Geotechnical Evaluation

Murrieta U-Haul Facility
41458 Los Alamos Road and 25086 Jefferson Avenue
Murrieta, California

Amerco Real Estate Company/U-Haul International

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

March 25, 2019 | Project No. 108673001





Geotechnical | Environmental | Construction Inspection & Testing | Forensic Engineering & Expert Witness

Geophysics | Engineering Geology | Laboratory Testing | Industrial Hygiene | Occupational Safety | Air Quality | GIS

Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

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March 25, 2019
Project No. 108673001

Ms. Sabrina Perez
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Phoenix, Arizona 85004

Subject: Geotechnical Evaluation
Murrieta U-Haul Facility
41458 Los Alamos Road and 25086 Jefferson Avenue
Murrieta, California

Dear Ms. Perez:

In accordance with our proposal dated September 12, 2018, and your authorization, Ninyo & Moore has performed a geotechnical evaluation for the new and existing U-Haul facilities in Murrieta, California. This report presents our findings, conclusions, and geotechnical recommendations for this project.

Ninyo & Moore appreciates the opportunity to be of service to you on this project.

Respectfully submitted,
NINYO & MOORE

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1 INTRODUCTION

In accordance with your authorization and our proposal dated September 12, 2018, we have performed a geotechnical evaluation for the proposed New U-Haul Facility located at 41458 Los Alamos Road and the adjacent existing U-Haul Facility at 25086 Jefferson Avenue in Murrieta, California (Figure 1). The purpose of this geotechnical evaluation was to assess the general soil and geologic conditions at the site and to develop conclusions and recommendations regarding potential geologic and seismic impacts associated with the proposed facility at 41458 Los Alamos Road and the existing facility at 25086 Jefferson Avenue. This report presents a summary of our findings and conclusions regarding the geotechnical conditions within the project area and our recommendations regarding design and construction of the proposed project.

2 EXECUTIVE SUMMARY

The following is an executive summary related to the project and our findings, conclusions and recommendations:

- The project consists of two adjacent lots. Lot one is an approximately 0.48-acre parcel located at Longitude -117.2037, Latitude 33.5561, on Assessor's Parcel Number (APN) 949-220-013, at 41458 Los Alamos Road, Murrieta, CA. Lot one is currently vacant. Lot two is an approximately 2.8-acre parcel located at Longitude -117.2034, Latitude 33.5556, on APN 949-220-055, at 25086 Jefferson Avenue, Murrieta, CA. Lot two is currently occupied by an existing U-Haul facility.

- The project is anticipated to include the construction of paved access roads and a potential storage warehouse. The warehouse is anticipated to be supported on shallow, spread footings or mat foundation.

- The Elsinore fault crosses the southwestern portion of the site. Accordingly, the site is located within the State of California Earthquake Fault Zone (EFZ) (formerly known as an Alquist-Priolo Special Studies Zone). The potential for surface ground rupture is considered high.

- In accordance with 2016 California Building Code (CBC) guidelines, the site is classified as seismic Site Class D and is located in a zone where the peak ground acceleration with 2 percent probability of being exceeded in 50 years is 0.85g.

- Groundwater was encountered as shallow as approximately 17 feet below ground surface, bgs (i.e., an elevation of approximately 1087 feet above mean sea level (MSL) in our borings. Previous subsurface evaluations performed at the site encountered groundwater at depths as shallow as approximately 13 feet (i.e., an elevation of approximately 1091 feet above MSL) (LGC Inland, 2011).

- The site is located in an area that is mapped as being potentially susceptible to liquefaction (DOC, 2018). Post-earthquake total settlement up to approximately 4 inches was calculated for the site. Differential settlements on the order of approximately 2 inches over a horizontal span of 40 feet should be expected. We understand that the proposed warehouse will not be designed to mitigate liquefaction induced settlement.

- Based on Federal Emergency Management Agency (FEMA), the project site is mapped as being in Zone X. As such, the site is considered an area of minimal flood hazard, having a 0.2 percent annual chance flood hazard, but may encounter surface waters during periods of heavy precipitation.

- Undocumented fill, extending up to approximately 10 feet bgs was encountered in our borings. This fill should be excavated and replaced with compacted, engineered fill beneath the building footprint, including new spread footings, mat foundations, and concrete slabs-on-grade.

- The on-site soils can be generally excavated using heavy duty earthmoving equipment in good working condition. However, cemented zones were encountered in our explorations and additional excavation effort should be anticipated.

- The on-site soils are generally considered suitable for re-use as engineered fill provided they meet the criteria provided herein.

- Based on the results of our soil corrosivity tests and Caltrans (2018a) criteria, the on-site soils would not be classified as corrosive.

- New pavements and flatwork should be supported on a zone of adequately moisture-conditioned and compacted engineered fill.

3 SCOPE OF SERVICES

Our scope of services for the project included:

- Performing geologic reconnaissance of the project site and reviewing readily available published and in-house geotechnical literature of the general site area including existing geotechnical reports, topographic maps, geologic and geologic hazard maps, fault maps, flood zone maps, stereoscopic aerial photographs, and information provided by the client.

- Conducting a site visit to select and mark out the proposed exploration locations. Underground Service Alert (USA) was notified prior to our subsurface evaluation. Additionally, a private utility locator was retained to clear exploration locations

- Drilling, logging, and sampling three exploratory borings to depths ranging from approximately 46.5 to 101.5 feet bgs.

- Advancing three Cone Penetration Tests (CPTs) to depths ranging from 45 to 100 feet bgs. The CPT logs are included in Appendix B.

- Collecting soil samples in the borings at selected intervals using ASTM International (ASTM) Methods D 1586 (standard penetration test (SPT) with split-barrel sampling of soils) and D3550 (ring-lined barrel sampling of soils) for laboratory testing and analysis.

- Performing laboratory testing to evaluate the in-situ moisture content and dry density, gradation analysis, Atterberg limits, consolidation, direct shear, expansion index, corrosivity (including pH, minimum electrical resistivity, and soluble sulfate and chloride content), and R-value. Laboratory test results for in-situ moisture content and dry density are shown on the boring logs in Appendix A and the results of the remaining tests are included in Appendix C.

- Preparing this report presenting our findings, conclusions and recommendations.

4 SITE DESCRIPTION

The project consists of the development of a new U-Haul facility on an approximately 0.48-acre parcel at Longitude -117.2037, Latitude 33.5561, on APN 949-220-013, at 41458 Los Alamos Road in Murrieta, California and the evaluation of an existing U-Haul facility on an approximately 2.8-acre parcel at Longitude -117.2034, Latitude 33.5556, on APN 949-220-055, at 25086 Jefferson Avenue in Murrieta, California (Figure 1).

Based on the preliminary project drawings, Sheet 2 and 3, dated September 22, 2018, Scale 1 inch = 30 feet, (Red Plains, 2018); the elevation at the project site ranges from roughly 1,103 to 1,120 feet, relative to MSL. The project site generally slopes gently from the northeast to the southwest. At the time of our field work, the project site was relatively flat, included a vacant parcel and a parcel developed with an existing U-Haul facility.

Aerial photographs from an Environmental Site Assessment (ESA) report for the site (Nova, 2018), historic aerial photographs (historicalaerials.com), and Google Earth © were reviewed for this project. Based on these historical aerial photographs, portions of the site have been developed since 1938 as agricultural land with farm houses. Since the 1950s, the site has been developed with residential houses. The site remained relatively unchanged until the early 2000s, until the existing building in the southwestern portion of the site was constructed between 2002 and 2005. Additionally, historic topographic maps show a drainage path going through the western portion of the site (historicaerials.com).

5 PROJECT DESCRIPTION

The project consists of the development of a new U-Haul facility on an approximately 0.48-acre parcel at Longitude -117.2037, Latitude 33.5561, on APN 949-220-013, at 41458 Los Alamos Road in Murrieta, California and the evaluation of an existing U-Haul facility on an approximately 2.8-acre parcel at Longitude -117.2034, Latitude 33.5556, on APN 949-220-055, at 25086 Jefferson Avenue in Murrieta, California (Figure 1).

We understand a conceptual site plan has not yet been developed for the proposed new U-Haul facility, however, the project is anticipated to include the construction of paved access roads and a potential storage warehouse. The warehouse is anticipated to be supported on shallow, spread footings or mat foundation. Wall and column loads are anticipated to be on the order of 5 to 10 kips per foot and 100 to 150 kips, respectively. Recreational vehicle (RV) canopies may also be constructed. Retaining walls are not anticipated to be constructed.

The site located at 41458 Los Alamos Road is currently vacant. Site grading plans were not available at the time this report was prepared; however, we understand the proposed ground floor level will be situated at or within 2 feet of existing site grades. New access drives and parking areas will be paved with concrete or asphalt. Traffic loads will be typical for RVs and fire trucks per Riverside County and/or City of Murrieta standards. The site located at 25086 Jefferson Avenue is currently occupied by an existing U-Haul facility. The existing U-Haul facility consists of a single story, approximately 5,775 square foot building and a two-story, approximately 24,460 square foot building. Additionally, the site includes associated parking and driveways.

The site is located in an Alquist-Priolo Earthquake Fault Zone and is in an area mapped as being susceptible to liquefaction, however, we understand the proposed storage warehouse will have less than 2,000 human occupancy hours per year, so fault evaluation is not included in this scope. Additionally, we understand that the proposed warehouse will not be designed to mitigate liquefaction induced settlement.

6 SUBSURFACE EXPLORATION AND LABORATORY TESTING

On October 16 and October 22, 2018, we performed field explorations to evaluate the subsurface conditions at the project site. Our field exploration program consisted of drilling, logging, and sampling three small-diameter exploratory borings, designated as B-1 through B-3, to depths of approximately 46.5 to 101.5 feet bgs, and advancing three CPTs, designated as CPT-1 thorough CPT-3, to depths of approximately 46 to 100 feet bgs. The borings were drilled using a CME-95 truck-mounted drill rig equipped with hollow-stem augers and the CPTs were advanced using a truck-mounted CPT rig. The approximate locations of the borings and CPTs are depicted on Figure 2. Logs of the borings and CPTs are included in Appendix A and B, respectively.

Soil samples were obtained at selected intervals by driving a sampler approximately 18 inches into the soil, using an automatic 140-pound hammer falling approximately 30 inches. Relatively undisturbed ring samples were obtained with a modified ring sampler (ASTM D 3550), and disturbed samples were obtained using an unlined SPT sampler (ASTM D 1586). Bulk samples consisting of auger cuttings of representative earth materials were obtained at selected locations.

Ninyo & Moore logged the borings in general accordance with the Unified Soil Classification System (USCS) by observing auger cuttings and samples. The ring samples were trimmed in the field, wrapped in plastic bags, and placed in moisture-tight cylindrical plastic containers. Larger bulk samples were collected at selected locations from the cuttings. Soil classifications and other pertinent data are presented on the boring logs in Appendix A.

The samples collected during our drilling activities were transported to the Ninyo & Moore laboratory in San Diego, California, for geotechnical laboratory testing. Geotechnical testing included in-situ moisture content and dry density, gradation, Atterberg limits, consolidation, direct shear, expansion index, corrosivity (including pH, minimum electrical resistivity, and soluble sulfate and chloride content), and R-value. The results of the in-situ moisture and density testing are presented on the boring logs in Appendix A. A description of each test method and the laboratory results of the remainder of the tests are presented in Appendix C.

7 GEOLOGY AND SUBSURFACE CONDITIONS

The following sections describe the geology at the site as well as potential geologic hazards.

7.1 Geologic Setting

The project area is situated in the inland section of the Peninsular Ranges Geomorphic Province. This geomorphic province encompasses an area that extends approximately 900 miles from the Transverse Ranges and the Los Angeles Basin south to the southern tip of Baja California (Norris and Webb, 1990). The province varies in width from approximately 30 to 100 miles. In general, the province consists of rugged mountains underlain by Jurassic metavolcanic and metasedimentary rocks, and Cretaceous igneous rocks of the southern California batholith. The portion of the province in Riverside County that includes the project area consists generally of uplifted and dissected Quaternary sedimentary rock.

The Peninsular Ranges Province is traversed by a group of sub-parallel faults and fault zones trending roughly northwest. Several of these faults, which are shown on Figure 3, Fault Location Map, are considered active faults. The San Jacinto and San Andreas faults are active fault systems located northeast of the project area and the Agua Blanca–Coronado Bank, San Clemente, Newport Inglewood and Elsinore faults are active faults located west of the project area. The project site is located within the Alquist-Priolo Earthquake Fault Zone (Figure 4). The Elsinore fault, the nearest active fault, has been mapped approximately 0.1 mile west of the project site. Major tectonic activity associated with these and other faults within this regional tectonic framework consists primarily of right-lateral, strike-slip movement. Further discussion of faulting relative to the site is provided in the Faulting and Seismicity section of this report.

7.2 Subsurface Conditions

Geologic units encountered during our subsurface exploration included fill soils and Pauba Formation (Kennedy and Morton, 2003). Although not encountered in our subsurface exploration, alluvial deposits have been mapped near the western portion project site (Kennedy and Morton, 2003). Our knowledge of the subsurface conditions at the project site is based on our field exploration and laboratory testing, and our understanding of the general geology of the area. The following sections provide a generalized description of the materials encountered. More detailed descriptions are presented on the boring logs in Appendix A and CPT logs in Appendix B. The geology of the site is shown on Figure 5.

7.2.1 Pavement

Pavement, composed of AC over AB was encountered in boring B-3 and CPT-1 through CPT-3. The pavement encountered generally consisted of approximately 2 inches of AC over approximately 6 inches of AB.

7.2.2 Fill

Undocumented fill, composed of loose to medium dense silty sand, decomposed granite, and firm sandy lean clay was encountered below the aggregate base in boring B-3 and at the surface in borings B-1 and B-2. The fill extended to a depth up to approximately 10 feet below existing grade in our borings.

7.2.3 Alluvium

While not encountered in our subsurface evaluation, young alluvial materials have been mapped near the western portion of the project site. These materials are generally expected to consist of unconsolidated sand, silt, and clay.

7.2.4 Pauba Formation

Materials of the Pauba Formation were encountered below the fill in our borings and extended to the total depths explored. As encountered, these materials generally consisted of various shades of brown, gray, and black, moist to wet, moderately cemented, silty fine to coarse grained sandstone, sandy siltstone, and weakly to moderately indurated, sandy claystone. Heaving sands were encountered in boring B-1 at approximately 45 feet below existing grade.

7.3 Groundwater

Groundwater was encountered between 17 feet and 24 feet during drilling (i.e., elevations of approximately 1087 and 1080 feet above MSL). Previous subsurface evaluations performed at the site encountered groundwater at depths as shallow as 13 feet (i.e., an elevation of approximately 1091 feet above MSL) (LGC Inland, 2011). However, it should be noted that groundwater levels could fluctuate due to seasonal variations, precipitation, irrigation, groundwater withdrawal or recharge, and in areas adjacent to and in ephemeral streams, and other factors.

7.4 Surface Water

The Federal Emergency Management Agency FEMA published flood map for the subject site is number 06065C2715G, effective on August 28, 2008. Based on this map the project site is mapped as being in Zone X. As such, the site is considered an area of minimal flood hazard, having a 0.2 percent annual chance flood hazard, but may encounter surface waters during periods of heavy precipitation.

8 GEOLOGIC HAZARDS

The following sections describe potential geologic hazards at the site, including faulting and seismicity, ground surface rupture, strong ground motion, liquefaction, tsunamis, landsliding, and tsunamis.

8.1 Faulting and Seismicity

The project area is considered to be seismically active. Based on our review of the referenced geologic maps and stereoscopic aerial photographs, as well as our geologic field mapping, the subject site is underlain by the Elsinore fault, which crosses the southwestern portion of the site. The subject site is located within a State of California Earthquake Fault Zone (EFZ) (formerly known as an Alquist-Priolo Special Studies Zone) (Hart and Bryant, 1997). Furthermore, the potential for strong ground motion is considered significant during the design life of the proposed structure. Figure 3 shows the approximate site location relative to the major faults in the region.

Table 1 lists selected principal known active faults that may affect the subject site, the approximate fault to site distance, and the maximum moment magnitude (M_{max}) and the fault types provided by the United States Geological Survey (USGS) National Seismic Hazard Maps – Fault Parameters website (USGS, 2008). The locations and magnitudes of the faults were calculated from near the center of the project site at Longitude -117.2037 and Latitude 33.5561.

Table 1 – Principal Active Faults

Faults	Approximate Fault-to-Site Distance miles (kilometers)	Maximum Moment Magnitude (M_{max})
Elsinore (Temecula Segment)	0.1 (0.16)	7.1
Elsinore (Glen Ivy Segment)	6.4 (10.2)	6.9
Elsinore (Julian Segment)	18.6 (30.0)	7.4
San Jacinto (Anza Segment)	19.8 (31.9)	7.3
San Jacinto (San Jacinto Valley Segment)	22.1 (35.6)	7.0
San Joaquin Hills	27.3 (44.0)	7.1
Newport-Inglewood (Offshore)	28.6 (46.0)	7.0
Elsinore (Whittier Segment)	29.2 (46.9)	7.0
San Jacinto (San Bernardino Valley Segment)	31.8 (51.2)	7.1
Rose Canyon	32.5 (52.3)	6.9
South San Andreas (South San Bernardino Segment)	35.5 (57.1)	7.0
South San Andreas (Banning/Garnet Hill Segment)	35.8 (57.6)	7.1
San Jacinto (Coyote Creek Segment)	38.6 (62.1)	7.0
San Jacinto (Clark Segment)	40.1 (64.5)	7.1
South San Andreas (North San Bernardino Segment)	40.9 (65.9)	6.9
Coronado Bank	43.9 (70.7)	7.4
Earthquake Valley	44.1 (71.0)	6.8
Pinto Mountain	44.2 (71.1)	7.3
Puente Hills (Coyote Hills)	44.9 (72.3)	6.9
Cucamonga	44.9 (72.3)	6.7
Palos Verdes	45.5 (73.1)	7.3
San Jose	47.4 (76.3)	6.7
Cleghorn	49.6 (79.8)	6.8
Sierra Madre	49.8 (80.1)	7.2
North Frontal (West)	52.6 (84.6)	7.2
Burnt Mountain	53.6 (86.2)	6.8
Puente Hills (Santa Fe Springs)	53.6 (86.2)	6.7
South San Andreas (South Mojave Segment)	56.0 (90.2)	7.3
Eureka Peak	56.9 (91.5)	6.7
South San Andreas (Coachella Segment)	57.4 (92.4)	7.0
Helendale-So Lockhart	57.5 (92.5)	7.4
North Frontal (East)	58.6 (94.3)	7.0
Clamshell-Sawpit	59.9 (96.4)	6.7

8.2 Surface Rupture

The closest known active fault is the Elsinore Fault which crosses the southwestern portion of the site. The site is located within the State of California Earthquake Fault Zone (EFZ) (formerly known as an Alquist-Priolo Special Studies Zone) (Hart and Bryant, 1997), as shown on Figure 4. The Elsinore Fault (Temecula segment) is capable of generating an earthquake magnitude of 7.1 (USGS, 2008).

Based on our review of the referenced literature, the potential for ground rupture due to faulting is considered high. Lurching or cracking of the ground surface as a result of nearby seismic events is possible.

As noted, we understand the proposed storage warehouse will have less than 2,000 human occupancy hours per year, so fault evaluation is not included in this scope.

8.3 Strong Ground Motion

The 2016 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. The MCE_R ground motion response accelerations are based on the spectral response accelerations for 5 percent damping in the direction of maximum horizontal response and incorporate a target risk for structural collapse equivalent to 1 percent in 50 years with deterministic limits for near-source effects. The horizontal peak ground acceleration (PGA) that corresponds to the MCE_R for the site was calculated as 0.83g using the United States Geological Survey (USGS, 2018b) seismic design tool (web-based).

The 2016 CBC specifies that the potential for liquefaction and soil strength loss be evaluated, where applicable, for the Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration with adjustment for site class effects in accordance with the American Society of Civil Engineers (ASCE) 7-10 Standard. The MCE_G peak ground acceleration is based on the geometric mean peak ground acceleration with a 2 percent probability of exceedance in 50 years. The MCE_G peak ground acceleration with adjustment for site class effects (PGA_M) was calculated as 0.85g using the USGS (USGS, 2018b) seismic design tool that yielded a mapped MCE_G peak ground acceleration of 0.85g for the site and a site coefficient (F_{PGA}) of 1.00 for Site Class D.

8.4 Seismic Design Considerations

Table 2 presents the seismic design parameters for the site in accordance with CBC guidelines and mapped spectral acceleration parameters (USGS, 2018b): These ground motion values are calculated for "Stiff Soil" sites, which correspond to a shear-wave velocity of approximately 600 to 1200 feet per second in approximately the top 100 feet bgs. Different soil or rock types may amplify or de-amplify these values. The proposed improvements should be designed in accordance with the requirements of governing jurisdictions and applicable building codes.

Table 2 – 2016 California Building Code Seismic Design Criteria

Seismic Design Factors	Value
Site Class	D
Site Coefficient, F_a	1.000
Site Coefficient, F_v	1.500
Mapped Spectral Response Acceleration at 0.2-second Period, S_s	2.082g
Mapped Spectral Response Acceleration at 1.0-second Period, S_1	0.849g
Spectral Response Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	2.082g
Spectral Response Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	1.274g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	1.388g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.849g

8.5 Liquefaction and Seismically Induced Settlement

Liquefaction is the phenomenon in which loosely deposited granular soils with silt and clay contents of less than approximately 35 percent and non-plastic silts located below the water table undergo rapid loss of shear strength 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, and 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 California Department of Conservation (DOC) Maps Data Viewer (DOC, 2018), the proposed site is located within an area mapped as being potentially susceptible to liquefaction. As noted in the previous sections, the site is underlain by fill soils, alluvium, and materials of the Pauba Formation, and groundwater was encountered as shallow as approximately 17 feet bgs in our borings. Accordingly, we evaluated the liquefaction potential at the project site. Liquefaction evaluation was performed using a maximum moment magnitude of

7.1 associated with Elsinore (Temecula Segment) fault and MCE_G peak ground acceleration with adjustment for site class effects (PGA_M) of 0.85g as discussed in previous sections. A groundwater depth of 13 feet was used in our analysis based on previous subsurface evaluations as discussed in the groundwater section. The liquefaction analysis was performed using the computer program LiquefyPro (CivilTech Software, 2007). The analysis was based on the National Center for Earthquake Engineering Research (NCEER) procedure (Youd, et al., 2001) using Modified Robertson Method (1997).

As a result of liquefaction, the proposed structure may be subject to several hazards, including liquefaction-induced settlement. In order to estimate the amount of post-earthquake settlement, the method proposed by Ishihara and Yoshimine (1992) was used for the evaluation of dynamic settlement using the computer program LiquefyPro (CivilTech Software, 2007). 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. Liquefaction calculations were performed based on our boring and CPT data. Post-earthquake total settlement up to approximately 4-inches was calculated for the site. Differential settlements on the order of approximately 2-inches over a horizontal span of 40 feet should be expected. Our liquefaction analysis results are presented in Appendix D. As noted, we understand that the proposed warehouse will not be designed to mitigate liquefaction induced settlement.

8.6 Landslides

Landslides, slope failures, and mudflows of earth materials generally occur where slopes are steep and/or the earth materials are too weak to support themselves. No landslides or indications of deep-seated landslides were noted underlying the project site during our field exploration or our review of available geologic literature and topographic maps. Based on the relatively level topography at the site, the potential for landslides or mudflows to affect the project site is considered low.

8.7 Tsunamis

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. Based on the inland location of the site, the potential for damage due to tsunamis is not a design consideration.

9 CONCLUSIONS

Based on our geotechnical evaluation, it is our opinion that construction of the proposed project is feasible from a geotechnical standpoint, provided the recommendations presented in this report are incorporated into the design and construction of the project. Geotechnical considerations and conclusions include the following:

- The site is generally underlain by fill soils, alluvium, and materials of the Pauba Formation. Fill soils were encountered in our borings to depths up to approximately 10 feet bgs. The fill materials are compressible and not considered unsuitable for support of structures in their current condition. Recommendations for remedial grading are presented in the following sections of this report.

- Groundwater was encountered as shallow as approximately 17 feet bgs (i.e., an elevation of approximately 1087 feet above MSL) in our borings. Previous subsurface evaluations performed at the site encountered groundwater at depths as shallow as approximately 13 feet (i.e., an elevation of approximately 1091 feet above MSL) (LGC Inland, 2011). Fluctuations in the depth to groundwater will occur due to tidal fluctuations, flood events, seasonal precipitation, variations in ground elevations, subsurface stratification, irrigation, groundwater pumping, storm water infiltration, and other factors.

- The contractor should be prepared to address issues associated with seepage, perched water conditions and groundwater such as excavation stability, dewatering, and the presence of wet subgrade soils that may require stabilization.

- The on-site soils can be generally excavated using heavy duty earthmoving equipment in good working condition. However, cemented zones were encountered in our explorations and additional excavation effort should be anticipated. As noted previously, the site was developed in the past with residential houses. Therefore, construction debris should also be anticipated.

- The on-site soils are considered suitable for re-use as engineered fill provided they meet the criteria provided herein.

- Caving should be anticipated by the contractor and excavations may require shoring if loose soils, seepage, perched water conditions, and groundwater are encountered.

- The Elsinore fault crosses the southwestern portion of the site. Accordingly, the site is located within the State of California Earthquake Fault Zone (EFZ) (formerly known as an Alquist-Priolo Special Studies Zone). The potential for surface ground rupture is considered high.

- The site is located in an area that is mapped as being potentially susceptible to liquefaction (DOC, 2018). Post-earthquake total settlement up to approximately 4-inches was calculated for the site. Differential settlements on the order of approximately 2-inches over a horizontal span of 40 feet should be expected. As noted previously, we understand that the proposed warehouse will not be designed to mitigate liquefaction induced settlement.

- Based on the results of our soil corrosivity tests and Caltrans (2018a) criteria, the on-site soils would not be classified as corrosive.

10 RECOMMENDATIONS

The following sections present our geotechnical recommendations for the proposed construction. The proposed improvements should be constructed in accordance with the following recommendations and the requirements of the applicable governing agencies. If the proposed construction is changed from that discussed in this report, Ninyo & Moore should be contacted for additional recommendations.

10.1 Earthwork

The following sections provide our earthwork recommendations for this project. If the site grade is planned to change by more than 2 feet vertically, Ninyo & Moore should be contacted for additional recommendations.

10.1.1 Pre-Construction Survey

Prior to construction activities, it may be desirable to recognize the condition of the existing utilities, underground structures, or other features that are near the planned construction and to survey or document (e.g., photographs, video, official documentation, etc.) their pre-construction condition. The findings of the survey could be used to document any damage that might result from this project.

10.1.2 Site Preparation

Vegetation, unsuitable materials, or debris from the clearing operation should be removed from the site and disposed of. Obstructions that extend below finish grade, if present, should be removed and the resulting voids filled with moisture-conditioned and compacted engineered fill.

After rough grade has been achieved and prior to further earthwork, the exposed subgrade should be proof-rolled and visually observed for the presence of debris, organic matter and other unsuitable materials. If unsuitable soils are encountered at subgrade level during earthwork operations, these soils should be removed to their full depth, and be replaced with engineered fill.

The geotechnical consultant should carefully evaluate any areas of loose, soft, or wet soils prior to placement of fill or other construction. Drying or over-excavation of some materials may be appropriate.

10.1.3 Remedial Grading for Building Pad

We recommend that the existing fill soils be overexcavated down to competent formational materials or 6 feet below the proposed grade, whichever is deeper. This overexcavation should extend to the horizontal limits of the structural footprint (including foundations for attached overhangs, canopies, and other building appurtenances) plus a horizontal distance of 5 feet. The extent and depths of removals and overexcavations should be evaluated by Ninyo & Moore's representative in the field based on the materials exposed. The resultant overexcavation surface should be scarified to a depth of approximately 8 inches, moisture conditioned and recompacted to a relative compaction of 90 percent as evaluated by the ASTM D 1557 prior to placing new fill. The resulting excavation should then be backfilled with granular soils with a very low to low expansion potential (i.e., an expansion index [EI] of 50 or less) up to 2 feet below bottom of slab-on-grade. The upper 2 feet below bottom of slab-on-grade should backfilled with granular soils with a very low expansion potential (i.e., an expansion index [EI] of 20 or less). These materials should be placed and compacted in accordance with the Fill Placement and Compaction section of this report.

10.1.4 Remedial Grading for Pavement and Flatwork

In the proposed pavement and flatwork areas, we recommend that the on-site soils be overexcavated to a depth of 1 foot below the subgrade elevation. The proposed overexcavations should extend outward horizontally 2 feet from the horizontal limits of the pavement or flatwork. The extent and depth of removals should be evaluated by Ninyo & Moore's representative in the field based on the material exposed. The resulting surface should be scarified 8 inches, moisture conditioned, and recompacted to a relative compaction of 90 percent as evaluated by ASTM D 1557. The overexcavation should then be filled with engineered fill. The engineered fill should be moisture conditioned to near optimum moisture content and compacted to a relative compaction of 90 percent as evaluated by ASTM D 1557. The upper 12 inches of subgrade soils beneath vehicular pavements should be placed at a relative compaction of 95 percent as evaluated by ASTM D 1557.

10.1.5 Excavation Characteristics

Our evaluation of the excavation characteristics of the on-site materials is based on the results of subsurface exploration, site observations, and experience with similar materials. Excavation of the surface materials can generally be accomplished using heavy-duty earthmoving equipment in good operating condition. However, very dense soils, and cemented materials were observed in our borings. These materials are anticipated to be more difficult to excavate and/or slow the rate of excavation during construction. Due to the relatively widely spaced nature of our borings and the past history of the site, differing subsurface conditions should also be anticipated during construction.

10.1.6 Temporary Slopes

The contractor should provide safely sloped excavations or an adequately constructed shoring system in compliance with Occupational Safety and Health Administration (OSHA) Regulations for employees working in an excavation that may expose them to the danger of moving ground. The contractor should make his own evaluation of OSHA soil type based on actual conditions encountered in the field. Based on the soil conditions encountered at the site during our field explorations, we recommend that OSHA Soil “Type C” classification be used for excavations in fill and alluvium and that OSHA Soil “Type B” classification be used for excavations in Pauba Formation. This corresponds to temporary slopes of 1.5H:1V (Horizontal to Vertical [H:V]) for OSHA Soil “Type C” and 1H:1V for OSHA Soil “Type B”. This side slope inclination is for excavations that are less than 20 feet deep. Excavations encountering seepage should be evaluated on a case-by-case basis. On-site safety of personnel is the responsibility of the contractor.

Excavations that encounter soils with low cohesion will not stand open without shoring or bracing. Temporary excavations that encounter groundwater seepage or surface runoff may need shoring or may be stabilized by placing sandbags or gravel along the base of the seepage zone. Excavations encountering seepage should be evaluated on a case-by-case basis. Slope stability for trenches deeper than 20 feet, though not anticipated, should be designed by the contractor’s engineer based on alignment-specific soil properties and settlement-sensitive features. If material is stored or equipment is operated near an excavation, stronger shoring should be used to resist the extra pressure due to superimposed loads.

Additionally, due to the heterogeneity of the site soil conditions, sloughing of the surficial native soils during construction should be anticipated where the excavation abuts fill soils from adjacent utilities and/or is subject to the influence of vibration from nearby traffic.

10.1.7 Temporary Shoring

In excavations where loose soils or soils with low cohesion, seepage, perched water conditions, and groundwater are encountered we recommend that a temporary earth retention system be utilized. Temporary earth retention systems may include braced systems, such as trench boxes or shields with internal supports or cantilever systems (e.g., soldier piles and lagging); however, the risk of excessive lateral deflection may render a cantilevered shoring system inappropriate for the project.

For preliminary design of the shoring system, the magnitude and distribution of lateral earth pressures presented on Figure 6 for cantilevered shoring and Figure 7 for braced shoring should be used. The recommended design earth pressures are based on the assumptions that the shoring system will be constructed without raising the ground surface elevation behind the shoring system, that there are no stockpiles of soil and/or construction materials, or other loads that act above a 1:1 (horizontal to vertical) plane extending up and back from the dredge line. For earth retention systems subjected to the above-mentioned surcharge loads, the contractor should include the effect of these loads on the design lateral earth pressures. In addition, where loose, low cohesion soils are encountered, the excavations may not stand open long enough to install the trench boxes. The contractor should be prepared to deal with these soil conditions and plan accordingly. Once installed, some sloughing is possible at the ends of the trench box; therefore, any loose material should be removed prior to backfilling of the trench. We recommend that an experienced structural engineer design the shoring system. The shoring parameters presented in this report should be considered as guidelines.

We anticipate that settlement of the ground surface will occur behind shoring systems during excavation. The amount of settlement will depend on the type of shoring system used, the contractor's workmanship, and soil conditions. We recommend that embankments, roadways, utilities, and other structures in the vicinity of the planned excavation be evaluated with regard to foundation support and tolerance to settlement. To reduce the potential for distress to these structures, we recommend that the shoring system be designed to limit the ground settlement behind it to ½-inch or less. Possible causes of settlement that should be addressed include settlement during excavation, construction vibrations, de-watering (if needed), and removal of the shoring system. We recommend that shoring installation be evaluated carefully by the contractor prior to construction and that ground vibration and settlement monitoring be performed during construction.

The contractor should evaluate the adequacy of the shoring parameters presented in this report, and make the appropriate modifications for their design. We recommend that the contractor take appropriate measures to protect the workers. OSHA requirements pertaining to workers' safety should be observed. We further recommend that the construction methods provided herein be carefully evaluated by a qualified specialty contractor prior to commencement of the construction.

10.1.8 Protection of Existing Structures/Utilities

Lateral movement of a shored excavation will depend on the type and relative stiffness of the system used and other factors beyond the scope of this study. The shoring designer should perform a deflection analysis for the proposed shoring system. A survey of existing utilities, embankments, and structures adjacent to those portions of the proposed excavation that will be shored should also be performed prior to construction. The purpose of the analysis and survey would be to evaluate the ability of existing structures, embankments, pipelines, or conduits to withstand anticipated horizontal and vertical movements associated with a shored excavation. If movements exceed the tolerance of existing project features (utilities, embankments, structures, etc.), alternative shoring systems employing the at-rest earth pressure, tie-backs, dead-man anchors, or cross bracing may be needed to reduce deflections to acceptable levels. The Contractor should anticipate repairing cracks in any improvements adjacent to shored portions of the excavation due to anticipated lateral displacements of the shoring system. Horizontal and vertical movements of the shoring system should be monitored by a surveyor and the results reviewed by the project Geotechnical Engineer.

10.1.9 Bottom Stability

Trench excavations that encounter wet soils or that are close to or below the water table may be unstable. In general, unstable bottom conditions may be mitigated by using a stabilizing geogrid (such as a Tensar Biaxial Grid Type 2, Triaxial Grid TX140 or an approved equivalent), overexcavating the excavation bottom to suitable depths and replacing with gravel wrapped in filter fabric, or other suitable method. Recommendations for stabilizing the excavation bottoms should be based on an evaluation on a case by case basis in the field by Ninyo & Moore at the time of construction.

The project site is mapped within FEMA Zone X. The site may encounter surface waters during periods of heavy precipitation. If the excavations are open during a heavy rain event, the trench material(s) might become saturated and unstable and a dewatering system may be needed for these conditions. Should this occur, further remedial measures may be needed.

Excavations that do encounter surface run-off (if any) could be dewatered by pumping the water out from the bottom and away from the excavation. However, heavily saturated units or perched water or groundwater, if encountered, may call for more aggressive means of dewatering.

10.1.10 Construction Dewatering

As noted in the previous sections, groundwater was encountered as shallow as 13 feet below ground surface at the site. In addition, significant fluctuations in the groundwater levels may also occur as noted in previous sections. Dewatering measures, if needed, during excavation operations should be planned by the contractor and reviewed by the design engineer. Considerations for construction dewatering should include geotechnical characteristics, fluctuations in groundwater depth, anticipated drawdown, volume of pumping, potential for settlement, and groundwater discharge. Disposal of groundwater should be permitted in accordance with guidelines of the Regional Water Quality Control Board (RWQCB).

10.1.11 Materials for Fill and Re-use of On-site Soils

On-site and imported granular soils that exhibit relatively low plasticity indices and a very low to low expansion potential are generally suitable for re-use as engineered fill. Relatively low plasticity indices, as evaluated by ASTM D 318, are defined as a Plasticity Index (PI) of 15 or less for this project. The Atterberg limits tests performed on soil samples obtained from our borings resulted in PI values of ranging from non-plastic (NP) to 13. The results of our expansion index test performed on a soil sample obtained from our borings resulted in very low expansion potential. As such, it is our opinion that some of the on-site soils are considered suitable for re-use as engineered fill for this project. We suggest additional field sampling and laboratory testing be conducted by the contractor either prior to or during construction to better evaluate the limits of suitable and unsuitable materials.

In addition, suitable engineered fill should not include construction debris, organic material, or other non-soil fill materials. Fill material should not contain rocks or lumps over approximately 3 inches in diameter, and not more than approximately 30 percent larger than ¾ inch. Large chunks, if generated during excavation, may be broken into acceptably sized pieces or disposed of offsite. Obstructions that extend below finish grade should be removed and the resulting holes filled with compacted soil. Unsuitable fill material should be disposed of offsite.

Engineered and imported fill material should also be non-corrosive in accordance with the Caltrans (2018a) corrosion guidelines, which is defined as a soil with an electrical resistivity value greater than 1,100 ohm-centimeters (ohm-cm), a chloride content of less than 500 parts per million (ppm), a soluble sulfate content of less than 1,500 ppm, and a pH greater than 5.5. In lieu of this, corrosion protection techniques (e.g., cathodic protection, pipe wrapping, etc.) can be implemented. A corrosion specialist should be consulted for recommendations of an appropriate corrosion protection technique.

The contractor should be responsible for the uniformity of import material brought to the site. We recommend that materials proposed for use as import fill be evaluated from a contractor's stockpile rather than in-place materials. Materials for use as fill should be evaluated by the project geotechnical consultant's representative prior to filling or importing. Do not import soils that exhibit a known risk to human health, the environment, or both.

10.1.12 Fill Placement and Compaction

Prior to placement of compacted fill, the contractor should request an evaluation of the exposed ground surface by the project geotechnical consultant. Unless otherwise recommended, the exposed ground surface should then be scarified to a depth of approximately 8 inches and watered or dried, as needed, to achieve moisture contents generally above the optimum moisture content. The scarified materials should then be compacted to a relative compaction of 90 percent as evaluated in accordance with ASTM D 1557. The evaluation of compaction by the geotechnical consultant should not be considered to preclude any requirements for observation or approval by governing agencies. It is the contractor's responsibility to notify the geotechnical consultant and the appropriate governing agency when project areas are ready for observation, and to provide reasonable time for that review.

Fill soils should be moisture conditioned to generally above the laboratory optimum moisture content prior to placement. The optimum moisture content will vary with material type and other factors. Moisture conditioning of fill soils should be generally consistent within the soil mass.

Prior to placement of additional compacted fill material following a delay in the grading operations, the exposed surface of previously compacted fill should be prepared to receive fill. Preparation may include scarification, moisture conditioning, and recompaction.

Compacted fill should be placed in horizontal lifts of approximately 8 inches in loose thickness. Prior to compaction, each lift should be watered or dried as needed to achieve a moisture content generally above the laboratory optimum, mixed, and then compacted by mechanical methods, to a relative compaction of 90 percent as evaluated by ASTM D 1557. Successive lifts should be treated in a like manner until the desired finished grades are achieved. The upper 12 inches of the subgrade materials underneath vehicular pavements should be placed to a relative compaction of 95 percent as evaluated by ASTM D 1557. Additionally, aggregate base materials underneath vehicular pavements should be compacted to a relative compaction of 95 percent relative density as evaluated by the current version of ASTM D 1557.

10.1.13 Pipe Bedding

We recommend that new pipelines, where constructed in an open excavation, be supported on 6 or more inches of granular bedding material. Granular pipe bedding should be provided to distribute vertical loads around the pipe. Bedding material and compaction requirements should be in accordance with this report. Pipe bedding typically consists of graded aggregate with a coefficient of uniformity of three or greater.

10.1.14 Modulus of Soil Reaction (E')

The modulus of soil reaction (E') is used to characterize the stiffness of soil backfill placed at the sides of buried flexible pipes for the purpose of evaluating deflection caused by the weight of the backfill over the pipe (Hartley and Duncan, 1987). A soil reaction modulus of 1,000 pounds per square inch (psi) may be used for an excavation depth of up to approximately 5 feet when backfilled with granular soil compacted to a relative compaction of 90 percent as evaluated by the ASTM D 1557. A soil reaction modulus of 1,400 psi may be used for trenches deeper than 5 feet.

The E' for native materials will vary with material type and stiffness of the trench sidewalls. Approximate values of E' for the materials generally encountered in our borings are presented in Table 3 below:

Table 3 – Modulus of Soil Reaction (E') for Onsite Soils

Trench Wall Soil Classification (USCS)	Approximate E' (psi)		
	Loose/Firm	Medium Dense/Stiff	Dense/Very Stiff/Hard
Silty Sand (SM)	400	700	2000

10.1.15 Utility Pipe Zone Backfill

The pipe zone backfill extends from the top of the pipe bedding material and continues to extend to 1 foot or more above the top of the pipe in accordance with the recent edition of the Standard Specifications for the Public Works Construction ("Greenbook"). Pipe zone backfill should have a Sand Equivalent (SE) of 30 or greater, and be placed around the sides and top of the pipe. Special care should be taken not to allow voids beneath and around the pipe. Compaction of the pipe zone backfill should proceed up both sides of the pipe.

It has been our experience that the voids within a crushed rock 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. If open-graded gravel is utilized as pipe zone backfill, this material should be separated from the adjacent trench sidewalls and overlying trench backfill with a geosynthetic filter fabric.

10.1.16 Utility Trench Zone Backfill

Utility trench zone backfill material should be generally free of trash, debris, roots, vegetation, or deleterious materials. Trench zone backfill should generally be free of rocks or hard lumps of material in excess of 3 inches in diameter. Rocks or hard lumps larger than about 3 inches in diameter should be broken into smaller pieces or should be removed from the site. Oversize materials should be separated from material to be used as trench backfill. Moisture conditioning (including drying and/or mixing) of existing on-site materials is anticipated if reused as trench backfill.

10.1.17 Thrust Blocks

Thrust restraint for buried pipelines may be achieved by transferring the thrust force to the soil outside the pipe through a thrust block. Thrust blocks may be designed using the magnitude and distribution of passive lateral earth pressures presented on Figure 8. Thrust blocks should be backfilled with granular backfill material and compacted following the recommendations presented in this report.

10.1.18 Controlled Low Strength Material (CLSM)

It is our opinion that the backfill zone may be filled with CLSM as an alternative to the material described in this report. CLSM consists of a fluid, workable mixture of aggregate, Portland cement, and water. The use of CLSM has some advantages:

- A narrower backfill zone can be used, thereby reducing the quantity of soil to be excavated and possibly reducing disturbance to the near-by traffic.
- Relatively higher E' values may be used (E'= 3,000 psi).
- The support given to the connecting pipes is generally better.
- Because little compaction is needed to place CLSM, there is less risk of damaging the connecting pipes.
- CLSM can be batched to flow into irregularities in the trench bottom and walls.

The CLSM design mix should be in accordance with current Standard Specifications for the Public Works Construction ("Greenbook"). Additional mix design information can be provided upon request. The 28-day strength of the material should be no less than 50 psi and no more than 120 psi.

Buoyant or uplift forces on the piping should be considered when using CLSM and prudent construction techniques may result in multiple pours to avoid inducing excessive uplift forces. Sufficient time should be provided to allow the CLSM to cure before placing additional lifts of CLSM or trench backfill.

10.1.19 Constructed Slopes

We recommend that constructed cut slopes, if any are planned for this project, and constructed embankment fill slopes be no steeper than 3:1 (horizontal to vertical). New embankment fills should be benched into existing embankments, where appropriate. Benches should be level and wide enough to allow operation of and compaction by, construction equipment. Fill slopes should be constructed in a manner (e.g., overfilling and cutting to grade) such that the recommended degree of compaction is achieved to the finished slope face. Cut and fill slopes should be protected from erosion. This should promote re-vegetation and a stable slope. Periodic maintenance of exposed slopes should be anticipated.

Unprotected slopes may rill and erode if exposed to running water. Silty soils and soils containing fine sand are more susceptible in this regard. Laying slopes back to 3:1 (horizontal to vertical) will decrease runoff velocity and decrease the likelihood of serious erosion. Adequate drainage and temporary erosion protection covering could minimize erosion problems and promote post-construction vegetation. Plating the slopes with gravelly material will reduce precipitation impact and slow the rate of erosion. Along longer slopes, brow ditches should be considered to reduce the amount of surface flow on the slope face. Where feasible, the existing vegetation should be salvaged and replaced.

10.2 Foundations

Based on the results of our field and laboratory evaluations and our understanding that the proposed warehouse building will not be designed to mitigate liquefaction induced dynamic settlement, it is our opinion that the proposed structures can be supported on shallow spread footings or mat foundations. Foundations should be designed in accordance with structural considerations and the following recommendations. In addition, requirements of the governing jurisdictions and applicable building codes should be considered in the design of the proposed structures.

10.2.1 Spread Footings

Spread footings should bear at a depth of 36 inches or more below the adjacent finished grade, on 3 feet or more of moisture-conditioned and compacted engineered fill as described in this report. Footings should have a width of 24 inches or more, and isolated spread footings should have a width of 30 inches or more. Spread footings should be reinforced in accordance with the recommendations of the structural engineer.

Spread and pad footings founded on engineered fill may be designed using a net allowable bearing capacity of 3,000 pounds psf. The allowable bearing capacity may be increased by one-third when considering loads of short duration such as wind or seismic forces. Total and differential settlement of up to about 1 inch and ½ inch over a horizontal distance of 40 feet, respectively, may occur.

Foundations bearing on moisture-conditioned, compacted engineered fill and subject to lateral loadings may be designed using an ultimate coefficient of friction of 0.35 (total frictional resistance equivalent to the coefficient of friction multiplied by the dead load). A passive resistance value of 350 psf per foot of depth can be used with a value of up to 3,500 psf. This value assumes that the ground is horizontal for a distance of 10 feet, or three times the height generating the passive pressure, whichever is greater. We recommend that the upper 1 foot of soil not protected by pavement or a concrete slab be neglected when calculating passive resistance. The lateral resistance can be taken as the sum of the frictional resistance and passive resistance, provided that the passive resistance does not exceed one-half of the total allowable resistance. The passive resistance may be increased by one-third when considering loads of short duration such as wind or seismic forces.

10.2.2 Mat Foundations

Mat foundations may be used as an alternative to spread footings for the proposed warehouse. We recommend that mat foundations be supported on 3 feet or more of moisture-conditioned and compacted engineered fill, as described in this report. A net allowable equivalent soil bearing pressure of 1,500 psf is recommended for mat foundations bearing on engineered fill. We recommend that a modulus of subgrade reaction, K_v, of 250 kips per cubic foot, be used for design. This value is based on a unit square foot area and should be adjusted for the planned mat size. Adjusted values of the modulus of subgrade reaction, K_b, can be obtained from the following equation for mats of various widths:

$$\mathbf{K_b = K_v\ [(B+1)/2B]^2}$$

The B in the above equation represents the width (i.e., the lesser dimension of the width and length) of the mat in feet.

Total settlements of the mat-supported area are estimated to be on the order of 1 inch. Differential settlements will depend upon the structural rigidity of the mat. We recommend that these settlements be considered during the design.

For resistance of mat foundation to lateral loads, we recommend a passive pressure of 350 psf per foot of depth be used with a value of up to 3,500 psf. This value assumes that the ground is horizontal for a distance of 10 feet, or three times the height generating the passive pressure, whichever is greater. We recommend that the upper 1 foot of soil not protected by pavement or a concrete slab be neglected when calculating passive resistance. The passive resistance values may be increased by one-third when considering loads of short duration such as wind or seismic forces.

For frictional resistance to lateral loads on mat, we recommend a coefficient of friction of 0.35 at the concrete-soil interface. The 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 resistance.

10.3 Floor Slabs

The design of the floor slabs is the responsibility of the structural engineer; however, we recommend that they be reinforced with steel re-bars. Placement of the steel reinforcement in the slab is vital for satisfactory performance. If moisture sensitive floor coverings are to be used, we recommend that slabs be underlain by a vapor retarder and capillary break system consisting of a 10-mil polyethylene (or equivalent) membrane placed over 4 inches of medium to coarse, clean sand or pea gravel. The steel reinforcements for the floor slabs shall be placed on the vapor retarder using chairs, as appropriate.

10.4 Concrete Flatwork

Exterior concrete flatwork should be 4 inches in thickness and should be reinforced with No. 3 reinforcing bars placed at 24 inches on-center both ways. A vapor retarder is not needed for exterior flatwork. To reduce the potential manifestation of distress to exterior concrete flatwork due to movement of the underlying soil, we recommend that such flatwork be installed with crack-control joints at appropriate spacing as designed by the design engineer. Before placement of concrete, remedial grading should be performed in accordance with the recommendations in this report. Positive drainage should be established and maintained adjacent to flatwork.

10.5 Preliminary Flexible Pavement Design

As part of the new construction, we anticipate that new pavements will be constructed. Our laboratory testing of a near surface soil sample at the project site indicated an R-value of 28. A preliminary design R-value of 20, along with assumed design Traffic Indices (TI) of 5, 6, 7, and 9.5 has been the basis of our preliminary flexible pavement design. The assumed TIs should be evaluated by the Civil Engineer based on anticipated traffic loading at the site. Actual pavement recommendations should be based on R-value tests performed on bulk samples of the soils that are exposed at the finished subgrade elevations across the site at the completion of the grading operations. The preliminary recommended flexible pavement sections are presented in Table 4.

Table 4 – Recommended Preliminary Flexible Pavement Sections			
Traffic Index (Pavement Usage)	**Design R-Value**	**Asphalt Concrete Thickness (inches)**	**Crushed Aggregate Base Thickness (inches)**
5 (Parking Areas)	20	3.0	7.5
6 (Light Traffic)	20	3.0	10.5
7 (Medium Traffic)	20	4.5	11.0
9.5 (Heavy Traffic/ Fire Lanes)	20	6.5	16.0

As indicated, these values assume TIs of 9.5 or less for site pavements. If traffic loads are different from those assumed herein, the pavement design should be re-evaluated. In addition, we recommend that the upper 12 inches of the subgrade and aggregate base materials be compacted to a relative compaction of 95 percent relative density as evaluated by the current version of ASTM D 1557.

10.6 Preliminary Rigid Pavement Design

Rigid PCC pavements are recommended for areas that will experience regular truck traffic, main ingress and egress areas, and in areas where vehicles will be turning or loading (e.g., adjacent to trash dumpsters). For rigid pavements Ninyo & Moore recommends sections composed of 9 inches of 600 psi flexural strength Portland cement concrete reinforced with No. 3 bars, 18-inches on center, be placed over 12 inches or more of aggregate base materials compacted to a relative compaction of 95 percent as evaluated by ASTM D 1557 at a moisture content at or near optimum. Additionally, the upper 12 inches of the subgrade beneath the aggregate base should be compacted to 95 percent of its Proctor density as evaluated by ASTM D 1557. The above section may also be used for fire lane PCC pavements.

For light and moderately trafficked vehicle pavements, we recommend 6 and 7½ inches of 600 psi flexural strength PCC, respectively, over 6 inches or more of aggregate base. Additionally, the upper 12 inches of the subgrade beneath the aggregate base should be compacted to 95 percent of its Proctor density as evaluated by ASTM D 1557. We also recommend that a qualified structural engineer be consulted for appropriate reinforcement of concrete pavement.

10.7 Corrosion

The corrosion potential of the on-site soils was tested to evaluate its potential effect on the foundations and structures. Our corrosion evaluation is based on the results of our field and laboratory testing done for this project. A corrosion specialist should perform their own analysis.

Laboratory testing consisted of pH, minimum electrical resistivity, and chloride and soluble sulfate contents. The pH and minimum electrical resistivity tests were performed in general accordance with California Test Method (CT) 643, while sulfate and chloride tests were performed in accordance with CT 417 and 422, respectively. The results of these corrosivity tests are presented in Appendix C.

The soil pH values of tested samples were measured at approximately 6.8 and 6.9, the electrical resistivities were measured at approximately 1,300 and 1,500 ohm-centimeters, the chloride contents were measured at approximately 70 and 95 parts per million (ppm), and the sulfate contents were measured at approximately 0.001 percent (i.e., 10 ppm). Based on the Caltrans (2018) corrosion criteria, the project site would not be classified as a corrosive site. Caltrans (2018) defines a corrosive site as having earth materials with chloride concentration of 500 ppm or greater, sulfates concentration of 0.15 percent or greater (i.e., 1,500 ppm or greater), a pH of 5.5 or less, and/or an electrical resistivity of 1,100 ohm-centimeters or less.

10.8 Concrete

Laboratory chemical tests performed on on-site soil samples indicated a soluble sulfate content of 0.001 percent (i.e., 10 ppm) by dry weight of soil. Based on American Concrete Institute criteria, the on-site soils should be considered to present a negligible sulfate exposure to concrete.

Notwithstanding the sulfate test results and due to the limited number of chemical tests performed, as well as our experience with similar soil conditions and regional practice, we recommend that “Type II/V” cement be used for the construction of concrete structures at this site. Due to potential uncertainties as to the use of reclaimed irrigation water, or topsoil that may contain higher sulfate contents, pozzolan or admixtures designed to increase sulfate resistance may be considered.

We recommend that the structural concrete have a water-cementitious materials ratio no more than 0.45 by weight for normal weight aggregate concrete. The structural engineer should ultimately select the concrete design strength based on the project specific loading conditions. Higher strength concrete may be selected for increased durability and resistance to slab curling and shrinkage cracking.

10.9 Site Drainage

Roof, pad, and slope drainage should be conveyed such that runoff water is diverted away from slopes and structures to suitable discharge areas by nonerodible devices (e.g., gutters, downspouts, concrete swales, etc.). Positive drainage adjacent to structures should be established and maintained. Positive drainage may be accomplished by providing drainage away from the foundations of the structure at a gradient of 5 percent or steeper for a distance of 10 feet or more outside the building perimeter, or 2 percent or steeper for a distance of 10 feet or more outside the building perimeter if paved. Drainage should be further maintained by a graded swale leading to an appropriate outlet, in accordance with the recommendations of the project civil engineer and/or landscape architect.

Surface drainage on the site should be provided so that water is not permitted to pond. A gradient of 2 percent or steeper should be maintained over the pad area and drainage patterns should be established to divert and remove water from the site to appropriate outlets.

Care should be taken by the contractor during final grading to preserve any berms, drainage terraces, interceptor swales or other drainage devices of a permanent nature on or adjacent to the property. Drainage patterns established at the time of final grading should be maintained for the life of the project. The property owner and the maintenance personnel should be made aware that altering drainage patterns might be detrimental to slope stability and foundation performance.

10.10 Pre-Construction Conference

We recommend that a pre-construction conference be held. Representatives of the owner, civil engineer, the geotechnical consultant, and the contractor should be in attendance to discuss the project plans and schedule. Our office should be notified if the project description included herein is incorrect, or if the project characteristics are significantly changed.

10.11 Plan Review and Construction Observation and Testing

The conclusions and recommendations presented in this report are based on analysis of observed conditions in widely spaced exploratory borings. If conditions are found to vary from those described in this report, Ninyo & Moore should be notified, and additional recommendations will be provided upon request. Ninyo & Moore should review the final project drawings and specifications prior to the commencement of construction. Ninyo & Moore should perform the needed observation and testing services during construction operations.

The recommendations provided in this report are based on the assumption that Ninyo & Moore will provide geotechnical observation and testing services during construction. In the event that it is decided not to utilize the services of Ninyo & Moore during construction, we request that the selected consultant provide the client with a letter (with a copy to Ninyo & Moore) indicating that they fully understand Ninyo & Moore's recommendations, and that they are in full agreement with the design parameters and recommendations contained in this report. Construction of proposed improvements should be performed by qualified subcontractors utilizing appropriate techniques and construction materials.

11 LIMITATIONS

The field evaluation and laboratory testing presented in this geotechnical report have been conducted in general accordance with current practice and the standard of care exercised by geotechnical consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the conclusions, recommendations, and opinions 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 does not provide sufficient data to prepare an accurate bid by contractors. 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.

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.

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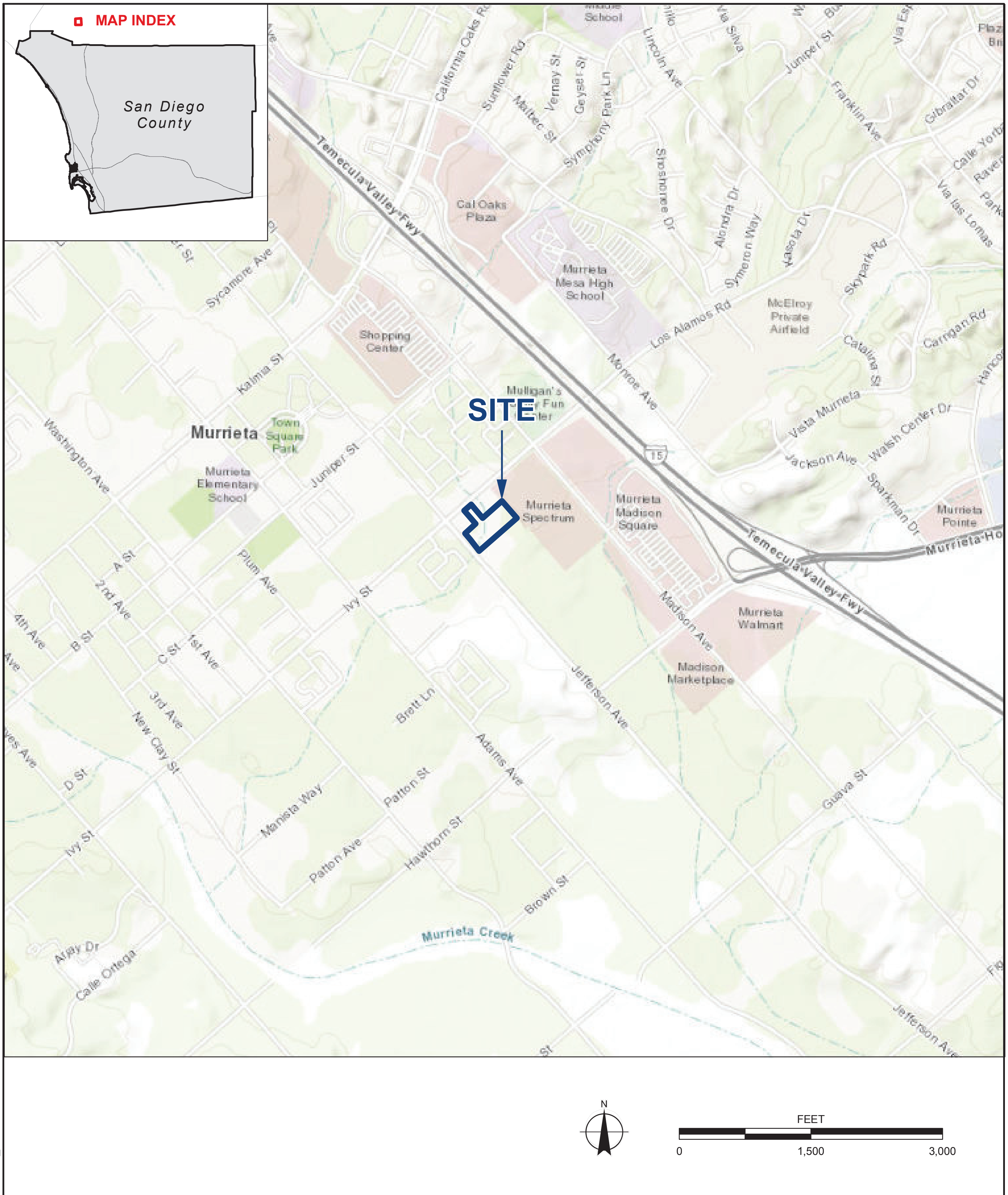
FIGURES



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NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCE: ESRI WORLD TOPO, 2017

FIGURE 1

SITE LOCATION

MURRIETA U-HAUL FACILITY
41458 LOS ALAMOS ROAD AND 25086 JEFFERSON AVENUE
MURRIETA, CALIFORNIA
108673001 | 3/19

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LEGEND

SITE BOUNDARY

B-3 TD=101.5 BORING TD=TOTAL DEPTH IN FEET

CPT-3 TD=45.7 CONE PENETRATION TEST TD=TOTAL DEPTH IN FEET

N

FEET

0 100 200

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCE: GOOGLE EARTH, 2017

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FIGURE 2

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BORING LOCATIONS

MURRIETA U-HAUL FACILITY

41458 LOS ALAMOS ROAD AND 25086 JEFFERSON AVENUE

MURRIETA, CALIFORNIA

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SOURCE: U.S. GEOLOGICAL SURVEY AND CALIFORNIA GEOLOGICAL SURVEY, 2006, QUATERNARY FAULT AND FOLD DATABASE FOR THE UNITED STATES.

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

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FIGURE 3

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FAULT LOCATIONS

MURRIETA U-HAUL FACILITY
41458 LOS ALAMOS ROAD AND 25086 JEFFERSON AVENUE
MURRIETA, CALIFORNIA
108673001 | 3/19

SOURCE: 1990, EARTHQUAKE ZONE OF REQUIRED INVESTIGATION, MURRIETA QUADRANGLE, CALIFORNIA GEOLOGICAL SURVEY.

LEGEND

Earthquake Fault Zones
Zone boundaries are delineated by straight-line segments; the boundaries define the zone encompassing active faults that constitute a potential hazard to structures from surface faulting or fault creep such that avoidance as described in Public Resources Code Section 2621.5(a) would be required.

N

FEET
0 2,000 4,000

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

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FIGURE 4

AP EARTHQUAKE FAULT ZONE
MURRIETA U-HAUL FACILITY
41458 LOS ALAMOS ROAD AND 25086 JEFFERSON AVENUE
MURRIETA, CALIFORNIA
108673001 | 3/19

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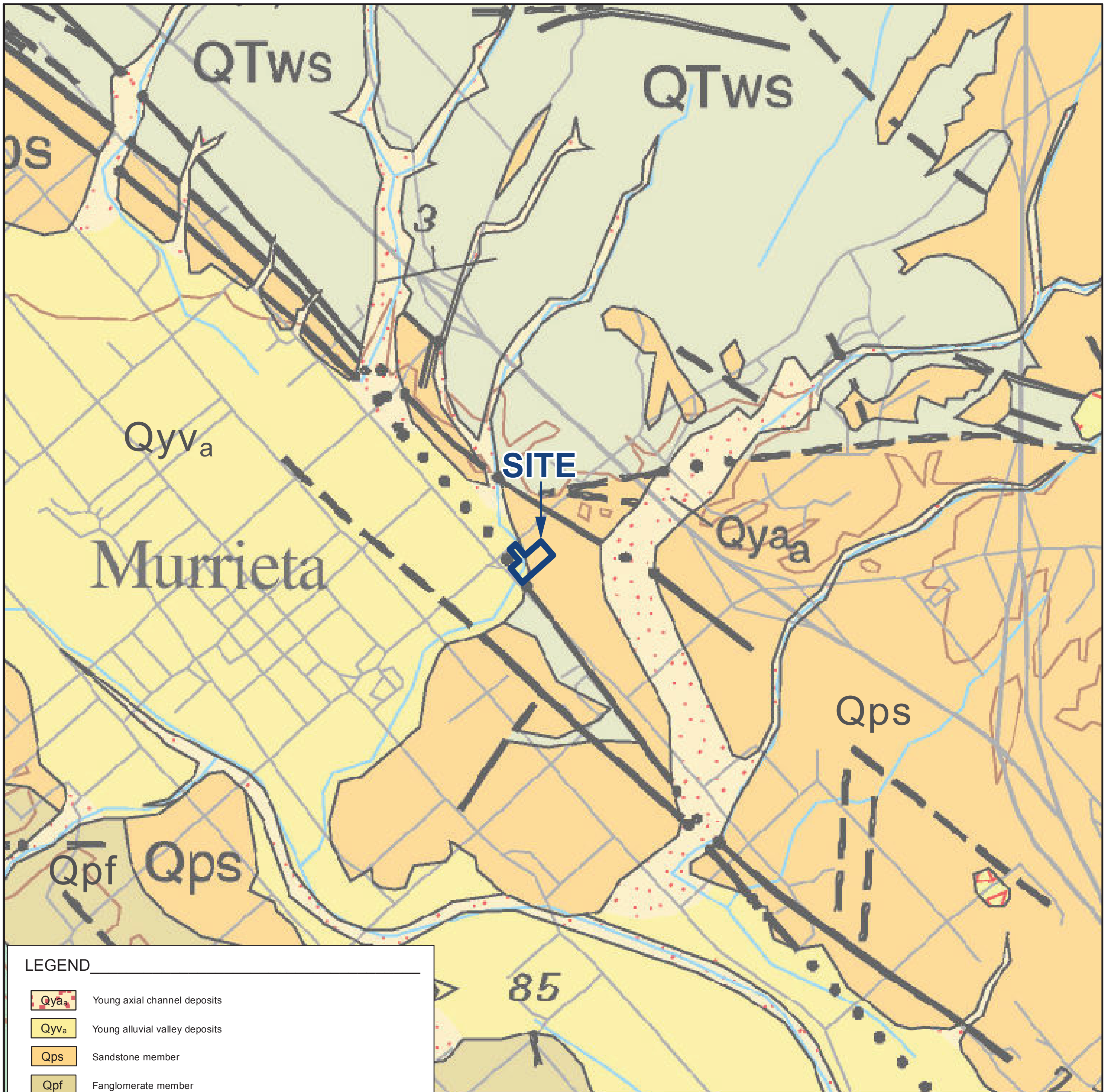

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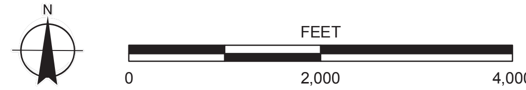

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NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

FIGURE 5

GEOLOGY

MURRIETA U-HAUL FACILITY
41458 LOS ALAMOS ROAD AND 25086 JEFFERSON AVENUE
MURRIETA, CALIFORNIA

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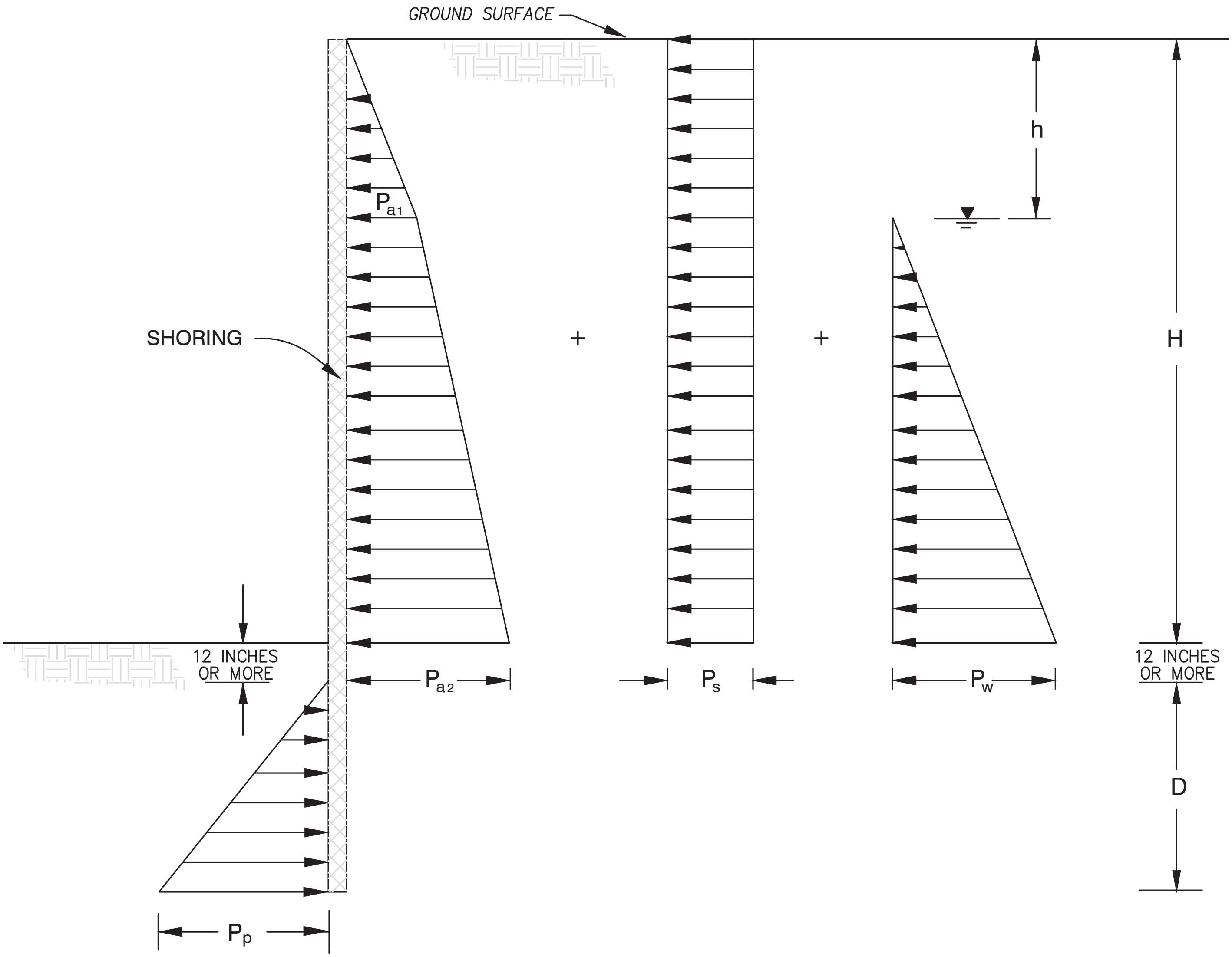

NOTES:

1. ACTIVE LATERAL EARTH PRESSURE, P_a
   P_{a1} = 40 H psf; $P_{a2} = P_{a1}$ + 20 (H - h) psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, P_s
   P_s = 120 psf
3. HYDROSTATIC PRESSURE, P_w
   P_w = 62.4 (H - h) psf
4. PASSIVE LATERAL EARTH PRESSURE, P_p
   P_p = 300 D psf
5. SURCHARGES FROM EXCAVATED SOIL OR CONSTRUCTION MATERIALS ARE NOT INCLUDED
6. H, h AND D ARE IN FEET
7. GROUNDWATER TABLE

NOT TO SCALE

6 108673001 D-CS.DWG

FIGURE 6

LATERAL EARTH PRESSURES FOR TEMPORARY CANTILEVERED SHORING BELOW GROUNDWATER

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41458 LOS ALAMOS ROAD AND 25086 JEFFERSON AVENUE
MURRIETA, CALIFORNIA

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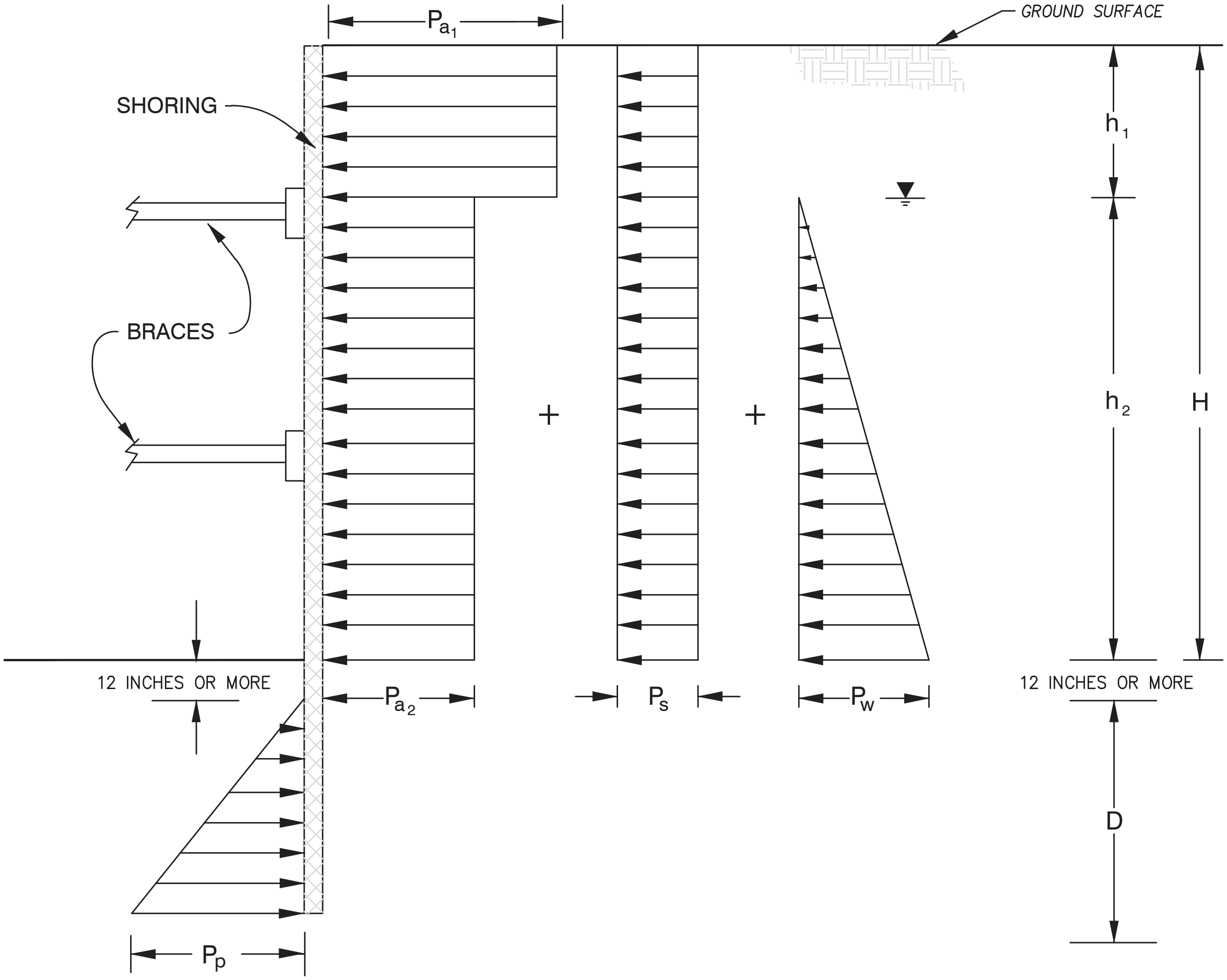

NOTES:

1. APPARENT LATERAL EARTH PRESSURES, $P_{a\,1}$ AND $P_{a\,2}$
   $P_{a\,1} = 26\ H$ psf
   $P_{a_2} = 13\ H$ psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, P_s
   $P_s = 120$ psf
3. WATER PRESSURE, P_w
   $P_w = 62.4\ h_2$ psf
4. PASSIVE PRESSURE, P_p
   $P_p = 300\ D$ psf
5. SURCHARGES FROM EXCAVATED SOIL OR CONSTRUCTION MATERIALS ARE NOT INCLUDED
6. H, h_1, h_2 AND D ARE IN FEET
7. ▼ GROUNDWATER TABLE

NOT TO SCALE

7 108673001 D-BEBG.DWG

FIGURE 7

LATERAL EARTH PRESSURES FOR BRACED EXCAVATION BELOW GROUNDWATER

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MURRIETA, CALIFORNIA

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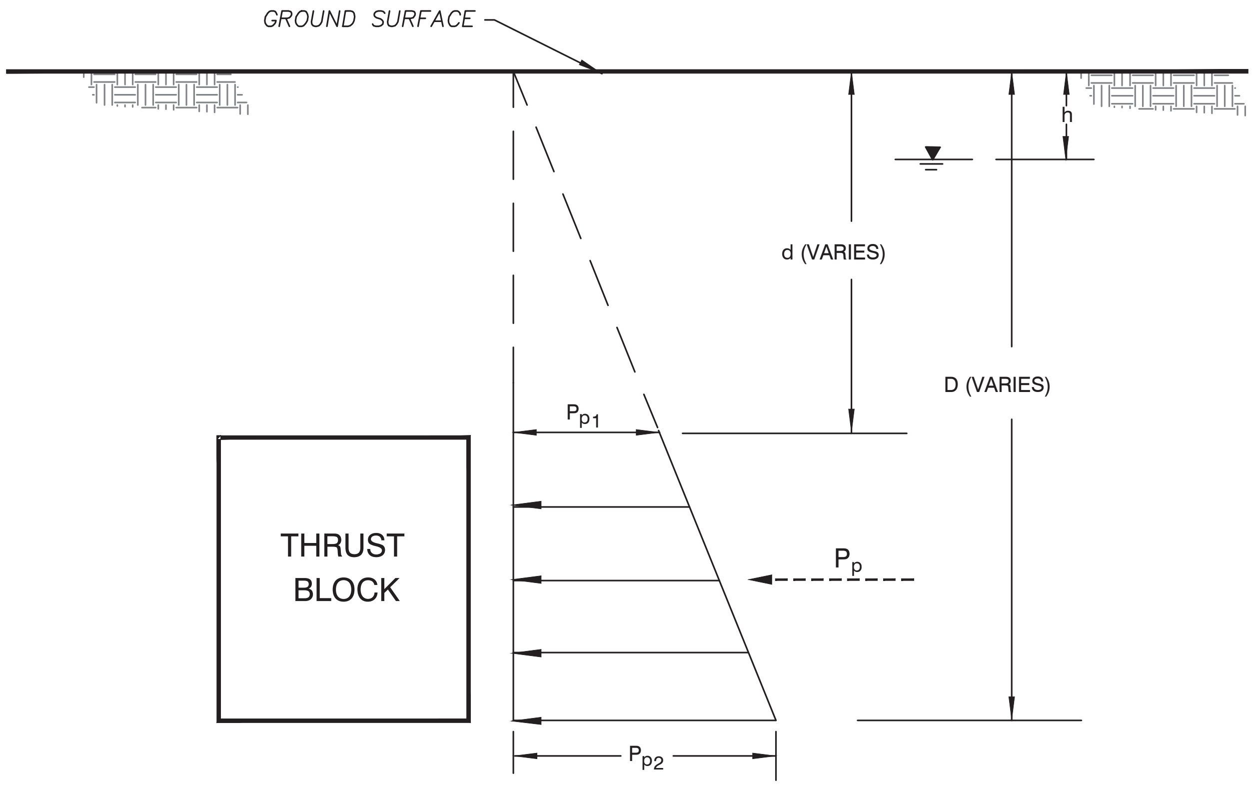

NOTES:

1. GROUNDWATER BELOW BLOCK

   $P_p = 175\ (D^2 - d^2)$ lb/ft

2. GROUNDWATER ABOVE BLOCK

   $P_p = 1.5\ (D - d)[\ 124.8h + 58\ (D + d)]$ lb/ft

3. ASSUMES BACKFILL IS GRANULAR MATERIAL
4. ASSUMES THRUST BLOCK IS ADJACENT TO COMPETENT MATERIAL
5. D, d AND h ARE IN FEET
6. GROUNDWATER TABLE

NOT TO SCALE

8 108673001 D-TB.DWG

FIGURE 8

THRUST BLOCK LATERAL EARTH PRESSURE DIAGRAM

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MURRIETA U-HAUL FACILITY
41458 LOS ALAMOS ROAD AND 25086 JEFFERSON AVENUE
MURRIETA, CALIFORNIA

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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 consisting of auger cuttings of representative earth materials were obtained from selected 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 12 to 18 inches with a 140-pound hammer free-falling 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 (California) 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 sample barrel was driven into the ground with a 140-pound hammer free-falling from a height of 30 inches in general accordance with ASTM D 1586. The samples were removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing.

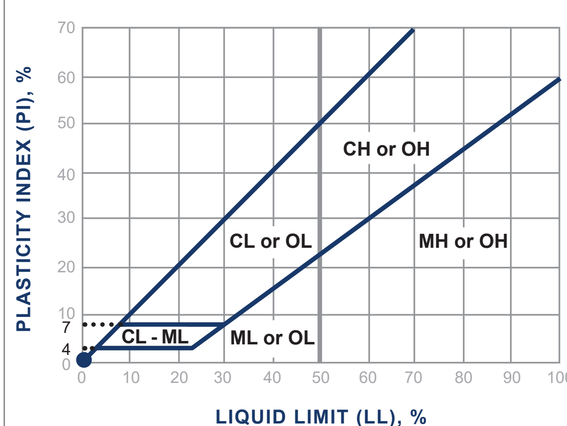
Soil Classification Chart Per ASTM D 2488

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
		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
								The total depth line is a solid line that is drawn at the bottom of the boring.
20								

DATE DRILLED	10/16/18
BORING NO.	B-1
GROUND ELEVATION	1,107' ± (MSL)
SHEET	1 OF 2
METHOD OF DRILLING	8" Diameter Hollow Stem Auger (Baja) (CME-95)
DRIVE WEIGHT	140 lbs. (Auto-Trip)
DROP	30"
SAMPLED BY	CMK
LOGGED BY	CMK
REVIEWED BY	CAT

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	FILL: Yellow and light brown, dry to moist, medium dense, silty SAND; decomposed granite.
								@ 3': Dark brown.
			15	11.6	115.0			@ 6': Reddish brown.
			52	15.0	116.8			PAUBA FORMATION: Reddish brown, moist, moderately indurated, sandy SILTSTONE.
10			30	14.0	109.8			
			33	14.6	111.9			
			28	10.5	120.2			Reddish brown, moist, moderately cemented, fine- to medium-grained SANDSTONE.
			39	15.4	113.6			
20			45	▽ 14.9	113.8			Wet.
			22					Coarse-grained.
30			32					Reddish brown, wet, moderately indurated, CLAYSTONE.
			51					Brown, wet, moderately cemented, fine- to coarse-grained SANDSTONE.
40								

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DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.

DATE DRILLED 10/16/18 BORING NO. B-1

GROUND ELEVATION 1,107' ± (MSL) SHEET 2 OF 2

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (Baja) (CME-95)

DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30"

SAMPLED BY CMK LOGGED BY CMK REVIEWED BY CAT

DESCRIPTION/INTERPRETATION

DEPTH (feet)	BLOWS/FOOT	DESCRIPTION/INTERPRETATION
40	48	PAUBA FORMATION: (Continued) Gray, wet, moderately cemented, fine- to coarse-grained SANDSTONE.
	58	Heaving sands.

Total Depth = 46.5 feet
Groundwater encountered at 20 feet during drilling.
Backfilled with grout shortly after drilling on 10/16/18.

Note: 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.

50

60

70

80

DATE DRILLED 10/16/18 BORING NO. B-2

GROUND ELEVATION 1,106' ± (MSL) SHEET 1 OF 2

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (Baja) (CME-95)

DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30"

SAMPLED BY CMK LOGGED BY CMK REVIEWED BY CAT

DEPTH (feet)	SAMPLES Bulk / Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0	Bulk					SM	FILL: Yellow and light brown, dry, medium dense, silty SAND; decomposed granite.
							PAUBA FORMATION: Dark brown, moist, moderately cemented, silty fine- to coarse-grained SANDSTONE.
	Driven	20	8.8	108.4			
10	Driven	29	10.3	117.8			
	Driven	41	17.8	113.5			
20	Driven	27	21.9	104.2			
			▽				Wet.
	Driven	18					Dark reddish brown; wet.
30	Driven	16					Brown, wet, weakly indurated, fine-grained sandy CLAYSTONE.
			▼				
	Driven	11					
40							

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MURRIETA U-HAUL FACILITY, 41458 LOS ALAMOS ROAD AND
25086 JEFFERSON AVENUE, MURRIETA, CALIFORNIA

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 10/16/18 BORING NO. B-2 GROUND ELEVATION 1,106' ± (MSL) SHEET 2 OF 2 METHOD OF DRILLING 8" Diameter Hollow Stem Auger (Baja) (CME-95) DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30" SAMPLED BY CMK LOGGED BY CMK REVIEWED BY CAT DESCRIPTION/INTERPRETATION
40			28					PAUBA FORMATION: (Continued) Brown, wet, moderately cemented, silty fine- to medium-grained SANDSTONE.
			35					
50			68					
								Total Depth = 51.5 feet Groundwater encountered at 24 feet during drilling and approximately 34.5 feet after drilling. Backfilled with grout shortly after drilling on 10/16/18. Note: 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	10/16/18	BORING NO.	B-3
GROUND ELEVATION	1,105' ± (MSL)	SHEET	1 OF 3
METHOD OF DRILLING	8" Diameter Hollow Stem Auger (Baja) (CME-95)		
DRIVE WEIGHT	140 lbs. (Auto-Trip)	DROP	30"
SAMPLED BY	CMK	LOGGED BY	CMK
REVIEWED BY	CAT		

DEPTH (feet)	SAMPLES Bulk / Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							ASPHALT CONCRETE: Approximately 2 inches thick.
	Bulk					CL	AGGREGATE BASE: Approximately 6 inches thick.
						SM	FILL: Olive gray, moist, firm, sandy lean CLAY; petroleum odor.
			14.7				Olive gray, moist, medium dense, silty SAND.
	Driven	16	15.9	109.8			Brown.
	Bulk/Driven	8					Loose.
10	Driven	18	10.9	115.1			PAUBA FORMATION: Brown, moist, moderately cemented, silty fine- to coarse-grained SANDSTONE.
	Driven	22	16.5	112.1			
	Driven	32					Clay lenses.
	Bulk/Driven	41	▼				Reddish brown, moist, moderately indurated, sandy CLAYSTONE.
20	Driven	28	24.9 ▽	99.0			Wet.
	Driven	46					Brown, wet, moderately cemented, silty coarse-grained SANDTONE.
30	Driven	26					Reddish brown, wet, moderately indurated, sandy CLAYSTONE.
	Driven	15					Light brown.
40							

DATE DRILLED 10/16/18 BORING NO. B-3

GROUND ELEVATION 1,105' ± (MSL) SHEET 2 OF 3

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (Baja) (CME-95)

DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30"

SAMPLED BY CMK LOGGED BY CMK REVIEWED BY CAT

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
40			28					PAUBA FORMATION: (Continued) Brown, wet, moderately cemented, silty fine- to coarse-grained SANDSTONE.
			21					Grayish brown, wet, weakly indurated, fine-grained sandy CLAYSTONE.
50			12					Gray, wet, weakly cemented, fine- to coarse-grained SANDSTONE.
			56					Moderately cemented.
60			39					
			59					Light gray; coarse-grained.
70			50/4"					Gray.
			34					Black, wet, moderately indurated, CLAYSTONE.
80								

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25086 JEFFERSON AVENUE, MURRIETA, CALIFORNIA

DATE DRILLED 10/16/18 BORING NO. B-3

GROUND ELEVATION 1,105' ± (MSL) SHEET 3 OF 3

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (Baja) (CME-95)

DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30"

SAMPLED BY CMK LOGGED BY CMK REVIEWED BY CAT

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
80		▼	51					PAUBA FORMATION: (Continued) Black, wet, moderately indurated, CLAYSTONE. Gray, wet, moderately to strongly cemented, fine- to coarse-grained SANDSTONE.
		▼	66					
90		▼	43					Grayish brown.
		▼	65					Coarse-grained.
100		▼	74					
110								
120								

Total Depth = 101.5 feet
Groundwater encountered at 21 feet during drilling and approximately 18 feet after drilling.
Backfilled with grout shortly after drilling on 10/16/18.

Note: 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.

APPENDIX B

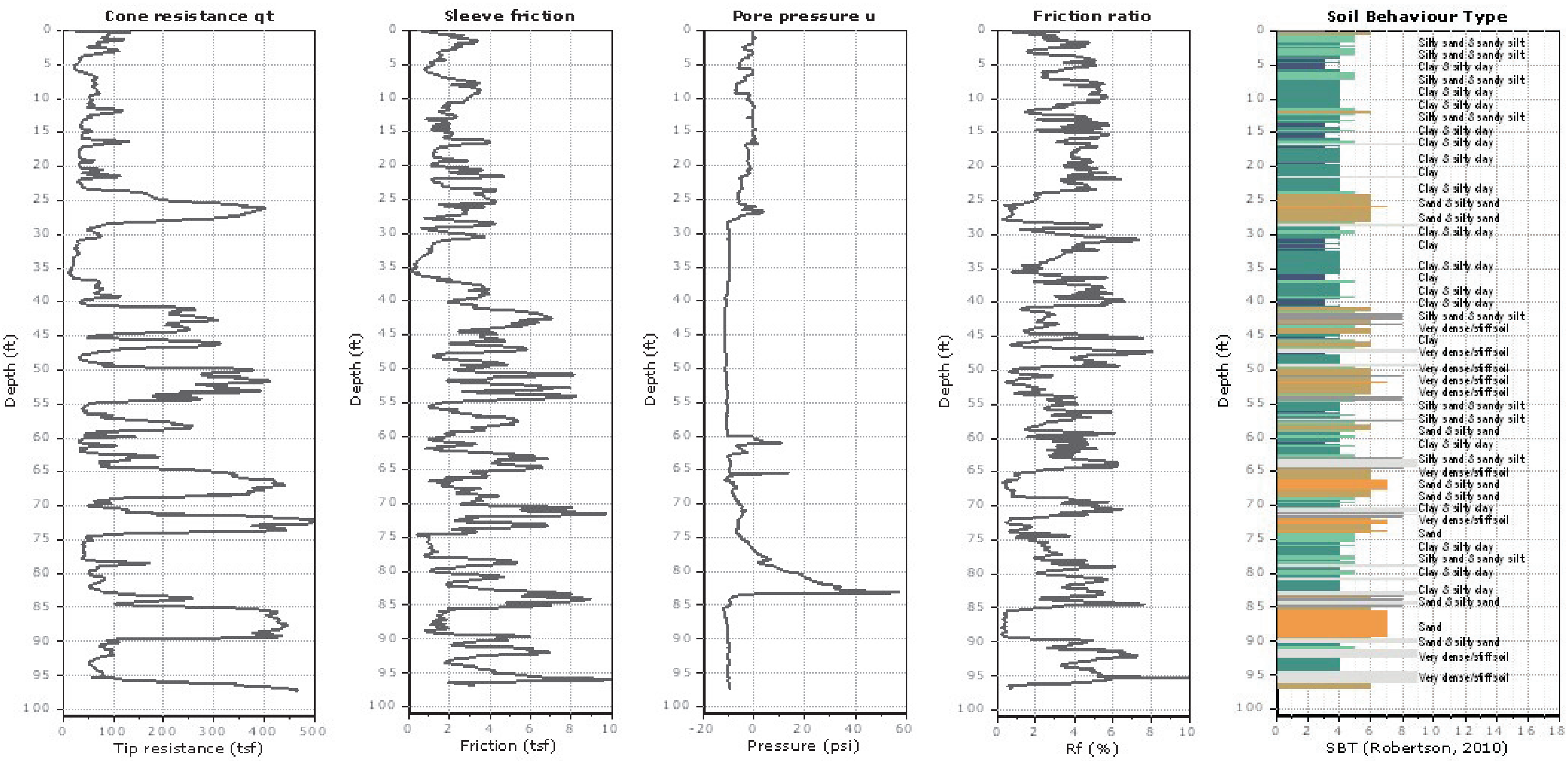
CPT Logs

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

CPT-1

Project: **Ninyo & Moore**
Location: **41458 Los Alamos Rd, Murrieta, CA**

Total depth: 97.38 ft, Date: 10/22/2018
Cone Type: Vertek



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

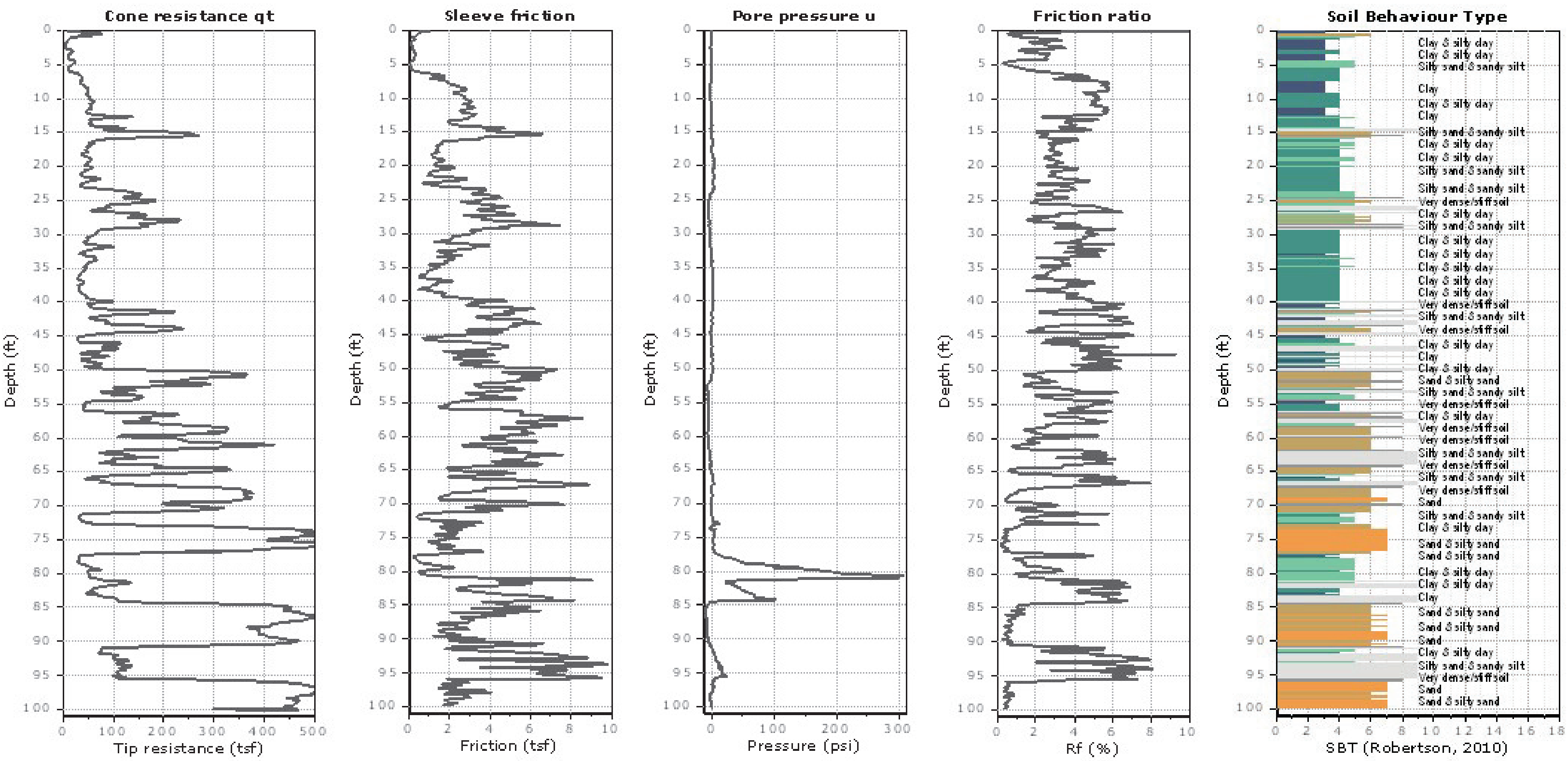
CPT-2

Total depth: 100.27 ft, Date: 10/22/2018

Cone Type: Vertek

Project: Ninyo & Moore

Location: 41458 Los Alamos Rd, Murrieta, CA

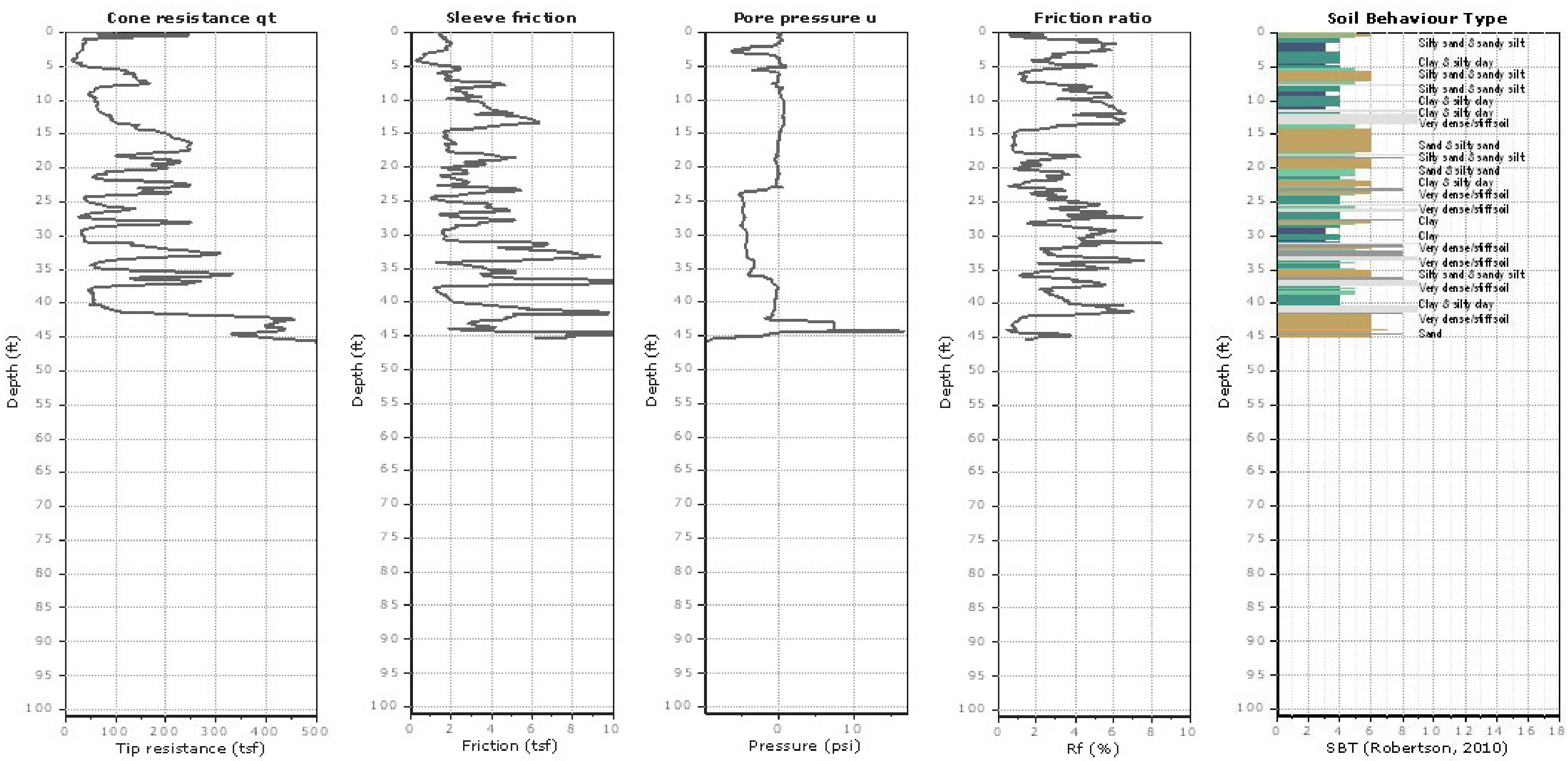


Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

CPT-3

Project: **Ninyo & Moore**
Location: **41458 Los Alamos Rd, Murrieta, CA**

Total depth: 45.74 ft, Date: 10/22/2018
Cone Type: Vertek



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 D2488. 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 excavations were evaluated in general accordance with ASTM D2937. The test results are presented on the logs of the exploratory excavations in Appendix A.

Gradation Analysis
Gradation analysis tests were performed on selected representative soil samples in general accordance with ASTM D422. The grain-size distribution curves are shown on Figures C-1 through C-3. These test results were utilized in evaluating the soil classifications in accordance with the USCS.

Atterberg Limits
Tests were performed on selected representative fine-grained soil samples to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D4318. These test results were utilized to evaluate the soil classification in accordance with the Unified Soil Classification System. The test results and classifications are shown on Figure C-4.

Consolidation Tests
A consolidation test was performed on a selected relatively undisturbed soil sample in general accordance with ASTM D2435-04. 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 tests are summarized on Figure C-5.

Direct Shear Strength Tests
Shear strength tests were performed on relatively undisturbed samples in general accordance with ASTM D3080 to evaluate the shear strength characteristics of selected materials. The samples were inundated during shearing to represent adverse field conditions. The test results are shown on Figures C-6 and C-7.

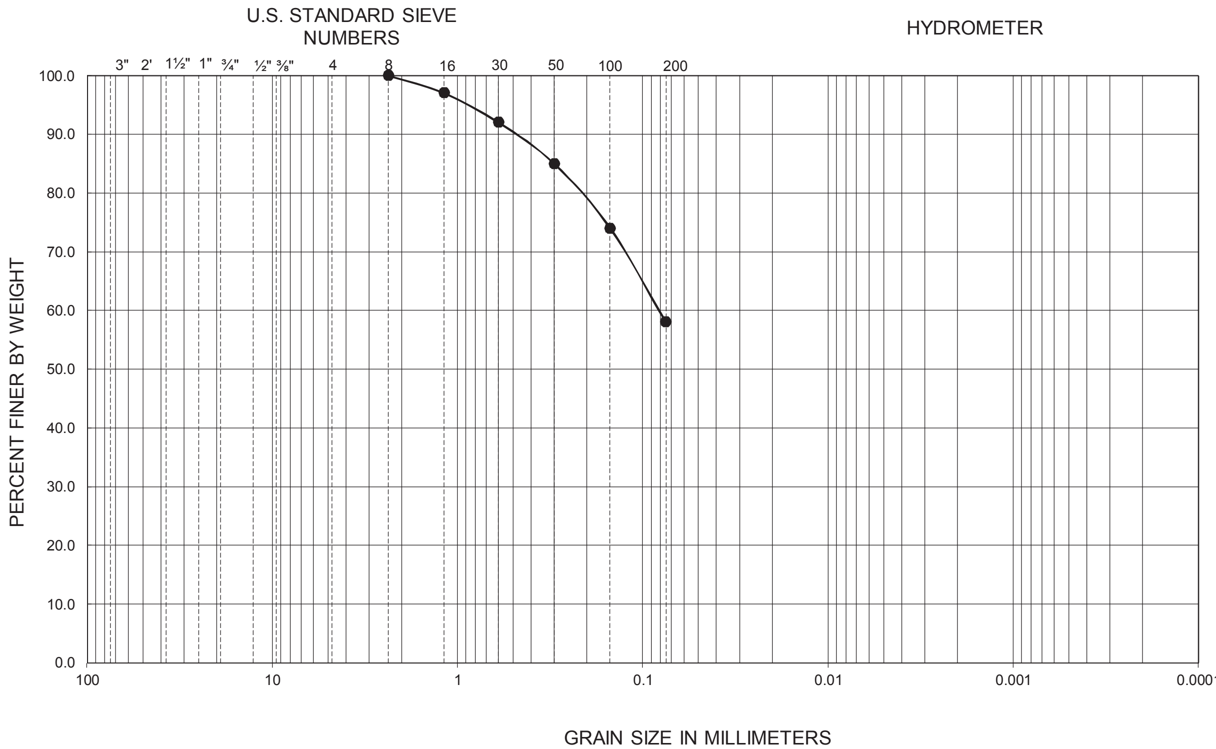
Expansion Index Test
The expansion index of selected material was evaluated in general accordance with ASTM D 4829. The specimen was molded under a specified compactive energy at approximately 50 percent saturation. The prepared 1-inch thick by 4-inch diameter specimen was loaded with a surcharge of 144 pounds per square foot and was inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The results of this test are presented on Figure C-8.

Soil Corrosivity Tests
Soil pH tests were performed on representative samples in general accordance with Arizona Test ARIZ 236b. The sulfate content and chloride contents of selected samples were also evaluated in general accordance with ARIZ 733 and 736, respectively. The test results are presented on Figure C-9.

R-Value

The resistance value, or R-value, for site soils were evaluated in general accordance with CT 301. A 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 results are shown on Figure C-10.

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	Equivalent USCS
●	B-1	10.0-11.5	--	--	--	--	--	--	--	--	58	ML

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE C-1

GRADATION TEST RESULTS

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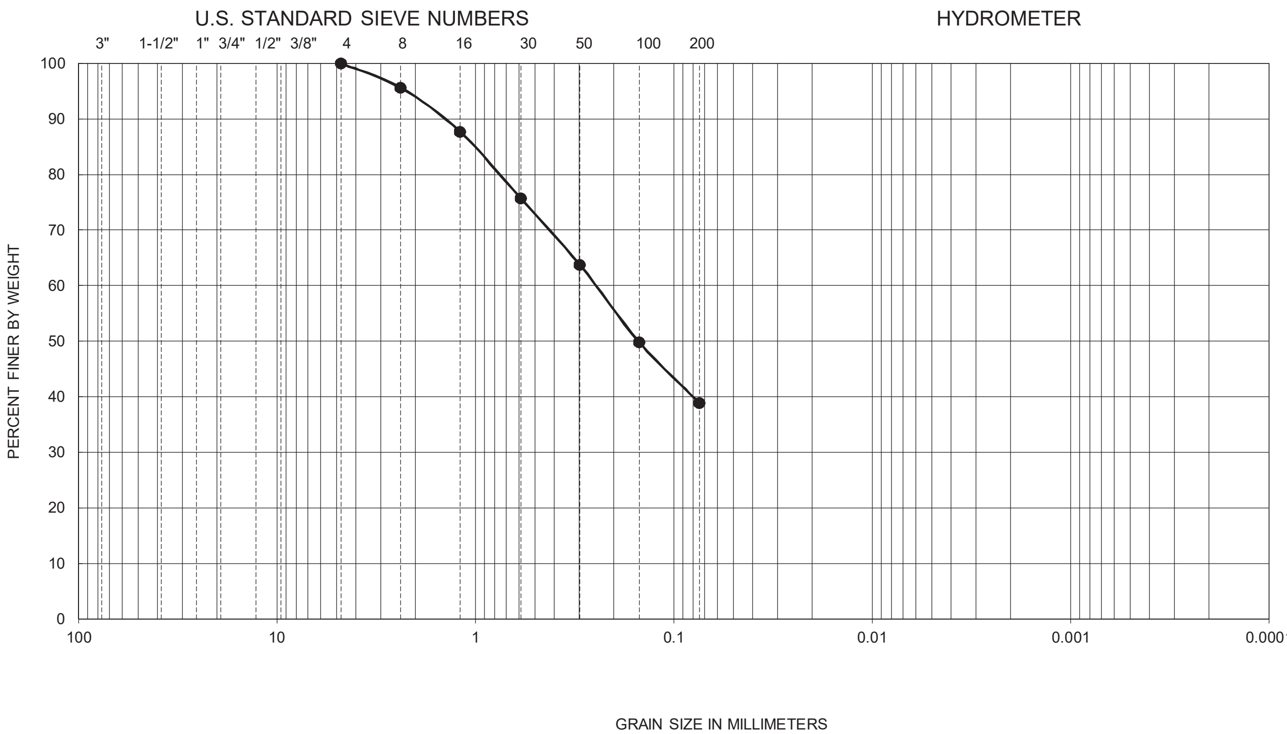
GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-2	0.0-5.0	--	--	--	--	--	--	--	--	39	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE C-2

GRADATION TEST RESULTS

MURRIETA U-HAUL FACILITY
41458 LOS ALAMOS ROAD AND 25086 JEFFERSON AVENUE, MURRIETA, CALIFORNIA

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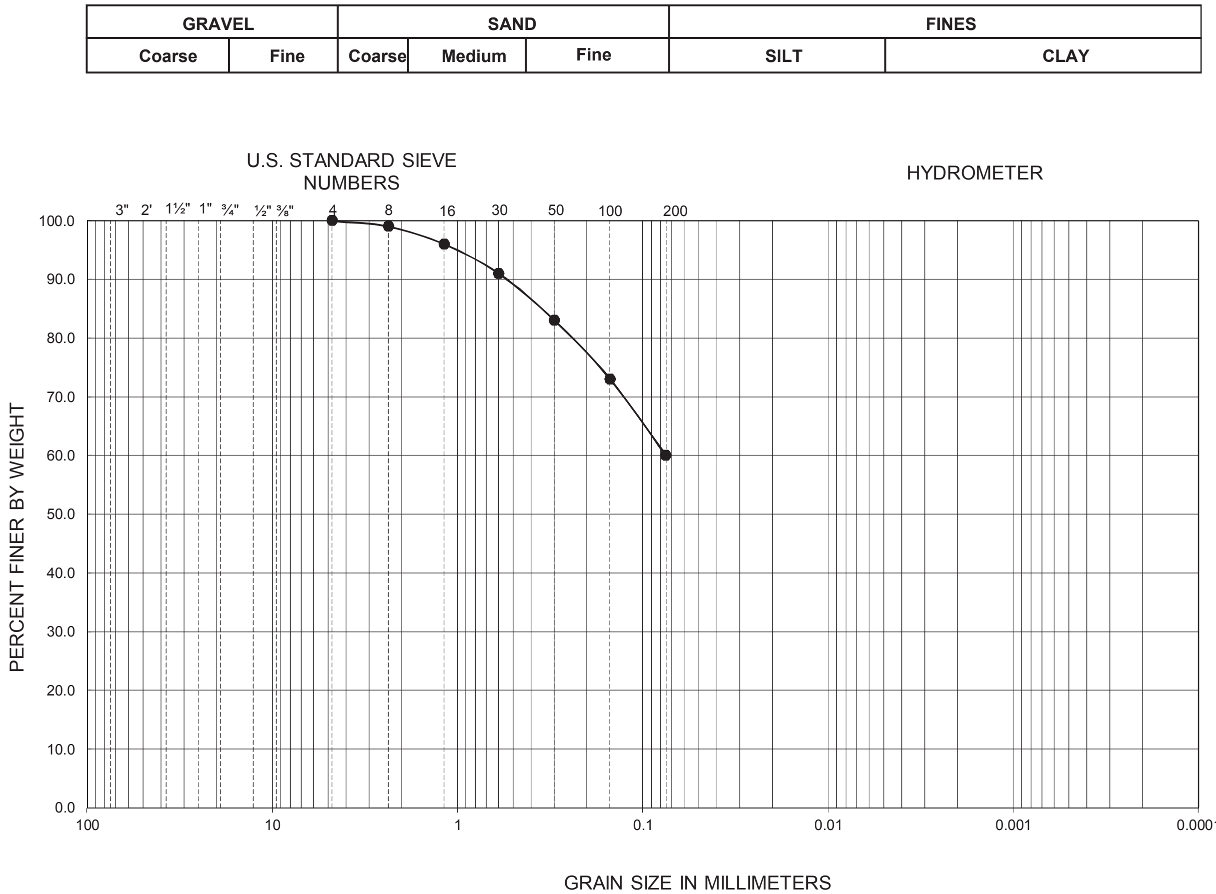

Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	Equivalent USCS
●	B-3	17.5-19.0	--	--	--	--	--	--	--	--	60	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE C-3

GRADATION TEST RESULTS

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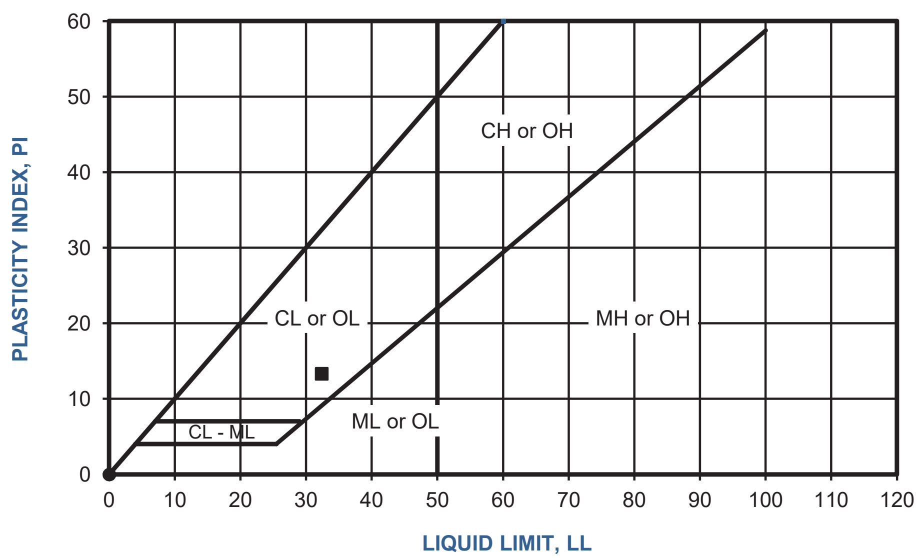
SYMBOL	LOCATION	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	Equivalent USCS
●	B-1	7.5-9.0	NP	NP	NP	ML	ML
■	B-3	35.0-36.5	32	19	13	CL	CL

NP - INDICATES NON-PLASTIC



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

FIGURE C-4

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ATTERBERG LIMITS TEST RESULTS

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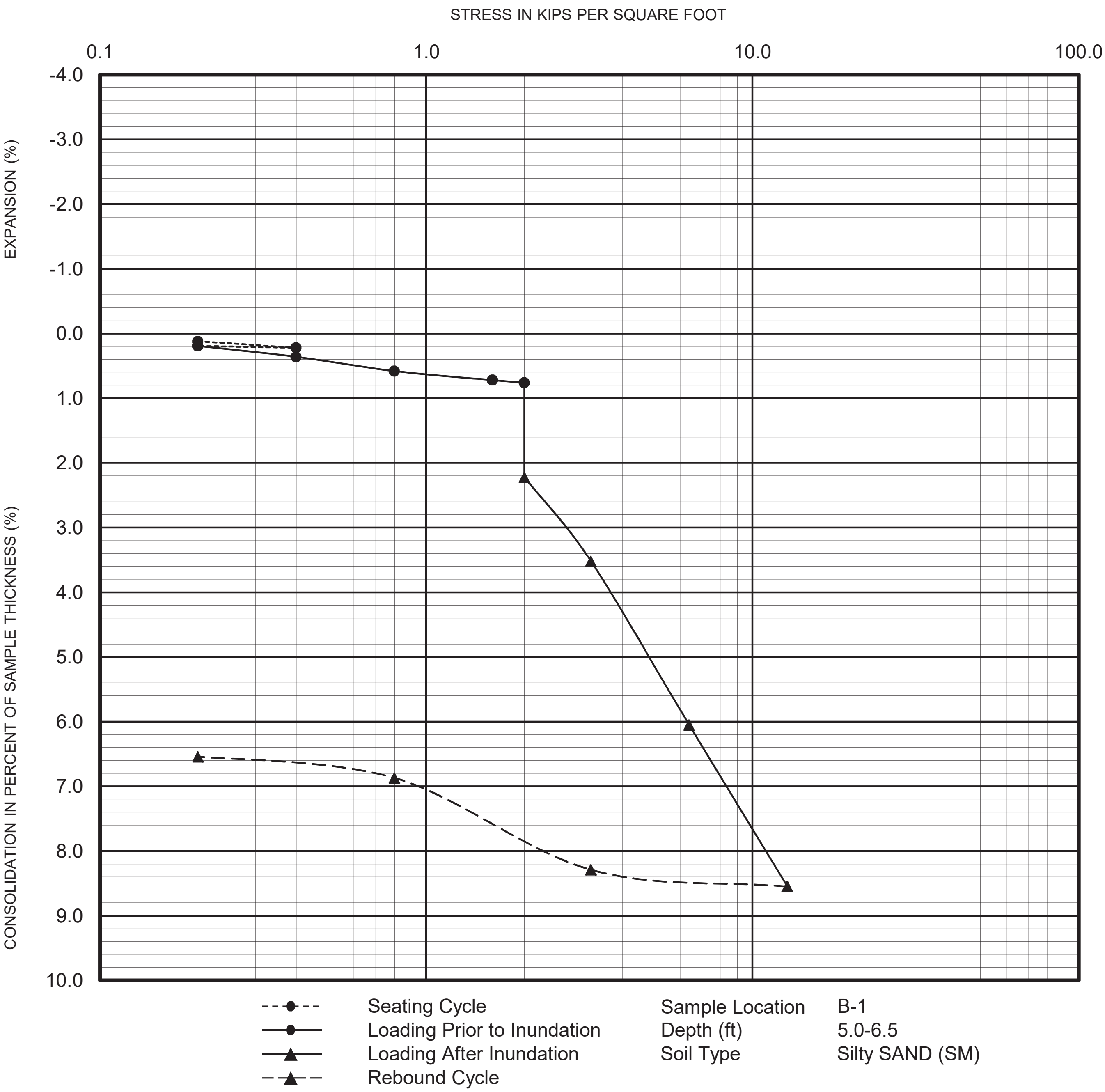

Sample Location	B-1
Depth (ft)	5.0-6.5
Soil Type	Silty SAND (SM)

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

FIGURE C-5

CONSOLIDATION TEST RESULTS

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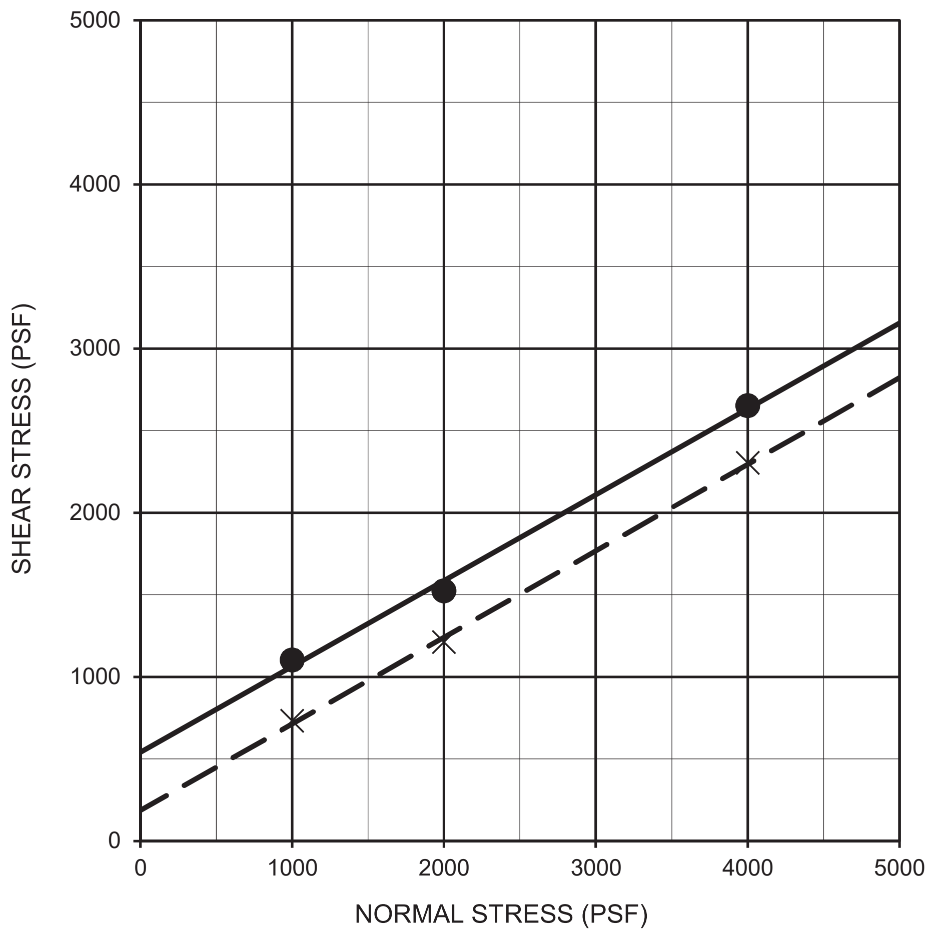
108673001_CONSOLIDATION B-1 @ 5.0-6.5.xlsx



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
Sandy SILTSTONE	——●——	B-1	7.5-9.0	Peak	540	28	Formation
Sandy SILTSTONE	– – X – –	B-1	7.5-9.0	Ultimate	190	28	Formation

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

FIGURE C-6

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DIRECT SHEAR TEST RESULTS

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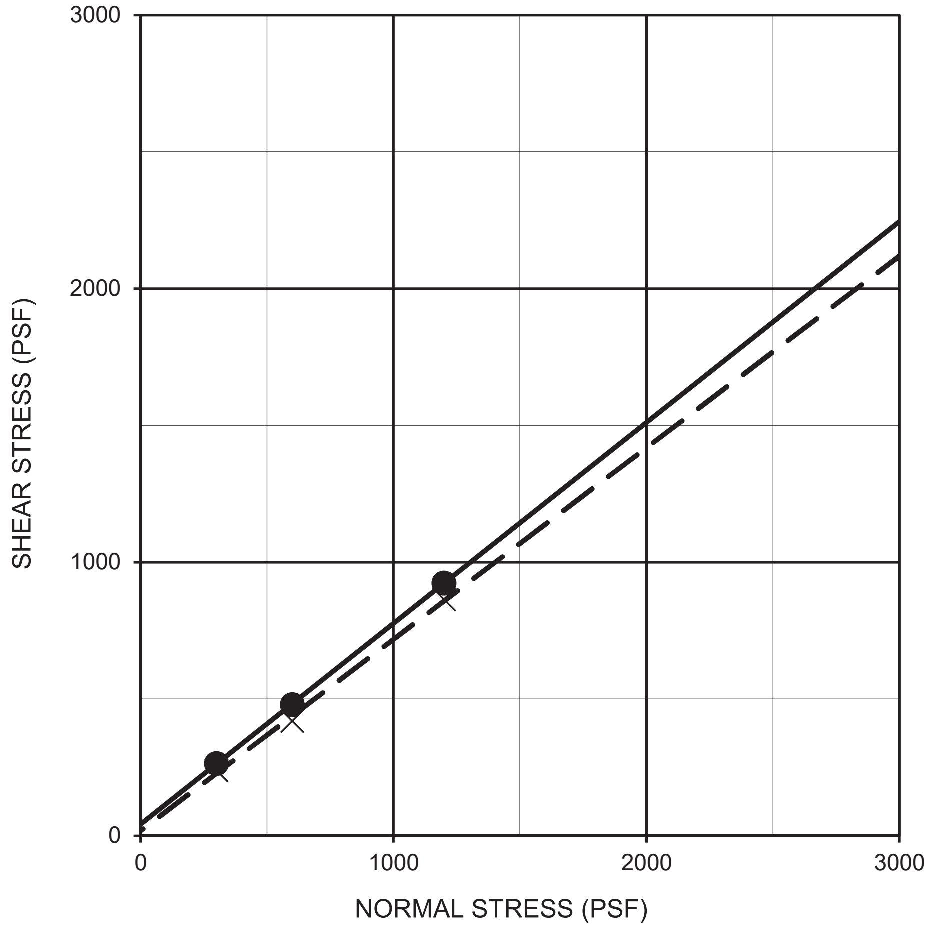

Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
Silty SANDSTONE	——●——	B-2	5.0-6.5	Peak	40	36	Formation
Silty SANDSTONE	– – X – –	B-2	5.0-6.5	Ultimate	20	35	Formation

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

FIGURE C-7

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DIRECT SHEAR TEST RESULTS

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	INITIAL MOISTURE (percent)	COMPACTED DRY DENSITY (pcf)	FINAL MOISTURE (percent)	VOLUMETRIC SWELL (in)	EXPANSION INDEX	POTENTIAL EXPANSION
B-1	0.0-5.0	8.5	116.6	21.0	0.002	2	Very Low

PERFORMED IN GENERAL ACCORDANCE WITH ☐ UBC STANDARD 18-2 ☑ ASTM D 4829

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EXPANSION INDEX 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-1	7.5-9.0	6.8	1,300	10	0.001	95
B-2	0.0-5.0	6.9	1,500	10	0.001	70

[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-9

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CORROSIVITY TEST RESULTS

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	SOIL TYPE	R-VALUE
B-1	0.0-5.0	Silty SAND	28

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2844/CT 301

FIGURE C-10

R-VALUE TEST RESULTS

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APPENDIX D

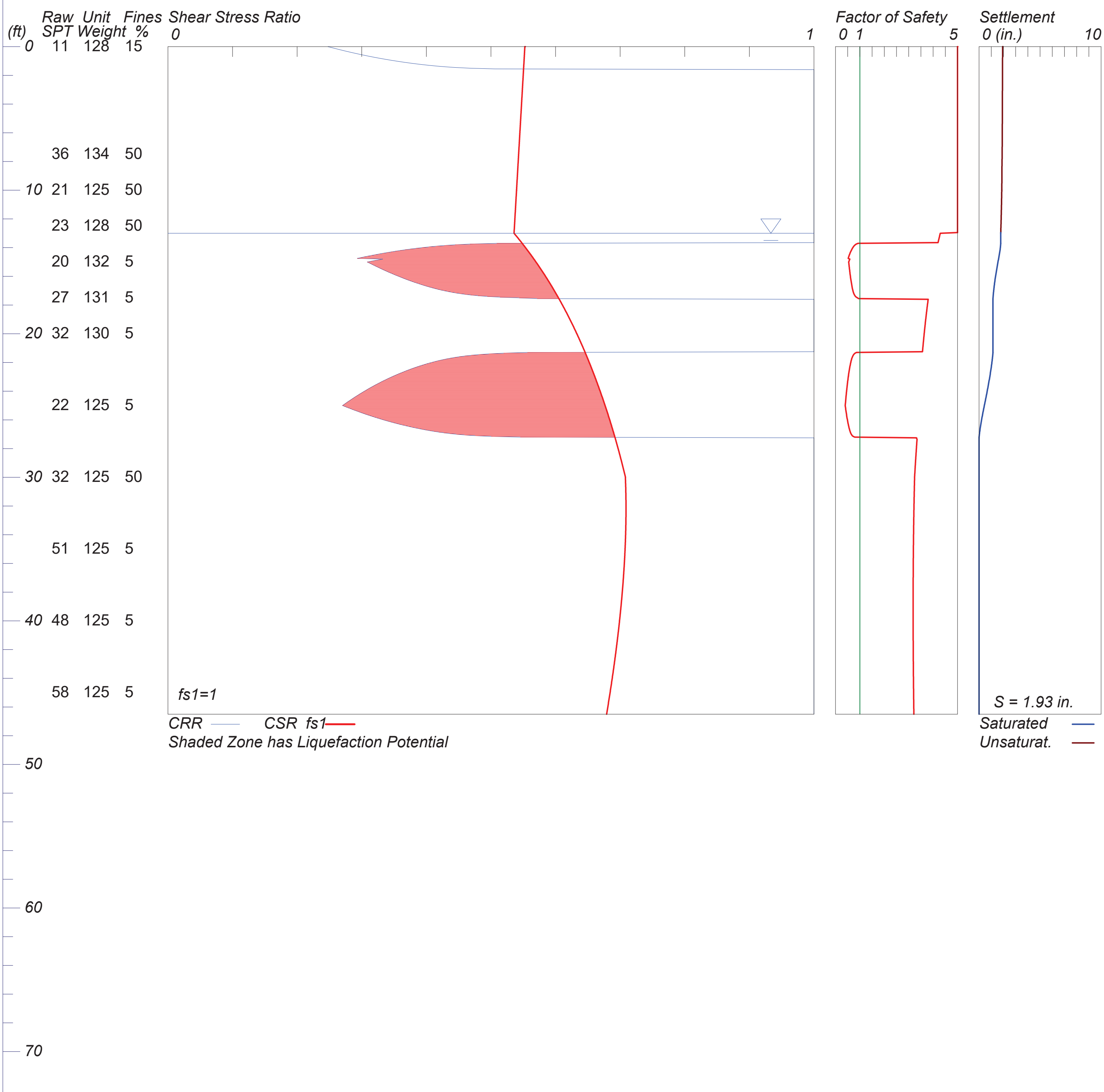
Liquefaction Analysis Results

DYNAMIC SETTLEMENT ANALYSIS

Murrieta U-Haul

Hole No.=B-1   Water Depth=13 ft   Surface Elev.=1107

Magnitude=7.1
Acceleration=0.85g



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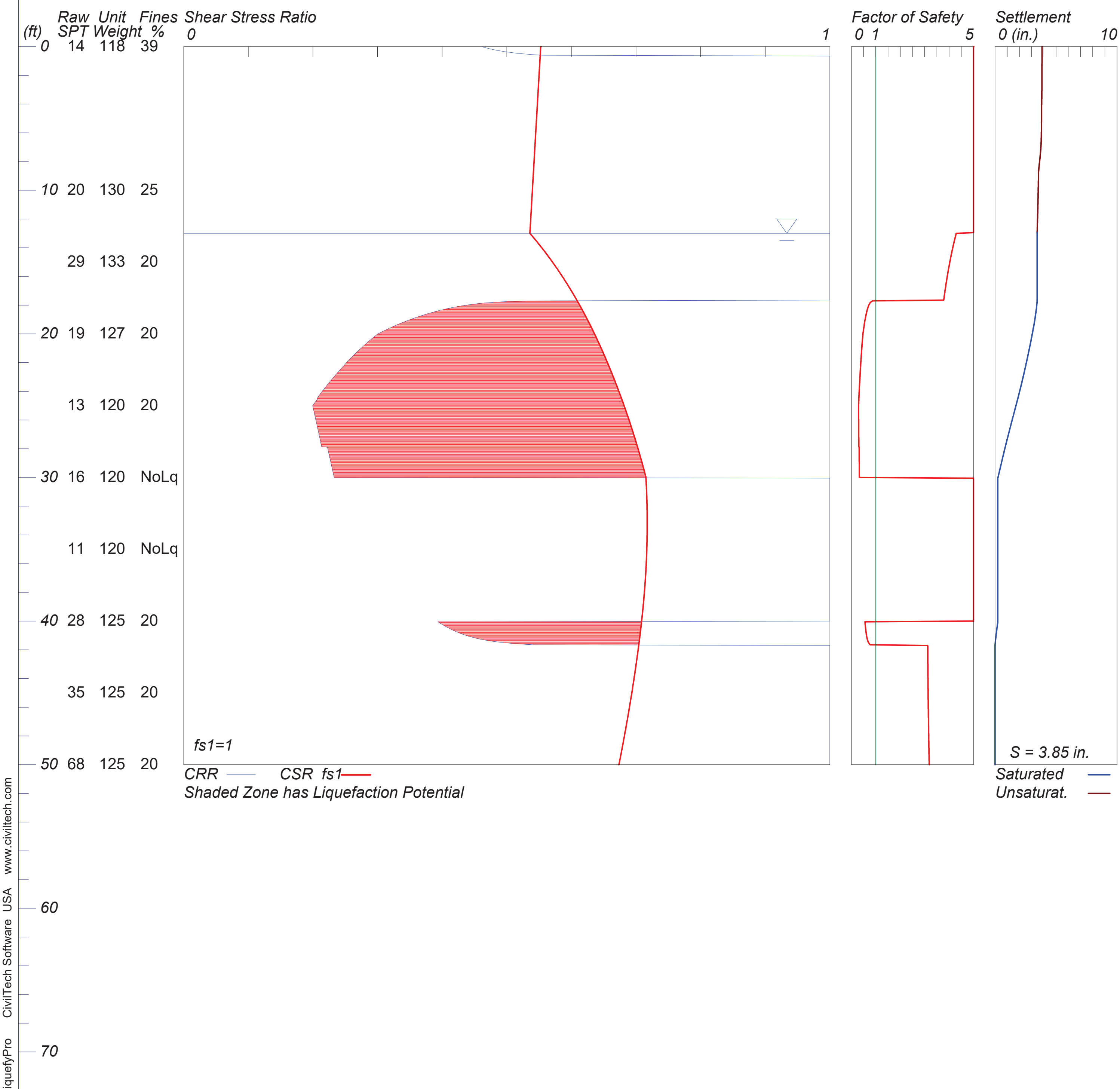
DYNAMIC SETTLEMENT ANALYSIS

Murrieta U-Haul

Hole No.=B-2   Water Depth=13 ft   Surface Elev.=1107

Magnitude=7.1
Acceleration=0.85g

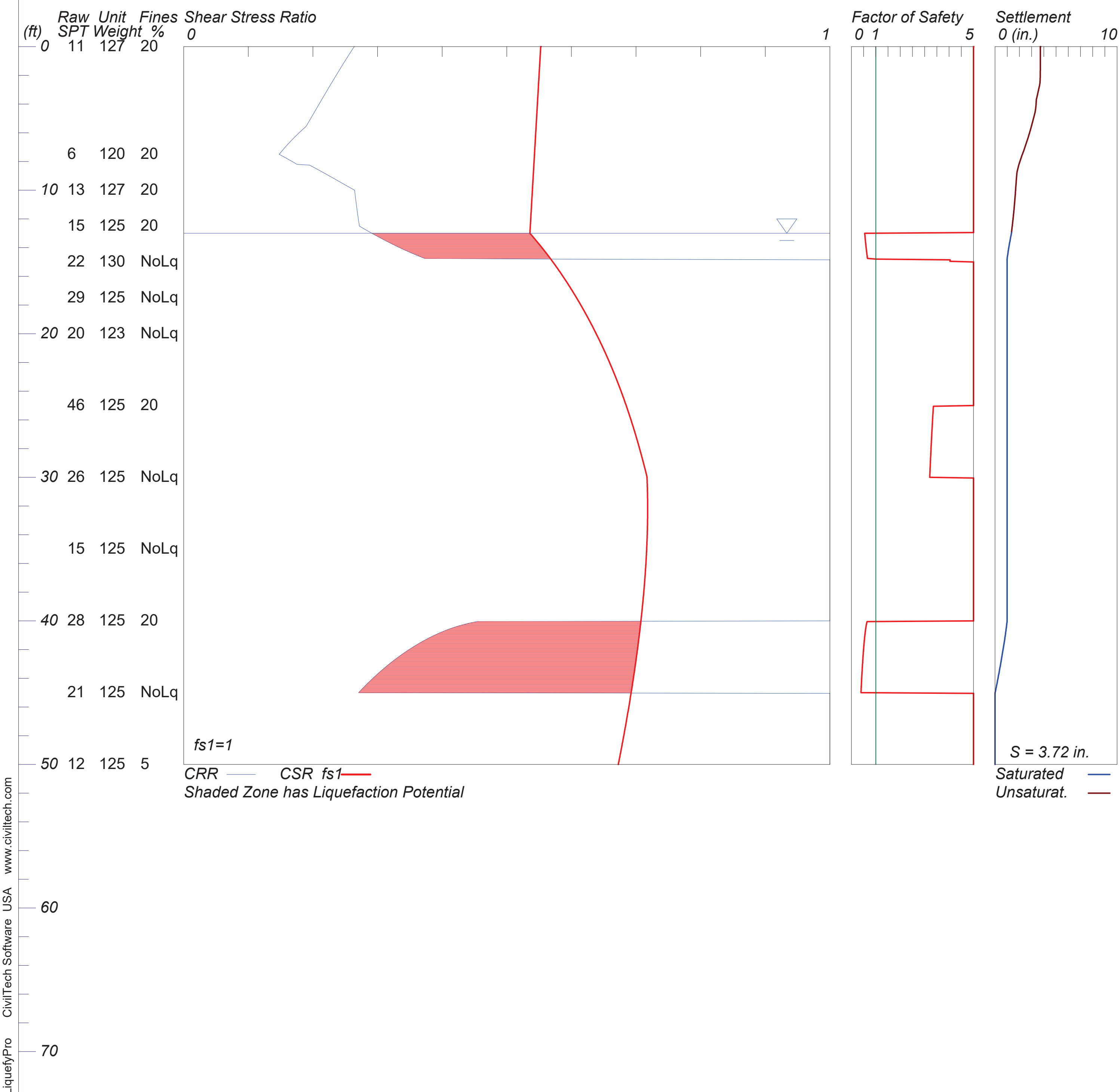


DYNAMIC SETTLEMENT ANALYSIS

Murrieta U-Haul

Hole No.=B-3 Water Depth=13 ft Surface Elev.=1105

Magnitude=7.1
Acceleration=0.85g



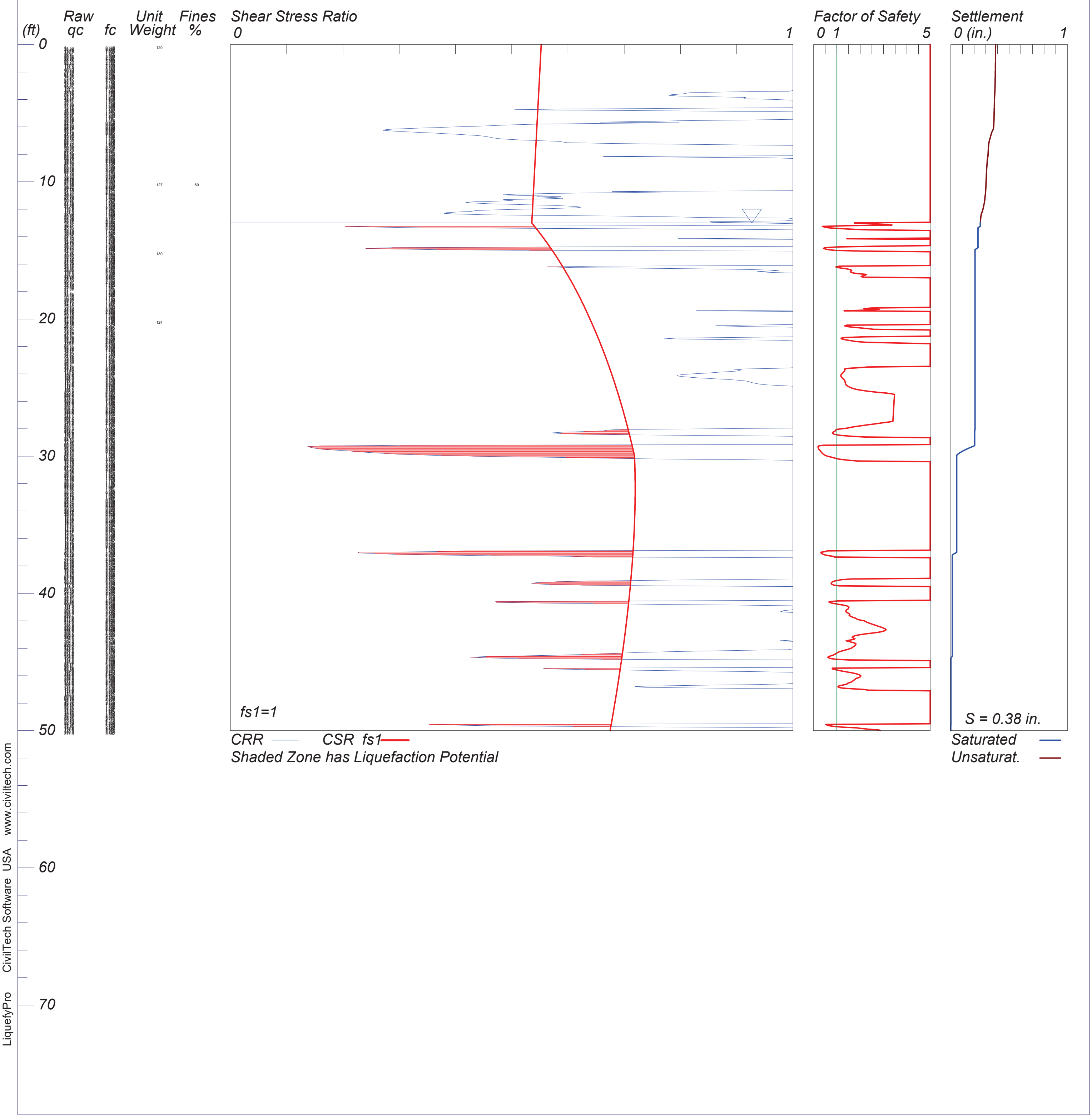
CivilTech Corporation **108673001**

DYNAMIC SETTLEMENT ANALYSIS

Murrieta U-Haul

Hole No.=CPT-1   Water Depth=13 ft   Surface Elev.=1105

Magnitude=7.1
Acceleration=0.85g



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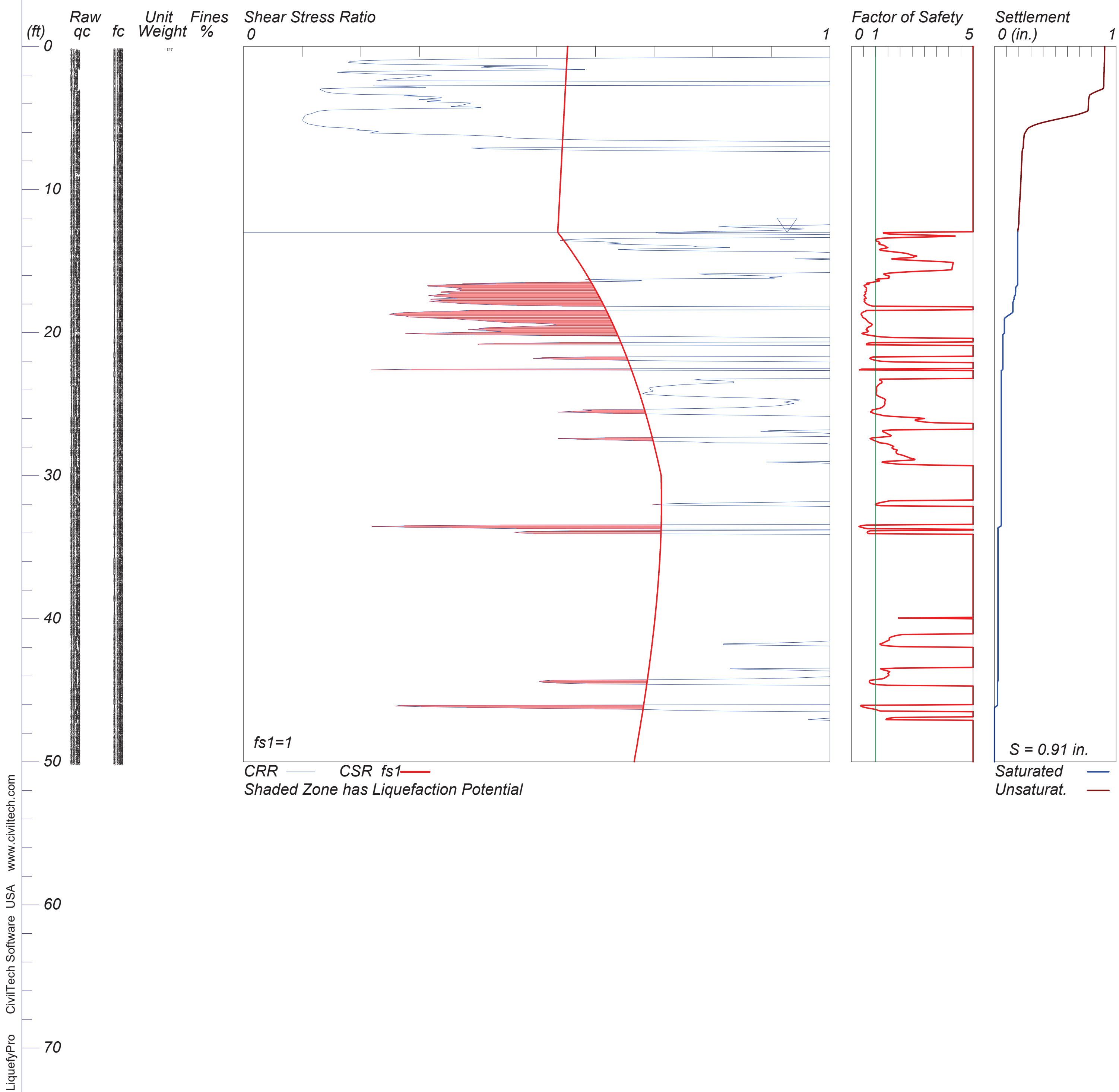
DYNAMIC SETTLEMENT ANALYSIS

Murrieta U-Haul

Hole No.=CPT-2 Water Depth=13 ft Surface Elev.=1108

Magnitude=7.1
Acceleration=0.85g

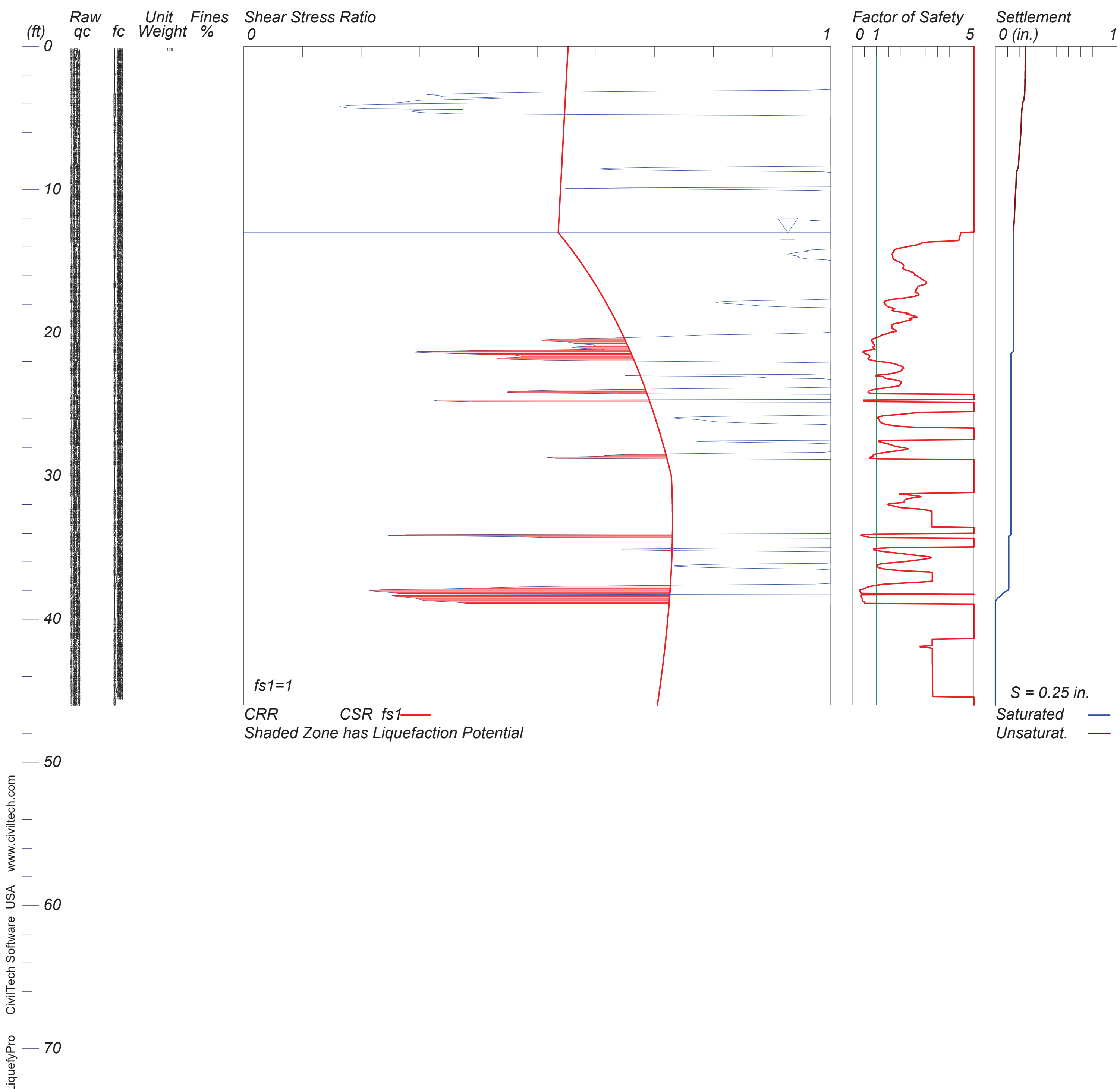


DYNAMIC SETTLEMENT ANALYSIS

Murrieta U-Haul

Hole No.=CPT-3 Water Depth=13 ft Surface Elev.=1110

Magnitude=7.1
Acceleration=0.85g



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