcavation recommended in this report, in our opinion, the use of RAPs is not justified economically due to the high mobilization costs of ground improvement construction equipment compared to grading equipment.

- Our laboratory corrosion testing indicates that the near-surface site soils can be classified as non-corrosive based on California Department of Transportation (Caltrans, 2021) corrosion guidelines.

- The site is not mapped within a State of California Seismic Hazards Zone as being potentially liquefiable (CGS, 1999). However, due to the relatively shallow depths to groundwater at the project site, we performed a site-specific evaluation. Our evaluation indicated that scattered layers of granular soils between depths of approximately 20 and 50 feet are susceptible to liquefaction during the design seismic event for a historic high groundwater depth of 20 feet.

- Liquefaction-induced dynamic ground settlement is estimated to be on the order of 0.3 inch. The differential dynamic settlement under such condition is estimated to be about 0.15 inch or less over a horizontal distance of 40 feet. Due to the relatively small magnitude of liquefaction-induced ground settlement estimated at this site, liquefaction is not considered to be a design concern.

- The subject site is not located within a State of California Earthquake Fault Zone (EFZ) (formerly known as an Alquist-Priolo Special Studies Zone). Based on our review of published geologic maps. There are no known active faults underlying the site. However, the active Newport-Inglewood fault is mapped approximately 1.7 miles northeast of the site. The potential for surface fault rupture at the site is considered to be low.

- Detailed construction drawings were not available for our review. When construction drawings are available, including foundation plans, they should be forwarded to Ninyo & Moore for review. Additional or revised recommendations may be appropriate. In addition, if storm water infiltration will be implemented as part of the project, additional subsurface exploration and percolation testing should be performed at the locations and depths of the planned infiltration facilities, which are presently unknown.

3 SCOPE OF SERVICES

Our scope of services included the following:

- Project coordination, planning, and scheduling of the subsurface exploration.

- Review of readily available background documents, including topographic maps, geologic maps and literature, fault and seismic hazard maps, stereoscopic aerial photographs, other in-house information, and previous geotechnical documents provided by the client.

- A site reconnaissance, performed on November 2, 2021, to evaluate the site conditions and mark proposed boring locations for utility clearance and Underground Service Alert.

- Acquisition of a Well Permit from the County of Los Angeles.

- Subsurface exploration consisting of drilling, logging, and sampling of three small-diameter borings with a truck-mounted drill rig. The borings were excavated to depths of up to approximately 51.5 feet below the ground surface. The borings were logged by a representative from our firm, and bulk and relatively undisturbed soil samples were collected at selected depth intervals for laboratory testing.

- Additional subsurface exploration consisting of one CPT sounding using a truck-mounted CPT equipment to a depth of approximately 100.4 feet below the ground surface.

- Laboratory testing on representative soil samples. Laboratory tests included evaluation of in-situ moisture content and dry density, percent of particles finer than the No. 200 sieve, Atterberg limits, consolidation characteristics, direct shear strength, R-value, and soil corrosivity (including pH, electrical resistivity, water soluble sulfates, and chlorides).

- Data compilation and engineering analysis of the information obtained from our background review, subsurface evaluation, and laboratory testing.

- Preparation of this geotechnical report presenting our findings, conclusions, and geotechnical recommendations for this project.

4 SITE DESCRIPTION AND PROPOSED CONSTRUCTION

The approximately 4.2-acre, rectangular site is located at 14206 Van Ness Avenue in Gardena, California (Figure 1). The site is located in a mixed residential, commercial, and industrial area and is bounded by Van Ness Avenue to the west, Rosecrans Avenue to the south, and commercial properties to the north and east. Current structures at the site include three, single-story commercial/industrial buildings. The ground surface is generally paved with asphalt concrete (AC). A masonry block wall borders the eastern side of the property, and a wrought iron fence borders the site to the north. The site is relatively flat with ground elevations ranging from approximately 48 to 51 feet above mean sea level (MSL) (Kimley Horn, 2021).

We understand, that the project includes the design and construction of a new five-story, at-grade, storage building with a footprint area of approximately 53,764 square feet. The building will include loading docks and an elevator pit. In addition, an at-grade, single-story retail/office building will also be constructed. The retail building will have a net footprint area of approximately 8,000 square feet. As part of the development, site improvements are anticipated to include new utilities, new pavements for access drives, parking areas, associated exterior flatwork and landscaping, light poles, and screen walls along the perimeter of the site (Amerco Real Estate Company, 2021b).

5 FIELD EXPLORATION AND LABORATORY TESTING

Our subsurface exploration was conducted on November 5, 2021, and consisted of drilling, logging, and sampling of three small-diameter borings to depths ranging from approximately 26.5 to 51.5 feet below ground surface and performing one CPT sounding to a depth of approximately 100.4 feet. The borings were drilled using a truck-mounted drill rig with 8-inch-diameter hollow-stem augers. The borings were drilled to evaluate the subsurface conditions and were logged by a representative from our firm. Bulk and relatively undisturbed soil samples were obtained at selected depths from the borings for laboratory testing. The CPT sounding was performed using a 30-ton truck-mounted CPT rig. A continuous soil profile, including cone tip resistance and sleeve friction, were recorded during the sounding. Seismic readings were also collected at approximately 5 feet depth intervals. The borings and CPT were backfilled with cement-bentonite grout. The approximate locations of the borings and CPT are presented on Figure 2. The boring logs and CPT sounding logs are presented in Appendices A and B, respectively.

Laboratory testing of representative soil samples included tests to evaluate in-situ moisture content and dry density, percent fines passing the No. 200 sieve, Atterberg limits, consolidation characteristics, direct shear strength, R-value, and soil corrosivity (including pH, electrical resistivity, water soluble sulfates, and chlorides). The results of the in-situ moisture content and dry density tests are presented on the boring logs in Appendix A. The remaining laboratory test results are presented in Appendix C.

6 GEOLOGY AND SUBSURFACE CONDITIONS

6.1 Regional Geologic Setting

The subject site is located in the Los Angeles Basin, which is situated at the northwest end of the Peninsular Ranges geomorphic provinces of southern California (Norris and Webb, 1990). The Los Angeles Basin has been divided into four structural blocks, which are generally bounded by prominent northwest-trending fault systems: the northwestern, southwestern, central, and northeastern blocks. The site is located in the southwestern block, which is bounded by the Newport-Inglewood fault to the northeast, the Palos Verdes Hills fault to the southwest, and the Santa Monica-Hollywood-Raymond fault system to the northwest. The block is underlain by up to approximately 20,500 feet of Miocene to Pleistocene-age marine sedimentary rock over basement rock consisting of the Mesozoic age Catalina Schist. Variable thicknesses of late Pleistocene to Holocene-age alluvial deposits associated with the ancestral Los Angeles and San Gabriel Rivers generally overlie the sedimentary rock (Norris and Webb, 1990).

6.2 Site Geology

Published geologic maps (Figure 3) indicate that the site is underlain by Holocene and late Pleistocene-age alluvial flood-plain deposits consisting of poorly consolidated and poorly sorted clay, silt and loose to moderately dense sand and silty sand (Saucedo et al., 2016). The materials encountered during our geotechnical exploration included AC, artificial fill, and alluvial deposits. Below is a general description of the materials encountered during geotechnical exploration. Detailed descriptions of the subsurface materials are presented on our boring logs in Appendix A.

6.2.1 Existing Pavement

AC was encountered in all borings and CPT-1. The AC thickness varied from approximately 3.2 to 7 inches.

6.2.2 Fill

Fill soils were encountered in all borings; the fill was observed to occur up to a depth of approximately 5½ feet in boring B-1, approximately 2 feet in boring B-2, and approximately 13 feet in Boring B-3. The fill generally consisted of moist, firm to very stiff, lean clay with sand, and medium dense, clayey sand, and silty sand. Wood fragments were observed in the fill in Boring B-1.

6.2.3 Alluvium

Alluvium was encountered underlying the fill in all borings and CPT-1 to the total depths explored of up to approximately 100.4 feet. The alluvial materials generally consisted of moist to wet, firm to hard, lean clay with sand, and most to wet, loose to very dense, sandy silt, clayey sand, and silty sand.

7 GROUNDWATER

Groundwater was measured during drilling in Boring B-3 at approximately 24.9 feet below the ground surface. The historic high depth to groundwater in the vicinity of the site has been mapped as between 20 and 30 feet below the existing ground (CGS, 1998b). Fluctuations in the level of groundwater may occur due to variation in ground surface topography, subsurface stratification, rainfall, irrigation practices, groundwater pumping, and other factors which may not have been evident at the time of our field evaluation.

8 FAULTING AND SEISMICITY

The site is located in a seismically active area, as is the majority of southern California, and the potential for strong ground motion in the project area is considered significant during the design life of the proposed improvement. Figure 4 shows the approximate site location relative to the principle faults in the region. Based on our background review and site reconnaissance, the project site is not transected by known active or potentially active faults nor is it located within a State of California EFZ (formerly known as an Alquist-Priolo Special Studies Zone) (Hart and Bryant, 2007).

Table 1 lists selected principal known active faults that may affect the project site, the maximum moment magnitude (M_{max}), and the calculated approximate fault-to-site distances using the United States Geological Survey (USGS) fault database (USGS, 2008). The active Newport-Inglewood fault zone is mapped approximately 1.7 miles northwest of the site. In addition, the active Puente Hills (Los Angeles segment) blind thrust fault is mapped approximately 6.8 miles north of site (USGS, 2008). Blind thrust faults are low-angle faults at depth that do not break the surface and are, therefore, not shown on Figure 4. Although blind thrusts do not have a surface trace, they are capable of generating damaging earthquakes and are included in Table 1.

Table 1 – Principal Active Faults		
Fault	Approximate Fault-to-Site Distance miles (kilometers)	Maximum Moment Magnitude (M_{max})
Newport Inglewood	1.7 (2.7)	7.5
Compton	8.4 (5.2)	7.3
Puente Hills (LA)	6.8 (11.0)	7.0
Palos Verdes	7.3 (11.8)	7.7
Puente Hills (Santa Fe Springs)	10.2 (16.5)	6.7
Santa Monica	12.1 (19.5)	7.4
Elysian Park (Upper)	12.3 (19.7)	6.7
Hollywood	13.5 (21.7)	6.7
Malibu Coast	14.8 (23.8)	7.0
Puente Hills (Coyote Hills)	15.7 (25.3)	6.9
Anacapa-Dume	15.8 (25.4)	7.2

Principal seismic hazards associated with seismic activity for this site are surface fault rupture, ground motion, and liquefaction. A brief description of these hazards and the potential for their occurrences on site are discussed below.

8.1 Surface Fault Rupture

Based on our review of the referenced literature and our site reconnaissance, no active faults are known to cross the project site. Therefore, the probability of damage from surface fault rupture is considered to be low. However, lurching or cracking of the ground surface as a result of nearby seismic events is possible.

8.2 Ground Motion

Considering the proximity of the site to active faults capable of producing a maximum moment magnitude of 6.0 or more, the project area has a high potential for experiencing strong ground motion. The 2019 California Building Code (CBC) specifies that the risk-targeted maximum considered earthquake (MCE_R) ground motion response accelerations be used to evaluate seismic loads for design of buildings and other structures. A $V_{s,30}$ (average shear wave velocity for the upper 30 meters of soil layers) of 250 meters per second (820 feet per second) was estimated for the site using the CPT-1 data. Accordingly, the site is classified as Site Class D. Per the 2019 CBC, site-specific ground motion hazard analysis needs to be performed following the guidelines presented in Sections 21.2 and 21.3 of American Society of Civil Engineers (ASCE) Publication 7-16 for soil deposits classified as Site Class D with mapped S_1 (spectral response acceleration at a period of 1 second) greater than or equal to 0.2g. Since the S_1 is 0.643g at the site (per ASCE 7-16, using the 2021 Applied Technology Council [ATC] web-based seismic design tool [ATC, 2021]), site-specific ground motion hazard analysis was performed to evaluate the ground motion characteristics at the site.

The site-specific ground motion hazard analysis consisted of the review of available seismologic information for nearby faults and performance of probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis (DSHA) to develop 5-percent-damped acceleration response spectrum (ARS) curves corresponding to the MCE_R. Prior to the site-specific ground motion hazard analysis, we obtained the mapped seismic ground motion values and developed the general MCE_R response spectrum for 5 percent damping in accordance with Section 11.4 of ASCE 7-16 (ATC, 2021).

The 2014 next generation attenuation (NGA) West-2 relationships were used to evaluate the site-specific ground motions. The NGA relationships used for developing the probabilistic and deterministic response spectra were those by Chiou and Youngs (2014), Campbell and Bozorgnia (2014), Boore, Stewart, Seyhan, and Atkinson (2014), and Abrahamson, Silva, and Kamai (2014). The Open Seismic Hazard Analysis software developed by USGS (USGS, 2020) was used for performing the PSHA. The Calculation of Weighted Average 2014 NGA Models spreadsheet developed by the Pacific Earthquake Engineering Research Center (PEER) was used for performing the DSHA (Seyhan, 2014).

PSHA was performed for earthquake hazards having a 2 percent probability of being exceeded in 50 years adjusted for the risk factors per ASCE 7-16. The maximum rotated components of 5-percent-damped ground motions were considered in PSHA. The DSHA considers accelerations from characteristic earthquakes on active faults within the region using the hazard curves and deaggregation plots at the site using the USGS Unified Hazard Tool application (USGS, 2021). A magnitude 7.3 event on the Compton fault was deemed to be the controlling earthquake. The DSHA was performed for the site using this event and corrections were applied to spectral accelerations for the 84th percentile of the maximum rotated component of ground motion with 5 percent damping.

The site-specific MCE_R response spectrum was considered as the lesser of the PSHA and DSHA spectral response acceleration at any period, and the site-specific design response spectrum was determined by taking two-thirds of the MCE_R response spectrum with some conditions in accordance with Section 21.3 of ASCE 7-16. Figure 5 presents the site-specific MCE_R and design response spectra as well as the general mapped design response spectra calculated in accordance with Section 11.4 of ASCE 7-16. The site-specific spectral acceleration parameters, obtained from ground motion hazard analysis, are presented in Section 12.2 for evaluation of seismic loads on buildings and other structures. The site-specific MCE_G (maximum considered earthquake geometric mean) peak ground acceleration, PGA_M, was calculated as 0.828g.

8.3 Liquefaction Potential

Liquefaction is the phenomenon in which loosely deposited granular soils and cohesionless fine-grained soils located below the water table undergo rapid loss of shear strength due to excess pore pressure generation when subjected to strong earthquake-induced ground shaking. Ground shaking of sufficient duration results in the loss of grain-to-grain contact due to a rapid rise in pore water pressure. This causes the soil to behave as a fluid for a short period of time. Liquefaction is known generally to occur in saturated or near-saturated cohesionless soils at depths shallower than 50 feet below the ground surface. Factors known to influence liquefaction potential include composition and thickness of soil layers, grain size, relative density, groundwater level, degree of saturation, and both intensity and duration of ground shaking.

According to the State of California (CGS, 1999), the site is not located in an area mapped as a potential liquefaction hazard zone. However, due to the relatively shallow depths to groundwater at the project site, we evaluated the liquefaction potential of the subsurface soils at the site. Our analysis was performed using the CPT data collected from CPT-1 and a historic high depth to groundwater of 20 feet below the existing ground surface. The liquefaction analysis was based on the National Center for Earthquake Engineering Research procedure (Youd, et al., 2001) using the computer program Liquefy Pro (CivilTech Software, 2008). In accordance with the site-specific ground motion study described above, a design earthquake moment magnitude of 7.3 and a PGA_M of 0.828g were used in the analysis. Our liquefaction analysis indicates that scattered layers of granular soil deposits occurring between depths of approximately 20 and 50 feet may be susceptible to liquefaction during the design seismic event. Results of our liquefaction analysis are presented in Appendix D.

8.4 Dynamic Settlement of Relatively Saturated Soils

As a result of liquefaction, the proposed building may be subject to liquefaction-induced settlement. In order to estimate the amount of post-earthquake settlement, the method proposed by Tokimatsu and Seed (1987) was used in which the seismically induced cyclic stress ratios and corrected N-values are related to the volumetric strain of the soil. The method proposed by Robertson & Wride (1998) was used to evaluate the cyclic resistance ratio from the CPT Data. The amount of soil settlement during a strong seismic event depends on the thickness of the liquefiable layers and the density and/or consistency of the soils. Total liquefaction-induced settlement of approximately 0.30 inch was calculated from the data obtained from CPT-1. Dynamic differential settlement is estimated to be about 0.15 inch or less over a horizontal distance of about 40 feet.

9 LANDSLIDING

There are no mapped landslides on site or in the vicinity, and the site is not mapped as having the potential for seismically induced landslides (CGS, 1999). Based on this information and the location of the site, landsliding is not considered to be a potential hazard at the site.

10 TSUNAMIS AND SEICHES

Tsunamis are long wavelength, seismic, sea waves (long compared to ocean depth) generated by the sudden movements of the ocean floor during submarine earthquakes, landslides, or volcanic activity. Seiches are waves generated in a large, enclosed body of water. The project area is not mapped within an area considered susceptible to tsunamis or seiche inundation. Therefore, damage due to tsunamis or seiches is not a design consideration.

11 FLOOD HAZARDS

Based on review of a Los Angeles County Geographic Information System Hazards Mapping, the site is mapped as lying outside the 500-year floodplain. The site is not mapped as located within the potential inundation zones due to dam failure (Federal Emergency Management Agency [FEMA], 2008). Based on this review, the potential for flooding of the site is considered to be low.

12 RECOMMENDATIONS

Based on our understanding of the project, the following sections present our geotechnical recommendations for design and construction of the proposed building and other site improvements. The recommendations are based on the results of our subsurface evaluation and laboratory testing, our review of the referenced geologic materials, and our geotechnical analysis. The proposed construction should be performed in conformance with the recommendations presented in this report, project specifications, and appropriate agency standards.

12.1 Earthwork

Based on our understanding of the project, earthwork at the site is anticipated to consist of site clearing, remedial grading to prepare the ground surface for the new building, pavements, trenching, and backfilling associated with underground utility installation. Earthwork should be performed in accordance with the requirements of the applicable governing agencies and the recommendations presented in the following sections.

12.1.1 Construction Plan Review and Pre-Construction Conference

We recommend that the project plans be submitted to Ninyo & Moore for review to evaluate conformance to the geotechnical recommendations provided in this report. We further recommend that a pre-construction conference be held in order to discuss the grading recommendations presented in this report. The owner and/or their representative, the governing agencies' representatives, the civil engineer, Ninyo & Moore, and the contractor should be in attendance to discuss the work plan, project schedule, and earthwork requirements

12.1.2 Site Preparation

Prior to performing the site excavations, the project area should be cleared of pavements, rubble and debris, surface obstructions, abandoned utilities, vegetation, and other deleterious materials. Existing utilities within the project limits should be re-routed or protected from damage by construction activities. Obstructions that extend below the pipeline subgrade, if any, should be removed and the resulting holes filled with compacted soils. Materials generated from the clearing operations should be removed from the project site and disposed at a legal dumpsite.

12.1.3 Treatment of Near-Surface Soils

In order to provide suitable support and reduce the potential for settlement of the proposed improvements, we recommend that the areas beneath the new buildings be overexcavated to remove undocumented fill and clayey native soils that were encountered in our exploratory borings. The depth of overexcavation beneath the building areas, including retaining walls for loading docks, should extend 6 feet below the existing ground surface or to a depth of approximately 3 feet below the bottoms of the proposed footings, whichever is deeper; the overexcavated soils should be replaced with engineered fill compacted to 90 percent relative compaction per ASTM D 1557. The lateral limits of overexcavation for the building area should extend to approximately 9 feet beyond the outside edge of the footings or to a distance equal to the depth of overexcavation, whichever is greater. We recommend that the overexcavated clayey soils be disposed of offsite or the clayey soils be treated with lime and placed back as compacted fill.

Following the recommended overexcavation, the bottom of the excavation should expose relatively dense or stiff soils. The excavation bottom should be evaluated by our representative during the excavation work. Additional overexcavation of loose, soft, and/or wet areas may be appropriate, depending on our observations during construction. Prior to placing new compacted fill, the exposed bottom should be scarified, moisture-conditioned, and re-compacted to a depth of approximately 8 inches. Care should be taken by the contractor to avoid undermining existing improvements located adjacent to the project site.

In order to provide suitable support and reduce the potential settlement of new and reconstructed pavements subject to vehicle traffic, we recommend overexcavating the upper approximately 24 inches of the surficial soils or to a depth that provides 12 inches of compacted fill beneath the pavement section, whichever is deeper. We recommend that the bottom of sidewalks and/or hardscapes be underlain by 12 inches of compacted fill. The lateral limits of overexcavation for pavements should extend to approximately 2 feet or to a distance equal to the depth of overexcavation, whichever is greater. The exposed subgrade should be evaluated by our representative during the excavation work. Loose, soft, and/or wet areas may need to be further overexcavated, depending on our observations during construction. Prior to placing new compacted fill, the exposed bottom should be scarified, moisture-conditioned, and re-compacted to a depth of approximately 8 inches.

12.1.4 Rammed Aggregate Piers

As an alternative to remedial grading to mitigate the unsuitable soil conditions at this site, the use of RAPs was considered. RAPs are vertical elements of crushed stone implemented at sites with soft/loose sediments that are prone to excessive settlement and/or possess high likelihood of liquefaction occurrence. RAPs (i.e., Geopier or equivalent) mainly serve to improve the shear strength of the foundation soils, reduce the settlement of the proposed structures, and provide additional drainage for subsurface layers. However, given the relatively shallow overexcavation depths recommended in this report, the use of RAPs is not considered to be economically justified for this project.

12.1.5 Excavations

We anticipate that excavations for removals, general grading, foundation construction, elevator pits, loading docks, and new utilities within the fill and alluvial soils at the site may be accomplished with backhoes, excavators, or other earthmoving equipment in good working condition. During excavations, the contractor should anticipate encountering oversize materials. The contractor should be prepared to take appropriate measures to address the presence of oversize materials in site soils.

12.1.6 Temporary Excavations

We recommend that trenches and excavations be designed and constructed in accordance with OSHA regulations. These regulations provide trench sloping and shoring design parameters for excavations up to 20 feet deep based on the soil types encountered. Excavations should be designed be the contractor’s engineer based on site-specific geotechnical analyses. For planning purposes, we recommend that on-site fill and surficial soils (if encountered) be considered as OSHA Type C soil.

Temporary excavations should be constructed in accordance with OSHA recommendations. For trench or other excavations, OSHA requirements regarding personnel safety should be met by using appropriate shoring (including trench boxes) or be laying back the slopes no steeper than 1½:1 (horizontal to vertical) or flatter. Depending on excavation depths, shoring may be appropriate. Continuous shoring may be appropriate for trenches in friable cohesionless sands. Excavations should be performed in accordance with OSHA's regulations.

If shoring systems are used for site excavations, they should be designed for the anticipated soil conditions using the lateral earth pressure values shown on Figures 6 and 7 for temporary cantilevered shoring and braced excavation, respectively. The recommended design pressures are based on the assumption that the shoring system is constructed without raising the ground surface elevation behind the shored sidewalls of the excavation, that there are no surcharge loads, such as soil stockpiles and construction materials, and that no loads act above a 1:1 (horizontal to vertical) plane ascending from the base of the shoring system. For a shoring system subjected to the above-mentioned surcharge loads, the contractor should include the effect of these loads on the lateral earth pressures acting on the shored walls.

We anticipate that settlement of the ground surface will occur behind the shored excavation. The amount of settlement depends heavily on the type of shoring system, the contractor's workmanship, and soil conditions. To reduce the potential for distress to adjacent improvements, we recommend that the shoring system be designed to limit the ground settlement behind the shoring system to ½ inch or less. Possible causes of settlement that should be addressed include settlement during installation of the shoring elements, excavation for structure construction, construction vibrations, and removal of the support system. We recommend that shoring installation be evaluated carefully by the contractor prior to construction.

The contractor should retain a qualified and experienced engineer to design the shoring system. The shoring parameters presented in this report are minimum requirements, and the contractor should evaluate the adequacy of these parameters and make appropriate modifications for their design. We recommend that the contractor take appropriate measures to protect workers. OSHA requirements pertaining to worker safety should be observed.

12.1.7 Fill Material

In general, the on-site soils are not suitable for reuse as general compacted fill, compacted structural fill, or trench backfill. We recommend that the overexcavated clayey soils be disposed of offsite and imported materials be used. Imported materials should consist of clean, non-expansive granular material that generally meets Greenbook criteria for structure backfill (Greenbook Section 217-3). Soil should also be tested for corrosive properties prior to importing. We recommend that imported materials meet the Caltrans (2021) criteria for non-corrosive soils (i.e., soils having a chloride concentration of 500 parts per million (ppm) or less, a soluble sulfate content of approximately 1,500 ppm or less, a pH value of 5.5 or higher, and a resistivity of 1,500 ohm-centimeters [ohm-cm] or higher). Materials for use as fill should be evaluated by the geotechnical consultant prior to importing. The contractor should be responsible for the uniformity of import material brought to the site.

As an alternative to removal and replacement of on-site soils, on-site clayey soils may be reused as general compacted fill, compacted structural sill, or trench backfill if they are treated with lime. The clayey soils should be blended with lime, hydrated multiple times, and replaced as compacted fill.

12.1.8 Fill Placement and Compaction

General compacted fill, compacted structural fill, and trench backfill should be placed and compacted in accordance with project specifications and sound construction practices. The materials should be moisture-conditioned to slightly above the optimum laboratory moisture content. The lift thickness for compacting fill soils will vary depending on the type of compaction equipment used, but should generally be placed in horizontal lifts not exceeding 8 inches in loose thickness. The materials should be compacted to a relative compaction of 90 percent as evaluated by ASTM D 1557. Special care should be taken to avoid pipe damage when compacting trench backfill above pipes. Compacted fill should be tested for specified compaction level by Ninyo & Moore.

12.2 Seismic Design Parameters

Seismic design of the proposed improvements should be performed in accordance with the requirements of the governing jurisdictions and applicable building codes. Table 2 presents the site-specific spectral response acceleration parameters in accordance with the CBC (2019) guidelines.

Table 2 – 2019 California Building Code Seismic Design Criteria	
Seismic Design Factors	**Value**
Site Class	D
Mapped Spectral Response Acceleration at 0.2-second Period, S_s	1.821g
Mapped Spectral Response Acceleration at 1.0-second Period, S_1	0.643g
Site-Specific Spectral Response Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	1.930g
Site-Specific Spectral Response Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	1.904g
Site-Specific Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	1.287g
Site-Specific Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	1.270g
Site-Specific Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Acceleration, PGA_M	0.828g

12.3 Foundations

The proposed building may be supported on shallow foundations including square and continuous footings, and mat foundations bearing on fill material compacted in accordance with the recommendations presented in the Earthwork section of this report. Foundations should be designed in accordance with structural considerations and the following recommendations. In addition, requirements of the appropriate governing jurisdictions and applicable building codes should be considered in the design of the structures.

12.3.1 Spread Footings

Square and continuous footings should be at least 24 inches wide and extend 24 inches or more below the adjacent finished grade. Spread footings should be reinforced with a minimum of two No. 4 steel reinforcing bars, one placed near the top and one placed near the bottom of the footings, and further detailed in accordance with the recommendations of the structural engineer.

Footings, bearing on compacted fill soils with low expansion potential, may be designed using a net allowable bearing capacity of 3,000 psf. The allowable bearing capacity may be increased by one-third when considering loads of short duration such as wind or seismic forces. The foundations should preferably be proportioned such that the resultant force from design loads, including lateral loads, falls within the kern (i.e., middle one-third of the footing base). Total and differential settlements for footings designed and constructed in accordance with the above recommendations are estimated to be less than approximately 1 inch and ½ inch over a horizontal span of 40 feet, respectively.

Footings bearing on compacted fill may be designed using a coefficient of friction of 0.35, where the total frictional resistance equals the coefficient of friction times the dead load. Footings may be designed using a passive resistance of 350 psf per foot of depth for level ground condition up to a value of 3,500 psf. The allowable lateral resistance can be taken as the sum of the frictional resistance and passive resistance provided the passive resistance does not exceed one-half of the total allowable resistance. In the event that the passive resistance is greater than one-half of the total allowable resistance, the passive resistance should be reduced to be the same value as the frictional resistance. The passive resistance may be increased by one-third when considering loads of short duration such as wind or seismic forces.

12.3.2 Pole Foundations

We recommend that light poles be supported by CIDH piles. Details of the light poles were not available at the time of this report. Light poles typically impose relatively light axial loads on pile foundations and the pile dimensions will generally be controlled by the lateral load demand.

The drilled piers with a diameter of 18 inches or more may be designed using an allowable side friction value of 80 psf under static compression and an allowable resistance of 50 psf for uplift starting at a depth of 2 foot below the ground surface. The lateral capacity of the drilled piers may be evaluated using a lateral bearing pressure of 300 psf per foot of depth, up to a value of 3,000 psf. The passive resistance may be considered to act on an area equal to the product of the effective width (two times the pier diameter) and the embedded length of the pier. The allowable bearing values may be increased by one-third when considering loading of short duration such as wind or seismic forces. These calculations assume that the poles have a minimum spacing of three times the pole diameter.

The pile dimensions (i.e., diameter and embedment) should be evaluated by the project structural engineer. CIDH piles should be designed to derive support from side friction. The bottoms of the drilled holes should be cleaned of loose materials prior to placing steel and pouring concrete. Piles should be installed within specified limits of vertical and horizontal alignment and should not exceed a batter of two percent over the length. Further, the top of the piles should be within 3 inches of the surveyed location.

12.3.3 Slab-On-Grade

Floor slabs subjected to dead and live loads should be designed by the project structural engineer based on the anticipated loading conditions. Floor slabs should be underlain by compacted soil prepared per the recommendations presented in this report. We recommend that slabs be, at a minimum, 5 inches thick and reinforced with No. 4 steel reinforcing bars placed 24 inches on-center (each way) placed near the mid-height of the slab. Placement of reinforcement in the slab is vital for its satisfactory performance. Floor slabs and foundations should be tied together by extending the slab reinforcements into the foundation.

Slabs should be underlain by a 4-inch-thick, or more, layer of sand or gravel with a particle size of approximately 3/8-inch or smaller. Soils underlying slabs should be moisture-conditioned and compacted in accordance with the recommendations presented in this report prior to concrete placement. Joints should be constructed at intervals designed by the structural engineer to help reduce random cracking of the slabs.

12.3.4 Mat Foundations

The proposed mat foundations may be designed using a net allowable bearing capacity of 3,000 pounds per square foot (psf) founded on newly compacted fill over competent underlying materials. The total and differential settlements corresponding to this bearing load are estimated to be less than approximately 1 inch and ½ inches over a horizontal span of 40 feet, respectively. Mat foundations typically experience some deflection due to loads placed on the mat and the reaction of the soils underlying the mat. A design modulus of subgrade reaction of 150 kips per cubic foot (kcf) may be used for the compacted subgrade soils in evaluating such deflections.

12.4 Lateral Earth Pressures

Retaining walls may be supported by spread footings designed in accordance with the recommendations presented in Section 12.3 of this report. The proposed elevator pit may be supported on spread or mat foundations. Lateral earth pressures recommended for the design of yielding retaining walls are provided on Figures 8. Passive pressures may be increased by one-third when considering loads of short duration, including wind and seismic loads. Below-grade walls for the proposed elevator pit may be considered to be restrained from lateral displacement under static loading conditions. Restrained walls subjected to lateral earth pressures should be designed using the parameters presented on Figures 9.

Retaining walls should be backfilled with free-draining, granular, non-expansive material (Expansion Index 20 or less). Measures should be taken to reduce the potential for build-up of moisture behind the retaining walls. Drainage design should include free-draining backfill materials and perforated drains as depicted on Figure 10.

12.5 Underground Utilities

We anticipate that utility pipelines will be supported on future compacted fill or native alluvial soils. The depths of the pipelines are not known; however, we anticipate that the pipe invert depths will not exceed 10 feet.

12.5.1 Pipe Bedding

We recommend that bedding material be placed around pipe zones 1 foot or more above the top of the pipe. The bedding material should be classified as sand, should be free of organic material, and have a sand equivalent of 30 or more. We do not recommend gravel be used for bedding material because of the silty nature of some of the subsurface material. It has been our experience that the voids within gravel material are sufficiently large to allow fines to migrate into the voids, thereby creating the potential for sinkholes and depressions to develop at the ground surface. Where soft, wet soil conditions are encountered, the trench excavation should be extended approximately 1 foot or more below the pipe invert and should be backfilled with gravel wrapped in filter fabric.

Special care should be taken not to allow voids beneath and around the pipe. Compaction of the bedding material and backfill should proceed uniformly up both sides of the pipe. Trench backfill, including bedding material, should be placed in accordance with the recommendations presented in the Earthwork section of this report.

12.5.2 Trench Backfill

Clayey soils encountered at the site are not suitable for reuse as backfill for trenches. We recommend that overexcavated clayey soils be disposed of offsite and imported soils be used as backfill for trenches. Imported materials should consist of clean, non-expansive granular material that generally meets Greenbook criteria for structure backfill (Greenbook Section 217-3). Soil should also be tested for corrosive properties prior to importing. We recommend that imported materials meet the Caltrans (2021) criteria for non-corrosive soils (i.e., soils having a chloride concentration of 500 parts per million (ppm) or less, a soluble sulfate content of approximately 1,500 ppm or less, a pH value of 5.5 or higher, and a resistivity of 1,500 ohm-centimeters [ohm-cm] or higher). Materials for use as fill should be evaluated by the geotechnical consultant prior to importing. The contractor should be responsible for the uniformity of import material brought to the site.

Trench backfill should be compacted to a relative compaction of 90 percent as evaluated by ASTM D 1557. Lift thickness for backfill will depend on the type of compaction equipment utilized, but fill should generally be placed in lifts not exceeding 8 inches in loose thickness. Special care should be exercised to avoid damaging utilities during compaction of the backfill.

12.5.3 Modulus of Soil Reaction

The modulus of soil reaction is used to characterize the stiffness of soil backfill placed on the sides of buried flexible pipelines for the purpose of evaluating lateral deflection caused by the weight of the backfill above the pipe. We recommend that a modulus of soil reaction of 1,000 pounds per square inch (psi) be used for design, provided that granular bedding material is placed adjacent to the pipe, as recommended in this report.

12.6 Sidewalks and Hardscape

We recommend that new exterior concrete sidewalks and flatwork (hardscape) have a thickness of 4 inches and be reinforced per the structural engineer's specifications. The hardscape should be underlain by 12 inches of compacted fill and installed with crack-control joints at an appropriate spacing as designed by the structural engineer to reduce the potential for shrinkage cracking. Positive drainage should be established and maintained adjacent to flatwork. To reduce the potential for differential offset, joints between the new hardscape and adjacent curbs, existing hardscape, building walls, and/or other structures, and between sections of new hardscape, should be doweled.

12.7 Preliminary Pavement Design

AC and Portland Cement Concrete (PCC) pavement sections have been designed for the proposed facility based on the subgrade soil conditions and our laboratory testing. The R-value of the subgrade soils was evaluated for a representative soil sample obtained from our exploratory boring B-1. Laboratory R-value testing indicated an R-value of 12 for the surficial materials encountered. Our flexible and rigid pavement analyses were performed using the methodology outlined in the Caltrans Highway Design Manual (Caltrans, 2020) and the Navy Pavement Design Manual (Naval Facilities Engineering Command [NAVFAC], 1979). The analyses assume a 20-year design life for new pavements and Traffic Indices (TI) of 5, 7, and 8. The TIs of 5 and 7 are generally used for parking areas and driveways subjected to light passenger vehicles and periodic truck traffic, respectively. We anticipate that a TI of 8 would be used for truck and trailer traffic. Based on the design R-value and TIs, the recommended pavement structural sections are listed in Table 3.

Table 3 – Preliminary Structural Pavement Sections			
Traffic Index	AC over CAB or AC over CMB (inches)	Full Depth AC (inches)	Full Depth PCC (inches)
≤5.0	4 over 7½	7½	6
7.0	5 over 12	10½	8½
8.0	6 over 14	12	10

Notes:
AC – Asphalt Concrete
CAB – Crushed Aggregate Base
CMB – Crushed Miscellaneous Base
PCC – Portland Cement Concrete

Subgrade soils in areas to be paved should be prepared as recommended in the Earthwork section of this report. Crushed aggregate base (CAB) or crushed miscellaneous base (CMB) material should conform to the latest edition of the Standard Specifications for Public Works Construction "Greenbook," Section 200. CAB/CMB should be compacted to a relative compaction of 95 percent as evaluated by ASTM D 1557, and should be placed at slightly above the optimum moisture content as evaluated by ASTM D 1557. AC should conform to Section 203 of the Greenbook and should be compacted to a relative compaction of 95 percent. Pavement sections should be selected based on actual anticipated traffic loading conditions and evaluation of the subgrade materials at the time of construction. We recommend that the paving operations be observed and tested by Ninyo & Moore, and that mix designs for the various pavements be made by a specialized engineering company.

12.8 Corrosivity

Laboratory testing was performed on one representative sample of near-surface soils to evaluate soil pH, electrical resistivity, water-soluble chloride content, and water-soluble sulfate content. The soil pH and electrical resistivity tests were performed in general accordance with California Test Method (CT) 643. Chloride content test was performed in general accordance with CT 422. Sulfate testing was performed in general accordance with CT 417. The laboratory test results are presented in Appendix C.

The pH value of the tested samples was 6.9 and the electrical resistivity was 3,193 ohm centimeters. The chloride content of the soil sample was 85 parts per million (ppm). The water-soluble sulfate content was 0.03 percent (i.e., 300 ppm). Based on our laboratory test results and Caltrans (Caltrans, 2021) corrosion criteria, the project site can be classified as a non-corrosive site, which is defined as having earth materials with less than 500 ppm chlorides, less than 0.15 percent sulfates (i.e., 1,500 ppm), a pH of 5.5 or more, or an electrical resistivity of more than 1,500 ohm-cm. If corrosion-susceptible improvements are planned on site, we recommend that a corrosion engineer be consulted for further evaluation and recommendations.

12.9 Concrete Placement

Concrete in contact with soil or water that contains high concentrations of water-soluble sulfates can be subject to premature chemical and/or physical deterioration. Based on the CBC criteria the potential for sulfate attack is negligible for water-soluble sulfate contents in soil ranging from 0.00 to 0.10 percent by weight, moderate for water-soluble sulfate contents ranging from 0.10 to 0.20 percent by weight, severe for water-soluble sulfate contents ranging from 0.20 to 2.00 percent by weight, and very severe for water-soluble sulfate contents over 2.00 percent by weight. The soil sample tested for this evaluation, using Caltrans Test Method 417, indicated a water-soluble sulfate content of 0.030 percent by weight (i.e., 300 ppm). Accordingly, the on-site soils are considered to have a negligible potential for sulfate attack. Due to the potential variability in soil conditions across the site and the possible use of reclaimed water, we recommend that Type II/V cement be considered for the project.

In order to reduce the potential for shrinkage cracks in the concrete during curing, we recommend that the concrete for the proposed improvements, if applicable, be placed with a slump of 4 inches based on ASTM C 143. The slump should be checked periodically at the site prior to concrete placement. We further recommend that concrete cover over reinforcing steel be provided in accordance with CBC (2019). The structural engineer should be consulted for additional concrete specifications.

12.10 Drainage

Positive surface drainage is imperative for satisfactory site performance. Positive drainage should be provided and maintained to transport surface water away from foundations and other site improvements. Positive drainage is defined as a slope of 2 percent or more over a distance of 5 feet or more away from the foundations. Surface water should not be allowed to pond adjacent to foundation elements.

13 CONSTRUCTION OBSERVATION

The recommendations provided in this report are based on our understanding of the proposed project and our evaluation of the data collected based on subsurface conditions disclosed by widely spaced exploratory borings. It is imperative that the interpolated subsurface conditions be checked by our representative during construction. Observation and testing of compacted fill and backfill should also be performed by our representative during construction. We further recommend that the project plans and specifications be reviewed by this office prior to construction. In addition, we should review the plans and specifications prior to construction. It should be noted that, upon review of these documents, some recommendations presented in this report might be revised or modified.

During construction, we recommend that the duties of the geotechnical consultant include, but not be limited to:

- Observing site clearing/demolition and removal of site improvements.
- Observing remedial grading bottoms and the placement and compaction of fill, including trench backfill, if applicable.
- Evaluating imported materials prior to their use as fill.
- Performing field tests to evaluate fill compaction.
- Observing foundation excavations for bearing materials and cleaning prior to placement of reinforcing steel or concrete.
- Observing retaining wall subdrain construction and backfill.

The recommendations provided in this report assume that Ninyo & Moore will be retained as the geotechnical consultant during the construction phase of this project. If another geotechnical consultant is selected, we request that the selected consultant indicate to the owner and to our firm in writing that our recommendations are understood and that they are in full agreement with our recommendations.

14 LIMITATIONS

The preliminary geotechnical evaluation presented in this report has been conducted in general accordance with current practice and the standard of care exercised by reputable geotechnical consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the information presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be encountered during construction. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation will be performed upon request. Please also note that our evaluation was limited to assessment of the geotechnical aspects of the project, and did not include evaluation of structural issues, environmental concerns, or the presence of hazardous materials.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report is intended for design purposes only. It is suggested that the bidders and their geotechnical consultant perform an independent evaluation of the subsurface conditions in the project areas. The independent evaluations may include, but not be limited to, review of other geotechnical reports prepared for the adjacent areas, site reconnaissance, and additional exploration and laboratory testing.

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified, and additional recommendations, if warranted, will be provided upon request. It should be understood that the conditions of a site could change with time as a result of natural processes or the activities of man at the subject site or nearby sites. In addition, changes to the applicable laws, regulations, codes, and standards of practice may occur due to government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Ninyo & Moore has no control.

This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

15 REFERENCES

Abrahamson, N.A., Silva, W.J., and Kamai, R., 2014, Summary of the ASK14 Ground Motion Relation for Active Crustal Regions. Earthquake Spectra: August 2014, Vol. 30, No. 3, pp. 1025-1055.

Allen, T.I., and Wald, D.J., 2007, Topographic slope as a proxy for global seismic site conditions ($V_{S,30}$) and amplification around the globe, U.S. Geological Survey Open-File Report 2007-1357, 69 p.

Amerco Real Estate Company, 2021a, Preliminary Site Plan, U-Haul of Gardena, 14206 Van Ness Avenue, Gardena, CA 90249, dated January 26.

Amerco Real Estate Company, 2021b, Project Development, Scope of Services, Design Assumptions, and Project Specifications: Proposal Acceptance and Terms of Agreement, dated February 23.

American Society of Civil Engineers (ASCE), 2016, Minimum Design Loads and Associated Criteria for Building and other Structures, Standard 7-16.

ASTM International (ASTM), 2018, Annual Book of ASTM Standards, West Conshohocken, Pennsylvania.

The Applied Technology Council (ATC), 2021, Hazards by Location, https://hazards.atcouncil.org/#/.

Atik, A.L., 2009, Calculation of Weighted Average 2008 NGA Models, PEER, https://apps.peer.berkeley.edu/ngawest/nga_models.html.

Boore, D.M., Stewart, J.P., Seyhan, E., and Atkinson, G.M., 2014, NGA-West2 Equations for Predicting PGA, PGV, and 5% Damped PSA for Shallow Crustal Earthquakes. Earthquake Spectra: August 2014, Vol. 30, No. 3, pp. 1057-1085.

Bowles, J.E., 1996, Foundation Analysis and Design, Fifth Edition, The McGraw-Hill Companies, Inc.

California Building Standards Commission, 2019, California Building Code (CBC): California Code of Regulations.

California Department of Transportation, 2020, Highway Design Manual, Seventh Edition, dated July 1.

California Department of Transportation, 2021, Corrosion Guidelines, Version 3.2, Division of Engineering Services, Materials Engineering and Testing Services, Corrosion Technology Branch, dated May.

California Geological Survey, State of California, 1986, Earthquake Fault Zones, Inglewood Quadrangle, 7.5-Minute Series: Scale 1:24,000, dated July 1.

California Geological Survey, 1998a, Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada: International Conference of Building Officials, dated February.

California Geological Survey, State of California, 1998b, Seismic Hazard Zone Report for the Inglewood 7.5-Minute Quadrangle, Los Angeles County, California: Seismic Hazard Zone Report 027.

California Geological Survey, State of California, 1999, Seismic Hazard Zones, Inglewood Quadrangle, 7.5-Minute Series: Scale 1:24,000, dated March 25.

California Geological Survey, 2008, Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117A, dated September 11.

Campbell, K.W., and Bozorgnia, Y., 2014, NGA-West Ground Motion Model for the Average Horizontal Components of PGA, PGV, and 5% Damped Linear Acceleration Response Spectra, Earthquake Spectra Volume 30, Issue 3, pp. 1087-1115, dated August.

CivilTech Software, 2008, Liquefy Pro (Version 5.5j), A computer program for liquefaction and settlement analysis.

Chiou, B. S.-J., and Youngs, R.R., 2014, Update of the Chiou and Young's NGA Model for the Average Horizontal Component of Peak Ground Motion and Response Spectra, Earthquake Spectra Volume 30, Issue 3, pp. 1117-1153, dated August.

Federal Emergency Management Agency (FEMA), 2008, Flood Insurance Rate Map, Los Angeles County, California and Incorporated Areas, FIRM Panel 06037C1790F, dated September 26.

GeoTracker, 2021, www.geotracker.waterboards.ca.gov/map.

Google Earth, 2021, http://earth.google.com.

Hart, E.W., and Bryant, W.A., 2007, Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zone Maps: California Department of Conservation, Division of Mines and Geology, Special Publication 42, with Supplements 1 and 2 Added in 2012, Supplement 2 added in 2014, Supplement 3 added in 2015, and Supplement 4 added in 2016.

Historic Aerials, 2021, http://www.historicaerials.com.

Jennings, C.W., and Bryant, W.A., 2010, Fault Activity Map: California Geological Survey, California Geologic Data Map Series, Map No. 6, Scale 1:750,000.

Joint Cooperative Committee of the Southern California Chapter of the American Public Works Association and Southern California Districts of the Associated General Contractors of California, 2018, "Greenbook," Standard Specifications for Public Works Construction: BNI Building News, Los Angeles, California.

Kimley Horn, 2021, Preliminary Grading Plan, 14206 Van Ness Avenue, dated August 20.

Naval Facilities Engineering Command (NAVFAC), 1979, Pavement Design Manual 5.4, Dated October.

Ninyo & Moore, 2021, Proposal for Geotechnical Evaluation, Proposed U-Haul Facility, 14206 Van Ness Avenue, Gardena, California, Proposal No. 04-03406, dated October 5.

Norris, R.M. and Webb, R.W., 1990, Geology of California: John Wiley & Sons.

Public Works Standard, Inc., 2018, The "Greenbook": Standard Specifications for Public Works Construction, with Errata No. 1, dated 2019: BNI Building News, Vista, California.

Robertson, P.K., & Wride, C.E., 1997, Cyclic Liquefaction and its evaluation based on SPT and CPT, Final contribution to the proceedings of the 1996 NCEER Workshop on Evaluation of Liquefaction Resistance.

Saucedo, G.J., Greene, H.G., Kennedy, M.P., and Bezore, S.P., 2016, Geologic Map of the Long Beach 30' x 60' Quadrangle, California, Version 2.0, Scale 1:100,000.

Seyhan, E, 2014, Weighted Average 2014 NGA West-2 GMPE, Pacific Earthquake Engineering Research Center.

Tokimatsu, K., and Seed, H.B., 1987, Evaluation of Settlements in Sands Due to Earthquake Shaking, Journal of the Geotechnical Engineering Division, ASCE, Vol. 113, No. 8, pp. 861-878.

United States Geological Survey, 2008 National Seismic Hazard Maps – Fault Parameters; https://earthquake.usgs.gov/cfusion/hazfaults_2008_search/query_main.cfm.

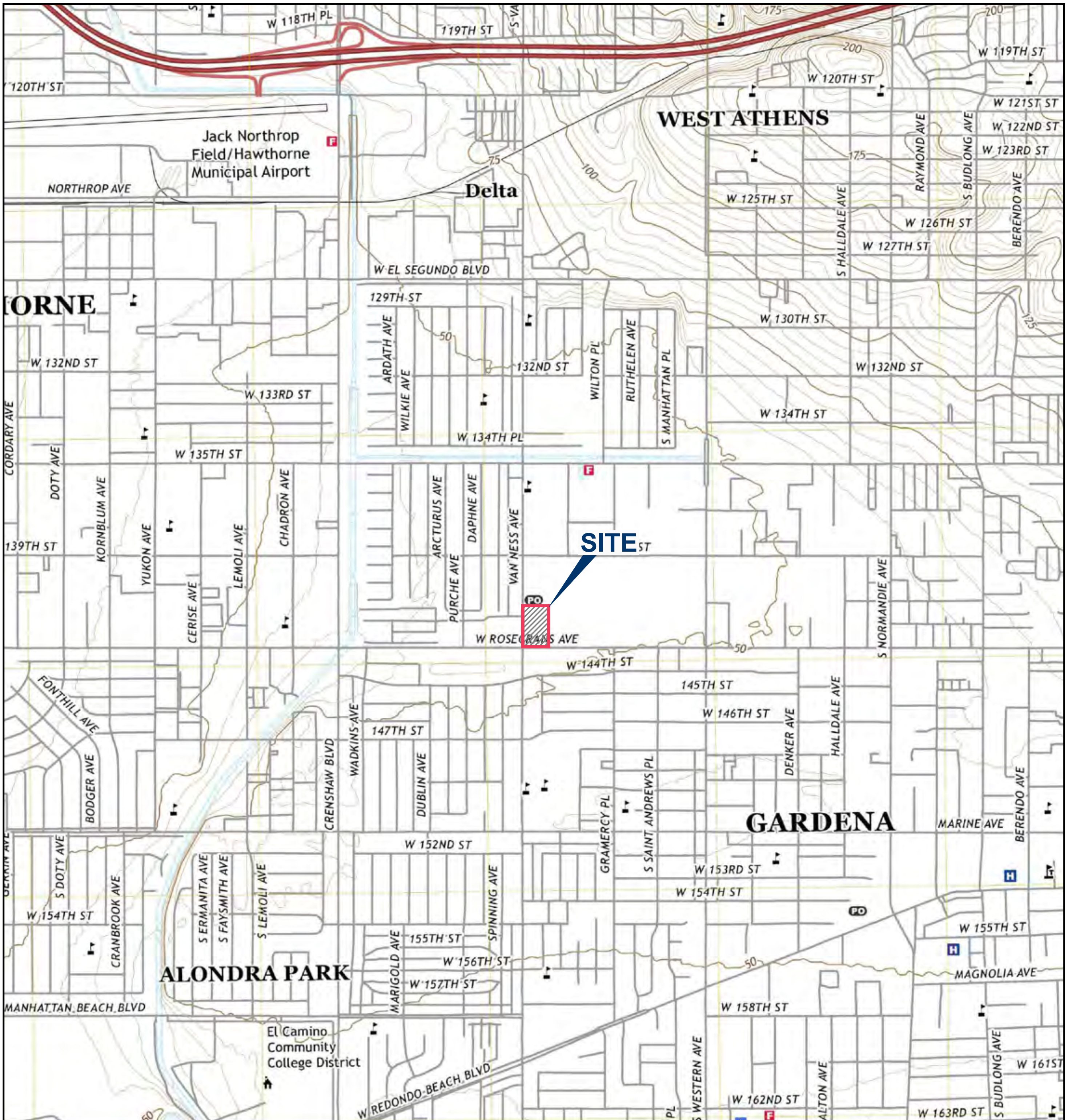
United States Geological Survey, 2018, Inglewood, California Quadrangle Map, 7.5 Minute Series: Scale 1: 24,000.

United States Geological Survey and Southern California Earthquake Center, 2020, Open Seismic Hazard Analysis, version 1.5.2, http://www.opensha.org/.

United States Geological Survey, 2021, Earthquake Hazards Program, Unified Hazard Tool, ttps://earthquake.usgs.gov/hazards/interactive/.

Youd, T.L., Idriss, I.M., Andrus, R.D., Arango, I., Castro, G., Christian, J.T., Dobry, R., Finn, W.D., Harder, L.F., Hynes, M.E., Ishihara, K., Koester, J.P., Liao, S.S.C., Marcuson, W.F., Martin, G.R., Mitchell, J.K., Moriwaki, Y., Power, M.S., Robertson, P.K., Seed, R.B., and Stokoe, K.H., II., 2001, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 124(10), 817-833.

FIGURES



NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: USGS, 2018.

N

FEET

0 2,000 4,000

FIGURE 1

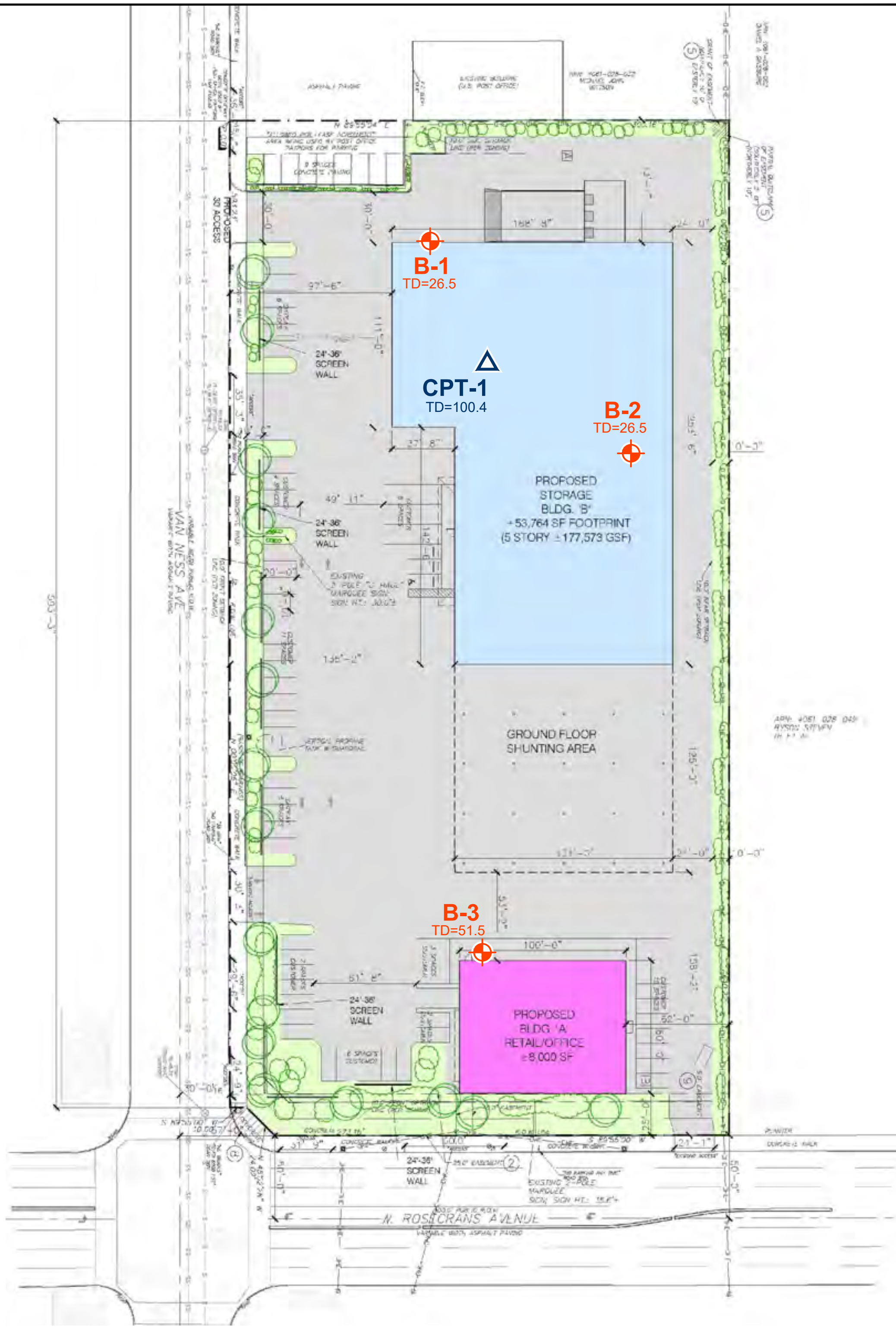
SITE LOCATION

NEW U-HAUL FACILITY
GARDENA, CALIFORNIA

211798001 I 12/21

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

1_211798001_SL.dwg 12/07/2021 GK



REFERENCE: AMERCO REAL ESTATE COMPANY, 2021.

LEGEND

B-3 TD=51.5	BORING; TD=TOTAL DEPTH IN FEET
CPT-1 TD=100.4	CONE PENETROMETER TEST; TD=TOTAL DEPTH IN FEET

N

FEET
0 100 200

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

2_211798001_SPBL.dwg 12/07/2021 GK

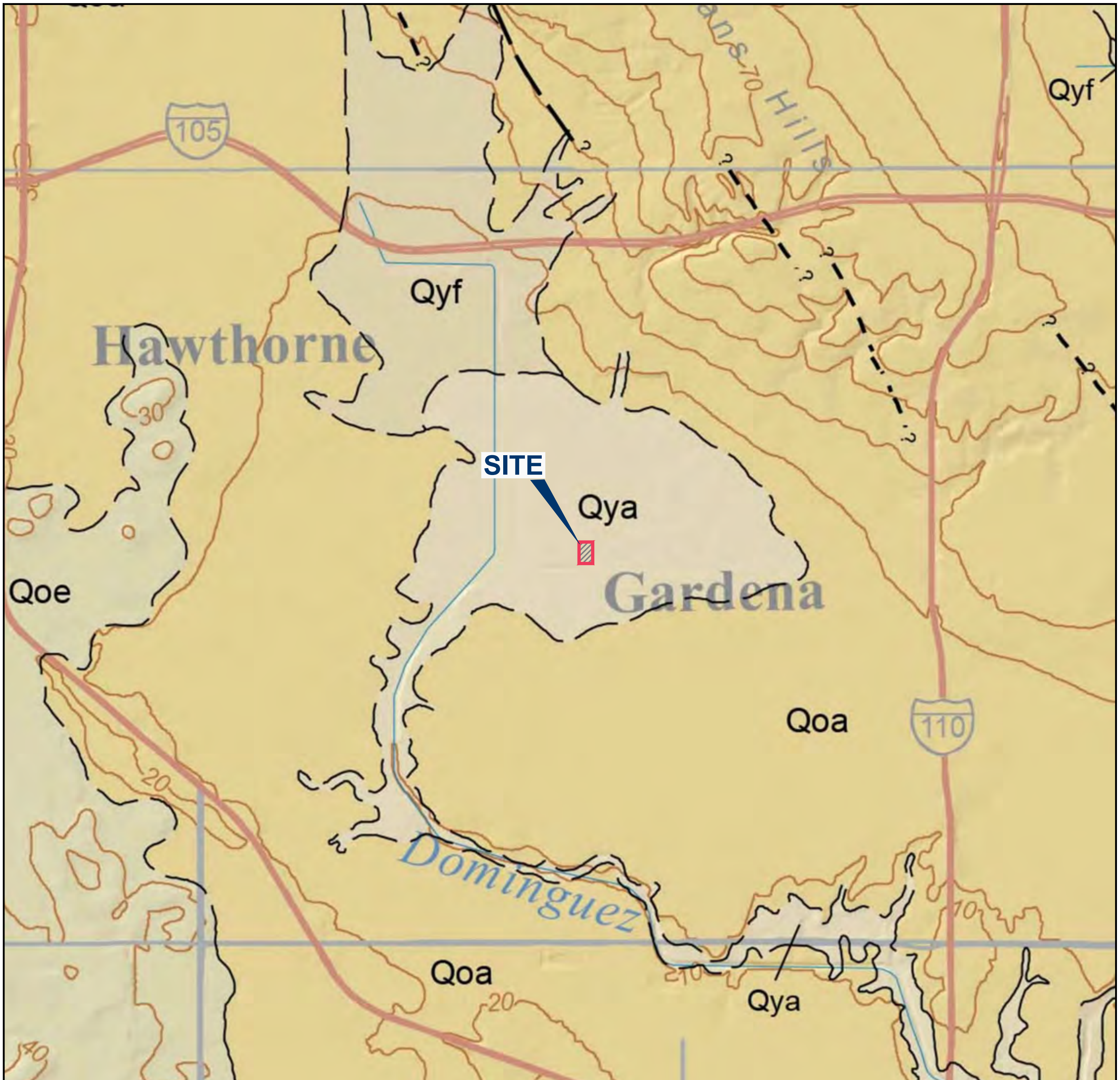
FIGURE 2

SITE PLAN AND BORING LOCATIONS

NEW U-HAUL FACILITY
GARDENA, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

211798001 I 12/21



REFERENCE: SAUCEDO AT AL., 2016, GEOLOGIC MAP OF THE LONG BEACH QUADRANGLE 30X60.

LEGEND

Qya	YOUNG ALLUVIUM
Qoa	OLD ALLUVIUM
Qyf	YOUNG ALLUVIAL FAN DEPOSITS
Qoe	OLD EOLIAN DEPOSITS
(solid line)	GEOLOGIC CONTACT
--------	FAULT

N

FEET

0 4,000 8,000

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

3_X211798001_RG.dwg 12/07/2021 GK

FIGURE 3

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

REGIONAL GEOLOGY

NEW U-HAUL FACILITY
GARDENA, CALIFORNIA

211798001 I 12/21

SOURCES: CALIFORNIA GEOLOGICAL SURVEY, ACCESSED DECEMBER 07, 2021, AT: https://www.usgs.gov/natural-hazards/earthquake-hazards/faults; ESRI, 2021.

N

MILES
0 8 16

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

211798001_FL.mxd 12/7/2021

FIGURE 4

FAULT LOCATIONS

NEW U-HAUL FACILITY
GARDENA, CALIFORNIA

211798001 | 12/21

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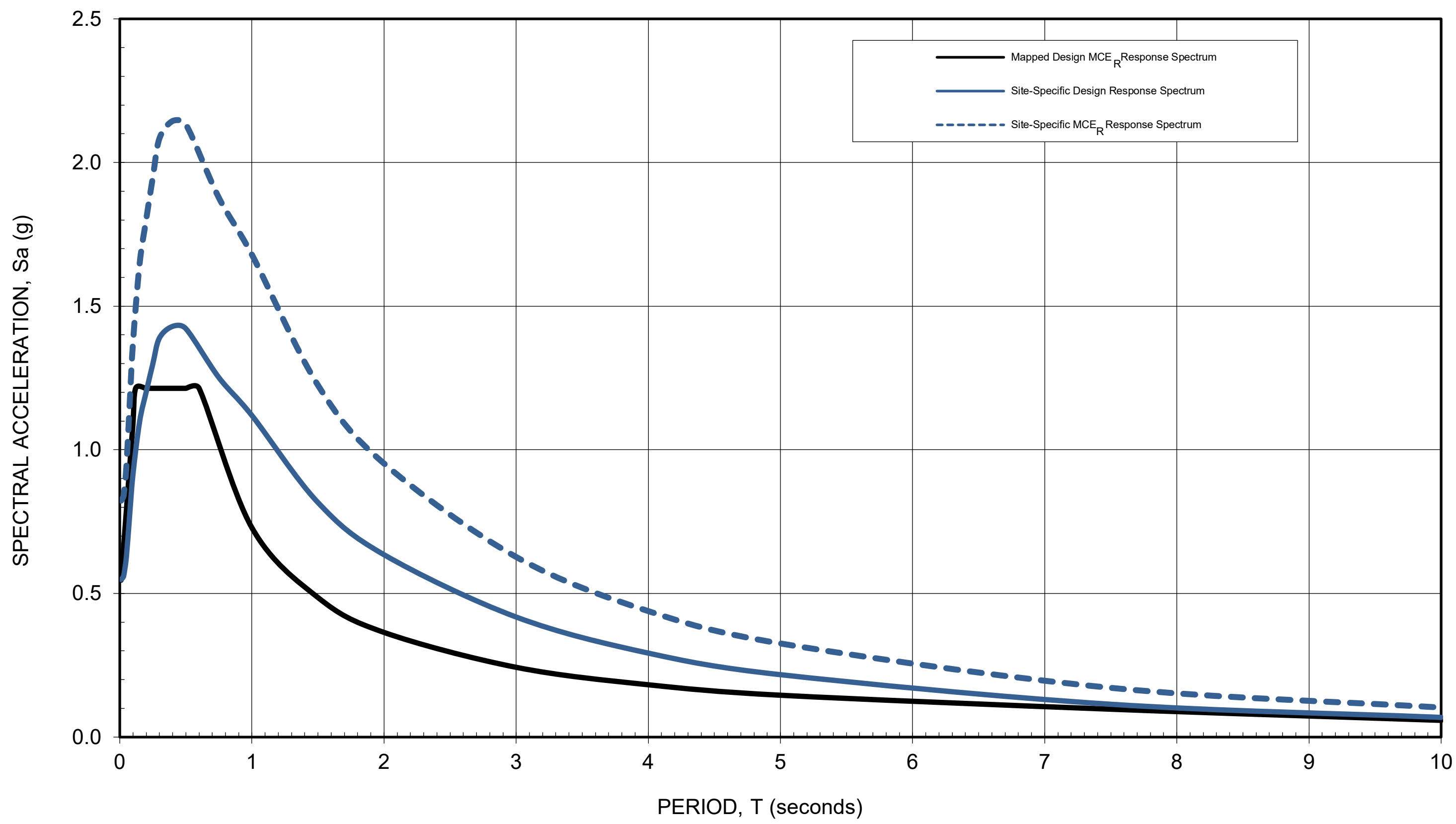
PERIOD (seconds)	SITE-SPECIFIC MCE_R RESPONSE SPECTRUM Sa (g)	SITE-SPECIFIC DESIGN RESPONSE SPECTRUM Sa (g)
0.010	0.823	0.549
0.020	0.830	0.553
0.030	0.843	0.562
0.050	0.941	0.627
0.075	1.161	0.774
0.100	1.373	0.915
0.150	1.647	1.098
0.200	1.802	1.201
0.250	1.948	1.299
0.300	2.084	1.389
0.400	2.145	1.430

PERIOD (seconds)	SITE-SPECIFIC MCE_R RESPONSE SPECTRUM Sa (g)	SITE-SPECIFIC DESIGN RESPONSE SPECTRUM Sa (g)
0.500	2.133	1.422
0.750	1.877	1.252
1.000	1.679	1.119
1.500	1.225	0.816
2.000	0.952	0.635
3.000	0.627	0.418
4.000	0.439	0.292
5.000	0.326	0.217
7.500	0.171	0.114
10.000	0.103	0.069

S_{DS} = 1.287 g	S_{D1} = 1.270 g	S_{MS} = 1.930 g	S_{M1} = 1.904 g	PGA_M = 0.828 g



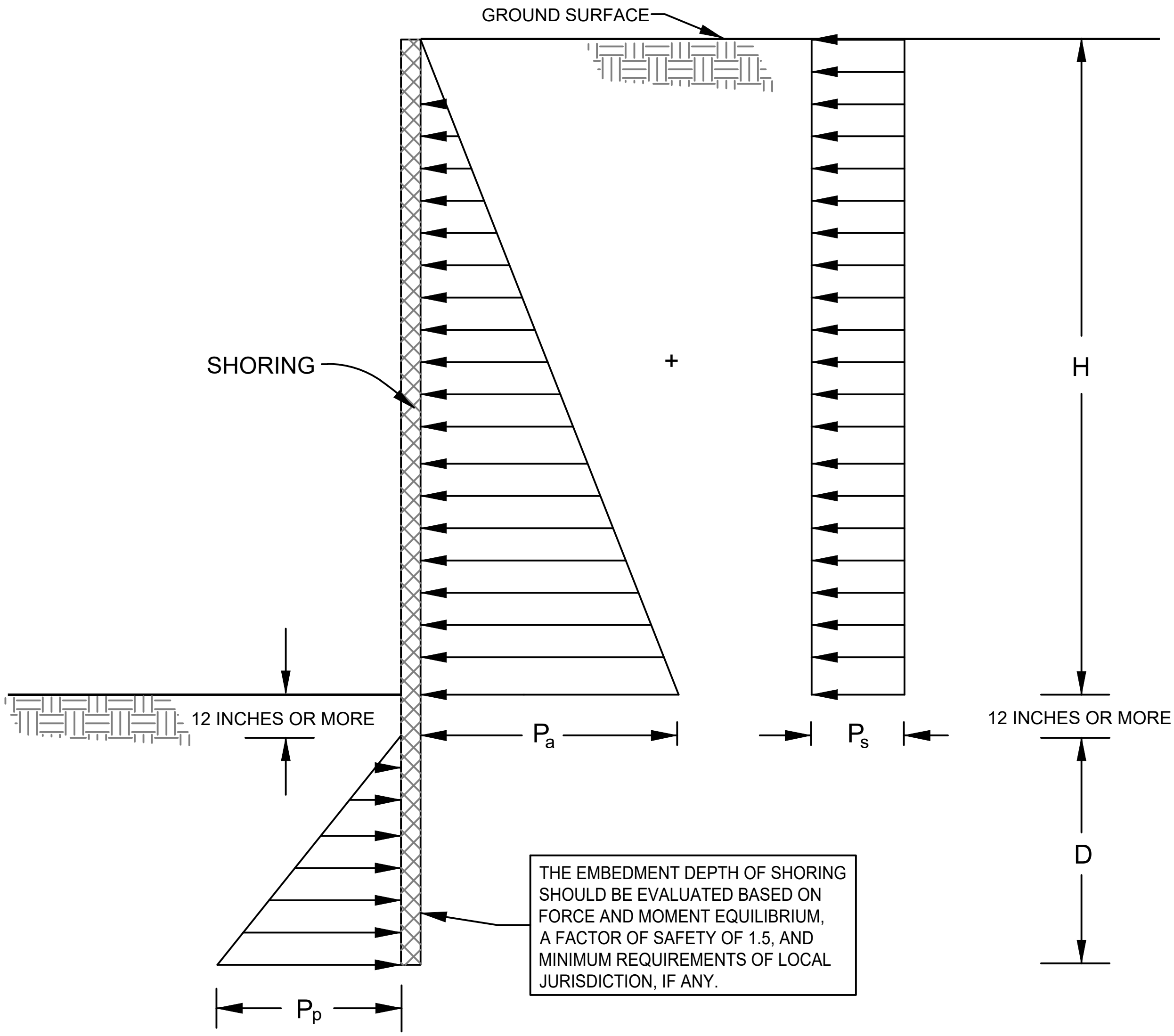
NOTES:

1 The probabilistic ground motion spectral response accelerations are based on the risk-targeted Maximum Considered Earthquake (MCE_R) having a 2% probability of exceedance in 50 years in the maximum direction using the Chiou & Youngs (2014), Campbell & Bozorgnia (2014), Boore et al. (2014), and Abrahamson et al. (2014) attenuation relationships and the risk coefficients.

2 The deterministic ground motion spectral response accelerations are for the 84th percentile of the geometric mean values in the maximum direction using the Chiou & Youngs (2014), Campbell & Bozorgnia (2014), Boore et al. (2014), and Abrahamson et al. (2014) attenuation relationships for deep soil sites considering a Mw 7.3 event on the Compton fault zone located 8.4 kilometers from the site. It conforms with the lower bound limit per ASCE 7-16 Section 21.2.2.

3 The Site-Specific MCE_R Response Spectrum is the lesser of spectral ordinates of deterministic and probabilistic accelerations at each period per ASCE 7-16 Section 21.2.3. The Site-Specific Design Response Spectrum conforms with lower bound limit per ASCE 7-16 Section 21.3.

4 The Mapped Design MCE_RResponse Spectrum is computed from mapped spectral ordinates modified for Site Class D (stiff soil profile) per ASCE 7-16 Section 11.4. It is presented for the sake of comparison.



NOTES:

1. ACTIVE LATERAL EARTH PRESSURE, P_a
   P_a = 50H psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, P_s
   P_s = 72 psf
3. PASSIVE LATERAL EARTH PRESSURE, P_p
   P_p = 300D psf
4. ASSUMES GROUNDWATER IS NOT PRESENT
5. H AND D ARE IN FEET

NOT TO SCALE

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FIGURE 6

LATERAL EARTH PRESSURES FOR TEMPORARY CANTILEVERED SHORING

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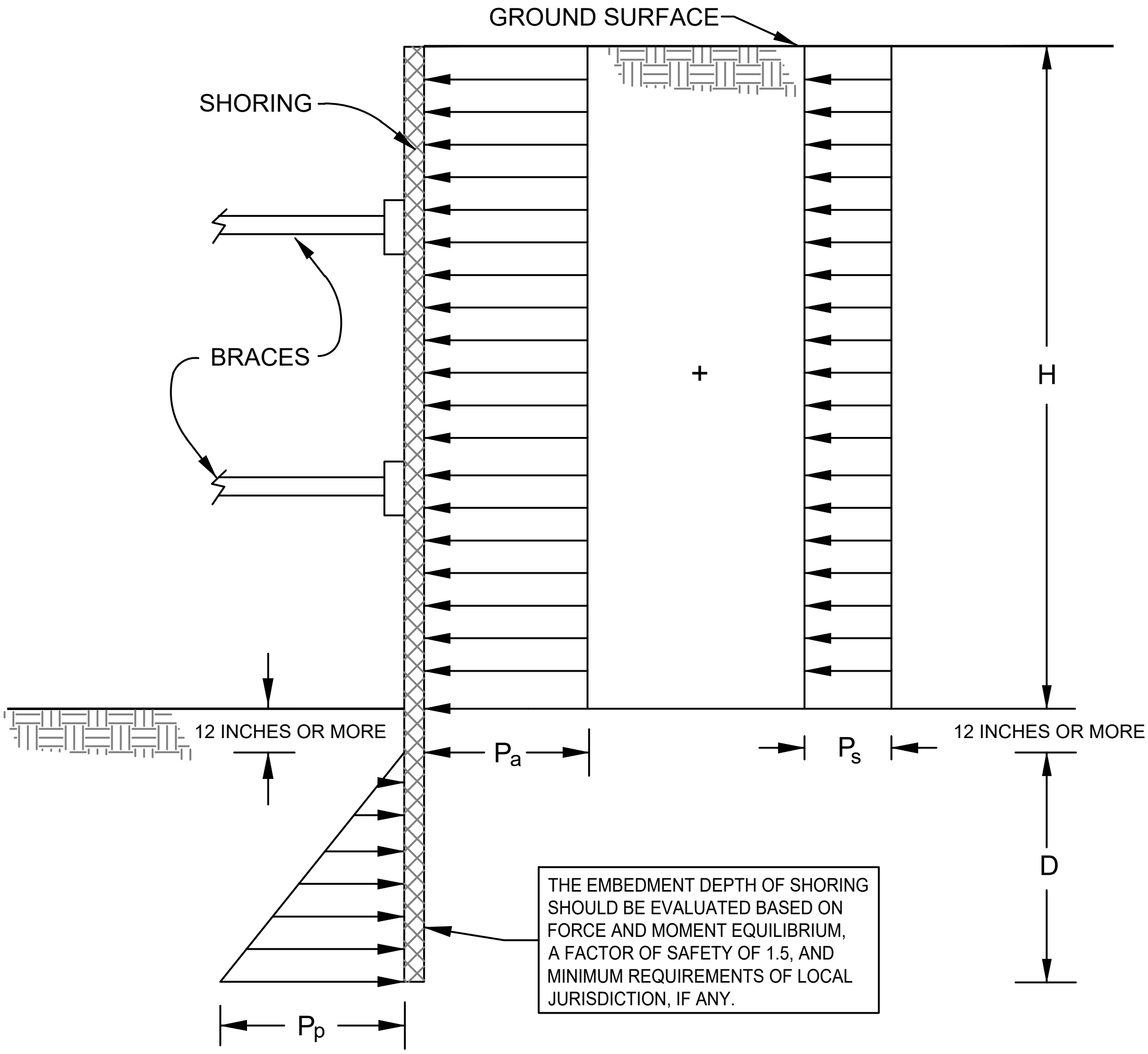

NOTES:

1. APPARENT LATERAL EARTH PRESSURE, P_a
   P_a = 50H psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, P_s
   P_s = 120 psf
3. PASSIVE LATERAL EARTH PRESSURE, P_p
   P_p = 300D psf
4. ASSUMES GROUNDWATER IS NOT PRESENT
5. SURCHARGES FROM EXCAVATED SOIL OR CONSTRUCTION MATERIALS ARE NOT INCLUDED
6. H AND D ARE IN FEET

NOT TO SCALE

FIGURE 7

LATERAL EARTH PRESSURES FOR BRACED EXCAVATION

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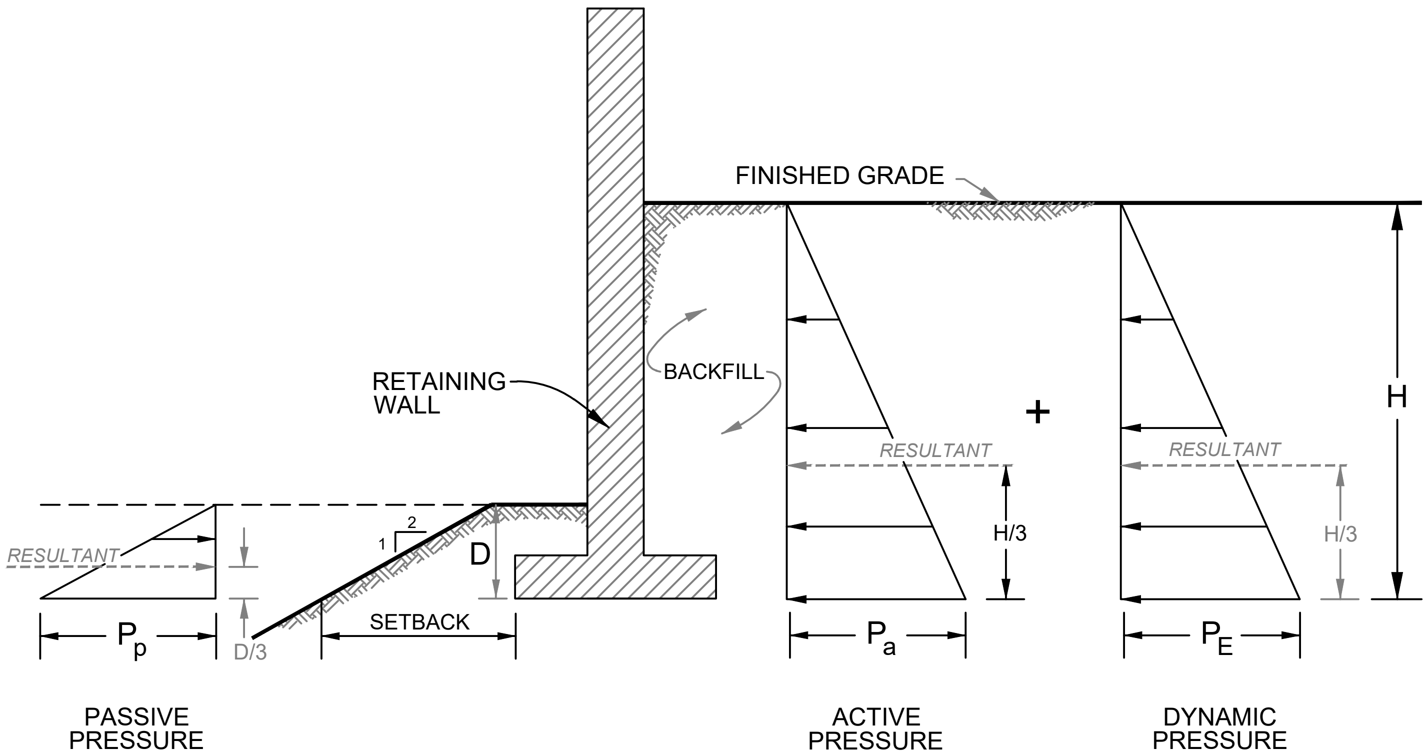

NOTES:

1. ASSUMES NO HYDROSTATIC PRESSURE BUILD-UP BEHIND THE RETAINING WALL
2. STRUCTURAL, GRANULAR BACKFILL MATERIALS AS SPECIFIED IN GREENBOOK SHOULD BE USED FOR RETAINING WALL BACKFILL
3. DRAINS AS RECOMMENDED IN THE RETAINING WALL DRAINAGE DETAIL SHOULD BE INSTALLED BEHIND THE RETAINING WALL
4. DYNAMIC LATERAL EARTH PRESSURE IS BASED ON A MAPPED DESIGN PEAK GROUND ACCELERATION OF 0.51 g
5. P_E IS CALCULATED IN ACCORDANCE WITH THE RECOMMENDATIONS OF MONONOBE AND MATSUO (1929), AND ATIK AND SITAR (2010)
6. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
7. H AND D ARE IN FEET
8. SETBACK SHOULD BE IN ACCORDANCE WITH FIGURE 1808.7.1 OF THE CBC (2019)

RECOMMENDED GEOTECHNICAL DESIGN PARAMETERS

Lateral Earth Pressure	Equivalent Fluid Pressure ($lb/ft^2/ft$)(1)	
P_a	Level Backfill with Granular Soils (2)	2H:1V Sloping Backfill with Granular Soils (2)
	37H	57H
P_E	20H	25H
P_p	Level Ground	2H:1V Descending Ground
	350D	140D

NOT TO SCALE

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FIGURE 8

LATERAL EARTH PRESSURES FOR YIELDING RETAINING WALLS

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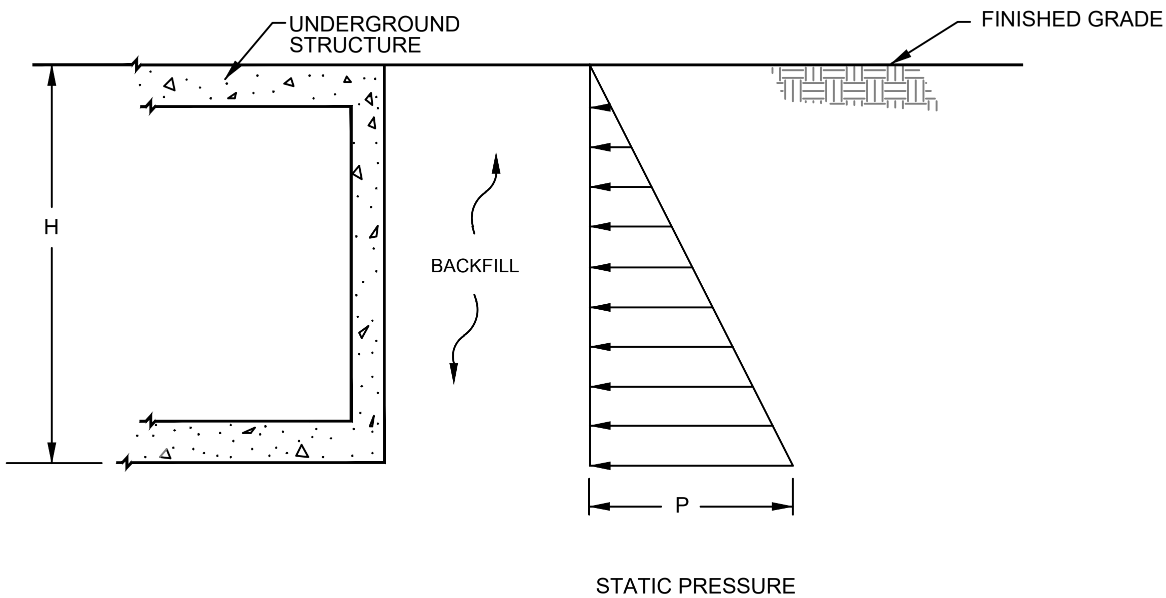

NOTES:

1. APPARENT LATERAL EARTH PRESSURES
   P= 56H psf
2. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
3. H IS IN FEET
4. STRUCTURAL, GRANULAR BACKFILL MATERIALS AS SPECIFIED IN GREENBOOK SHOULD BE USED
5. ASSUMES GROUNDWATER IS NOT PRESENT

NOT TO SCALE

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FIGURE 9

LATERAL EARTH PRESSURES FOR UNDERGROUND STRUCTURES

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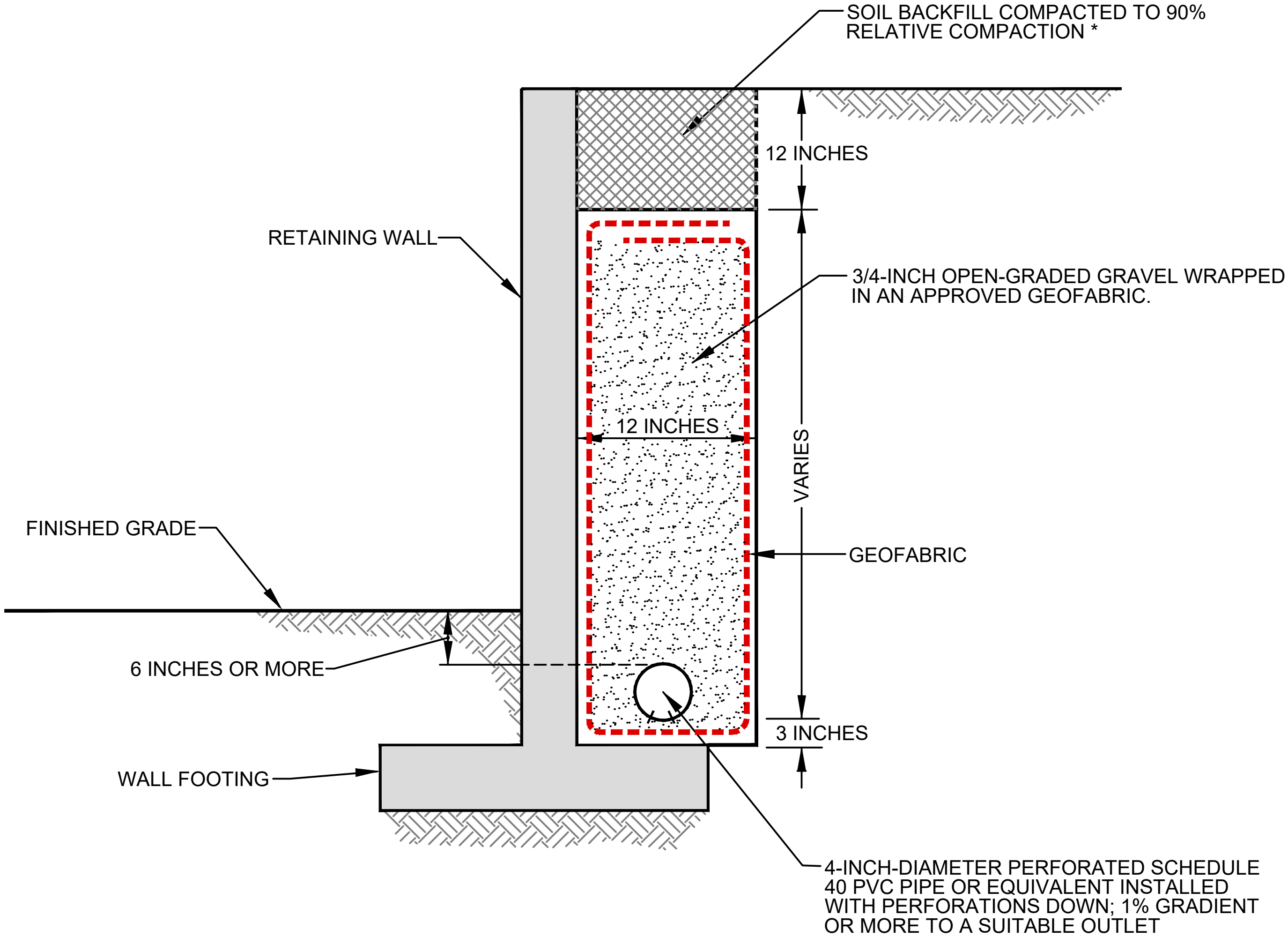

*BASED ON ASTM D1557

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NOT TO SCALE

FIGURE 10

RETAINING WALL DRAINAGE DETAIL

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APPENDIX A

Boring Logs

APPENDIX A

BORING LOGS

Field Procedure for the Collection of Disturbed Samples

Disturbed soil samples were obtained in the field using the following methods.

Bulk Samples

Bulk samples of representative earth materials were obtained from the exploratory borings. The samples were bagged and transported to the laboratory for testing.

The Standard Penetration Test (SPT) Sampler

Disturbed drive samples of earth materials were obtained by means of a Standard Penetration Test sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of 1 $^{3}/_{8}$ inches. The sampler was driven into the ground 18 inches with a 140-pound hammer falling freely from a height of 30 inches in general accordance with ASTM D 1586. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration. Soil samples were observed and removed from the sampler, bagged, sealed and transported to the laboratory for testing.

Field Procedure for the Collection of Relatively Undisturbed Samples

Relatively undisturbed soil samples were obtained in the field using the following method.

The Modified Split-Barrel Drive Sampler

The sampler, with an external diameter of 3.0 inches, was lined with 1-inch-long, thin brass rings with inside diameters of approximately 2.4 inches. The sampler barrel was driven into the ground with the weight of a 140-pound hammer in general accordance with ASTM D 3550. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer or bar, and the number of blows per foot of driving are presented on the boring logs as an index to the relative resistance of the materials sampled. The samples were removed from the sampler barrel in the brass rings, sealed, and transported to the laboratory for testing.

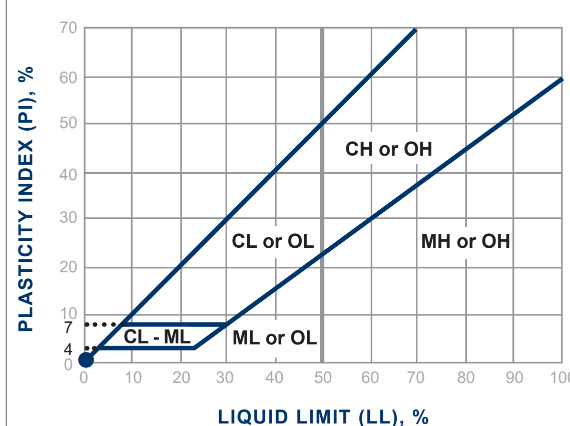
Soil Classification Chart Per ASTM D 2488

Primary Divisions			Secondary Divisions: Group Symbol	Secondary Divisions: Group Name
COARSE-GRAINED SOILS more than 50% retained on No. 200 sieve	GRAVEL more than 50% of coarse fraction retained on No. 4 sieve	CLEAN GRAVEL less than 5% fines	GW	well-graded GRAVEL
			GP	poorly graded GRAVEL
		GRAVEL with DUAL CLASSIFICATIONS 5% to 12% fines	GW-GM	well-graded GRAVEL with silt
			GP-GM	poorly graded GRAVEL with silt
			GW-GC	well-graded GRAVEL with clay
			GP-GC	poorly graded GRAVEL with clay
		GRAVEL with FINES more than 12% fines	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND 50% or more of coarse fraction passes No. 4 sieve	CLEAN SAND less than 5% fines	SW	well-graded SAND
			SP	poorly graded SAND
		SAND with DUAL CLASSIFICATIONS 5% to 12% fines	SW-SM	well-graded SAND with silt
			SP-SM	poorly graded SAND with silt
			SW-SC	well-graded SAND with clay
			SP-SC	poorly graded SAND with clay
		SAND with FINES more than 12% fines	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOILS 50% or more passes No. 200 sieve	SILT and CLAY liquid limit less than 50%	INORGANIC	CL	lean CLAY
			ML	SILT
			CL-ML	silty CLAY
		ORGANIC	OL (PI > 4)	organic CLAY
			OL (PI < 4)	organic SILT
	SILT and CLAY liquid limit 50% or more	INORGANIC	CH	fat CLAY
			MH	elastic SILT
		ORGANIC	OH (plots on or above "A"-line)	organic CLAY
			OH (plots below "A"-line)	organic SILT
	Highly Organic Soils		PT	Peat

Grain Size

Description		Sieve Size	Grain Size	Approximate Size
Boulders		> 12"	> 12"	Larger than basketball-sized
Cobbles		3 - 12"	3 - 12"	Fist-sized to basketball-sized
Gravel	Coarse	3/4 - 3"	3/4 - 3"	Thumb-sized to fist-sized
	Fine	#4 - 3/4"	0.19 - 0.75"	Pea-sized to thumb-sized
Sand	Coarse	#10 - #4	0.079 - 0.19"	Rock-salt-sized to pea-sized
	Medium	#40 - #10	0.017 - 0.079"	Sugar-sized to rock-salt-sized
	Fine	#200 - #40	0.0029 - 0.017"	Flour-sized to sugar-sized
Fines		Passing #200	< 0.0029"	Flour-sized and smaller

Plasticity Chart



Apparent Density - Coarse-Grained Soil

Apparent Density	Spooling Cable or Cathead: SPT (blows/foot)	Spooling Cable or Cathead: Modified Split Barrel (blows/foot)	Automatic Trip Hammer: SPT (blows/foot)	Automatic Trip Hammer: Modified Split Barrel (blows/foot)
Very Loose	≤ 4	≤ 8	≤ 3	≤ 5
Loose	5 - 10	9 - 21	4 - 7	6 - 14
Medium Dense	11 - 30	22 - 63	8 - 20	15 - 42
Dense	31 - 50	64 - 105	21 - 33	43 - 70
Very Dense	> 50	> 105	> 33	> 70

Consistency - Fine-Grained Soil

Consistency	Spooling Cable or Cathead: SPT (blows/foot)	Spooling Cable or Cathead: Modified Split Barrel (blows/foot)	Automatic Trip Hammer: SPT (blows/foot)	Automatic Trip Hammer: Modified Split Barrel (blows/foot)
Very Soft	< 2	< 3	< 1	< 2
Soft	2 - 4	3 - 5	1 - 3	2 - 3
Firm	5 - 8	6 - 10	4 - 5	4 - 6
Stiff	9 - 15	11 - 20	6 - 10	7 - 13
Very Stiff	16 - 30	21 - 39	11 - 20	14 - 26
Hard	> 30	> 39	> 20	> 26

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USCS METHOD OF SOIL CLASSIFICATION

BORING LOG EXPLANATION SHEET

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	
0								Bulk sample.
								Modified split-barrel drive sampler.
								No recovery with modified split-barrel drive sampler.
								Sample retained by others.
								Standard Penetration Test (SPT).
5								No recovery with a SPT.
			XX/XX					Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.
								No recovery with Shelby tube sampler.
								Continuous Push Sample.
								Seepage.
10								Groundwater encountered during drilling.
								Groundwater measured after drilling.
							SM	MAJOR MATERIAL TYPE (SOIL): Solid line denotes unit change.
							CL	Dashed line denotes material change.
15								Attitudes: Strike/Dip b: Bedding c: Contact j: Joint f: Fracture F: Fault cs: Clay Seam s: Shear bss: Basal Slide Surface sf: Shear Fracture sz: Shear Zone sbs: Shear Bedding Surface
20								The total depth line is a solid line that is drawn at the bottom of the boring.

DATE DRILLED	11/5/21
BORING NO.	B-1
GROUND ELEVATION	50' ± (MSL)
SHEET	1 OF 1
METHOD OF DRILLING	8" Hollow-Stem Auger (ABC Liovin Drilling)
DRIVE WEIGHT	140 lbs. (Auto. Trip Hammer)
DROP	18"
SAMPLED BY	VAM
LOGGED BY	VAM
REVIEWED BY	RDH

DEPTH (feet)	SAMPLES	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						ASPHALT CONCRETE: Approximately 7 inches thick.
	Bulk				CL	FILL: Brown, gray, and dark grayish brown, moist, firm, lean CLAY with sand; wood fragments; mottled.
	Driven	30	14.4	103.0	SC	Reddish yellow, moist, medium dense, clayey SAND.
					SC	ALLUVIUM: Dark gray, moist, medium dense, clayey SAND.
						Yellowish red; dense; oxide staining.
10	Driven	43	12.3	122.3		
	Driven	28	20.2	101.7		Olive brown.
	Bulk				CL	Olive brown, moist, hard, lean CLAY with sand; iron oxide staining.
20	Driven	26				
	Driven	22				
						Total Depth = 26.5 feet. Groundwater was not encountered during drilling. Backfilled with cement-bentonite grout and capped with concrete on 11/5/21.
30						Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
40						

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DATE DRILLED	11/5/21	BORING NO.	B-2
GROUND ELEVATION	51' ± (MSL)	SHEET	1 OF 1
METHOD OF DRILLING	8" Hollow-Stem Auger (ABC Liovin Drilling)		
DRIVE WEIGHT	140 lbs. (Auto. Trip Hammer)	DROP	18"
SAMPLED BY	VAM	LOGGED BY	VAM
REVIEWED BY	RDH		

DEPTH (feet)	SAMPLES Bulk / Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							ASPHALT CONCRETE: Approximately 6 inches thick.
	Bulk					CL	FILL: Brown, gray, and dark grayish brown, moist, firm, lean CLAY with sand.
	Bulk					CL	ALLUVIUM: Olive brown, moist, firm, lean CLAY with sand; iron oxide staining.
	Driven	20	21.4	103.9		SC	Reddish yellow, moist, medium dense, clayey SAND.
						CL	Dark gray, moist, very stiff, lean CLAY.
10	Bulk / Driven	32	12.4	119.2			Yellowish red; hard.
	Driven	17					Very Stiff.
			19.3	107.8		ML	Olive brown, moist, medium dense, sandy SILT; iron oxide staining.
						CL	Brown, moist, very stiff, lean CLAY with sand; iron oxide staining.
20	Driven	16					
						ML	Olive brown, moist, loose, sandy SILT; iron oxide staining.
	Driven	4					
30							
40							

Total Depth = 26.5 feet.
Groundwater was not encountered during drilling.
Backfilled with cement-bentonite grout and capped with concrete on 11/5/21.

Notes:
Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.

FIGURE A- 2

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DATE DRILLED	11/5/21	BORING NO.	B-3
GROUND ELEVATION	50' ± (MSL)	SHEET 1 OF 2	
METHOD OF DRILLING	8" Hollow-Stem Auger (ABC Liovin Drilling)		
DRIVE WEIGHT	140 lbs. (Auto. Trip Hammer)	DROP	18"
SAMPLED BY	VAM	LOGGED BY VAM	REVIEWED BY RDH

DEPTH (feet)	SAMPLES Bulk / Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0					SM	ASPHALT CONCRETE: Approximately 3.2 inches thick.
					CL	FILL: Yellowish red, moist, medium dense, silty SAND. Grayish brown, moist, firm, lean CLAY.
		14	18.5	106.6		Very stiff; red brick fragments; brown inclusions.
					SC	Yellowish red, moist, medium dense, clayey SAND.
10		25	12.5	119.2		
					ML	ALLUVIUM: Brownish yellow, moist, medium dense, sandy SILT; slickenside surfaces; iron oxide staining.
		21	34.6	100.5		
20		15				No slickenside surfaces.
					SC	Olive brown, moist, dense, clayey SAND.
		22				@ 24.9': Groundwater measured after 24 minutes. @ 25.5': Groundwater encountered during drilling.
					ML	Olive brown, wet, medium dense, sandy SILT; iron oxide staining.
30		15				
					SM	Olive brown, wet, dense, silty SAND; iron oxide staining.
		22			ML	Olive brown, wet, dense, sandy SILT; iron oxide staining.
40						

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DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
40		■	36				ML	ALLUVIUM: (Continued) Olive brown, wet, very dense, sandy SILT; iron oxide staining.
							SM	Olive brown, wet, medium dense, silty SAND; iron oxide staining.
		■	14					
							ML	Olive brown, wet, medium dense, sandy SILT; iron oxide staining.
50		■	20					
								Total Depth = 51.5 feet. Groundwater was encountered during drilling at approximately 25.5 feet. Groundwater was measured after 23 minutes at approximately 24.9 feet. Backfilled with cement-bentonite grout and capped with concrete on 11/5/21. Notes: Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
60								
70								
80								

DATE DRILLED 11/5/21 BORING NO. B-3

GROUND ELEVATION 50' ± (MSL) SHEET 2 OF 2

METHOD OF DRILLING 8" Hollow-Stem Auger (ABC Liovin Drilling)

DRIVE WEIGHT 140 lbs. (Auto. Trip Hammer) DROP 18"

SAMPLED BY VAM LOGGED BY VAM REVIEWED BY RDH

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APPENDIX B

Cone Penetration Test Data

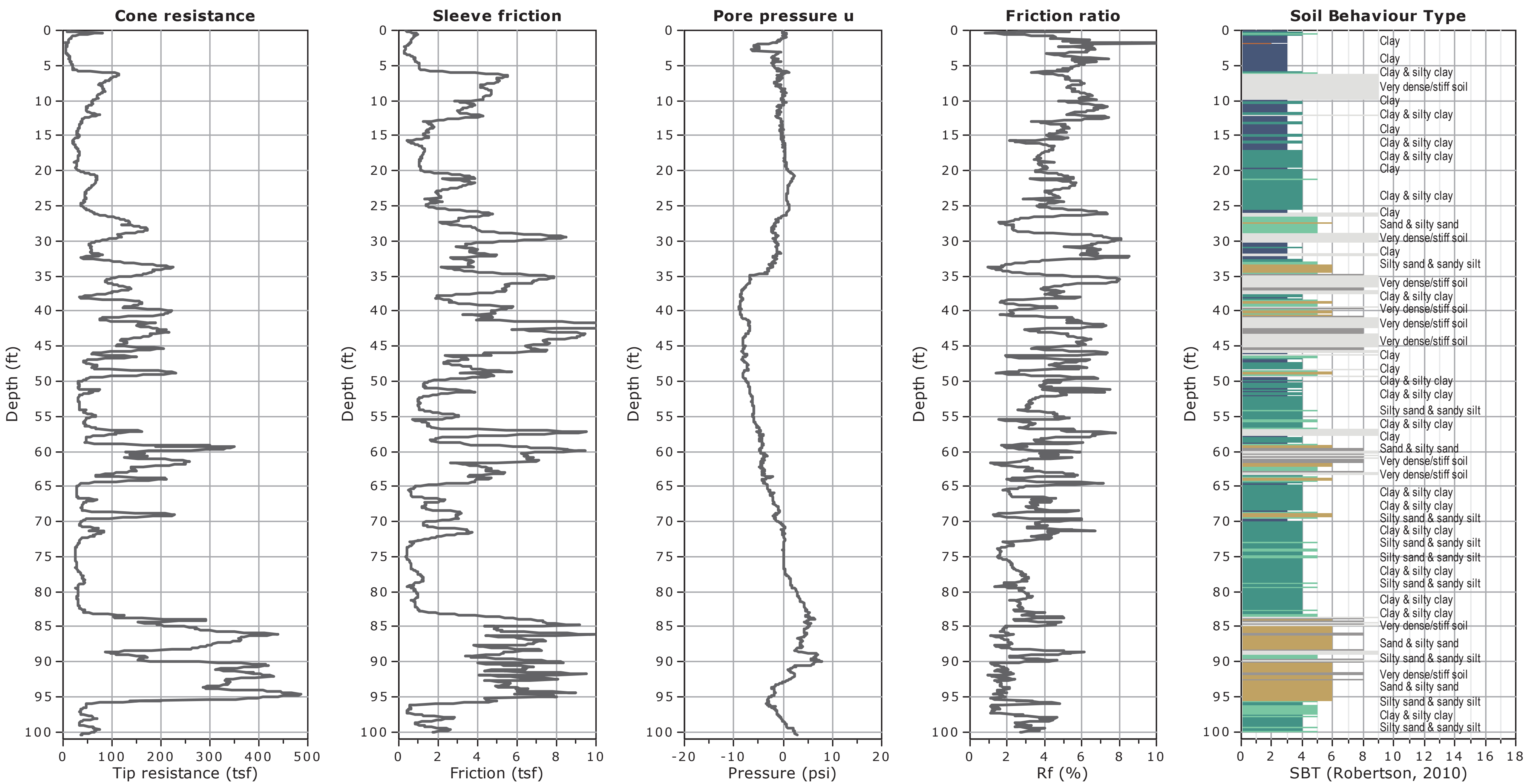
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: **Ninyo & Moore**

Location: **14206 Van Ness Ave, Gardena, CA**

CPT-1

Total depth: 100.41 ft, Date: 11/5/2021



APPENDIX C

Laboratory Testing

APPENDIX C

LABORATORY TESTING

Classification

Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488. Soil classifications are indicated on the logs of the exploratory borings in Appendix A.

In Place Moisture and Density Tests

The moisture content and dry density of relatively undisturbed samples obtained from the exploratory borings were evaluated in general accordance with ASTM D 2937. The test results are presented on the logs of the exploratory borings in Appendix A.

Percent Finer than No. 200 Sieve

An evaluation of the percentage of particles finer than the No. 200 sieve in selected soil samples was performed in general accordance with ASTM D 1140. The results of these tests are presented on Figure C-1.

Atterberg Limits

Tests were performed on a selected representative fine-grained soil sample to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D 4318. These test results were utilized to evaluate the soil classification in accordance with the USCS. The test results and classifications are shown on Figure C-2.

Consolidation Test

Consolidation test was performed on a selected relatively undisturbed soil sample in general accordance with ASTM D 2435. The sample was inundated during testing to represent adverse field conditions. The percent of consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The results of the test are summarized on Figure C-3.

Direct Shear Test

A direct shear test was performed on a relatively undisturbed sample in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of the selected material. The sample was inundated during shearing to represent adverse field conditions. The results are shown on Figure C-4.

R-Value

The resistance value, or R-value, for site soil was evaluated in general accordance with California Test (CT) 301. The sample was prepared and evaluated for exudation pressure and expansion pressure. The equilibrium R-value is reported as the lesser or more conservative of the two calculated results. The test result is shown on Figure C-5.

Soil Corrosivity Tests

Soil pH and resistivity tests were performed on a representative sample in general accordance with California Test (CT) 643. The soluble sulfate and chloride contents of the selected sample were evaluated in general accordance with CT 417 and CT 422, respectively. The test results are presented on Figure C-6.

SAMPLE LOCATION	SAMPLE DEPTH (ft)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	USCS (TOTAL SAMPLE)
B-1	5.0-5.3	CLAYEY SAND	92	39	SC
B-1	10.0-11.5	CLAYEY SAND	99	35	SC
B-1	15.0-16.5	CLAYEY SAND	100	46	SC
B-1	25.0-26.5	LEAN CLAY	100	56	CL
B-2	8.5-13.0	LEAN CLAY	100	51	CL
B-2	16.0-16.5	SANDY SILT	100	60	ML
B-3	14.0-17.0	SANDY SILT	100	72	ML
B-3	25.0-26.5	CLAYEY SAND	100	42	SC
B-3	36.0-36.5	SANDY SILT	100	58	ML

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1140

FIGURE C-1

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NO. 200 SIEVE ANALYSIS TEST RESULTS

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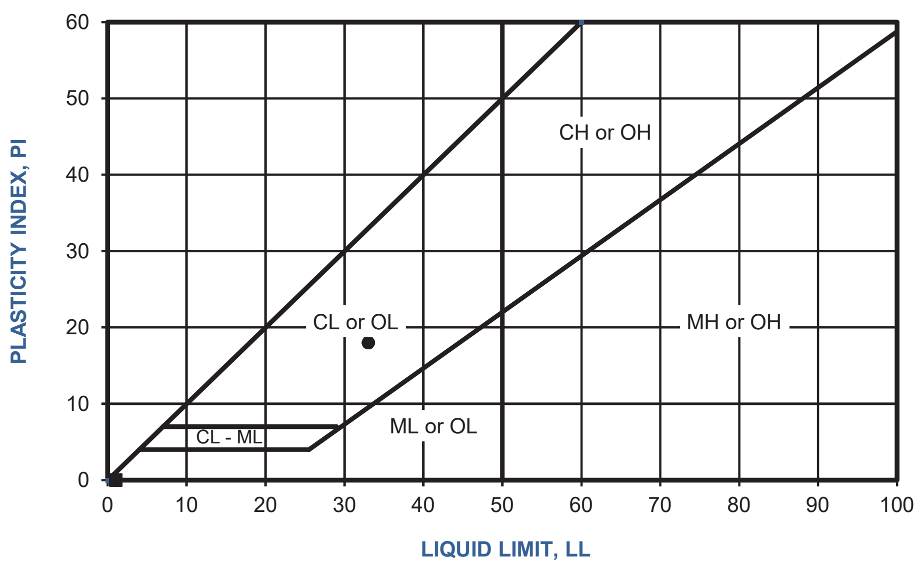
SYMBOL	LOCATION	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS
●	B-1	20.0-21.5	33	15	18	CL	CL

NP - INDICATES NON-PLASTIC



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

FIGURE C-2

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ATTERBERG LIMITS TEST RESULTS

NEW U-HAUL FACILITY
GARDENA, CALIFORNIA

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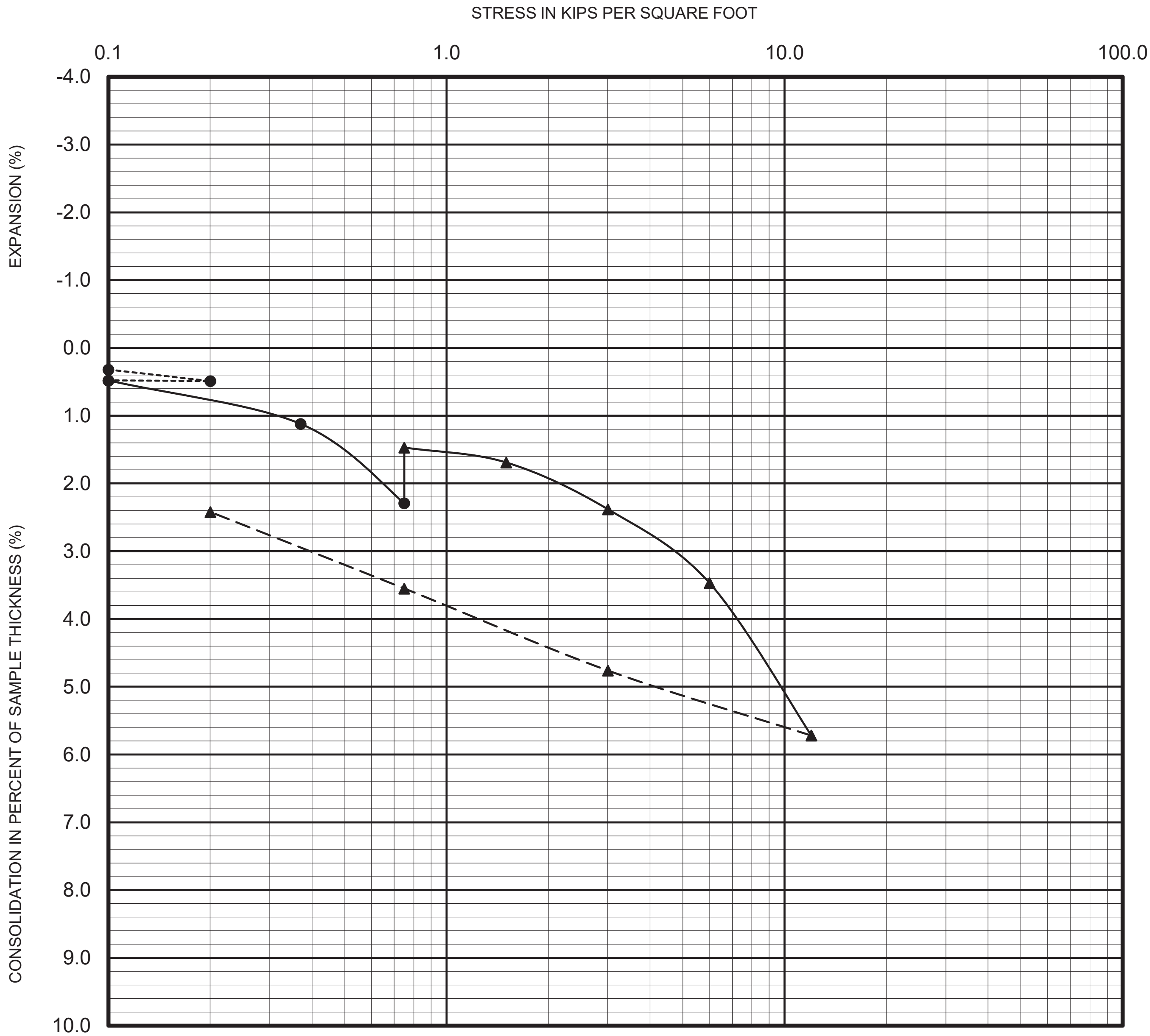

Seating Cycle		Sample Location	B-1
Loading Prior to Inundation		Depth (ft)	5.3-6.5
Loading After Inundation		Soil Type	SC
Rebound Cycle			

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

FIGURE C-3

CONSOLIDATION TEST RESULTS

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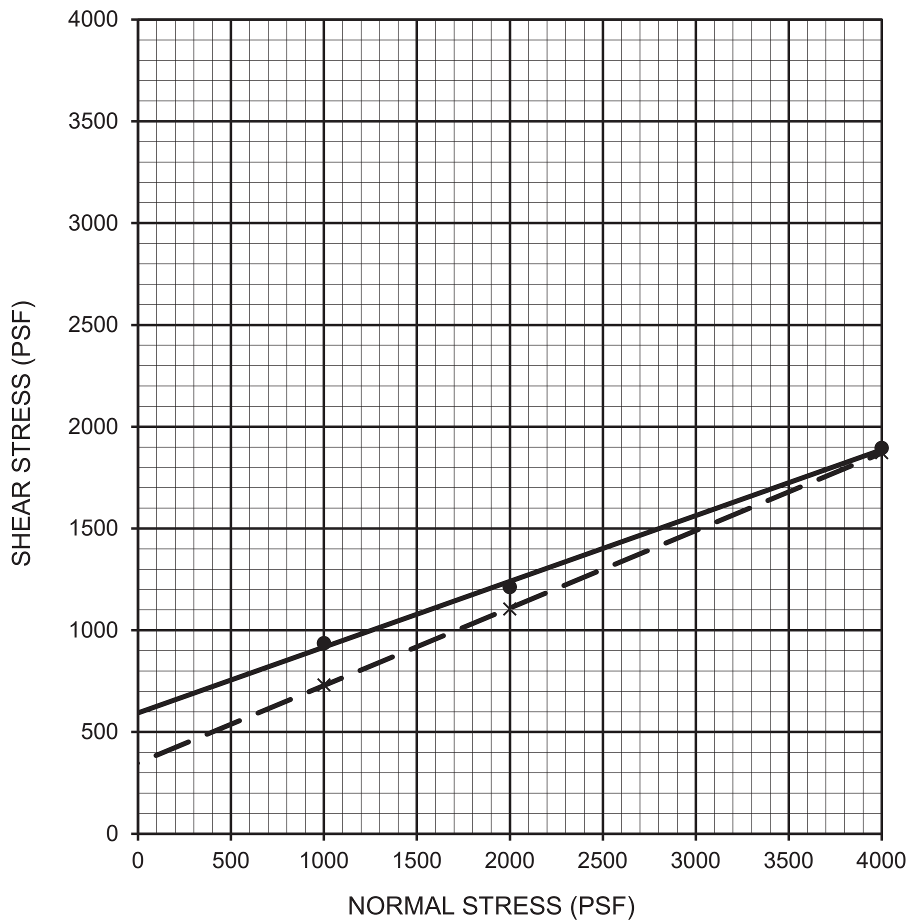

Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
LEAN CLAY	——●——	B-3	5.0-6.5	Peak	594	18	CL
LEAN CLAY	– – X – –	B-3	5.0-6.5	Ultimate	348	21	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

FIGURE C-4

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DIRECT SHEAR TEST RESULTS

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	SOIL TYPE	R-VALUE
B-1	0.6-4.0	LEAN CLAY	12

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2844/CT 301

FIGURE C-5

R-VALUE TEST RESULTS

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	pH [1]	RESISTIVITY [1] (ohm-cm)	SULFATE CONTENT [2]		CHLORIDE CONTENT [3] (ppm)
				(ppm)	(%)	
B-3	1.1-5.0	6.9	3,193	300	0.030	85

[1] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 643
[2] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 417
[3] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 422

FIGURE C-6

CORROSIVITY TEST RESULTS

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APPENDIX D

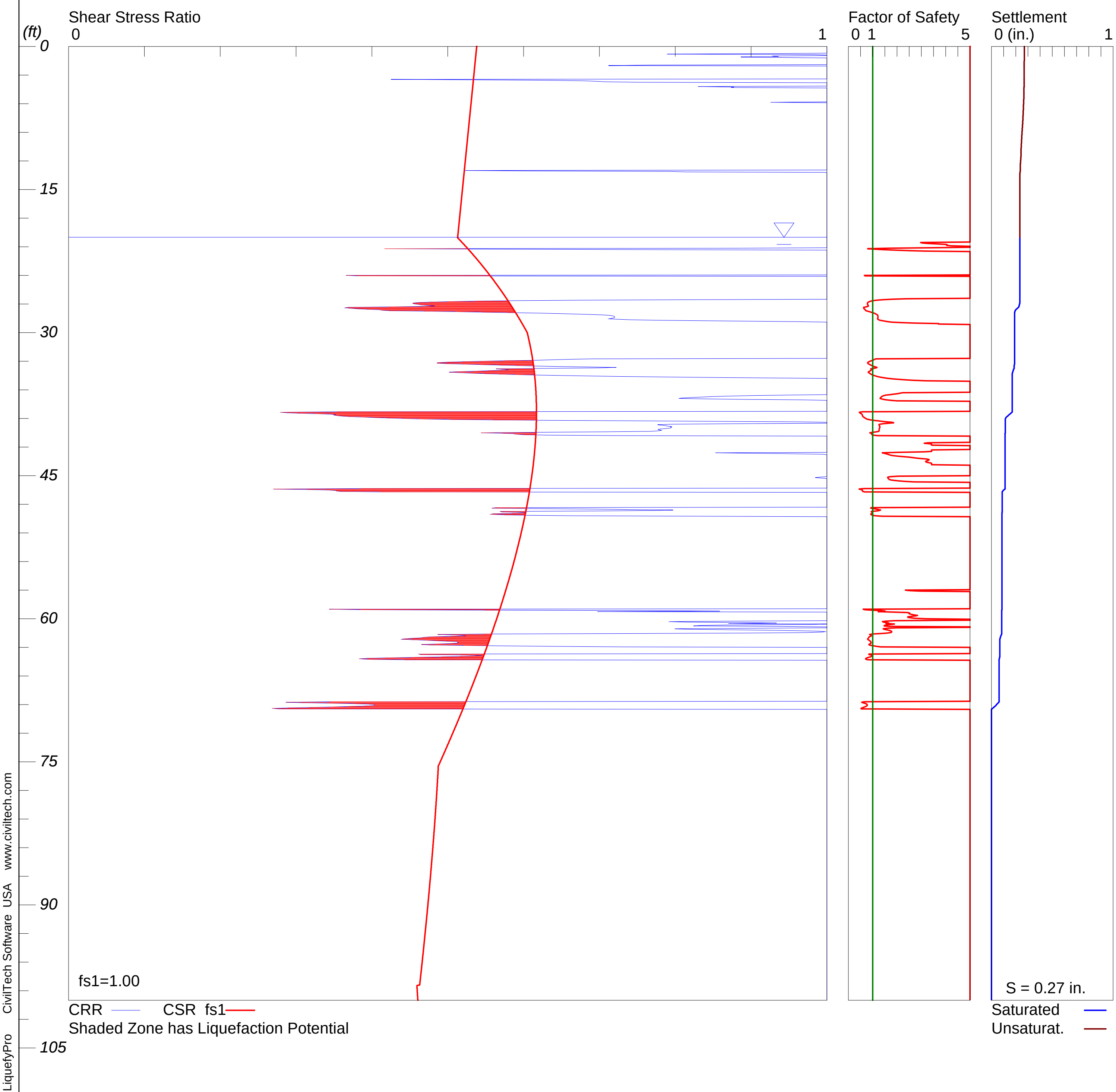
Liquefaction Analysis

LIQUEFACTION ANALYSIS

Uhaul Gardena

Hole No.=CPT-1 Water Depth=20 ft

Magnitude=7.3
Acceleration=0.828g



APPENDIX E

Amerco Real Estate Company Geotechnical Requirements

AMERCO
REAL ESTATE COMPANY

Project Development, Scope of Services, Design Assumptions, and Project Specifications:
Proposal Acceptance and Terms of Agreement

This page is required to be included as an Appendix item for reference in your Geotechnical Services proposal provided to AMERCO Real Estate Co./U-Haul International, Inc. (AREC/UHI) on every project, every time.

The following details our Standard Operating Procedures (SOPs) for services provided by our Geotechnical Engineering consultants:

Please be advised that the formal NTP is to follow the initial Kickoff (KO) Meeting that will be scheduled (unless otherwise determined in writing) for every project within 2 days after proposal acceptance is provided by the Owner; Right of Entry (ROE) is not granted with the signed agreement. A Certificate of Insurance (COI) is required for the ROE to be provided and shall be submitted to the Owner (identifying the Seller and/or additional parties identified in the KO Meeting) as the additionally insured.

It is the responsibility of the Consultant to directly contact Sabrina Perez (602-263-6502 ext. 516409) to schedule the KO meeting for every project, every time. During the KO meeting, project specifics will be discussed in detail (including review of the below and anticipated boring/exploration locations) with the National Account Manager (NAM) and/or local office Principal/Geotechnical Engineer.

***Scope of Services*:**
Please provide a scope of services to provide soils investigations and resulting recommendations for the current planning phase of development. The scope of services should include, but is not specifically limited to, the following:

- Building design uses an integral system (thickened perimeter ftgs [monolithic w/slab] and isolated cols.)
- Approximated infiltration ranges from the boring log stratigraphy to be requested during permit application (CD) phase
- Bearing capacity (min. 3,000 psf) and anticipated settlement (differential ¾" max.[between adj. columns/along 40' length])
- Corrosivity characteristics (including pH, minimum electrical resistivity, and soluble sulfate and chloride contents).
- Excavation conditions
- Expansion and collapse potential and mitigation measures
- Excavation/trench stability including bracing
- Flexible and rigid pavement design and construction (w/reliability of 90-95%, min.)
- Floor slab design criteria and construction requirements (subgrade modulus,etc)
- Foundation options (shallow and deep) for proposed improvements (to be discussed during KO Meeting)
- Geology, surface conditions, and subsurface conditions
- Groundwater conditions
- Laboratory test results
- Lateral earth pressures (including at rest, active, passive, adhesion, and coefficients of sliding friction between dissimilar materials)
- Liquefaction potential
- On-site soil suitability and structural fill recommendations (acceptable soil/material types, compaction, loose lift thickness, etc.)
- Seismic hazards (including Seismic Site Classification and Spectral Response Accelerations per current IBC code)
- Site preparation and grading
- Soil boring/exploration procedure, exploration/boring logs, and map(s) depicting final exploration/boring locations
- Subgrade improvement and site drainage
- Retaining wall design info (including for varying backfill conditions [horizontal, sloping, etc.])
    - **This information needs to be included within the Executive Summary**
- Design recommendations for shallow piers (light poles, covered parking, etc.)
    - **This information needs to be included within the Executive Summary**

Field Procedures:
It is the responsibility of the Consultant to ensure the following conditions are met for all field work performed as it relates to the services provided within the attached agreement:

- GPS coordinates are not to be identified within the proposal for review of the exploration locations.
- Consultant to request all reference material (i.e., ALTA surveys [w/topography], existing utility locations, As-Builts, Environmental Site Assessment (ESA) reports, etc.) before drilling operations occur.
- Stakeout of the exploration locations is required in advance of coordinating 811; private utility locating services will be contracted with the Owner directly but coordinated with the field consultant.
- Photo documentation is requested during stakeout of exploration locations before drilling takes place.
- Every staked location must be cleared of 5' radius around exploration location by utility locators (private, min. 25').
- Certificate of Insurance (COI) includes AMERCO/UHI and other parties (determined during KO Meeting) as additionally insured prior to field work commencing.
- Right of Entry (ROE) protocol is discussed during KO Meeting prior to field work commencing.
- Confirmation of coring for locations underlain by Portland cement concrete (PCC) to be confirmed after exploration locations are staked; and
- Should anything out of the ordinary/concerning occur during drilling, the Owner shall be notified immediately.

***Assumptions*:**
Detailed structural loading conditions and final site plans may/may not be available at this time. However, please assume the following:

- Up to four-story buildings are proposed, (unless otherwise specified).
- Anticipated settlement – differential ¾" max. (between adj. columns/along 40' length).
- No below grade/lower building level, (unless otherwise specified).
- Below grade loading docks and elevator pits may/may not be anticipated.
- Recreation vehicle (RV) canopies may/may not be anticipated.
- Wall and column load on the order of 5 to 10 kips/ft. and 150 to 200 kips, respectively, (unless otherwise specified).
- Min. soil bearing capacity desired is 3,000 psf.
- Ground floor level to be at or within 2 feet of existing site grade, (unless otherwise specified); and
- Typical traffic loads to be assumed for construction and RVs and Fire Trucks per State, County, or local agency (to include up to 90,000lb Fire Apparatus): Light Traffic Loads (~50,000 ESALs), Medium Traffic Loads (~110,000 ESALs) and Heavy Traffic Loads (~180,000 ESALS)

Project Schedule:
It is the responsibility of the Consultant to ensure that the following conditions are met for the project as it relates to the services provided within the attached agreement:

- Consultant NTP is provided by Owner during the KO Meeting, only (ROE is not granted with the signed agreement).
- The KO Meeting is to be scheduled by the Consultant within two days of the proposal acceptance: and
- Project milestones are to be discussed during every project KO Meeting including, but not limited to, anticipated fieldwork (stakeout and drilling) dates, COI entities, ROE protocol, project milestone dates, and draft/final report delivery dates.

Project Deliverables:

- A draft report and final report will be delivered by the Consultant. (The draft report will be finalized after having been reviewed by AREC/UHI and/or once a preliminary conceptual Site Plan [SP] has been made available to the Consultant [unless otherwise stated in writing.])
- An email summary of the encountered field conditions v. preliminary SP layout is required within 3 business days of reference material having been sent to the Consultant*. (The purpose of this is for the Consultant to provide input on layout/final design.)
- **An Executive Summary is required as a project synopsis within the first few pages of the report.****
- A preliminary proposal for Construction Materials Testing (CMT) is requested as a separate deliverable – preferred, but optional and at your Company's discretion.

*If the preliminary SP is unavailable, please consult with Sabrina Perez for further direction.
****The Executive Summary is a very critical page in the report, summarizing findings and conclusions by the Professional; the information presented will be used to assess the site from feasibility and construction cost perspectives by Board Members and team members, alike**. Please be thorough, consistent, accurate and concise with this page of the report, as it will demonstrate your suitability as a Professional to assist us as a National developer.

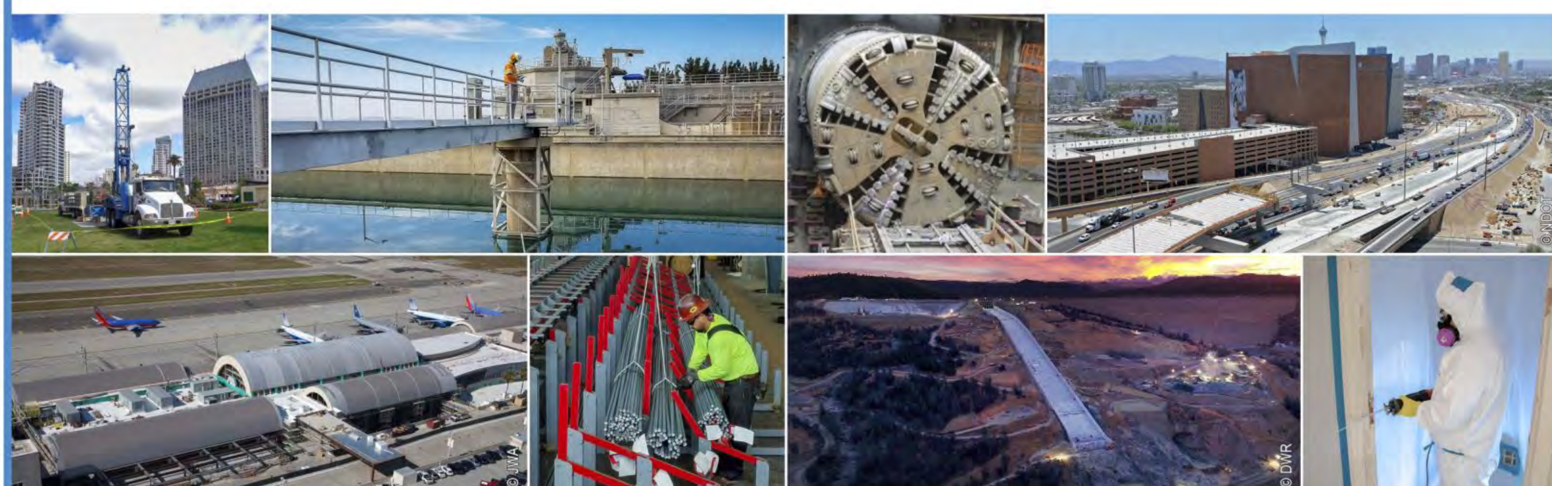
Proposal acceptance infers the above shall be met, unless otherwise provided in written correspondence. Final reviewed Boring/Exploration Map to be included as attachment with every proposal.

Thank you for your team effort.

I have read, understand, and agree to all the requirements presented in this contract between the Owner and my Company.

____________________________________ _______________
Consultant signature Date

____________________________________ _______________
AMERCO/UHI Representative Signature Date



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