APPENDIX B

GEOTECHNICAL INVESTIGATION REPORT

TWINING

Engineering a Better Tomorrow

Geotechnical Investigation Report

Proposed Fire Station No. 9
4101 Long Beach Boulevard
Long Beach, California

Prepared for:

City of Long Beach Public Works Department
411 West Ocean Boulevard
Long Beach, California 90807

July 1, 2021
Project No.: 210377.1

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Subject: **Geotechnical Investigation Report**
Proposed Fire Station No. 9
4101 Long Beach Boulevard
Long Beach, California

Dear Mr. McMahon,

In accordance with your request and authorization, we are presenting the results of our geotechnical investigation for the proposed Fire Station No. 9 project located at 4101 Long Beach Boulevard in Long Beach, California. The purpose of our investigation is to characterize subsurface conditions of the site, evaluate seismic and geohazards at the site, and provide geotechnical engineering recommendations for the proposed improvements, including recommendations for foundations and earthwork.

This report was prepared in accordance with the requirements of the 2019 California Building Code (2019 CBC) and ASCE 7-16 (ASCE, 2017). Based on our findings, the proposed project is geotechnically feasible, provided that the recommendations in this report are incorporated into the design and are implemented during construction of the project.

We appreciate the opportunity to be of service on this project. Should you have any questions regarding this report or if we can be of further service, please do not hesitate to contact the undersigned.

Respectfully submitted,
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Appendix A – Field Exploration
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1. INTRODUCTION

This report presents the results of the geotechnical investigation performed by Twining, Inc. (Twining) for the proposed Fire Station No. 9 project located at 4101 Long Beach Boulevard in Long Beach, California. A description of the site and the proposed improvements is provided in the following section. The objectives of this investigation have been to characterize subsurface conditions of the site, evaluate seismic and geohazards at the site, and provide geotechnical recommendations for design and construction of the proposed development, including recommendations for foundations and earthwork. Our investigation was performed in conformance with the 2019 California Building Code (2019 CBC) and ASCE 7-16 (ASCE, 2017).

2. SITE DESCRIPTION AND PROPOSED DEVELOPMENT

The project site is located at 4101 Long Beach Boulevard in Long Beach, California, as shown on Figure 1 – Site Location Map. The approximate site coordinates are latitude 33.83248°N and longitude 118.18966°W, on the Long Beach, California 7½-Minute Quadrangle, according to the United States Geological Survey (USGS) topographic maps (USGS 2018). The site is bound by an alley and residences on the north, Long Beach Boulevard on the east, E. Randolph Place on the south, and residences on the west. The site is relatively flat with a surface elevation at approximately 95 feet above mean sea level (msl).

The site is currently occupied by a one-story building, concrete pavement, and minor landscaping. Based on information from City of Long Beach Public Works Department, it is our understanding that the existing building will be demolished. The proposed project will consist of the construction of a fire station, drainage basin, and improvements to the adjacent alley. Associated improvements such as utility trenches and pavements are anticipated. The locations and footprint of the proposed construction are depicted on Figure 2 – Site Plan and Boring Location Map.

3. SCOPE OF WORK

Our scope of work included review of background information, pre-field activities and field exploration, laboratory testing, engineering analyses and report preparation. These tasks are described in the following subsections.

3.1. Literature Review

We reviewed readily available background data including proposed site improvement plans, published geologic maps, topographic maps, aerial photographs, seismic hazard maps and literature, and flood hazard maps relevant to the subject site. Relevant information has been incorporated into this report. A partial list of literature reviewed is presented in the “Selected References” section of this report.

3.2. Pre-Field Activities

Before starting our exploration program, we performed a geotechnical site reconnaissance to observe the general surficial conditions at the site and to select field exploration locations. After exploration locations were delineated, Underground Service Alert was notified of the planned locations a minimum of 72 hours prior to excavation. The locations were cleared of buried utilities by a private utility locator. We obtained a permit for the field exploration from the Department of Health and Human Services of the City of Long Beach (LBDHHS).

3.3. Field Exploration

The field exploration consisted of drilling, testing, sampling, and logging of 8 exploratory borings (B-1 through B-6, P-1, and P-2) and percolation testing in 2 of the borings (P-1 through P-2) conducted at the site on June 4, 2021. The approximate exploration locations are shown on Figure 2 – Site Plan and Boring Location Map.

The borings were advanced to approximate depths of 5 to 81.5 feet below ground surface (bgs) using a CME-75 truck-mounted drill rig equipped with 8-inch-diameter hollow-stem-auger (HSA). All borings were first excavated to 5 feet bgs using a hand-auger to clear potential underground utilities.

Drive samples of the soils were obtained from the borings using a Standard Penetration Test (SPT) sampler without room for liner and a modified California split-spoon sampler. The samplers were driven using a 140-pound automatic hammer falling approximately 30 inches. The blow counts to drive the samplers were recorded, and subsurface conditions encountered in the borings were logged by a Twining field engineer under the supervision of a California Registered Engineering Geologist. Bulk samples were collected from the upper 5-foot soil cuttings. The samples were transported to Twining's geotechnical engineering laboratory in Long Beach, California for examination and testing.

In-situ percolation testing was performed in boring P-1 and P-2, which were advanced to 5 feet bgs, to provide estimates of infiltration rate of the site soils.

Upon completion of exploration, the borings deeper than 5 feet were backfilled with lean concrete grout. The 5-foot-deep borings were backfilled with soil cuttings. The surface was repaired to match existing conditions.

Detailed descriptions of the field exploration, soils encountered during drilling, and the LBDHHS permit are presented in Appendix A – Field Exploration.

3.4. Geotechnical Laboratory Testing

Laboratory tests were performed on selected samples obtained from the borings to aid in the soil classification and to evaluate the engineering properties of site soils. The following tests were performed in general accordance with ASTM and Caltrans standards:

- In-situ moisture and density (ASTM D2937),
- #200 Wash (ASTM D1140),
- Atterberg Limits (ASTM D4318),
- Expansion Index (ASTM D4829),
- Consolidation (ASTM D2435),
- Direct shear (ASTM D3080),
- Maximum dry density and optimum moisture content (ASTM D1557),
- Resistance value (R-value) (ASTM D2844), and
- Corrosivity (Caltrans test methods CT417, CT422, and CT 643).

Detailed laboratory test procedures and results are presented in Appendix B – Laboratory Testing.

3.5. Engineering Analyses and Report Preparation

We compiled and analyzed the data collected from our field exploration and laboratory testing. We performed engineering analyses based on our literature review and data from field exploration and laboratory testing programs. Our analyses included the following:

- Site geology, and subsurface conditions,
- Groundwater conditions,
- Geologic hazards and seismic design parameters,
- Liquefaction potential and seismic settlement,
- Soil corrosion potential,
- Soil collapse and expansion potential,
- Site preparation and earthwork,
- Project feasibility and suitability of on-site soils for foundation support,
- Foundation design parameters including bearing capacity, settlement, and lateral resistance,
- Concrete slab-on-grade support,
- Modulus of subgrade reaction for concrete slab-on-grade design,
- Temporary excavations, and
- Pavement section recommendations.

We prepared this report to present our conclusions and recommendations from this investigation.

4. GEOLOGY AND SUBSURFACE CONDITIONS

The regional and site geology and subsurface conditions are described in this section, based on our data review and field investigation. A portion of the geologic map is reproduced as Figure 3 – Geologic Map. Detailed subsurface conditions are presented in Appendix A – Field Exploration.

4.1. Regional Geology

According to the Geologic Map of the Long Beach 30' × 60' quadrangle (Saucedo et al., 2016), the project the site is underlain by Old Shallow Marine Deposits on Wave-Cut Surface (geologic map symbol Qom) that are late to middle Pleistocene in age. The deposits consist of poorly sorted, somewhat permeable siltstone, sandstone, and conglomerate that are reddish-brown in color (Saucedo et al., 2016). These deposits accumulated in strandline, beach, and estuarine environments and rest on platforms that have been carved by wave action and pushed up from below the water by regional uplift (Saucedo et al., 2016). A portion of the geologic map is reproduced as Figure 3 – Geologic Map.

4.2. Surface and Subsurface Conditions

As described earlier, the site is currently occupied by a one-story building, concrete pavement, and landscaping. The pavement section encountered in the borings consisted of 3 to 6 inches of concrete underlain by approximately 2.5 feet of fill materials. The fill consisted of slightly moist lean clay and sandy lean clay.

The native materials encountered below the fill materials consisted primarily of lean clay and sandy silt in the upper 60 feet with layers of silty sand between 15 and 20 feet bgs and between 45 and 50 feet bgs. The materials encountered below 60 feet bgs consisted of silty sand.

The consistency of the lean clay and silt varied from stiff to very stiff and hard. Relative density of the silty sand was dense below 45 feet bgs and medium dense in the upper layers. The color of the materials varied from medium brown to reddish brown, strong brown, olive brown and dark yellowish brown. The materials were slightly moist. Detailed descriptions of the soils encountered during drilling are presented in Appendix A – Field Exploration.

4.3. Groundwater

No groundwater was encountered to the maximum exploration depth of approximately 81.5 feet bgs. The Seismic Hazard Zone report (California Department of Conservation, Division of Mines and Geology, 1998) presented the historically highest groundwater contour map for the Long Beach Quadrangle. However, the historical high groundwater level at the site is not well defined on the contour map. We researched historical water level data in the vicinity of the site. Based on the groundwater well database of Los Angeles County Department of Public Works (LADPW), historical groundwater level between August 1, 1934 and May 6, 2021 is available from a groundwater well located approximately 0.43 miles northwest of the site (Well ID 906D and State Well ID 4S13W12K01). Groundwater level in the well decreased over the years, and the highest level was deeper than 70 feet recorded at elevation 14 feet msl on April 17, 1935.

Groundwater conditions may vary across the site due to stratigraphic and hydrologic conditions and may change over time as a consequence of seasonal and meteorological fluctuations, or of activities by humans at this and nearby sites.

5. GEOLOGIC HAZARDS AND SEISMIC DESIGN CONSIDERATIONS

The site is located in a seismically active area, as is the majority of southern California, and the potential for strong ground motion in the project area is considered high during the design life of the proposed development. The hazards associated with seismic activity in the vicinity of the site area discussed in the following sections.

5.1. Active Faulting and Surface Fault Rupture

The subject site is not located within a State of California Earthquake Fault Zone (formerly known as a Special Studies Zone) (Hart and Bryant, 1997). The boundary of the closest Alquist-Priolo EFZs is located approximately 0.6 miles southwest of the site associated with the Long Beach fault zone (part of the Newport-Inglewood fault zone). Figure 4 shows the location of the fault zone with respect to the site. The current general plans of the City of Long Beach and the County of Los Angeles do not identify any additional hazardous faults in the immediate site vicinity.

Based on our review of geologic and seismologic literature and our site evaluation, it is our opinion that the likelihood of surface fault rupture at the site during the life of the proposed project is remote.

5.2. Liquefaction and Seismic Settlement Potential

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 generally known to occur in loose, saturated, relatively clean, fine-grained cohesionless soils at depths shallower than approximately 50 feet. Factors to consider in the evaluation of soil liquefaction potential include groundwater conditions, soil type, grain size distribution, relative density, degree of saturation, and both the intensity and duration of ground motion. Other phenomena associated with soil liquefaction include sand boils, ground oscillation, and loss of foundation bearing capacity.

The project site is not within a state-designated Zone of required investigation for liquefaction according to CGS (2016). Based on the great depth of groundwater and site subsurface conditions, it is our opinion that liquefaction potential and seismic settlement at the site is low.

5.3. Landslides

The area of the project site is not within an area with the potential for earthquake-induced landslides. Considering the site is flat and not close to significant slopes, the potential for earthquake-induced landslides to occur at the site is considered negligible.

5.4. Tsunamis and Seiches

Tsunamis are waves generated by massive landslides near or under sea water. Based on California Official Tsunami Inundation Maps, the site is not located on any State of California Tsunami Inundation Map for Emergency Planning. The potential for the site to be adversely impacted by earthquake-induced tsunamis is considered to be negligible.

Seiches are standing wave oscillations of an enclosed water body after the original driving force has dissipated. The potential for the site to be adversely impacted by earthquake-induced seiches is considered to be negligible due to the lack of any significant enclosed bodies of water located in the vicinity of the site.

5.5. Flooding

The Federal Emergency Management Agency (FEMA) has prepared flood insurance rate maps (FIRMs) for use in administering the National Flood Insurance Program, effective September 26, 2008. Based on our review of online FEMA flood mapping, the site is located within Zone X with minimal flood hazard.

5.6. Deaggregated Seismic Source Parameters

We performed a seismic hazard de-aggregation analysis for the peak ground acceleration with a probability of exceedance of 2% in 50 years. The analysis used the USGS Unified Hazard Tool based on the 2014 USGS seismic source model. The results of the analysis indicate the controlling modal moment magnitude and fault distance are 7.3 Mw and 3.8 miles (6.1 km), respectively.

5.7. Site Class for Seismic Design

According to our field exploration program, the average SPT resistance for the upper 80 feet is in the range between 15 and 50 blows per foot. Using the SPT resistance obtained from the field exploration, we estimated the shear-wave velocity (V_S) profile and an average V_S for the upper 100 feet of the soil profile (V_{S30}) of approximately 932 feet/sec or 284 m/sec. Based on the SPT resistance and the V_{S30} value, it is our opinion that Site Class D may be used for the project seismic design according to Chapter 20 of ASCE 7-16.

5.8. Seismic Design Parameters

Seismic design for new buildings should be based on the 2019 CBC and ASCE 7-16. As the site is classified as seismic Site Class D and the mapped spectral acceleration parameter at period 1-second, S_1, is greater than 0.2 g, the 2019 CBC requires a site-specific ground motion hazard analysis following Section 11.4.7 of ASCE 7-16 for new buildings. The site-specific ground motion hazard analysis is presented in Section 5.9.

Alternatively, Exception 2 in Section 11.4.8 of ASCE 7-16 may be used for the project new buildings in lieu of the site-specific ground motion hazard analysis. For seismic design of new buildings based on this exception, seismic design parameters in Table 1 may be used, based on site coordinates of latitude 33.83248°N and longitude 118.18966°W.

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Table 1 – Seismic Design Parameters Based on 2019 CBC and ASCE 7-16 for Design Based on Exception 2 in Section 11.4.8 of ASCE 7-16

Design Parameters	Value
Site Class	D
Mapped Spectral Acceleration Parameter at Period of 0.2-Second, S_s (g)	1.663
Mapped Spectral Acceleration Parameter at Period 1-Second, S_1 (g)	0.598
Site Coefficient, F_a	1
Site Coefficient, F_v	1.702
Adjusted MCE_R[1] Spectral Response Acceleration Parameter, S_{MS} (g)	1.663
Adjusted MCE_R[1] Spectral Response Acceleration Parameter, S_{M1} (g)	1.0
Design Spectral Response Acceleration Parameter, S_{DS} (g)	1.109
Design Spectral Response Acceleration Parameter, S_{D1} (g)	0.679
Risk Coefficient, C_{RS}	0.903
Risk Coefficient, C_{R1}	0.901
Peak Ground Acceleration, PGA_M[2] (g)	0.796
Seismic Design Category[3]	D
Long-Period Transition Period, T_L (seconds)	8
$T_S = S_{D1} / S_{DS}$	0.612

When using the above parameters for seismic design, the seismic design coefficient C_s should be calculated as follows:
For $T \leq 1.5T_S$, $C_S = S_{DS}/(R/I_e)$
For $T_L \geq T > 1.5T_S$, $C_S = 1.5\ S_{D1}/(T\ R/I_e)$
For $T > T_L$, $C_S = 1.5\ (S_{D1}\ T_L)/(T^2\ R/I_e)$
where
T = the fundamental period of the structure(s) determined in Section 12.8.2 of ASCE 7-16;
R = the response modification factor determined in Table 12.2-1 of ASCE 7-16; and
I_e = the importance factor determined in accordance with Section 11.5.1 of ASCE 7-16.

Notes: [1] Risk-Targeted Maximum Considered Earthquake.
[2] Peak Ground Acceleration adjusted for site effects.
[3] For S_1 greater than or equal to 0.75g, the Seismic Design Category is E for risk category I, II, and III structures and F for risk category IV structures.

5.9. Site-Specific Seismic Hazard Analysis and Seismic Design Parameters

The site-specific ground motion hazard analysis was performed in accordance with Section 21.2 of ASCE 7-16 based on a 2% probability of exceedance in 50 years. To develop the site-specific design response spectrum, we performed probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis (DSHA) to compute the risk-targeted maximum considered earthquake (MCE_R) response accelerations. Our PSHA and DSHA used four NGA-West2 ground motion prediction equations (GMPEs) developed by Abrahamson et al. (2014), Boore et al. (2014), Campbell and Bozorgnia (2014), and Chiou and Youngs (2014), respectively. The analyses were based on the Uniform California Earthquake Rupture Forecast Version 3 (UCERF3) developed by the Working Group on California Earthquake Probabilities (WGCEP). UCERF3 is the California portion of the 2014 USGS national seismic source model (Petersen et al. 2014). Our analyses included treatment of maximum direction spectra and adjustment for risk targeting.

The analyses were performed assuming a $V_{S,30}$ value of 932 feet/sec or 284 m/sec discussed in Section 5.7 and site coordinates of latitude 33.83248°N and longitude 118.18966°W described in Section 2. The site-specific design response spectrum is presented in Section 5.9.3, along with the MCE_R ground motions from our PSHA and DSHA. The site-specific design response spectrum is presented in Section 5.9.4. The detailed analysis description and results are presented below.

5.9.1. Probabilistic Seismic Hazard Analysis

A site-specific PSHA was performed to evaluate probabilistic MCE_R ground motions. The probabilistic spectral response accelerations are taken as the spectral response accelerations in the direction of maximum horizontal response represented by a 5% damped acceleration response spectrum that is expected to achieve a 1% probability of collapse within a 50-year period. In this report, ordinates of the probabilistic ground motion response spectrum were determined by Method 1 of Section 21.2.1.1 of ASCE 7-16.

The PSHA was first performed using the Hazard Spectrum Calculator by OpenSHA.org (http://www.opensha.org/apps-HazardSpectrumLocal) to obtain an average spectrum of the geometric-mean acceleration response spectra from the four NGA-West2 GMPEs. The spectra were calculated for 5-percent damped and a 2 percent probability of exceedance within a 50-year period. The average spectrum was converted to the maximum response ground motion using scale factors described in Section 21.2 of ASCE 7-16. The scale factors are 1.1 for spectral response periods less than or equal to 0.2 s, 1.3 for a period of 1.0 s, 1.5 for periods greater than or equal to 5.0 s, and between these periods are obtained by linear interpolation. The maximum response ground motion was then multiplied by a risk coefficient C_R to obtain the probabilistic MCE_R ground motion response spectrum. The values of C_R are C_{RS} for periods less than or equal to 0.2 s and C_{R1} for periods greater than or equal to 1.0 s. For periods between periods 0.2 s and 1.0 s, C_R is based on linear interpolation of C_{RS} and C_{R1}. The values of C_{RS} and C_{R1} for this project are presented in Table 1.

5.9.2. Deterministic Seismic Hazard Analysis

A site-specific DSHA was performed to evaluate the deterministic MCE_R ground motions. The deterministic MCE_R response acceleration at specified periods was calculated as the 84th percentile of the maximum rotated component of ground motion computed at each period for characteristic earthquakes on known active faults within the region.

The controlling active faults and their parameters used in our DSHA are provided in Table 2. The DSHA was performed for each fault to obtain the 5-percent-damped deterministic pseudo-absolute acceleration response spectrum using the four NGA-West2 GMPEs implemented in a Microsoft Excel spreadsheet available from the Pacific Earthquake Engineering Research Center (https://peer.berkeley.edu/research/data-sciences/databases).

Table 2 - Seismic Source Parameters

Fault Name	Newport-Inglewood alt 1	Newport-Inglewood alt 2	Compton	Palos Verdes
Slip Sense	Strike Slip	Strike Slip	Reverse	Strike Slip
M_W	7.2	7.2	6.9	7.3
Dip, (deg)	88	90	20	90
Z_{TOR} (km)	0	0	5.2	0
Z_{BOT}, (km)	15	10.2	15.6	13.6
W (km)	15.0	10.2	30.4	13.6
R_{RUP} (km)	1.49	1.17	7.88	11.3
R_{JB} (km)	1.49	1.17	0	11.3
R_X (km)	1.49	1.17	8.75	11.3
F_{NM}	0	0	0	0
F_{RV}	0	0	1	0

Notes:

M_W = Moment magnitude.
Z_{TOR} = The depth to the top of the rupture plane.
Z_{BOT} = The depth to the bottom of the rupture plane.
W = Fault rupture width.
R_{RUP} = Closest distance to coseismic rupture.
R_{JB} = Closest distance to surface projection of coseismic rupture.
R_X = Horizontal distance from top of rupture measured perpendicular to fault strike.
F_{RV} = Reverse-faulting factor: 0 for strike-slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust.
F_{NM} = Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal.

The resulting 84th percentile geometric-mean acceleration response spectra for the earthquakes were used to develop a deterministic response spectrum based on the greatest spectral acceleration at each period, and then converted into maximum rotated components of ground motion using the scale factors described in Section 21.2 of ASCE 7-16 as discussed in Section 5.9.1 of this report. The final deterministic MCE_R is taken as the maximum rotated deterministic response spectrum scaled by a single factor equal to the greater of $1.5F_a/S_{a,max,max}$ and 1, where $S_{a,max,max}$ is the maximum spectral acceleration of the maximum rotated deterministic response spectrum, and F_a is determined to be 1 using Table 11.4.1 of ASCE 7-16.

5.9.3. Site-Specific Design Response Spectrum

The site-specific MCE_R spectral response acceleration was calculated at each period to be the lesser of the spectral response accelerations from the probabilistic and deterministic MCE_R, but not less than 1.5 times 80 percent of the spectral acceleration evaluated in accordance with Sections 11.4.6 and 21.3 of ASCE 7-16. In order to calculate the 80 percent of the spectral acceleration, values of S_{DS}, S_{D1} and the design spectrum were calculated using the mapped values presented in Table 1, except that S_{M1} and S_{D1} at this step were based on an F_v value of 2.5, in accordance with Section 21.3 of ASCE 7-16.

Finally, the site-specific design spectral response acceleration at each period was calculated as two-thirds of the site-specific MCE_R spectral acceleration. The site-specific design response spectrum and relevant response spectral data are presented in Table 3 and Figure 5.

Table 3 - Site-Specific Design Response Spectrum Data

Period T (sec)	General Procedure Design Response Spectrum for Exception 2 of ASCE 7-16 (g)	Risk Coefficient C_R	Site-Specific Ground Motion Analysis Spectral Accelerations (g)						
			Maximum direction 2%-in-50-years Probabilistic Spectrum	Probabilistic MCE_R	Maximum direction 84th-percentile Deterministic Spectrum	Deterministic MCE_R	80% General Procedure Design Response Spectrum with Fv=2.5	Site Specific MCE_R	Site-Specific Design Response Spectrum
0.01	0.498	0.903	0.861	0.778	1.025	1.025	0.384	0.778	0.518
0.02	0.552	0.903	0.865	0.781	1.030	1.030	0.414	0.781	0.521
0.03	0.606	0.903	0.888	0.802	1.041	1.041	0.444	0.802	0.535
0.05	0.715	0.903	1.015	0.916	1.156	1.156	0.503	0.916	0.611
0.075	0.851	0.903	1.263	1.140	1.364	1.364	0.577	1.140	0.760
0.1	0.987	0.903	1.483	1.339	1.573	1.573	0.651	1.339	0.893
0.122	1.109	0.903	1.606	1.450	1.694	1.694	0.717	1.450	0.967
0.15	1.109	0.903	1.757	1.586	1.843	1.843	0.799	1.586	1.058
0.18	1.109	0.903	1.846	1.667	1.980	1.980	0.887	1.667	1.111
0.2	1.109	0.903	1.907	1.722	2.073	2.073	0.887	1.722	1.148
0.25	1.109	0.903	2.034	1.836	2.260	2.260	0.887	1.836	1.224
0.3	1.109	0.903	2.131	1.924	2.457	2.457	0.887	1.924	1.283
0.4	1.109	0.903	2.141	1.932	2.618	2.618	0.887	1.932	1.288
0.5	1.109	0.902	2.076	1.873	2.567	2.567	0.887	1.873	1.249
0.612	1.109	0.902	1.929	1.740	2.453	2.453	0.887	1.740	1.160
0.75	0.905	0.902	1.749	1.577	2.314	2.314	0.887	1.577	1.051
0.899	0.755	0.901	1.592	1.435	2.174	2.174	0.887	1.435	0.957
1	0.679	0.901	1.486	1.339	2.079	2.079	0.797	1.339	0.893
1.5	0.452	0.901	1.033	0.931	1.511	1.511	0.532	0.931	0.621
2	0.339	0.901	0.773	0.697	1.143	1.143	0.399	0.697	0.464
3	0.226	0.901	0.500	0.451	0.771	0.771	0.266	0.451	0.301
4	0.170	0.901	0.352	0.317	0.535	0.535	0.199	0.317	0.211
5	0.136	0.901	0.274	0.247	0.392	0.392	0.159	0.247	0.165

5.9.4. Site-Specific Seismic Design Parameters

The site-specific seismic design parameters are provided in Table 4. These parameters were determined from the site-specific design response spectrum presented in Table 3 following Section 21.4 of ASCE 7-16.

It should be noted that for use with the equivalent lateral force procedure in structural design, the site specific design spectral acceleration, Sa (the last column in Table 3 of this report), at period T may replace S_{D1}/T and $S_{D1}T_L/T^2$ in ASCE 7-16 Eqs. (12.8-3) and (12.8-4), respectively. The site-specific seismic design parameter S_{DS} shown in Table 4 of this report may be used in ASCE 7-16 Eqs. (12.8-2), (12.8-5), (15.4-1), and (15.4-3). The mapped value of S_1 in Table 1 of this report should be used in ASCE 7-16 Eqs. (12.8-6), (15.4-2), and (15.4-4).

Table 4 - Site-Specific Seismic Design Parameters

Site-Specific Seismic Design Parameters	Design Values (g)
Spectral Response Acceleration 0.2-second period, S_{MS}	1.74
Spectral Response Acceleration 1-second period, S_{M1}	1.40
Design Spectral Response Acceleration for short period, S_{DS}	1.16
Design Spectral Response Acceleration for 1-second period, S_{D1}	0.93
MCE Geomatric Mean (MCE_G) Peak Ground Acceleration, PGA_M	0.78

6. GEOTECHNICAL ENGINEERING RECOMMENDATIONS

Based on the results of our literature review and the field exploration, laboratory testing, and engineering analyses, it is our opinion that the proposed construction is feasible from a geotechnical standpoint, provided that the recommendations in this report are incorporated into the design plans and are implemented during construction.

6.1. General Considerations

Geotechnical engineering recommendations presented in this report for the proposed project are based on our understanding of the proposed development, subsurface conditions encountered during our field exploration, the results of laboratory testing on soil samples taken from the site, and our engineering analyses. Based on our field exploration, the site is covered by 3 to 6 inches of concrete pavement underlain by approximately 2.5 feet of fill materials consisting off slightly moist lean clay and sandy lean clay.

The following sections present our conclusions and recommendations pertaining to the engineering design for this project. If the design substantially changes, then our geotechnical engineering recommendations would be subject to revision based on our evaluation of the changes.

6.2. Soil Expansion and Collapse Potential

Based on our field exploration and laboratory testing results, the risk of soil expansion and collapse is low at the site. Soil expansion and collapse potentials are considered to have negligible effects on the design and construction of the project.

6.3. Corrosive Soil Evaluation

In accordance with the County of Los Angeles (2014) criteria, corrosive soil is defined as the soil has minimum electrical resistivity less than 1,000 ohm-centimeters, or chloride concentration greater than 500 ppm, or sulfate concentration in soils greater than 2,000 ppm, or a pH less than 5.5.

The potential for the near-surface on-site materials to corrode buried steel and concrete improvements was evaluated. Laboratory testing was performed on one selected near-surface soil to evaluate pH and electrical resistivity, as well as chloride and sulfate contents. The pH and electrical resistivity tests were performed in accordance with California Test 643, and the sulfate and chloride tests were performed in accordance with California Tests 417 and 422, respectively. These laboratory test results are presented in Appendix B – Laboratory Testing.

Discussions of corrosion protection for reinforced concrete and buried metal is provided below. Further interpretation of the corrosivity test results and associated corrosion design and construction recommendations are within the purview of a corrosion specialist. It is recommended that a qualified corrosion engineer be retained to review our corrosivity test results, to evaluate the general corrosion potential with respect to construction materials at the site, and to review the proposed design.

6.3.1. Reinforced Concrete

Laboratory tests indicate that the soil has less than 1,000 ppm or 0.1% of water soluble sulfate (SO_4) by weight. Based on ACI 318, concrete in contact with the site soils will have a sulfate exposure class S0. As a minimum, we recommend that Type II cement and a water-cement ratio of no greater than 0.50 be used on the project.

Test results indicate that the soil has less than 500 ppm of water soluble chlorides by weight and the potential for chloride attack of reinforcing steel in concrete structures and pipes in contact with soil is negligible.

6.3.2. Buried Metal

A factor for evaluating corrosivity to buried metal is electrical resistivity. The electrical resistivity of a soil is a measure of resistance to electrical current. Corrosion of buried metal is directly proportional to the flow of electrical current from the metal into the soil. As resistivity of the soil decreases, the corrosivity generally increases. Test results indicate the site soils have minimum electrical resistivity value of 3,600 ohm-centimeters. According to the County of Los Angeles (2014) criteria, the site soils are not corrosive.

Correlations between resistivity and corrosion potential published by the National Association of Corrosion Engineers (NACE, 1984) indicate that the soils have a mildly corrosive potential to buried metals. For design based on the NACE (1984) criteria, corrosion protection for metal in contact with site soils should be considered. Corrosion protection may include the use of epoxy or asphalt coatings.

6.4. Site Preparation and Earth Work

In general, earthwork should be performed in accordance with the recommendations presented in this report. Twining should be contacted for questions regarding the recommendations or guidelines presented herein.

6.4.1. Site Preparation

Site preparation should begin with the removal of utility lines, asphalt, concrete, vegetation, and other deleterious debris from areas to be graded. Tree stumps and roots should be removed to such a depth that organic material is not present. Clearing and grubbing should extend to the outside edges of the proposed excavation and fill areas. We recommend that unsuitable materials such as organic matter or oversized material be removed and disposed offsite. The debris and unsuitable material generated during clearing and grubbing should be removed from areas to be graded and disposed of at a legal dump site away from the project area.

6.4.2. Existing Underground Utilities

Existing underground utilities are expected in the project area, and some of them may cross proposed footings for the new buildings. Relocation of either the lines or footings to avoid the lines crossing the new footings is recommended as the footings will induce pressure on the lines. If relocation is not possible, existing utilities should be protected in place, and greater care should be exercised during excavation to avoid damaging the utilities. Utilities below a footing or the 1:1 plane projected out and down from the closest bottom edge of the footing should be encased. The encasement should have a minimum clearance of one inch all-around between the protected utility lines and the casing pipe. The casing pipe should be sealed at both ends.

Utilities in other areas should meet the minimum requirements for clearance and depth of cover for the County of Los Angeles; otherwise, encasement protection is recommended to provide a minimum clearance of one inch all-around between the protected utility lines and the casing pipe.

6.4.3. Temporary Excavations

Temporary excavations for the project are expected. We anticipate that unsurcharged excavations with vertical side slopes less than 4 feet high will generally be stable; however, if excavation extends to the sandy soil layers, some sloughing of cohesionless sandy materials encountered at the site should be expected.

Where space is available, temporary, un-surcharged excavation sides over 4 feet in height should be sloped no steeper than an inclination of 1.5H:1V (horizontal:vertical). Where sloped excavations are created, the tops of the slopes should be barricaded so that vehicles and storage loads are away from the top edge of the excavated slopes with a distance at least equal to the height of the slopes. A greater setback may be necessary when considering heavy vehicles, such as concrete trucks and cranes. Twining should be advised of such heavy vehicle loadings so that specific setback requirements can be established. If the temporary construction slopes are to be maintained during the rainy season, berms are recommended to be graded along the tops of the slopes in order to prevent runoff water from entering the excavation and eroding the slope faces.

Excavations shall not undermine the existing adjacent footings. Where space for sloped excavations is not available, slot-cut or temporary shoring may be utilized. Shoring recommendations are provided in Section 6.11.

Personnel from Twining should observe the excavations so that any necessary modifications based on variations in the encountered soil conditions can be made. All applicable safety requirements and regulations, including CalOSHA requirements, should be met. Stability of temporary excavations is the responsibility of the contractor.

6.4.4. Over-Excavation and Subgrade Preparation

Proposed structures may be supported by conventional shallow foundations. To minimize potential differential settlement, the foundations should all bear on at least 2 feet of non-expansive engineered fill or all on native soils, depending on embedment of foundations and thickness of undocumented fill encountered during construction. If the bottom of fill is deeper than the bottom of foundation, foundation excavation should extend to the bottom of undocumented fill or at least 2 feet below the bottom of foundation, whichever is deeper; if the bottom of fill is not deeper than the bottom of foundation, no over-excavation is necessary.

For minor structures and slabs-on-grade that are structurally separated from the building, the over-excavation should extend to at least 2 feet below the bottom of the footing of the minor structures and slabs-on-grade. Excavation for pavements and hardscape should be over-excavated at least 1 foot as measured from the bottom of the pavement or hardscape section. However, over-excavation may terminate at a shallower depth if native soils are encountered.

Where feasible, excavation should extend laterally beyond the foundation limits a minimum distance equal to 3 feet or the depth of over-excavation, whichever is greater. Excavation for other improvements (e.g., concrete walkways, flatwork, pavement) should extend laterally at least 2 feet beyond the limits of the improvements.

The extent and depths of all removal should be evaluated by Twining's representative in the field based on the materials exposed. Should excavations expose soft soils or soils considered unsuitable for use as fill by a Twining representative, additional removals may be recommended.

For example, deeper removal may be required in areas where soft, saturated, or organic materials are encountered.

The exposed excavation bottom should be evaluated and approved by Twining. Prior to placement of fill or placement of reinforcing steel or concrete for foundations, the bottom should be scarified to a minimum depth of 6 inches and moisture conditioned to achieve generally consistent moisture contents approximately 2 percent above the optimum moisture content. The scarified bottom should be compacted to at least 90 percent relative compaction in accordance with the latest version of ASTM Test Method D1557 and then evaluated and approved by Twining. However, the scarification and re-compaction may not be performed, if the bottom is firm and consists of undisturbed native soils and the relative compaction is tested at least 90%, in which case, the bottom should be rolled, and measures should be taken to prevent subgrade disturbance.

Fill and backfill materials should be compacted fill in accordance with Sections 6.4.5 and 6.4.6 of this report. Prior to placement of any fill, the geotechnical engineer or their representative should review the bottom of the excavation for conformance with the recommendations of this report.

6.4.5. Materials for Fill

In general, on-site soils expected to be excavated consist of lean clay with varying amounts of fines and a very low expansion potential and are considered suitable for use as fill. All fill soils should be free of organics, debris, rocks or lumps over three inches in largest dimension, other deleterious material, and not more than 40 percent larger than ¾ inch. Larger chunks, if generated during excavation, may be broken into acceptably sized pieces or may be disposed of offsite.

Any imported fill material should consist of granular soil having a “very low” expansion potential (i.e., expansion index of 20 or less). Import material should also have low corrosion potential (that is, chloride content less than 500 parts per million [ppm], soluble sulfate content of less than 0.1 percent, and pH of 5.5 or higher).

All fill soils should be evaluated and approved by a Twining representative prior to importing or filling.

6.4.6. Compacted Fill

Unless otherwise recommended, the exposed excavation bottom to receive fill should be prepared in accordance with Section 6.4.4 of this report. Prior to placement of compacted fill, the contractor should request Twining to evaluate the exposed excavation bottoms.

Compacted fill should be placed in horizontal lifts of approximately 8 to 10 inches in loose thickness, depending on the equipment used. Prior to compaction, each lift should be moisture conditioned, mixed, and then compacted by mechanical methods. The moisture content should be approximately 2 percent above the optimum moisture content. Fill materials should be compacted to a minimum relative compaction of 95 percent within the upper one foot below new vehicle trafficked pavement sections, and 90 percent in all other areas, unless indicated otherwise. The relative compaction should be determined by ASTM D1557. Successive lifts should be treated in the same manner until the desired finished grades are achieved.

6.4.7. Excavation Bottom Stability

Recommendations for stabilizing excavation bottoms should be based on evaluation in the field by the geotechnical consultant at the time of construction. In general, we anticipate that bottoms of the excavations will be stable and should provide suitable support for the proposed improvements. Conditions of the excavation bottom should be evaluated by Twining during the scarification and re-compaction efforts. Soft bottom conditions can be identified by surface yielding under rubber-tired equipment loading and the inability to achieve proper compaction. Recommendations for stabilizing excavation bottoms should be based on evaluation in the field by the geotechnical consultant at the time of construction.

6.4.8. Backfill for Utility Trench

When adjacent to any footings, utility trenches and pipes should be laid above an imaginary 1:1 (H:V) line projected down from the closest bottom edges of any footings. Otherwise, the pipe should be encased as described in Section 6.4.2 to accept the lateral effect from the footing load.

Utility trench excavations to receive backfill should be free of trash, debris or other unsatisfactory materials at the time of backfill placement. At locations where the trench bottom is yielding or otherwise unstable, pipe support may be improved by placing a minimum 6 inches of bedding materials. Remedial earthwork at the trench bottom should be performed where oversize materials (rocks or clods greater than 3 inches) are present. Removal of oversize materials to a depth of 6 inches below the bottom of the pipeline and replacement with fill material compacted to at least 90% relative compaction is recommended. The trench should be backfilled with bedding material extending to at least one foot over the top of pipe. The bedding material should be placed over the full width of the trench. After placement of the pipe, the bedding should be brought up uniformly on both sides of the pipe to reduce the potential for unbalanced loads. No void or uncompacted areas should be left beneath the pipe haunches.

The bedding materials may consist of clean sand having a minimum sand equivalent (SE) of 30, crushed rock, or 2-sack sand-cement slurry, and should meet the specifications provided in the latest edition of the “Greenbook” Standard Specifications for Public Works Construction. Samples of materials proposed for use as bedding material should be provided to the project geotechnical engineer for inspection and testing before the material is imported for use on the project. The onsite materials can only be used following the requirement of “Greenbook” bedding specification when the SE is not less than 30.

Above pipe bedding, trench backfill may be onsite soils and should not contain rocks or lumps over 3 inches in largest dimension. Larger chunks, if generated during excavation, may be broken into acceptably sized pieces or may be disposed offsite. The moisture content should be approximately 2 percent above the optimum moisture content. However, within the upper 12 inches of subgrade in areas of concrete slabs-on-grade, concrete pavement, and concrete flatwork, trench backfill should not consist of onsite soils with expansion potential greater than 20.

Backfill may be placed and compacted by mechanical means and should be compacted to 90 percent of the laboratory maximum dry density as per ASTM Standard D1557. Within pavement areas, the upper 12 inches of subgrade soils and the overlying aggregate base should be compacted to 95 percent.

Jetting or flooding of pipe bedding and backfill material is not recommended.

6.4.9. Rippability

The earth materials underlying the site should be generally excavatable with heavy-duty earthwork equipment in good working condition. Some gravels, cobbles and man-made debris should be anticipated.

6.4.10. Construction Dewatering

As discussed earlier, not groundwater was encountered to the maximum exploration depth of approximately 81.5 feet. Construction of the project is anticipated to occur above the groundwater. The possibility to encounter groundwater is low during earthwork and foundation preparation for the proposed structures, and the need for dewatering is not anticipated for construction of foundations and utility trenches.

If needed, considerations for construction dewatering should include anticipated drawdown, volume of pumping, potential for settlement of nearby structures, and groundwater discharge. Disposal of groundwater should be performed in accordance with guidelines of the Regional Water Quality Control Board.

6.4.11. Soil Export

In case the project generates excess soil in need of export from the site, evaluating the environmental quality of soil to be exported should be considered to protect the liability of both the sending and receiving parties. Environmental quality of the export soils could significantly affect soil export costs. Considering the potential liability, it is generally good practice to sample the soil that is planned for export regardless of the findings of a Phase I Environmental Site Assessment (ESA) and/or Preliminary Assessment (PA). Due diligence and project planning are key to managing costs associated with the export of soil. A qualified environmental professional should be consulted to assist with these efforts since each site is unique.

6.5. Foundation Recommendations for Proposed Building

Based upon the excavation/over-excavation and backfill recommendations, the proposed building may be supported on continuous strip footings or isolated footings designed in accordance with the geotechnical recommendations presented below. Structural design of foundations should be performed by the structural engineer and should conform to the 2019 California Building Code.

6.5.1. Bearing Capacity and Settlement

Proposed new footings for the building should be placed on the subgrade prepared in accordance with the requirements for the building pad as described in Section 6.4. The building load information is not currently available for our review. Based on our experience with similar projects, it is assumed that the maximum load will not exceed 150 kilo-pounds (kips) on isolated footings and 20 kips per foot on continuous footing. Geotechnical parameters presented in Table 5 may be used in the footing design. Twining should be contacted for footing dimensions, allowable bearing pressures, and settlements that are outside the indicated applicable ranges.

6.5.2. Lateral Resistance

Lateral loads may be resisted by footing base friction and by the passive resistance of the soils based on recommendations provided in Table 5. The total lateral resistance can be taken as the sum of the friction at the base of the footing and passive resistance. The upper one foot of soil should be neglected when calculating the passive resistance.

Table 5 - Geotechnical Design Parameters for Shallow Foundations

Minimum Footing Dimensions	• Width: 24 inches for square footings and 18 inches for continuous footings. • Minimum embedment: 24 inches measured from the lowest adjacent grade to the bottom of the footing. • Minimum thickness: 6 inches
Net Allowable Bearing Pressure	• Footings should all bear on at least 2 feet of engineered fill or all directly on undisturbed competent native soils. • Allowable bearing pressures of 3,000 and 4,000 pounds per square foot (psf) may be used for continuous and square footings, respectively. • The allowable bearing values may be increased by one-third for transient loads from wind or earthquake.
Estimated Static Settlement	• Approximately one inch of total settlement with differential settlement on the order of ½ inches over 30 feet for similarly loaded footings.
Allowable Coefficient of Friction Below Footings	0.3
Allowable Lateral Passive Resistance	• 240 pcf (equivalent fluid pressure), to a maximum pressure of 3,600 psf. • The upper one foot of soil should be neglected when calculating the passive resistance. • The allowable passive resistance value may be increased by one-third for transient loads such as wind or earthquake loads.

6.6. Foundation Recommendations for Minor Structures

Proposed minor structures structurally separated from the building may be supported on continuous strip footings or isolated footings designed in accordance with the geotechnical recommendations presented below. Structural design of foundations should be performed by the structural engineer and should conform to the 2019 CBC.

6.6.1. Bearing Capacity and Settlement

Proposed minor structures placed on subgrade prepared in accordance with the requirements as described in Section 6.4 may be designed using the geotechnical parameters presented in Table 6.

6.6.2. Lateral Resistance

Lateral loads may be resisted by footing base friction and by the passive resistance of the soils based on recommendations provided in Table 6. The total lateral resistance can be taken as the sum of the friction at the base of the footing and passive resistance.

Table 6 - Geotechnical Design Parameters for Shallow Foundations for Minor Structures

Minimum Footing Dimensions	• Width: 12 inches. • Minimum embedment: 12 inches measured from the lowest adjacent grade to the bottom of the footing. • Minimum thickness: 6 inches
Allowable Bearing Pressure	• An allowable bearing pressure of 1,500 psf may be used. • The allowable bearing values may be increased by one-third for transient loads from wind or earthquake.
Estimated Static Settlement	• Approximately one inch of total settlement with differential settlement on the order of ½ inches over 50 feet for similarly loaded footings. • The static settlement of the foundation system is expected to complete on initial application of loading.
Allowable Coefficient of Friction Below Footings	0.25.
Allowable Lateral Passive Resistance	• 100 pcf (equivalent fluid pressure), up to 1,500 psf. • The upper one foot of soil should be neglected when calculating the passive resistance. • The allowable passive resistance value may be increased by one-third for transient loads such as wind or earthquake loads.

6.7. Below-Grade Walls

For walls below grade, recommendations for wall lateral loads, backfill, and drainage are provided below. Foundation excavation, bearing capacity and lateral resistance for below-grade walls may be based on recommendations for the building provided in Sections 6.4 and 6.5 of this report. Below-grade walls should be designed to have a factor of safety of 1.5 for static stability and 1.1 for stability due to transient loads from wind or seismic.

6.7.1. Backfill and Drainage of Walls

The backfill material behind walls should consist of granular non-expansive material and be approved by the project geotechnical engineer. Based on the soil materials encountered during our exploration, most on-site soils will meet this requirement.

Wall backfill should be adequately drained. Adequate backfill drainage is essential to provide a free-drained backfill condition and to limit hydrostatic buildup behind walls. Drainage behind walls may be provided by a geosynthetic drainage composite such as TerraDrain, MiraDrain, or equivalent, attached to the outside perimeter of the wall and installed in accordance with the manufacturer's recommendations. The drainage system should meet the minimum requirements of Sections 1805.4.2 and 1805.4.3 of 2019 CBC.

In addition, walls sensitive to moisture buildup on the interior sides due to water migration from soils touching the walls should have appropriate waterproofing applied for the full height of the walls and meeting the minimum requirements of Section 1805.3 of 2019 CBC.

6.7.2. Lateral Earth Pressure

The values presented below assume that the supported grade is level, and Twining should be contacted for sloping backfill conditions. The recommended design lateral earth pressure is calculated assuming that a drainage system will be installed behind retaining walls in accordance with Sections 1805.4.2 and 1805.4.3 of 2019 CBC and that external hydrostatic pressure will not develop behind the walls. Where wall backfill does not have adequate drainage, the full hydrostatic pressure should be added to the lateral earth pressures provided below in design.

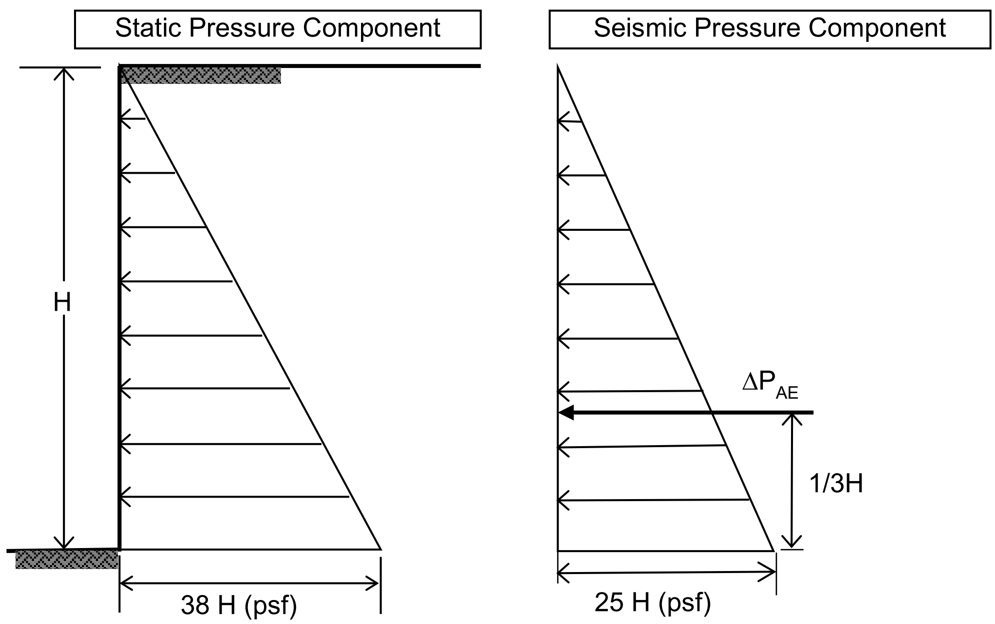
Walls that are free to move and rotate at the top (such as cantilevered walls) and have adequate drainage may be designed for the active earth pressure equivalent to a fluid weighing 38 pcf, if height of retained soil is no more than 15 feet.

Walls that are restricted to move horizontally at the top (such as by a floor deck) and have adequate drainage may be designed for the "at-rest" earth pressure equivalent to a fluid weighing 72 pcf.

Vertical surcharge loads within a 1:1 plane projected from the bottom of the wall distributed over retained soils should be considered as additional uniform horizontal pressures acting on the wall. These additional pressures can be estimated as approximately 41% and 58% of the magnitude of the vertical surcharge pressures for the "active" and "at-rest" conditions, respectively.

6.7.3. Seismic Lateral Earth Pressure

Walls retaining more than 6 feet high earth should be designed for seismic lateral earth pressure. The seismic pressure distribution may be considered a triangle with the maximum pressure at the bottom. We estimated the seismic earth pressure increment for walls retaining level ground based on Seed and Whitman (1970) and a horizontal seismic coefficient (k_h) equivalent to one-half of two-thirds of PGA_M provided in Table 1. The following combination of static and incremental seismic pressures shown in the following diagram may be used for seismic design for both cantilever and restrained walls.



where H is in feet and is no more than 15 feet.

Diagram 1 - Seismic Earth Pressure Distribution on Walls

6.8. Modulus of Subgrade Reaction

The modulus of subgrade reaction k for combined footing design and slabs-on-grade may be obtained from the following equation.

$$K = \frac{k_1}{B}\left(\frac{2L + B}{3L}\right)$$

where: k_1 = modulus for a 1-foot by 1-foot plate = 100 pounds per cubic inch (pci);
B = width of combined footing or slab in feet; and
L = length of combined footing or slab in feet.

6.9. Pole Foundations

Pole foundations for flagpoles, fences, and signposts may be designed using an allowable skin friction of 450 psf, and an allowable end bearing resistance of 4,000 psf. This value may be increased by 33 percent for seismic or transient wind load. The upper 2 feet of the foundation frictional resistance should be neglected.

Lateral resistance for conditions with and without lateral constraint provided at the ground surface conditions are provided below based on 2019 CBC.

6.9.1. Non-Constrained Ground

The embedment of pole foundations where no lateral constraint is provided at or above the ground surface should be calculated using Equation 18-1 of 2019 CBC (shown below) or a minimum 3 feet below the ground surface, whichever is deeper.

$$D = \frac{A}{2}\left(1 + \sqrt{1 + \frac{4.36h}{A}}\right) \qquad \text{(Equation 18-1 of 2019 CBC)}$$

where:

A = $2.34P/(S_1 * b)$

b = Diameter of round post or footing or diagonal dimension of square post or footing, feet

d = Depth of embedment in earth in feet but not over 12 feet for purpose of computing lateral pressure.

H = Distance in feet from ground surface to point of application of "P".

P = Applied lateral force in pounds.

S_1 = Allowable lateral soil-bearing pressure based on a depth of one-third the depth of embedment in psf.

An allowable passive earth pressure of 240 pcf up to a maximum of 3,600 psf may be used for design provided the upper one foot of passive resistance is neglected in the structural design. Pole foundations spaced at least 3 diameters of the maximum pole foundation may be designed using an allowable lateral resistance equal to 2 times of the allowable passive pressure.

6.9.2. Constrained Ground

The embedment of pole foundations where lateral constraint is provided at the ground surface, such as by a rigid floor or pavement, should be calculated using Equation 18-2 of 2019 CBC (shown below) or a minimum 3 feet below the ground surface, whichever is deeper.

$$D = \sqrt{\frac{4.25Ph}{S_3 b}} \qquad \text{(Equation 18-2 of 2019 CBC)}$$

where:

b = Diameter of round post or footing or diagonal dimension of square post or footing, feet

d = Depth of embedment in earth in feet but not over 12 feet for purpose of computing lateral pressure.

H = Distance in feet from ground surface to point of application of "P".

P = Applied lateral force in pounds.

S_3 = Allowable lateral soil-bearing pressure based on a depth of one-third the depth of embedment in psf.

An allowable passive earth pressure of 240 pcf up to a maximum of 3,600 psf may be used for design provided the upper one foot of passive resistance is neglected in the structural design. Pole foundations spaced at least 3 diameters of the maximum pole foundation may be designed using an allowable lateral resistance equal to 2 times of the allowable passive pressure.

6.10. Concrete Slabs

Slabs should be supported on non-expansive engineered fill in accordance with Section 6.4 of this report. For design of concrete slabs, the subgrade modulus k calculated from the equation in Section 6.8 may be used.

Floor slabs should be designed and reinforced in accordance with the structural engineer's recommendations. However, for slabs not supporting heavy loads, we recommend that the concrete should have a thickness of at least 4 inches, a 28-day compressive strength of at least 3,000 pounds per square inch (psi), a water-cement ratio of 0.50 or less, and a slump of 4 inches or less. Slabs should be reinforced with at least No. 3 reinforcing bars placed longitudinally at 18 inches on center. The reinforcement should extend through the control joints to reduce the potential for differential movement. Control joints should be constructed in accordance with recommendations from the structural engineer or architect. For slabs supporting equipment, a minimum thickness of 5 inches is recommended. Additional thickness and reinforcement recommendations may be provided by the structural engineer.

The topmost 8 inches below the slab subgrade should be maintained in a moisture condition of approximately 0 to 2 percent above optimum moisture content. The slab subgrade should be tested for moisture and compaction immediately prior to placement of the gravel or sand base, if any. All underslab materials should be adequately compacted prior to the placement of concrete. Care should be taken during placement of the concrete to prevent displacement of the underslab materials. The underslab material should be dry or damp and should not be saturated prior to the placement of concrete. The concrete slab should be allowed to cure properly and should be tested for moisture transmission prior to placing vinyl or other moisture-sensitive floor covering. In moisture sensitive areas, the floor slabs should be dampproofed in accordance with Section 1805A.2 of 2019 CBC. Specific recommendations can be provided by a waterproofing consultant.

Table 7 provides general recommendations for various levels of protection against vapor transmission through concrete floor slabs placed over a properly prepared subgrade. Care should be taken not to puncture the plastic membrane during placement of the membrane itself and the overlying silty sand.

The above recommendations are intended to reduce the potential for cracking of slabs; however, even with the incorporation of the recommendations presented herein, slabs may still exhibit some cracking. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics.

Table 7 - Options for Subgrade Preparation below Concrete Floor Slabs

Primary Objective	Recommendation
Enhanced protection against vapor transmission	• Concrete floor slab-on-grade placed directly on a 15-mil-thick moisture vapor retarder that meets the requirements of ASTM E1745 Class C (Stego Wrap or similar) • The moisture vapor retarder membrane should be placed directly on the subgrade (ACI302.1R-67); if required for either leveling of the subgrade or for protection of the membrane from protruding gravel, then place about 2 inches of silty sand[1] under the membrane
Above-standard protection against vapor transmission	This option is available if the slab perimeter is bordered by continuous footings at least 24 inches deep, OR if the area adjacent and extending at least 10 feet from the slab is covered by hardscape without planters: • 2 inches of dry silty sand[1]; over • Waterproofing plastic membrane 10 mils in thickness; over • At least 4 inches of ¾-inch crushed rock[2] or clean gravel[3] to act as a capillary break
Standard protection against vapor transmission	• 2 inches of dry silty sand[1]; over • Waterproofing plastic membrane 10 mils in thickness • If required for either leveling of the subgrade or for protection of the membrane from protruding gravel, place at least 2 inches of silty sand[1] under the membrane.

Notes:

[1] The silty sand should have a gradation between approximately 15 and 40 percent passing the No. 200 sieve and a plasticity index of less than 4.

[2] The ¾-inch crushed rock should conform to Section 200-1.2 of the latest edition of the "Greenbook" Standard Specifications for Public Works Construction (Public Works Standards, Inc., 2012).

[3] The gravel should contain less than 10 percent of material passing the No. 4 sieve and less than 3 percent passing the No. 200 sieve.

6.11. Temporary Shoring

If the project involves excavations that lack sufficient space for sloped excavations, cantilevered shoring or braced- or tieback shoring should be considered and designed.

For vertical excavations less than approximately 15 feet in height, cantilevered shoring may be used. Where cantilevered shoring is used for deeper excavations, the total deflection at the top of the wall tends to exceed acceptable magnitudes. Shoring of excavations deeper than approximately 15 feet should be accomplished with the aid of internal bracing or tieback earth anchors.

The shoring design should be provided by a California Registered Civil Engineer experienced in the design and construction of shoring under similar conditions. Once the final excavation and shoring plans are complete, the plans and the design should be reviewed by Twining Laboratories for conformance with the design intent and recommendations. Further, the shoring system should satisfy applicable requirements of CalOSHA.

6.11.1. Lateral Earth Pressures

For design of cantilevered shoring for excavations less than 15 feet in height, a triangular distribution of lateral earth pressure may be used. It may be assumed that the drained soils, with a level surface behind the cantilevered shoring, will exert an equivalent fluid pressure of 38 pcf.

For the design of braced- or tieback-shoring, a rectangular pressure distribution where the pressure may be used. The design pressure should be 25H psf, where H is the retained soil height in feet.

Any surcharge (live, including traffic, or dead load) located within a 1:1 plane projected upward from the base of the shored excavation, including adjacent structures, should be added to the lateral earth pressures. The lateral contribution of a uniform surcharge load located immediately behind the temporary shoring may be calculated by multiplying the vertical surcharge pressure by 41% for cantilevered shoring and 58% for braced- or tieback- shoring. Lateral load contributions of surcharges located at a distance behind the shored wall may be provided once the load configurations and layouts are known. As a minimum, a 250 psf vertical uniform surcharge is recommended to account for nominal construction and/or traffic loads. More detailed lateral pressure and loading information can be provided, if needed, for specific loading scenarios as recognized through the design process.

6.11.2. Soldier Pile Design

The soldier piles for support of shoring should be designed in accordance with the geotechnical parameters presented in Table 8. Soldier piles should be spaced no closer than 3D on center, where D is the diameter of the drilled shaft for the soldier piles. Soldier piles may consist of either cast-in-place concrete caissons or pre-drilled steel beams encased in concrete (below the bottom of the excavation) and slurry (above the bottom of the excavation).

Table 8 - Geotechnical Design Parameters for Soldier Piles

The allowable lateral resistance of an isolated soldier pile drilled into the on-site soils can be calculated using equivalent fluid pressure (EFP)	240 pcf
Increase (multiplier) of the allowable lateral passive resistance due to arching (this value is applicable for soldier piles that are spaced no closer than 3 diameters)	2

The downward component of a tieback anchor load transferred to the soldier pile may be supported by frictional resistance between the soldier piles and the retained earth, and the skin fiction of the pile shaft below finished excavation grade. The allowable frictional resistance between the soldier piles and the retained earth may be taken as 200 psf. The allowable downward capacity of a soldier pile below the excavated level may be estimated using an average allowable unit skin friction of 25 psf per foot below bottom of excavation. This allowable unit skin friction incorporates a factor of safety of 1.5. The upper 1.5D should be neglected when calculating the axial capacity below the excavated level.

Continuous timber lagging should be used between the soldier piles. If treated timber is used, the lagging may remain in place. To develop the full lateral resistance, provisions should be taken to assure firm contact between the soldier piles and the soils; for this, we recommend that 1-½-sack sand-cement slurry infill behind the lagging be used. For drilled piles, we recommend that piles adjacent to one another be drilled alternately on different days to minimize disturbance to the open excavations.

Drilling of soldier pile shafts can be accomplished using conventional drilling equipment. Caving should be anticipated where layers of clean sand or silty sand occurs. In the event of soil caving, it may be necessary to use casing and/or drilling mud to permit the installation of the soldier piles. Drilled holes for soldier piles should not be left open overnight. Concrete for piles should be placed immediately after the drilling of the hole and placement of the steel pile (or rebar cage) is complete. The concrete should be pumped to the bottom of the drilled shaft using a tremie. Once concrete pumping is initiated, the bottom of the tremie should remain below the surface of the concrete to prevent contamination of the concrete by soil inclusions. If steel casing is used, the casing should be removed as the concrete is placed. The concrete placed in the soldier pile excavations may be a lean mix concrete above the elevation of the bottom of the excavation. However, the concrete that is placed in the portion of the soldier pile that is below the deepest planned excavated level should have a minimum 28-day compressive strength of at least 2,500 psi. The contractor may also consider the use of driven piles or piles that are vibrated into place in lieu of drilled piles to address potential issues related to caving of drilled shafts.

6.11.3. Tieback Design

Excavations deeper than 15 feet may require tieback anchors to be used to resist lateral loads. For design purposes, it may be assumed that the failure wedge adjacent to the shoring is defined by a plane projected up at approximately 30 degrees from the vertical from the toe of the wall. The anchors should extend at least 15 feet beyond the potential failure wedge; however, the shoring engineer should evaluate the bonded length required beyond the failure wedge based on the loading on the shoring and the allowable skin friction provided. The bonded length should commence no less than 3 feet beyond the failure wedge.

We recommend using a soil/anchor bond friction of 450 psf along the anchors in the bonded zone. Only friction developed beyond the active wedge should be considered when determining the tieback resistance. If the anchors are spaced at least 6 feet on center, no reduction in the capacity of the anchors need be considered due to group action.

As the tieback shoring system is intended for temporary use, provisions should be made in the design to de-tension and abandon the tiebacks when the subgrade walls are able to support the lateral loads.

6.11.4. Anchor Installation

The anchors may be installed at angles of 15 to 30 degrees below the horizontal. Caving may occur during the drilling of tiebacks if loose cohesionless materials are encountered. The contractor should implement appropriate measures to stabilize the drilled hole such as the installation of steel casing for loose cohesionless materials or the use of drilling mud. The anchors should be filled with concrete placed by pumping from the tip out. The portion of the anchor tendons within the failure wedge should be sleeved in plastic. If the anchor tendons are sleeved, it is acceptable to grout the entire length of the anchor.

6.11.5. Lagging and Sheeting

Continuous lagging will be required between the soldier piles. The soldier piles and anchors should be designed for the full anticipated lateral pressure. However, where lagging is relatively flexible to wales or soldier beams, the pressure on the lagging will be less due to arching in the soils. We recommend that the lagging be designed for a semi-circular distribution of earth pressure where the maximum pressure is 500 psf at the mid-line between soldier piles, and 0 psf at the soldier piles.

6.11.6. Lateral Deflection and Settlement

Excessive deflection could result in settlement or undermining of surrounding structures. Shoring should be adequately designed, installed, and monitored to limit the amount of lateral deflection of the shoring system and settlement behind the shoring to the allowable values of adjacent structures and improvements. The amount of deflection of the shoring system and the allowable deflections and settlements should be determined by the shoring designer. The allowable deflections and settlements should be based on the proximity of adjacent structures and improvements and the potential negative effects on those structures. If it is desired to reduce the deflection, a greater lateral pressure could be used in shoring design. If greater than anticipated deflection occurs during construction, additional bracing or tiebacks may be necessary to minimize deflection of existing adjacent improvements.

Settlement of structures or facilities founded adjacent to the shoring will occur in proportion to both the distance between the shoring and the facilities, and the amount of horizontal deflection of the shoring system. The vertical settlement will be a maximum at the shoring face and decrease as the horizontal distance from the shoring increases. Beyond a distance from the shoring equal to the height of the shoring, the settlement is expected to be negligible. The maximum vertical settlement is expected to be about 75 percent of the maximum horizontal deflection on top of the shoring system.

6.11.7. Monitoring

For excavations in close proximity to existing improvements, some means of monitoring the performance of the shoring system is recommended. Monitoring should consist of periodic surveying of lateral and vertical locations at the tops of all soldier piles. We will be pleased to discuss this further with the design consultants and the contractor when the design of the shoring system has been finalized.

6.12. Pavement Recommendations

Pavement section should be constructed on top of properly prepared subgrade in accordance with Section 6.4 of this report and aggregate base (AB) section compacted to 95 percent of the maximum dry density in accordance with ASTM D1557.

We performed laboratory R-value testing for preliminary pavement section design. The test indicates an R value of 10, which was used in our pavement structural calculations. Sections 6.12.1 and 6.12.2 present our recommendations for preliminary design of flexible and rigid pavement sections, respectively. Final pavement design should be based on field observations, additional R-value tests during construction should the materials exposed differ than what is expected based on our field exploration, and the anticipated traffic index as determined by the project civil engineer.

6.12.1. Flexible Pavement Design

Our flexible pavement structural design is in accordance with Chapter 630 of the Caltrans Highway Design Manual, which is based on a relationship between the gravel equivalent (GE) of the pavement structural materials, the traffic index (TI), and the R-value of the underlying subgrade soil. For preliminary design of flexible pavement section, Table 9 provides recommended minimum thicknesses for hot mix asphalt (HMA) and aggregate base sections for different traffic indices.

Table 9 – Recommended Minimum HMA and Base Section Thicknesses

Traffic Index	5.0	6.0	7.0
HMA Thickness (in)	4	5	6
Aggregate Base Thickness (in)	7	9	12

6.12.2. Rigid Pavement Design

For preliminary design of rigid pavement section, Table 10 provides recommended minimum thicknesses for Portland cement concrete (PCC) pavement section and Class 2 Aggregate Base (AB) section for different traffic indices. The recommended values are based on a minimum 28-day concrete compressive strength of 3,500 psi. Positive drainage should be provided away from all pavement areas to prevent seepage of surface and/or subsurface water into the pavement base and/or subgrade.

Table 10 – Recommended Minimum Rigid Pavement Thicknesses

Traffic Index	5.0	6.0	7.0
JPCP Thickness (in)	5	6	7
Aggregate Base Thickness (in)	6	6	6

6.13. Stormwater Infiltration

Percolation testing will be required based on the actual location and depth of the planned system. The design of stormwater infiltration facility should be based on percolation test results with an appropriate reduction factor to account for test method, site variability, and long-term siltation.

For preliminary design of stormwater infiltration devices, we performed percolation testing at the site at two locations at a depth of approximately 5 feet bgs. Details of the percolation tests are presented in Appendix A. Infiltration rates with a reduction factor of 3 from our percolation tests are summarized in Table 11. The results indicate that stormwater infiltration is not feasible at the P-1 and P-2 locations and depth due to the rate being less than the required minimum rate of 0.3 inches per hour. However, based on subsurface conditions encountered in the other borings, additional percolation tests at approximately 15 feet bgs may be performed to study the feasibility of stormwater infiltration at greater depth.

Table 11 – Infiltration Rate with a Reduction Factor of 3

Location	Depth (feet)	Infiltration Rate (in/hour)
P-1	5	0.03
P-2	5	0.04

Proposed infiltration facility should have a minimum setback from property lines and foundations recommended in Table 12. In addition, the bottom of the infiltration facility should be at least 10 feet above the seasonal high groundwater, according to the requirements of Los Angeles County Low Impact Development Standards Manual (2014).

Table 12 – Recommended Minimum Infiltration Facility Setback

Setback from	Distance
Property lines & public right of way	5 feet
Foundations	the greater of 15 feet or a 1:1 plane drawn up from the bottom of foundation
Seasonal high groundwater	10 feet minimum depth from invert of infiltration device
Face of slope	the greater of 5 feet or one half of the slope height
Water wells	100 feet

6.14. Drainage Control

The control of surface water is essential to the satisfactory performance of the building and site improvements. Surface water should be controlled so that conditions of uniform moisture are maintained beneath the improvements, even during periods of heavy rainfall. The following recommendations are considered minimal:

- Ponding and areas of low flow gradients should be avoided.
- If bare soil within 5 feet of the structure is not avoidable, then a gradient of 5 percent or more should be provided sloping away from the improvement. Corresponding paved surfaces should be provided with a gradient of at least 1 percent.
- The remainder of the unpaved areas should be provided with a drainage gradient of at least 2 percent.
- Positive drainage devices, such as graded swales, paved ditches, and/or catch basins should be employed to accumulate and to convey water to appropriate discharge points.
- Concrete walks and flatwork should not obstruct the free flow of surface water.
- Brick flatwork should be sealed by mortar or be placed over an impermeable membrane.
- Area drains should be recessed below grade to allow free flow of water into the basin.
- Enclosed raised planters should be sealed at the bottom and provided with an ample flow gradient to a drainage device. Recessed planters and landscaped areas should be provided with area inlet and subsurface drainpipes.
- Planters should not be located adjacent to the structures wherever possible. If planters are to be located adjacent to the structures, the planters should be positively sealed, should incorporate a subdrain, and should be provided with free discharge capacity to a drainage device.
- Planting areas at grade should be provided with positive drainage. Wherever possible, the grade of exposed soil areas should be established above adjacent paved grades. Drainage devices and curbing should be provided to prevent runoff from adjacent pavement or walks into planted areas.
- Gutter and downspout systems should be provided to capture discharge from roof areas. The accumulated roof water should be conveyed to off-site disposal areas by a pipe or concrete swale system.

Landscape watering should be performed judiciously to preclude either soaking or desiccation of soils. The watering should be such that it just sustains plant growth without excessive watering. Sprinkler systems should be checked periodically to detect leakage and they should be turned off during the rainy season.

7. DESIGN REVIEW AND CONSTRUCTION MONITORING

Geotechnical review of plans and specifications is of paramount importance in engineering practice. The poor performance of many structures has been attributed to inadequate geotechnical review of construction documents. Additionally, observation and testing of the subgrade will be important to the performance of the proposed development. The following sections present our recommendations relative to the review of construction documents and the monitoring of construction activities.

7.1. Plans and Specifications

The design plans and specifications should be reviewed by Twining, Inc. prior to bidding and construction, as the geotechnical recommendations may need to be reevaluated in light of the actual design configuration and loads. This review is necessary to evaluate whether the recommendations contained in this report and future reports have been properly incorporated into the project plans and specifications. Based on the work already performed, this office is best qualified to provide such review.

7.2. Preconstruction Surveys

We recommend that preconstruction surveys be performed on the adjacent improvements prior to commencement of excavation activities for the subject project. The surveys should include written and photographic (or videographic) documentation of the existing conditions, as well as performance of floor level surveys or establishment of elevation monuments. Documentation of other structures and sensitive instruments within approximately 50 feet of the excavation(s) should also be performed.

7.3. Construction Monitoring

Site preparation, removal of unsuitable soils, assessment of imported fill materials, fill placement, foundation installation, and other site grading operations should be observed and tested, as appropriate. The substrata exposed during the construction may differ from that encountered in the test excavations. Continuous observation by a representative of Twining, Inc. during construction allows for evaluation of the soil conditions as they are encountered and allows the opportunity to recommend appropriate revisions where necessary.

8. LIMITATIONS

The recommendations and opinions expressed in this report are based on Twining, Inc.'s review of available background documents, on information obtained from field explorations, and on laboratory testing. It should be noted that this study did not evaluate the possible presence of hazardous materials on any portion of the site. In the event that any of our recommendations conflict with recommendations provided by other design professionals, we should be contacted to aid in resolving the discrepancy.

Due to the limited nature of our field explorations, conditions not observed and described in this report may be present on the site. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation and laboratory testing can be performed upon request. It should be understood that conditions different from those anticipated in this report may be encountered during grading operations, for example, the extent of removal of unsuitable soil, and that additional effort may be required to mitigate them.

Site conditions, including groundwater elevation, can change with time as a result of natural processes or the activities of man at the subject site or at nearby sites. Changes to the applicable laws, regulations, codes, and standards of practice may occur as a result of 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 Twining, Inc. has no control.

Twining's recommendations for this site are, to a high degree, dependent upon appropriate quality control of subgrade preparation, fill placement, and foundation construction. Accordingly, the recommendations are made contingent upon the opportunity for Twining to observe grading operations and foundation excavations for the proposed construction. If parties other than Twining are engaged to provide such services, such parties must be notified that they will be required to assume complete responsibility as the geotechnical engineer of record for the geotechnical phase of the project by concurring with the recommendations in this report and/or by providing alternative recommendations.

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. Twining should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report has been prepared for the exclusive use by the client and its agents for specific application to the proposed project. Land use, site conditions, or other factors may change over time, and additional work may be required with the passage of time. Based on the intended use of this report and the nature of the new project, Twining may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the Client or anyone else will release Twining from any liability resulting from the use of this report by any unauthorized party.

Twining performed its evaluation using the degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical professionals with experience in this area in similar soil conditions. No other warranty, either express or implied, is made as to the conclusions and recommendations contained in this report.

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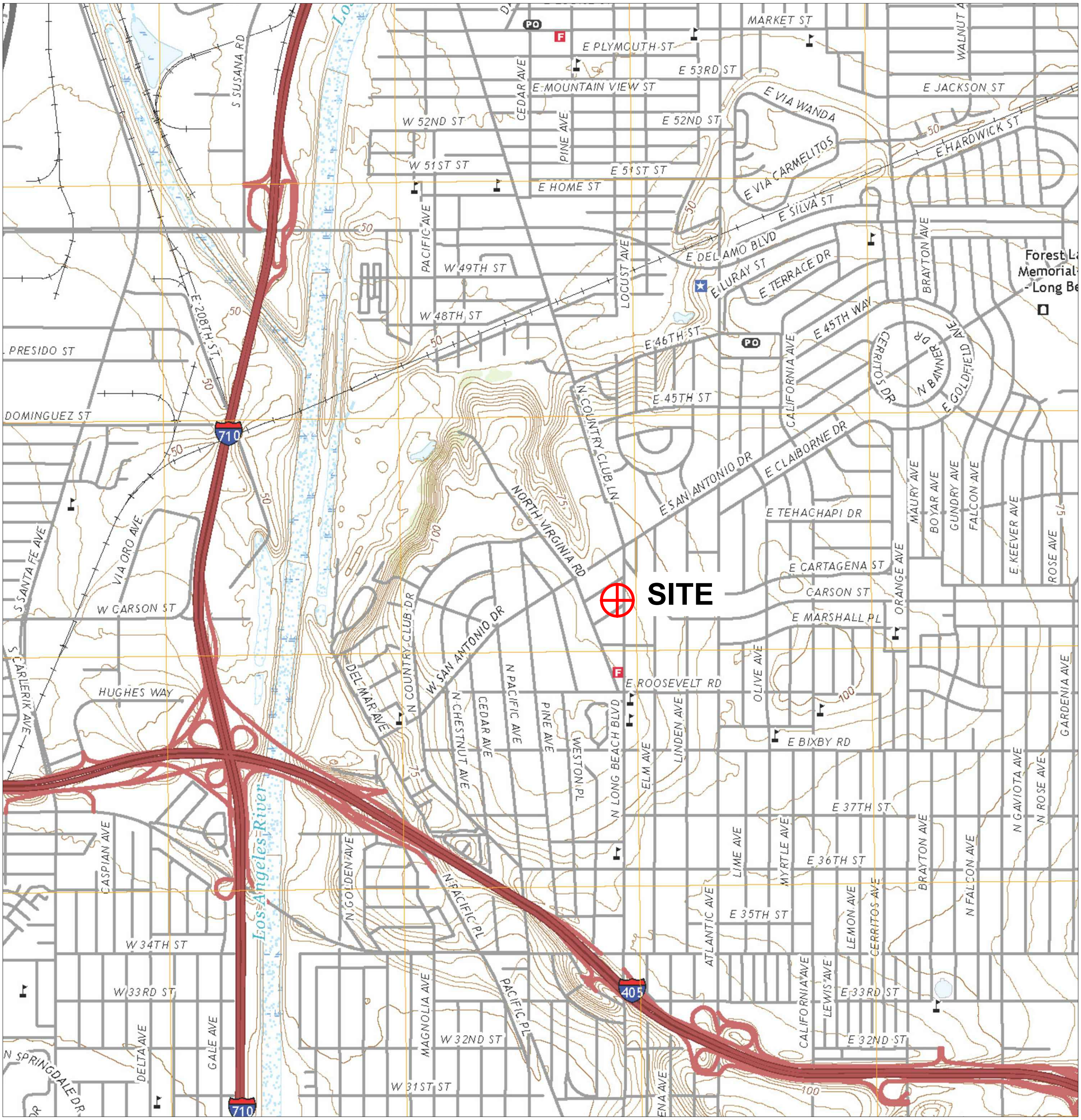
FIGURES



N

APPROXIMATE LOCATION OF PROJECT

SCALE IN FEET

0 2000 4000

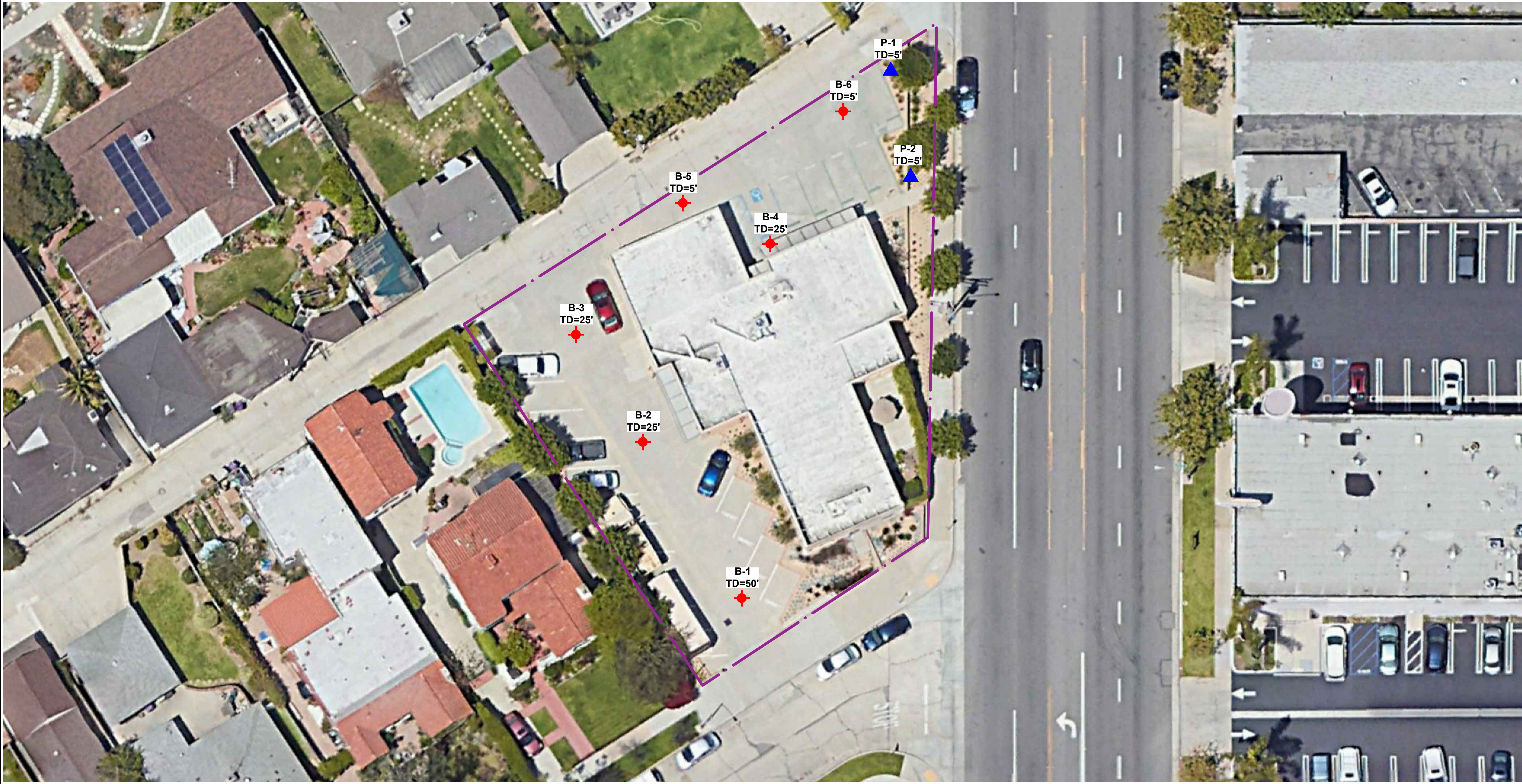
REFERENCE: USGS (2018)

TWINING

SITE LOCATION MAP

FIRE STATION NO. 9
4101 LONG BEACH BOULEVARD
LONG BEACH, CA

PROJECT NO.	REPORT DATE	
210377.1	July 2021	FIGURE 1



0 30 60 feet

NOTE: ALL DIMENSIONS AND LOCATIONS ARE APPROXIMATE

LEGEND

B-1 TD=50' — APPROXIMATE LOCATION OF PROPOSED BORING BY TWINING TOTAL DEPTH IN FEET

P-1 TD=5' — APPROXIMATE LOCATION OF PROPOSED PERCOLATION TEST BY TWINING TOTAL DEPTH IN FEET

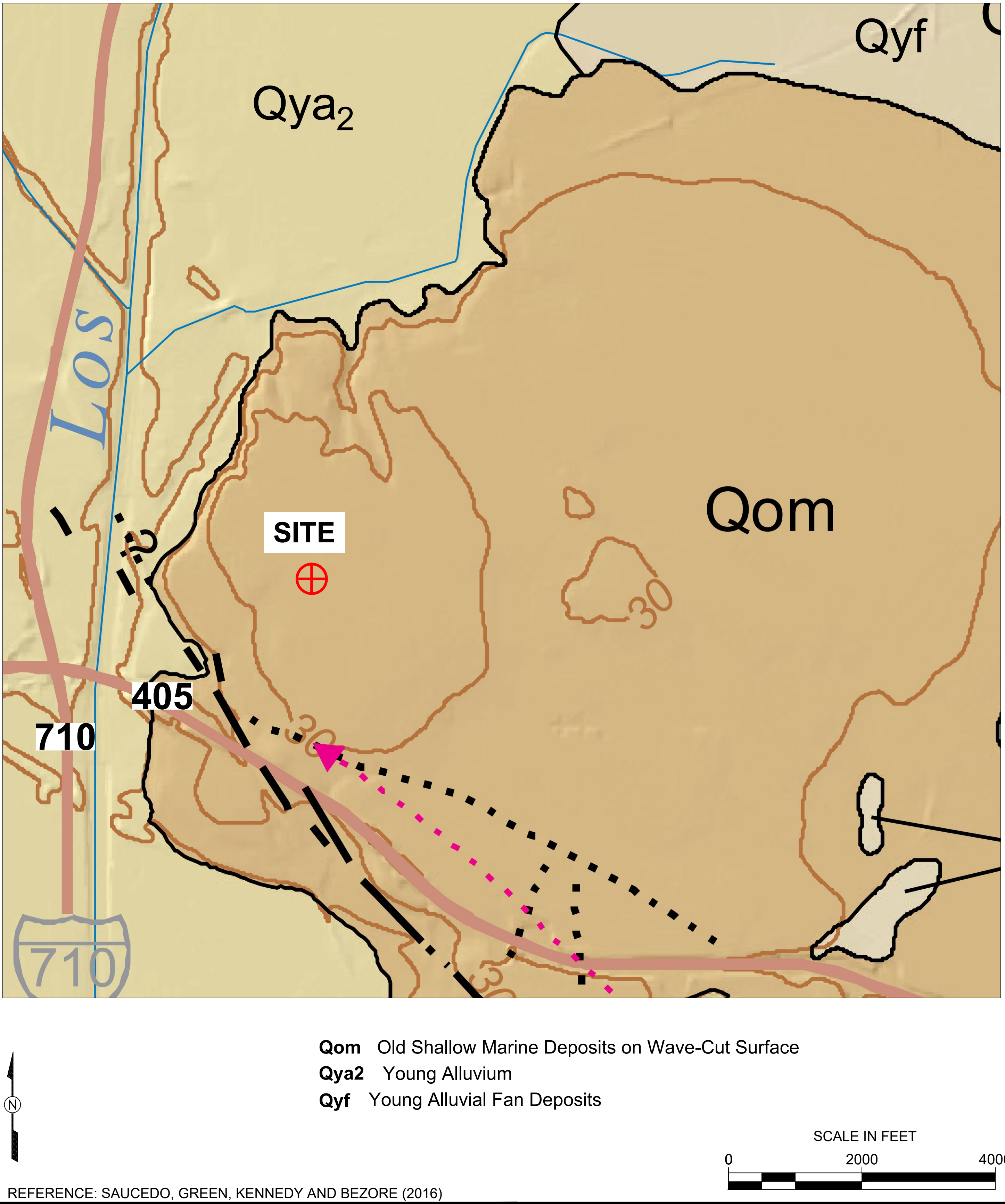
APPROXIMATE LOCATION OF PROPERTY LINE

TWINING

N

REFERENCE: GOOGLE EARTH (2021)

SITE PLAN AND BORING LOCATION MAP		
FIRESTATION NO. 9 4101 LONG BEACH BOULEVARD LONG BEACH, CA		
PROJECT No. 210377.1	REPORT DATE July 2021	FIGURE 2



Qom Old Shallow Marine Deposits on Wave-Cut Surface

Qya2 Young Alluvium

Qyf Young Alluvial Fan Deposits

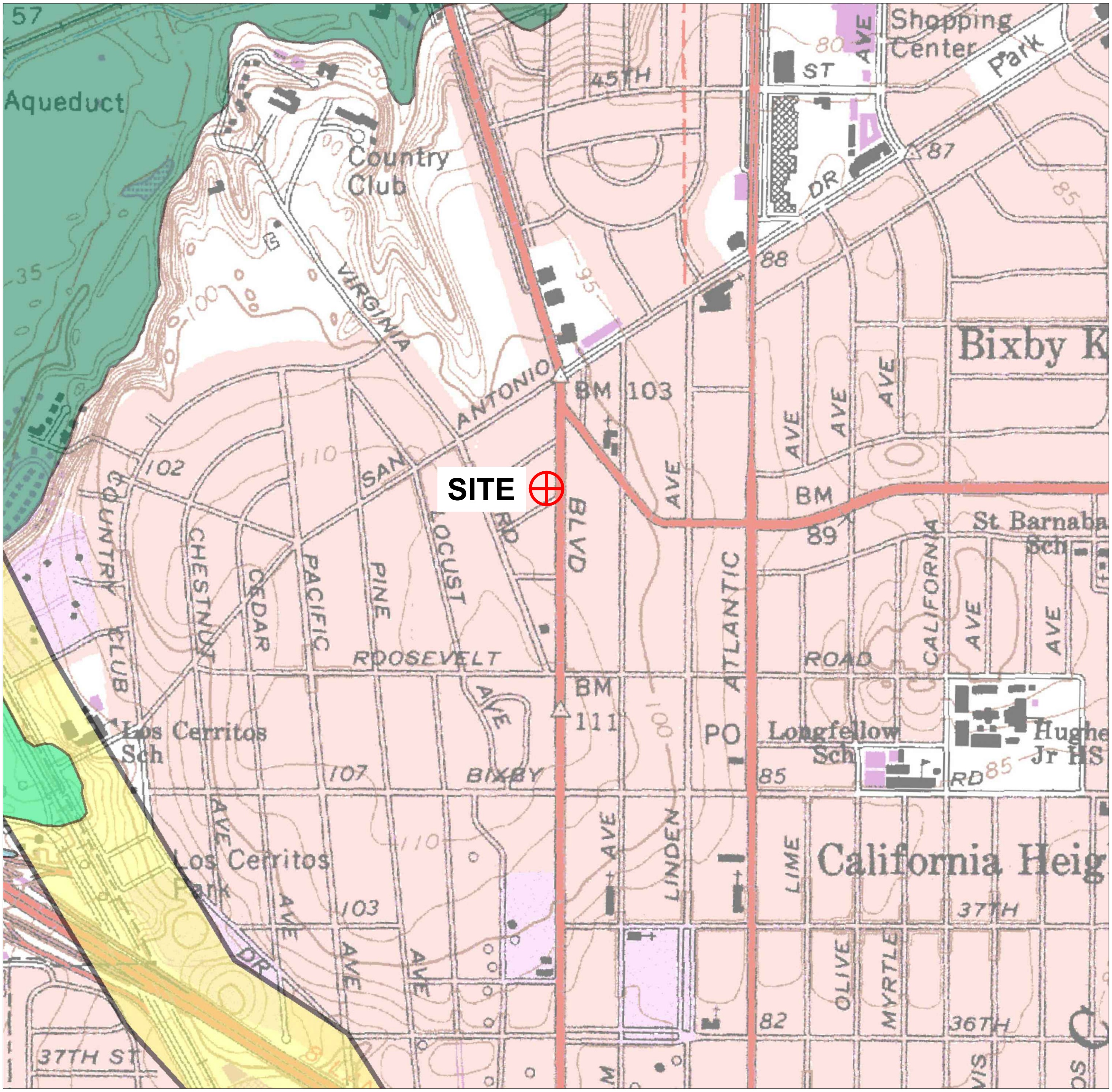
REFERENCE: SAUCEDO, GREEN, KENNEDY AND BEZORE (2016)

TWINING

GEOLOGIC MAP

FIRE STATION NO. 9
4101 LONG BEACH BOULEVARD
LONG BEACH, CA

PROJECT NO.	REPORT DATE	FIGURE 3
210377.1	July 2021	



MAP EXPLANATION

EARTHQUAKE FAULT ZONES

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.

1906 C ? ?

Active Fault Traces
Faults considered to have been active during Holocene time and to have potential for surface rupture: Solid Line in Black or Red where Accurately Located; Long Dash in Black or Solid Line in Purple where Approximately Located; Short Dash in Black or Solid Line in Orange where Inferred; Dotted Line in Black or Solid Line in Rose where Concealed; Query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by fault creep.

SEISMIC HAZARD ZONES

Liquefaction Zones
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslide Zones
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

OVERLAPPING EARTHQUAKE FAULT AND SEISMIC HAZARD ZONES

Overlap of Earthquake Fault Zone and Liquefaction Zone
Areas that are covered by both Earthquake Fault Zone and Liquefaction Zone.

Overlap of Earthquake Fault Zone and Earthquake-Induced Landslide Zone
Areas that are covered by both Earthquake Fault Zone and Earthquake-Induced Landslide Zone.

Note: Mitigation methods differ for each zone –
AP Act only allows avoidance; Seismic Hazard Mapping Act allows mitigation by engineering/geotechnical design as well as avoidance.

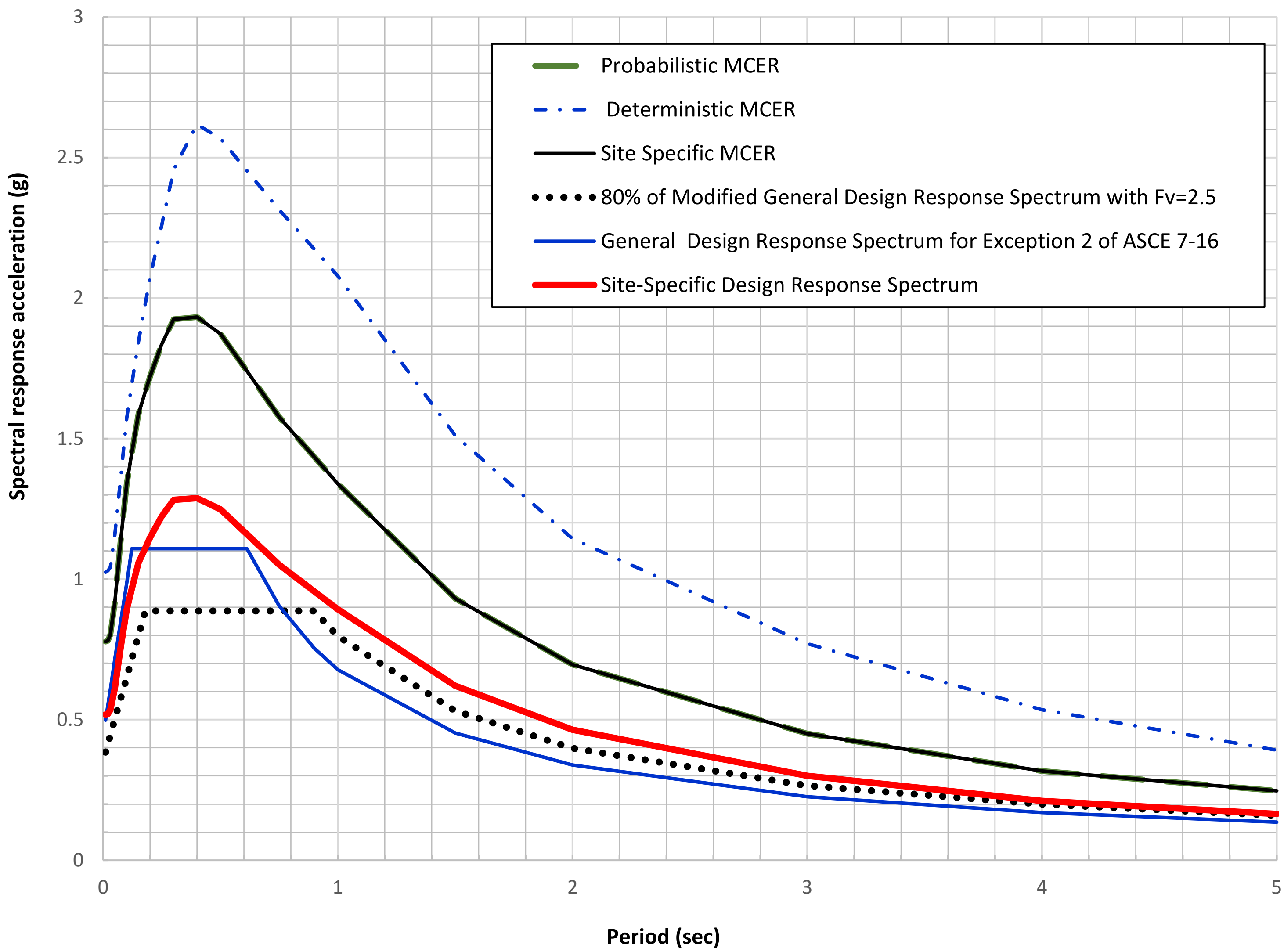
REFERENCE: CGS (1999)

TWINING

SEISMIC HAZARD ZONES MAP		
FIRE STATION NO.9 4101 LONG BEACH BOULEVARD LONG BEACH, CA		
PROJECT NO. 210377.1	REPORT DATE July 2021	FIGURE 4

PERIOD (seconds)	SITE-SPECIFIC DESIGN SPECTRAL ACCELERATION Sa, (g)
0.01	0.518
0.02	0.521
0.03	0.535
0.05	0.611
0.075	0.760
0.1	0.893
0.122	0.967
0.15	1.058
0.180	1.111
0.2	1.148
0.25	1.224
0.3	1.283

PERIOD (seconds)	SITE-SPECIFIC DESIGN SPECTRAL ACCELERATION Sa, (g)
0.4	1.288
0.5	1.249
0.612	1.160
0.75	1.051
0.899	0.957
1	0.893
1.5	0.621
2	0.464
3	0.301
4	0.211
5	0.165



Note: See Table 3 of the report for ordinates of the various curves.

TWINING

SITE-SPECIFIC DESIGN RESPONSE SPECTRUM

Proposed Fire Station No. 9
4101 Long Beach Boulevard
Long Beach, California

PROJECT NO.	DATE	FIGURE 5
210377.1	July 2021	

TWINING

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APPENDIX A
FIELD EXPLORATION

TWINING

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Appendix A
Field Exploration

General

The field exploration for the proposed project consisted of drilling, testing, sampling, and logging of eight exploratory borings (B-1 through B-6, P-1, and P-2) and performing percolation testing in two of the borings (P-1 and P-2). The approximate locations of the exploration are shown on Figure 2 – Site Plan and Boring Location Map.

We obtained permits for the borings from the Long Beach Department of Public Health (LBDPH). The permits are included at the end of this appendix.

The borings were first excavated to 5 feet below ground surface (bgs) using a hand-auger to clear potential underground utilities. Upon completion of exploration, borings B-1 through B-4 were backfilled with neat cement and the others with soil cuttings. The surface of all locations was repaired to match existing conditions, and the paved locations were patched with Portland cement concrete to match existing conditions.

Exploratory Borings

Drilling operation for the borings was performed by 2R Drilling of Chino, California using a CME-75 truck-mounted drill rig equipped with 8-inch diameter hollow-stem-auger (HSA). The borings were advanced to a maximum depth of 5.0 to 81.5 feet bgs on June 4, 2021.

An explanation of the boring logs is presented as Figure A-1. The boring logs are presented as Figures A-2 through A-9. The boring logs describe the earth materials encountered, samples obtained, and show the field and laboratory tests performed. The logs also show the boring number, drilling date, and the name of the logger and drilling subcontractor. The borings were logged by a Twining engineer using the Unified Soil Classification System under the supervision of a registered California Geotechnical Engineer. The boundaries between soil types shown on the logs are approximate because the transition between different soil layers may be gradual. Drive and bulk samples of representative earth materials were obtained from the borings.

Disturbed samples were obtained from select depths using a Standard Penetration Test (SPT) sampler. This sampler consists of a 2-inch O.D., 1.4-inch I.D. split barrel shaft without room for liner. Soil samples obtained by the SPT sampler were retained in plastic bags. A California modified sampler was also used to obtain drive samples of the soils from select depths. This sampler consists of a 3-inch outside diameter (O.D.), 2.4-inch inside diameter (I.D.) split barrel shaft. The samples were retained in brass rings for laboratory testing.

When the boring was drilled to a select depth, the sampler was lowered to the bottom of the boring and then driven a total of 18 inches into the soil using an automatic hammer weighing 140 pounds dropped from a height of 30 inches. The number of blows required to drive the samplers the final 12 inches is presented on the boring logs. Where sampler refusal is encountered and the sampler does not advance 18 inches, the total number of blows per number of inches advanced is presented. The blow counts given are field raw blow counts that have not been modified to account for field and/or depth conditions.

TWINING

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Percolation Testing

Percolation testing were performed in borings P-1 and P-2. After being advanced to 5 feet bgs using a hand-auger, the borings were drilled to 5 feet bgs again using an 8 inch-diameter, truck-mounted, hollow-stem auger. The borings were drilled under the observation of a field engineer who logged the subsurface conditions encountered and collected samples of the subsurface materials encountered.

The percolation test holes were prepared by placing approximately 1 inch of gravel at the bottom of the hole. A 3-inch diameter perforated PVC pipe wrapped in filter sock was placed at the bottom of the hole and the annular space around the pipe was backfilled with gravel.

After preparing the percolation test holes, the percolation was performed in accordance with the requirements of Los Angeles County. After presoaking, the test holes were filled with water to at least 12 inches above the bottom of the excavation. Measurements were recorded at least 30-minute intervals for a total of 6 or until percolation rates stabilized. The average drop that occurred over the last 3 readings was used to determine the percolation rate at each test location. Detailed test data is attached to this appendix.

The infiltration rate was calculated by dividing the measured percolation rate by a surface area factor to account for discharge of water from the sides of the boring (i.e., non-vertical flow), which was then divided by a reduction factor to account for test method, site variability, and long-term siltation as described in the County of Los Angeles GS200.2 manual. The following formula were used:

The average drop that occurred over the final 3 readings was used to determine the infiltration rate at each test location. Based on the County of Los Angeles GS200.2 manual, a reduction factor of 3 was applied to the measured infiltration rate to obtain the design infiltration rate. A summary of test results is presented in Table A-1, and the detailed test data is attached to the end of this appendix.

Table A-1 – Infiltration Rate with a Reduction Factor of 3

Location	Depth (feet)	Infiltration Rate (in/hour)
P-1	5	0.03
P-2	5	0.04

UNIFIED SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	**GRAVEL AND GRAVELLY SOILS** MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	**CLEAN GRAVELS** (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	**CLEAN SANDS** (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	**SILTS AND CLAYS**	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

COARSE-GRAINED SOILS / FINE-GRAINED SOILS

Relative Density	SPT (blows/ft)	Relative Density (%)	Consistency	SPT (blows/ft)
Very Loose	<4	0 - 15	Very Soft	<2
Loose	4 - 10	15 - 35	Soft	2 - 4
Medium Dense	10 - 30	35 - 65	Medium Stiff	4 - 8
Dense	30 - 50	65 - 85	Stiff	8 - 15
Very Dense	>50	85 - 100	Very Stiff	15 - 30
			Hard	>30

NOTE: SPT blow counts based on 140 lb. hammer falling 30 inches

Sample Symbol	Sample Type	Description
	SPT	1.4 in I.D., 2.0 in. O.D. driven sampler
	California Modified	2.4 in. I.D., 3.0 in. O.D. driven sampler
	Bulk	Retrieved from soil cuttings
	Thin-Walled Tube	Pitcher or Shelby Tube

LABORATORY TESTING ABBREVIATIONS

ATT	Atterberg Limits
C	Consolidation
CORR	Corrosivity Series
DS	Direct Shear
EI	Expansion Index
GS	Grain Size Distribution
K	Permeability
MAX	Moisture/Density (Modified Proctor)
O	Organic Content
RV	Resistance Value
SE	Sand Equivalent
SG	Specific Gravity
TX	Triaxial Compression
UC	Unconfined Compression

TWINING

EXPLANATION FOR LOG OF BORINGS

Fire Station No. 9
4101 Long Beach Boulevard
Long Beach, California

PROJECT NO.	REPORT DATE	
210377.1	June 2021	FIGURE A-1

DATE DRILLED	6/4/2021	LOGGED BY	CDD	**BORING NO.**	**B-1**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	
DRILLING METHOD	8" HSA	DRILLER	2R Drilling	SURFACE ELEVATION (ft.)	95 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
									4" of concrete; no base
								CL	**FILL** Lean CLAY; reddish brown; slightly moist; firm
		Bulk		10.6		EI, CORR, MAX, DS		CL	**NATIVE** Lean CLAY; reddish brown; slightly moist; stiff
90	5	Driven	58	13.8	120.3	CONSOL		CL	-- same; hard
85	10	Driven	11			#200, ATT		SM	Silty SAND; medium brown; slightly moist; medium dense; abundant mica
80	15	Driven	49	12.7	103.3	DS		SM	-- same
75	20	Driven	28					SM	-- same; dark reddish brown
70	25	Driven	82	25.9	99.9			ML	Sandy SILT; strong brown; slightly moist; hard; some iron oxide staining
65	30	Driven	26			#200, ATT		ML	SILT; dark yellowish brown; slightly moist; very stiff
60	35								

TWINING

LOG OF BORING

Fire Station No. 9
4101 Long Beach Boulevard
Long Beach, California

PROJECT NO.	REPORT DATE	
210377.1	June 2021	FIGURE A - 2

DATE DRILLED	6/4/2021	LOGGED BY	CDD	**BORING NO.**	**B-1**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	
DRILLING METHOD	8" HSA	DRILLER	2R Drilling	SURFACE ELEVATION (ft.)	95 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk / Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
			40	23.0	101.6	CONSOL		CL	Lean CLAY, medium brown, slightly moist; very stiff
55	40		19			#200, ATT		ML	SILT; medium brown; slightly moist; very stiff
50	45		83	14.8	115.6			SM	Silty SAND; yellowish brown; slightly moist; dense
45	50		19					ML	Sandy SILT, light olive brown; slightly moist; very stiff
40	55								
35	60		34					SM	Silty SAND, olive brown; slightly moist, dense
30	65								
25	70								

TWINING

LOG OF BORING

Fire Station No. 9
4101 Long Beach Boulevard
Long Beach, California

PROJECT NO.	REPORT DATE	
210377.1	June 2021	FIGURE A - 2

DATE DRILLED	6/4/2021	LOGGED BY	CDD	**BORING NO.**	**B-1**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	
DRILLING METHOD	8" HSA	DRILLER	2R Drilling	SURFACE ELEVATION (ft.)	95 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
				38					SM	Silty SAND, olive brown; slightly moist, dense *(continued)*
20	75									
15	80			47					SM	-- same; gray
10	85									
5	90									
0	95									
-5	100									
-10	105									

Total Depth = 81.5 feet
Backfilled on 6/4/2021
No Groundwater was encountered.
Backfilled with neat cement and patched with PCC at completion.

TWINING

LOG OF BORING

Fire Station No. 9
4101 Long Beach Boulevard
Long Beach, California

PROJECT NO.	REPORT DATE	
210377.1	June 2021	FIGURE A - 2

DATE DRILLED	6/4/2021	LOGGED BY	CDD	**BORING NO.**	**B-2**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	
DRILLING METHOD	8" HSA	DRILLER	2R Drilling	SURFACE ELEVATION (ft.)	95 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
									CL	3" of concrete; no base **FILL** Lean CLAY with sand; reddish brown; slightly moist
		X							CL	**NATIVE** Lean CLAY; reddish brown; slightly moist
90	5		X	23			ATT		CL	-- same; very stiff
85	10		X	25	10.4	105.6	#200, DS		ML	Sandy SILT; yellowish brown; slightly moist; medium dense
80	15		X	10					SM	Silty SAND; light brown; slightly moist; medium dense
75	20		X	49	16.3	100.2			ML	Sandy SILT; yellowish brown; slightly moist; hard; some caliche veins
70	25		X	27					SM	Silty SAND; yellowish brown; slightly moist; medium dense; some mica
										Total Depth = 26.5 feet Backfilled on 6/4/2021 No Groundwater was encountered. Backfilled with neat cement and patched with PCC at completion.
65	30									
60	35									

TWINING

LOG OF BORING

Fire Station No. 9
4101 Long Beach Boulevard
Long Beach, California

PROJECT NO.	REPORT DATE	
210377.1	June 2021	FIGURE A - 3

DATE DRILLED	6/4/2021	LOGGED BY	CDD	**BORING NO.**	**B-3**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	
DRILLING METHOD	8" HSA	DRILLER	2R Drilling	SURFACE ELEVATION (ft.)	95 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
										5" of concrete; no base
									CL	**FILL** Lean CLAY; reddish brown; slightly moist
									CL	**NATIVE** Lean CLAY; reddish brown; slightly moist
90	5			64	12.8	122.0			CL	-- same; hard
85	10			15			#200		ML	Sandy SILT; reddish brown; slightly moist; stiff
80	15			27	6.6	104.5			SM	Silty SAND; light yellowish brown; slightly moist; medium dense
75	20			21			ATT		ML	Sandy SILT; dark yellowish brown; slightly moist; very stiff
70	25			68	31.5	93.8			ML	-- same; dark grayish brown; hard; some mica
										Total Depth = 26.5 feet Backfilled on 6/4/2021 No Groundwater was encountered. Backfilled with neat cement and patched with PCC at completion.
65	30									
60	35									

TWINING

LOG OF BORING

Fire Station No. 9
4101 Long Beach Boulevard
Long Beach, California

PROJECT NO.	REPORT DATE	
210377.1	June 2021	FIGURE A - 4

DATE DRILLED	6/4/2021	LOGGED BY	CDD	**BORING NO.**	**B-4**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	
DRILLING METHOD	8" HSA	DRILLER	2R Drilling	SURFACE ELEVATION (ft.)	95 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
										6" of concrete; no base
									CL	**FILL** Lean CLAY; dark brown; slightly moist
									CL	**NATIVE** Lean CLAY; reddish brown; slightly moist
90	5			20					CL	-- same; stiff
85	10			37	24.3	102.6			ML	Sandy SILT; light brown; slightly moist; very stiff
80	15			19			#200		ML	SILT with sand; light yellowish brown; slightly moist; very stiff
75	20			60	6.9	102.2			SM	Silty SAND; light brownish gray; slightly moist; dense
70	25			28					ML	Sandy SILT; yellowish brown; slightly moist; very stiff; some mica
										Total Depth = 26.5 feet Backfilled on 6/4/2021 No Groundwater was encountered. Backfilled with neat cement and patched with PCC at completion.
65	30									
60	35									

BORING LOG 210377.1 - POLB FIRE STATION NO. 9.GPJ TWINING LABS.GDT 6/28/21

TWINING

LOG OF BORING

Fire Station No. 9
4101 Long Beach Boulevard
Long Beach, California

PROJECT NO.	REPORT DATE	
210377.1	June 2021	FIGURE A - 5

DATE DRILLED	6/4/2021	LOGGED BY	CDD	**BORING NO.**	**B-5**
DRIVE WEIGHT	N/A	DROP	N/A	DEPTH TO GROUNDWATER (ft.)	
DRILLING METHOD	HA	DRILLER	2R Drilling	SURFACE ELEVATION (ft.)	95 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
										4.5" of concrete; no base
							#200; ATT; R-Value		CL	**FILL** Sandy Lean CLAY; reddish brown; slightly moist
									CL	**NATIVE** Lean CLAY; reddish brown; slightly moist
90	5									Total Depth = 5.0 feet Backfilled on 6/4/2021 No Groundwater was encountered. Backfilled with cuttings and patched with PCC at completion.
85	10									
80	15									
75	20									
70	25									
65	30									
60	35									

TWINING

LOG OF BORING

Fire Station No. 9
4101 Long Beach Boulevard
Long Beach, California

PROJECT NO.	REPORT DATE	
210377.1	June 2021	FIGURE A - 6

DATE DRILLED	6/4/2021	LOGGED BY	CDD	**BORING NO.**	**B-6**
DRIVE WEIGHT	N/A	DROP	N/A	DEPTH TO GROUNDWATER (ft.)	
DRILLING METHOD	HA	DRILLER	2R Drilling	SURFACE ELEVATION (ft.)	95 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
									6" of concrete; no base
								CL	**FILL** Lean CLAY; reddish brown; slightly moist
								CL	**NATIVE** Lean CLAY; reddish brown; slightly moist
90	5								Total Depth = 5.0 feet Backfilled on 6/4/2021 No Groundwater was encountered. Backfilled with cuttings and patched with PCC at completion.
85	10								
80	15								
75	20								
70	25								
65	30								
60	35								

TWINING

LOG OF BORING

Fire Station No. 9
4101 Long Beach Boulevard
Long Beach, California

PROJECT NO.	REPORT DATE	
210377.1	June 2021	FIGURE A - 7

DATE DRILLED	6/4/2021	LOGGED BY	CDD	**BORING NO.**	**P-1**
DRIVE WEIGHT	N/A	DROP	N/A	DEPTH TO GROUNDWATER (ft.)	
DRILLING METHOD	HA	DRILLER	2R Drilling	SURFACE ELEVATION (ft.)	95 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
								CL	**FILL** Lean CLAY; medium brown; dry; firm
								CL	**NATIVE** Lean CLAY; reddish brown, slightly moist; stiff
90	5								Total Depth = 5.0 feet Backfilled on 6/4/2021 No Groundwater was encountered. Backfilled with cuttings at completion.
85	10								
80	15								
75	20								
70	25								
65	30								
60	35								

LOG OF BORING

Fire Station No. 9
4101 Long Beach Boulevard
Long Beach, California

TWINING

PROJECT NO.	REPORT DATE	FIGURE A - 8
210377.1	June 2021	

DATE DRILLED	6/4/2021	LOGGED BY	CDD	**BORING NO.**	**P-2**
DRIVE WEIGHT	N/A	DROP	N/A	DEPTH TO GROUNDWATER (ft.)	
DRILLING METHOD	HA	DRILLER	2R Drilling	SURFACE ELEVATION (ft.)	95 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
								CL	**FILL** Lean CLAY; medium brown; slightly moist; firm
								CL	**NATIVE** Sandy Lean CLAY; reddish brown; slightly moist; stiff
90	5								Total Depth = 5.0 feet Backfilled on 6/4/2021 No Groundwater was encountered. Backfilled with cuttings at completion.
85	10								
80	15								
75	20								
70	25								
65	30								
60	35								

TWINING

LOG OF BORING

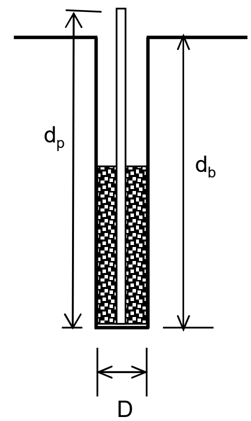
Fire Station No. 9
4101 Long Beach Boulevard
Long Beach, California

PROJECT NO.	REPORT DATE	
210377.1	June 2021	FIGURE A - 9

BORING PERCOLATION FIELD Log

Project No.: 210377.1
Project Name: Fire Station No. 9

Boring No.: **P-1**
Diameter of Boring (D): 8.0 inches
Depth of Boring (d_b): 5.0 feet = 60 inches
Diameter of Perc. Pipe : 3.5 inches
Length of Pipe (d_p) : 5.0 feet = 60 inches



PRE-SOAK

Date: 6/4/2021
Start Time: 7:50 AM
Elapsed Time: 30.00 minutes
Water Remaining: Yes

REDUCTION FACTOR

Reduction Factor 3.00

PERCOLATION TEST **Test Date:** 6/4/2021 **Test Performer:** JAB **Calculated by:** DHC

Reading Number	Initial Time T_i	Final Time T_f	Elapsed Time ΔT (min)	Initial depth to water surface dw_i (inches)	Final depth to water surface dw_f (inches)	Initial height of water column d_i (inches)	Drop of water column Δd (inches)	Water height drop rate $k_i = \Delta d / \Delta T$ (inch/hr)	Surface area factor S_f	Raw Percolation Rate $k = k_i / S_f$ (inch/hr)
1	8:20 AM	9:00 AM	40	15.6	17.4	44.4	1.8	2.70	22.8	0.12
2	9:00 AM	9:30 AM	30	17.4	18.0	42.6	0.6	1.20	22.2	0.05
3	9:30 AM	10:00 AM	30	18.0	18.8	42.0	0.8	1.68	21.8	0.08
4	10:00 AM	10:30 AM	30	18.8	19.7	41.2	0.8	1.68	21.4	0.08
5	10:30 AM	11:00 AM	30	19.7	20.5	40.3	0.8	1.68	21.0	0.08

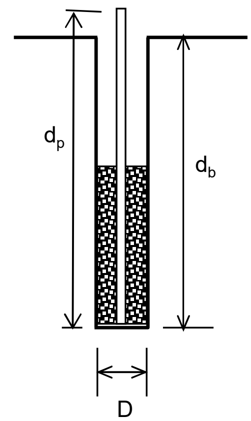
Measured Percolation Rate $k_{measured}$ (inch/hr) = **0.08**

Design Infiltration rate (inch/hr) = $k_{measured}$/RF = **0.03**

Reference: Los Angeles County Guidelines For Design, Investigation, and Reporting LID Stormwater Infiltration, GS200.2, dated 06/30/17
City of Los Angeles, Board of Public Works, Development BMP Handbook, Part B Planning Activities, 5th edition

BORING PERCOLATION FIELD Log

Project No.: 210377.1
Project Name: Fire Station No. 9



Boring No.: **P-2**
Diameter of Boring (D): 8.0 inches
Depth of Boring (d_b): 5.0 feet = 60 inches
Diameter of Perc. Pipe : 3.5 inches
Length of Pipe (d_p) : 5.0 feet = 60 inches

PRE-SOAK

Date: 6/4/2021
Start Time: 7:57 AM
Elapsed Time: 30.00 minutes
Water Remaining: Yes

REDUCTION FACTOR

Reduction Factor 3.00

PERCOLATION TEST **Test Date:** 6/4/2021 **Test Performer:** JAB **Calculated by:** DHC

Reading Number	Initial Time T_i	Final Time T_f	Elapsed Time ΔT (min)	Initial depth to water surface dw_i (inches)	Final depth to water surface dw_f (inches)	Initial height of water column d_i (inches)	Drop of water column Δd (inches)	Water height drop rate $k_i = \Delta d / \Delta T$ (inch/hr)	Surface area factor S_f	Raw Percolation Rate $k = k_i / S_f$ (inch/hr)
1	8:27 AM	8:57 AM	30	25.2	27.0	34.8	1.8	3.60	18.0	0.20
2	8:57 AM	9:27 AM	30	27.0	28.8	33.0	1.8	3.60	17.1	0.21
3	9:27 AM	9:57 AM	30	28.8	29.6	31.2	0.8	1.68	16.4	0.10
4	9:57 AM	10:27 AM	30	29.6	30.6	30.4	1.0	1.92	15.9	0.12
5	10:27 AM	10:57 AM	30	30.6	31.4	29.4	0.8	1.68	15.5	0.11

Measured Percolation Rate $k_{measured}$ (inch/hr) = **0.11**

Design Infiltration rate (inch/hr) = $k_{measured}$/RF = **0.04**

Reference: Los Angeles County Guidelines For Design, Investigation, and Reporting LID Stormwater Infiltration, GS200.2, dated 06/30/17
City of Los Angeles, Board of Public Works, Development BMP Handbook, Part B Planning Activities, 5th edition

Health and Human Services Department
Bureau of Environmental Health
CPP – Water Quality Program
2525 Grand Avenue #220, Long Beach, CA 90815
Office (562) 570-4132 | Fax (562) 570-4038

WELL PERMIT

PERMIT#: **2877**

DATE ISSUED: ***May 27, 2021***
PROPOSED DRILLING DATE: ***June 2, 2021***

All work must be completed in accordance with Water Well Bulletin 74-81 and 74-90.

PLEASE NOTIFY INSPECTOR 48 HOURS BEFORE DRILLING AND SUBMIT THE DRILLERS WELL COMPLETION REPORT (WCR) TO vanna.kho@longbeach.gov (OR MAIL/FAX AT ADDRESS ABOVE) AND THE DEPARTMENT OF WATER RESOURCES ONLINE AT https://civicnet.resources.ca.gov/DWR_WELLS/.

Site Address:	***4101 Long Beach Boulevard*** ***Long Beach, CA 90807***
Owner:	***City of Long Beach, Jonathon Bolin***
Owner Address:	*411 West Ocean Boulevard* *Long Beach, CA 90802*
Consulting Firm:	***Twining Inc.***
Consulting Firm Address	*2883 East Spring Street, Suite 300* *Long Beach, CA 90806*
Drilling Company:	***2R Drilling***
Drilling Co. Address:	*6939 Schaefer Avenue, Suite D-304* *Santa Fe Springs, CA 91710*
Type Of Permit:	***Soil Boring***
Type Of Well:	***Soil Boring***
Total Number Of Well/Soil Boring:	***8 Borings***

THIS PERMIT IS VALID FOR ONE YEAR FROM DATE ISSUED ABOVE

Vanna Kho (Digitally signed by Vanna Kho Date: 5/27/21-CM)

Inspector Name
Cross-Connection/Water Quality

CITY OF LONG BEACH

Department of Health and Human Services
Bureau of Environmental Health | Water Quality Program
2525 Grand Avenue #220 | Long Beach, CA 90815
(562) 570-4132 | Fax (562) 570-4038

WELL PERMIT APPLICATION

☐ ***EXPEDITE***
(FEE'S APPLY; SEE PG. 1)

Date: 05/18/2021 **Proposed Drilling Date:** 06/02/2021

Site Address: 4101 Long Beach Boulevard, Long Beach CA

Permit Delivery: ☐ Mail ☐ Fax ☐ Pick Up ☒ E-mail: dcrayton@twininginc.com

Permit Type: ☒ New Well Construction ☐ Destruction ☐ Other: ______

Well Type: ☐ Monitoring ☐ Cathodic ☐ Private Domestic ☐ Public Domestic ☐ Vapor Extraction
☒ Soil Boring ☐ Sparging ☐ Nested

Total # of: Wells ____ Borings 8 Total Cost: $ 435.00

Well Owner Name: City of Long Beach, Jonathon Bolin **Phone:** (562) 570-2855

Well Owner Address: 411 W Ocean Blvd, 5th Floor, Long Beach, CA 90802
City State Zip Code

Consulting Firm Name: Twining Inc **Phone:** (562) 426-3355

Consulting Firm Address: 2883 E Spring St, Suite 300, Long Beach, CA 90806
City State Zip Code

Drilling Company Name: 2R Drilling **Phone:** (909) 490-0530

Drilling Company Address: 6939 Schaefer Avenue Ste D-304, Chino, CA 91710
City State Zip Code

CA License #: C57 709029

Submit

PROVIDE PLOT PLAN LOCATING EACH WELL CONSTRUCTED OR ABANDONED

Construction/Destruction Method Type of casing, method of sealing, etc. (Use additional sheet or attachments)
See attached for details.

I hereby agree to comply in every respect with all regulations of the Long Beach Department of Health and Human Services and with all ordinance and laws of the City of Long Beach and of the State of California pertaining to well construction, reconstruction and destruction. Upon completion of well and within ten days perforations in casing, and any other data deemed necessary by other city agencies.

Print Name: Doug Crayton ***Applicants Signature:*** ______

Telephone: (208) 720-7972 ***Fax Number:*** (562) 426-6424 ***E-mail:*** dcrayton@twininginc.com

FOR OFFICE USE ONLY

■ *Approved* ☐ *Denied* *Received by:* EMAIL *Approved by:* ______ *Date:* ______

☐ *Approved with Conditions* ______

Permit # **2877**

TWINING

2883 East Spring Street
Suite 300
Long Beach CA 90806

Tel 562.426.3355
Fax 562.426.6424

APPENDIX B
LABORATORY TESTING

TWINING

2883 East Spring Street
Suite 300
Long Beach CA 90806

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Fax 562.426.6424

Appendix B
Laboratory Testing

Laboratory Moisture Content and Density Tests

The moisture content and dry densities of selected driven samples obtained from the exploratory borings were evaluated in general accordance with the latest version of ASTM D2937. The results are shown on the boring logs in Appendix A, and also summarized in Table B-1.

No. 200 Wash Sieve

The amount of fines passing the No. 200 sieve was evaluated in accordance with ASTM D1140. The results are presented in Table B-2.

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 are summarized in on Figure B-1 and Table B-3.

Resistance Value (R-value)

R-value testing was performed on a select bulk sample of the near-surface soils encountered at the site. The test was performed in general accordance with ASTM D2844. The result is summarized in Table B-4.

Expansion Index

The expansion index of a select soil sample was evaluated in general accordance with ASTM D4829. 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 result of expansion index test is presented in Table B-5.

Direct Shear

Direct shear tests were performed on a remolded sample and representative modified-California soil samples in general accordance with the latest version of ASTM D3080 to evaluate the shear strength characteristics of the selected materials. The samples were inundated during shearing to represent adverse field conditions. Test results are presented on Figures B-2 through B-4.

Consolidation

Consolidation tests were performed on selected modified-California soil samples in general accordance with the latest version of ASTM D2435. The samples were inundated during testing to represent adverse field conditions. The percent consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. Test results are presented on Figures B-5 through B-6.

Maximum Dry Density-Optimum Moisture Content

One selected bulk sample was tested to evaluate the maximum dry density and its optimum moisture content. The test was performed in general accordance with ASTM test method D1557. The result is presented on Figure B-7.

Corrosivity

Soil pH and resistivity tests were performed by Anaheim Test Lab, Inc. (ATLI) of Anaheim, California on a representative soil sample. The resistivity of the soil assumes saturated soil conditions. The chloride and sulfate contents of the selected samples were evaluated in general accordance with the latest versions of Caltrans test methods CT417, CT422, and CT 643. The test results are presented on Table B-6 and the ATLI report included in this appendix.

Table B-1 - Moisture Content and Dry Density

Boring No.	Depth (feet)	Moisture Content (%)	Dry Density (pcf)
B-1	2-5	10.6	--
B-1	5	13.8	120.3
B-1	15	12.7	103.3
B-1	25	25.9	99.9
B-1	35	23.0	101.6
B-1	45	14.8	115.6
B-2	10	10.4	105.6
B-2	20	16.3	100.2
B-3	5	12.8	122.0
B-3	15	6.6	104.5
B-3	25	31.5	93.8
B-4	10	24.3	102.6
B-4	20	6.9	102.2

Table B-2 - Number 200 Wash Results

Boring No.	Depth (feet)	Percent Passing #200
B-1	10	43.1
B-1	30	88.9
B-1	40	95.6
B-2	10	53.1
B-3	10	67.3
B-4	15	80.0
B-5	1-5	69.1

TWINING

2883 East Spring Street
Suite 300
Long Beach CA 90806

Tel 562.426.3355
Fax 562.426.6424

Table B-3 - Atterberg Limits Results

Boring No.	Depth (feet)	Liquid Limit	Plastic Limit	Plasticity Index	U.S.C.S. Classification
B-1	10	0	0	0	Silty Sand (SM)
B-1	30	0	0	0	Silt (ML)
B-1	40	44	29	15	Silt (ML)
B-2	5	44	16	28	Lean Clay (CL)
B-3	20	0	0	0	Sandy Silt (ML)
B-5	1-5	23	15	8	Sandy Lean Clay (CL)

Table B-4 Resistance Value (R-value)

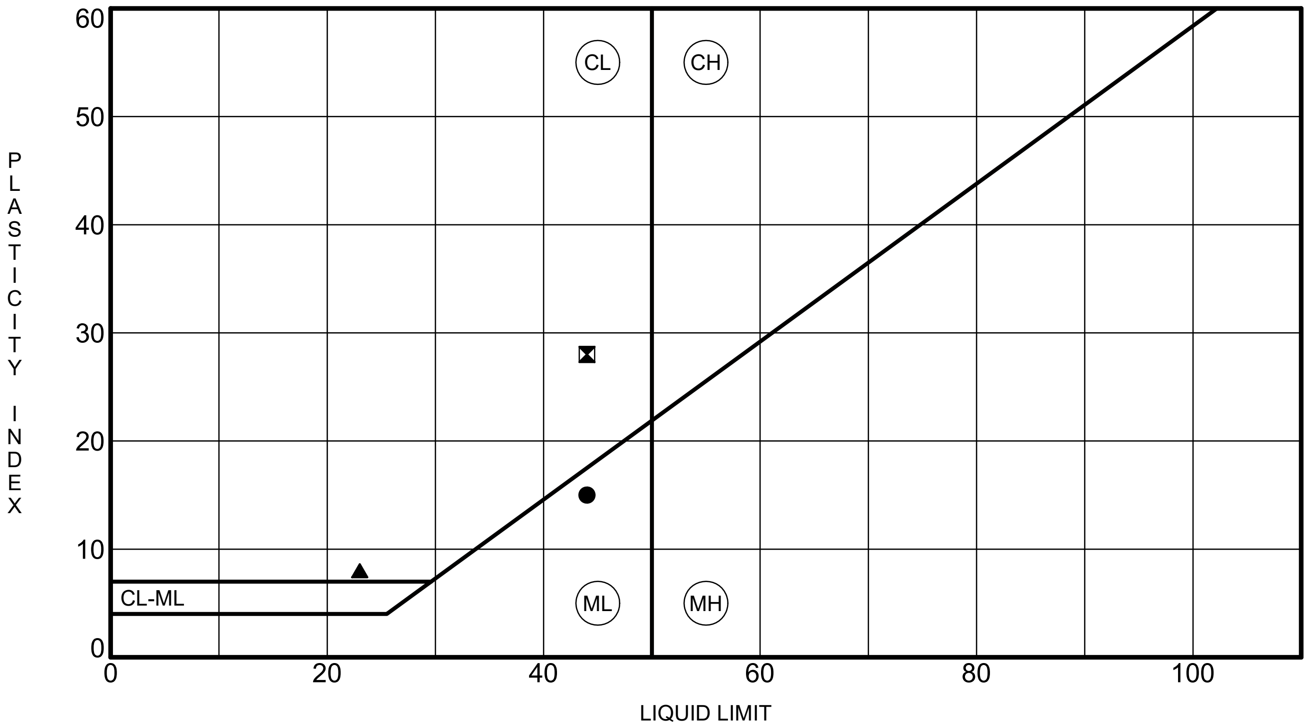
Boring No.	Depth (feet)	R Value
B-5	1 - 5	10

Table B-5 - Expansion Index

Boring No.	Depth (feet)	Expansion Index	Expansion Potential
B-1	2 - 5	14	Very low

Table B-6 - Corrosivity Test Results

Boring No.	Depth (feet)	pH	Minimum Resistivity (ohm-cm)	Water Soluble Sulfate (ppm)	Water Soluble Chloride (ppm)
B-1	2-5	7.5	3,600	489	81



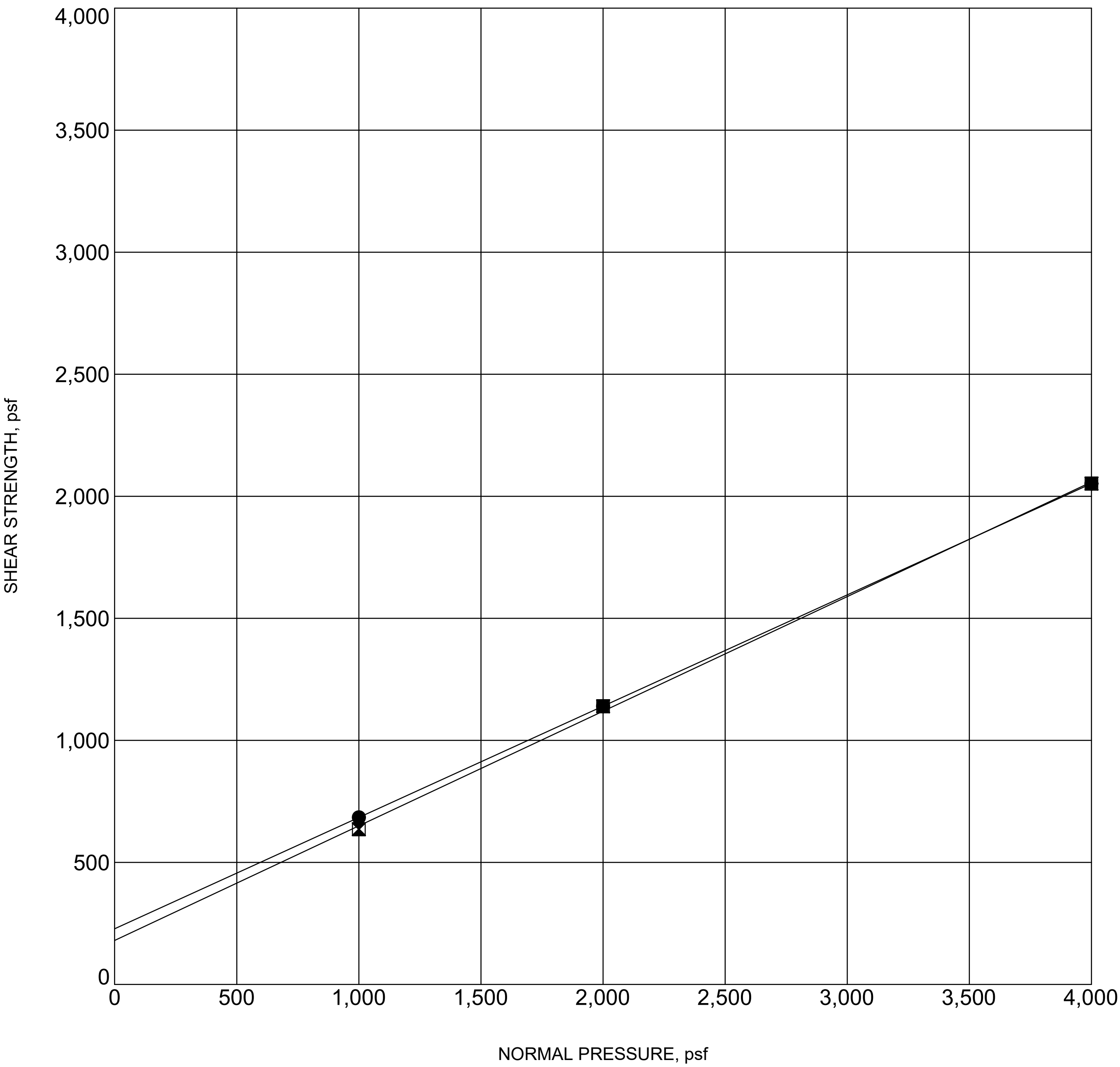
	Sample Location	LL	PL	PI	U.S.C.S. Classification
●	**B-1 at 40 ft**	**44**	**29**	**15**	**SILT**
⧗	**B-2 at 5 ft**	**44**	**16**	**28**	**Lean CLAY**
▲	**B-5 at 1 ft**	**23**	**15**	**8**	**Sandy Lean CLAY**

TWINING

ATTERBERG LIMITS

Fire Station No. 9
4101 Long Beach Boulevard
Long Beach, California

PROJECT NO.	REPORT DATE	FIGURE B-1
210377.1	June 2021	



Shear Strength Parameters

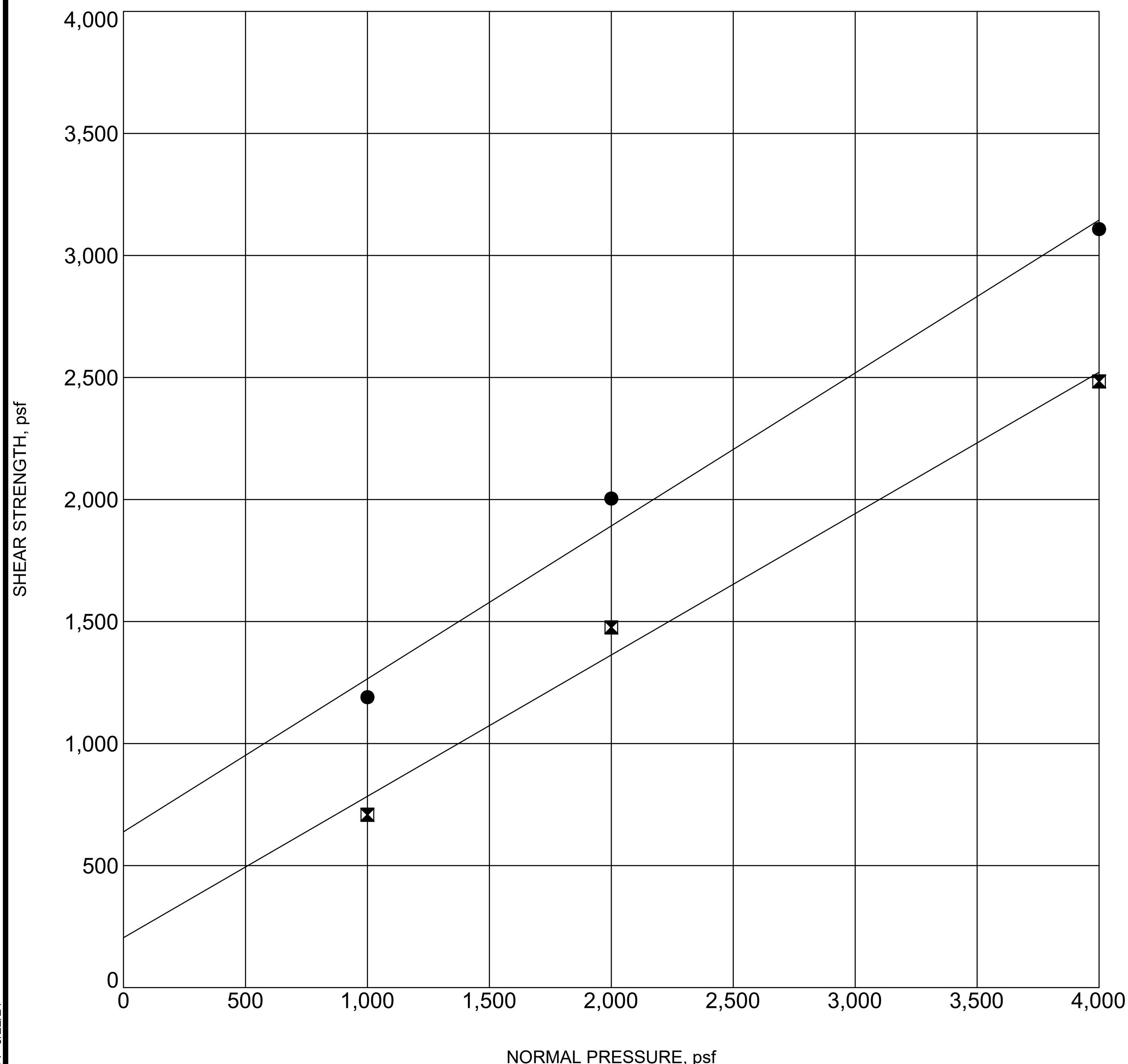
				Peak	**Ultimate**
Boring No.:	B-1		**Cohesion, C (psf):**	228	180
Sample Depth (ft):	2		**Friction Angle, Ø (deg):**	25	25
Sample Description:	Lean CLAY		**Initial Moisture (%):**	9.9	
Strain Rate (in./min):	0.005		**Final Moisture (%):**	10.6	
Dry Density (pcf):	115.2				

TWINING

DIRECT SHEAR TEST

Fire Station No. 9
4101 Long Beach Boulevard
Long Beach, California

PROJECT NO.	REPORT DATE	FIGURE B-2
210377.1	June 2021	



Boring No.:	B-1
Sample Depth (ft):	15
Sample Description:	Silty SAND
Strain Rate (in./min):	0.005
Dry Density (pcf):	103.3

Shear Strength Parameters

	Peak	Ultimate
Cohesion, C (psf):	638	204
Friction Angle, Ø (deg):	32	30

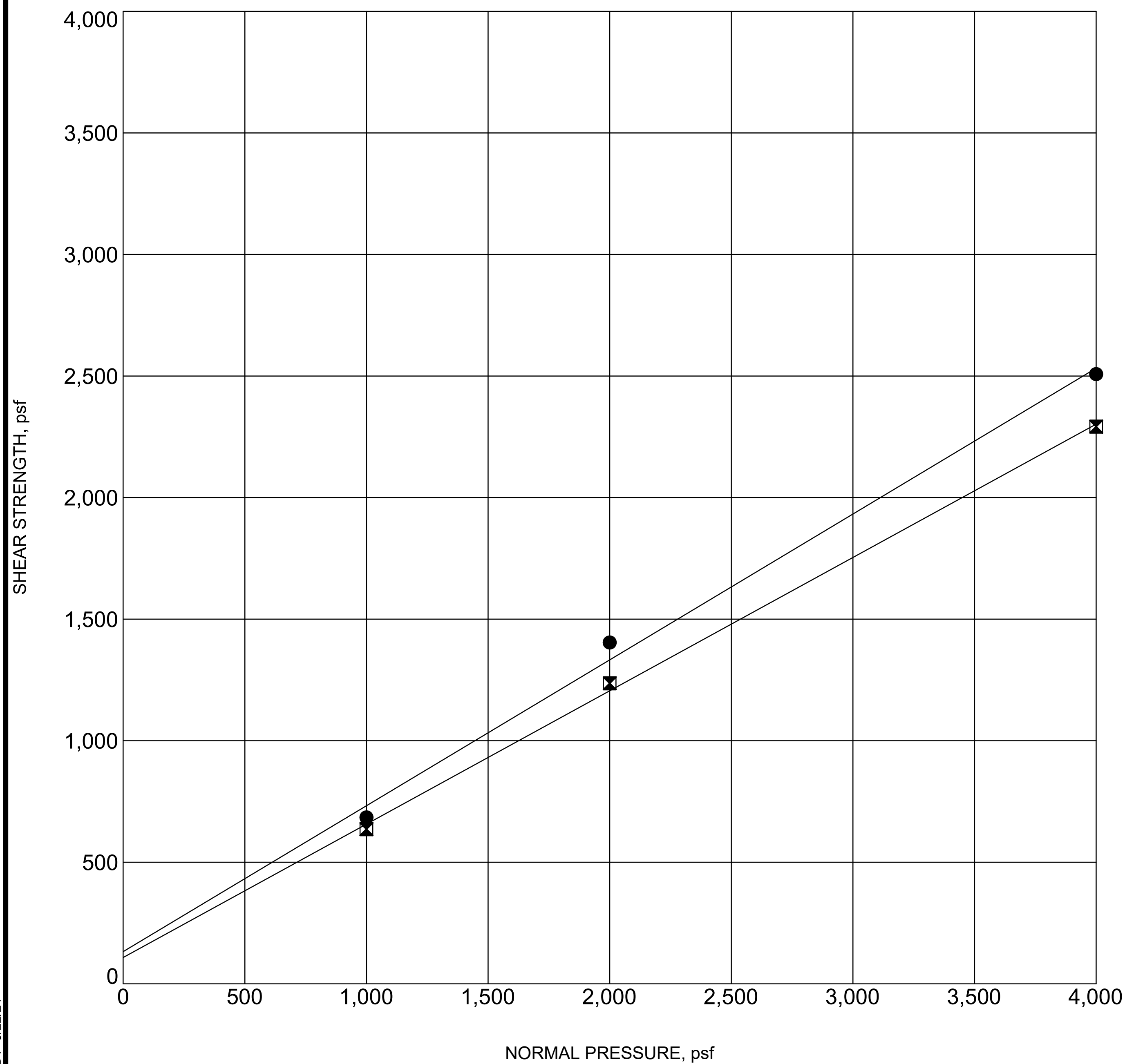
Initial Moisture (%): 12.7
Final Moisture (%): 17.9

TWINING

DIRECT SHEAR TEST

Fire Station No. 9
4101 Long Beach Boulevard
Long Beach, California

PROJECT NO.	REPORT DATE	FIGURE B-3
210377.1	June 2021	

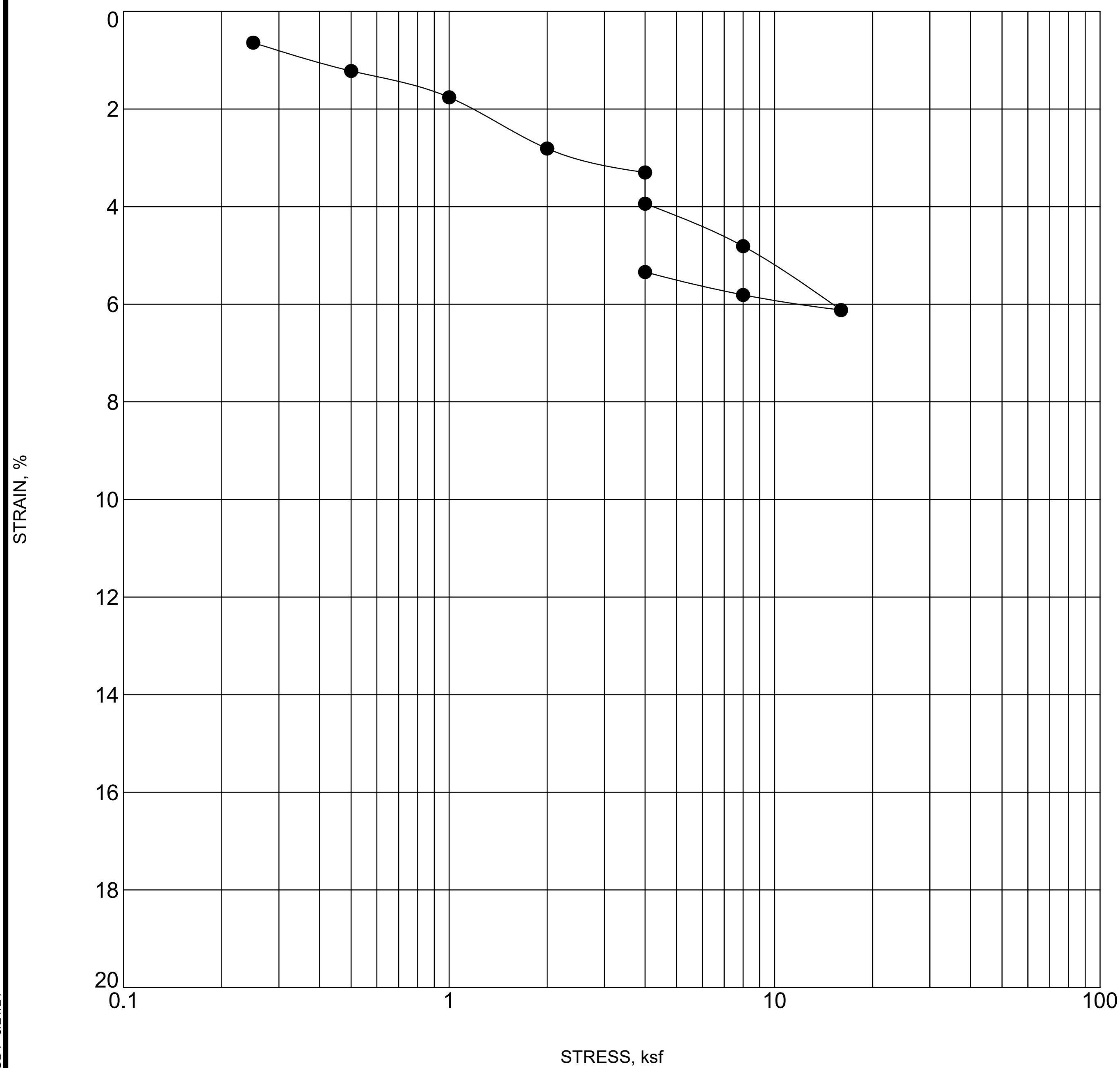


Boring No.:	B-2
Sample Depth (ft):	10
Sample Description:	Sandy SILT
Strain Rate (in./min):	0.005
Dry Density (pcf):	105.6

Shear Strength Parameters

	Peak	**Ultimate**
Cohesion, C (psf):	132	108
Friction Angle, Ø (deg):	31	29

Initial Moisture (%): 10.4
Final Moisture (%): 17.7



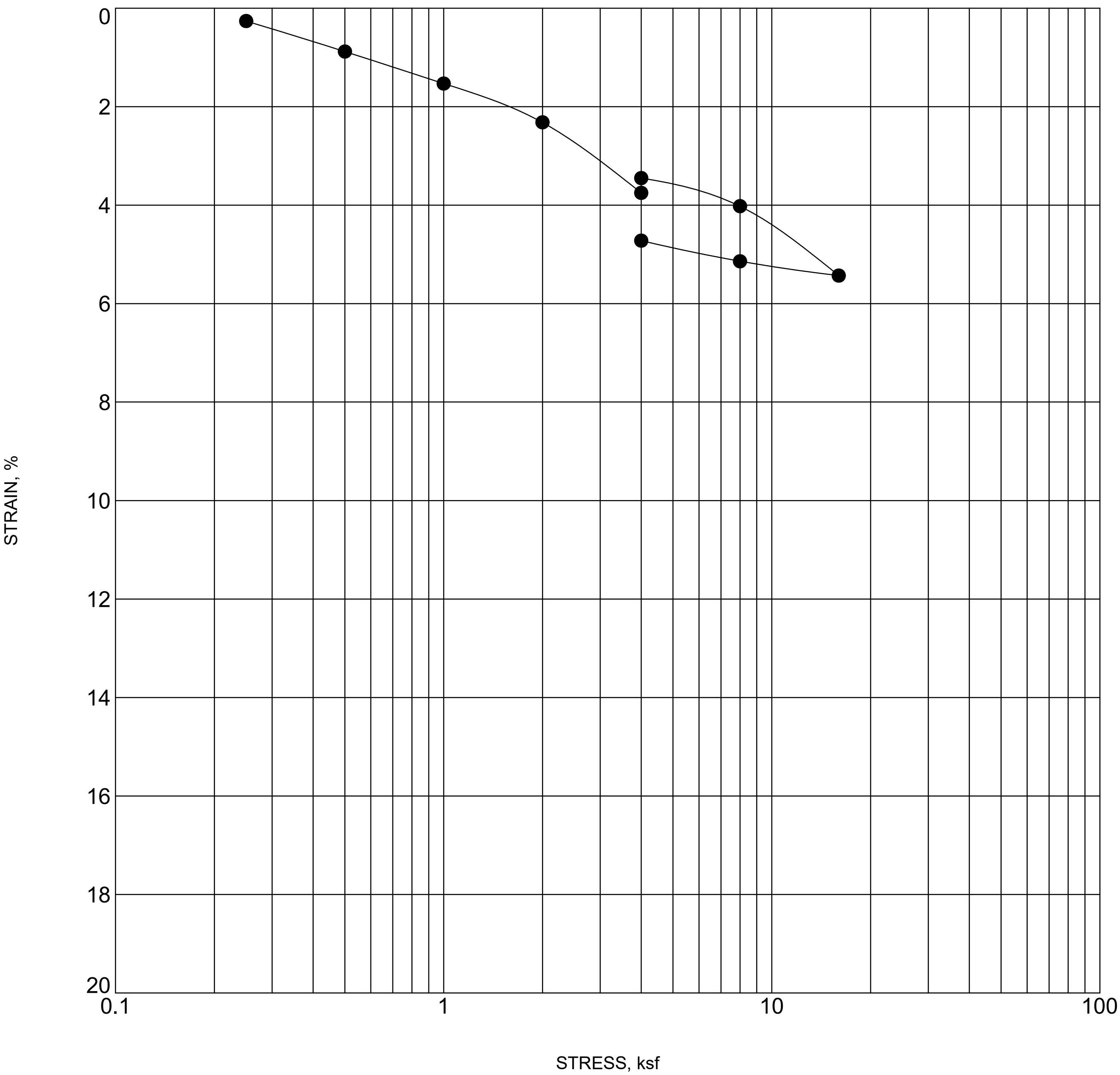
	Sample Location	Soil Description	Dry Density (pcf)	Moisture Content (%)
●	**B-1 at 5 ft**	**Lean CLAY**	**120.3**	**13.8**

TWINING

CONSOLIDATION TEST

Fire Station No. 9
4101 Long Beach Boulevard
Long Beach, California

PROJECT NO.	REPORT DATE	FIGURE B-5
210377.1	June 2021	



	Sample Location	Soil Description	Dry Density (pcf)	Moisture Content (%)
●	**B-1 at 35 ft**	**Lean CLAY**	**101.6**	**23.0**

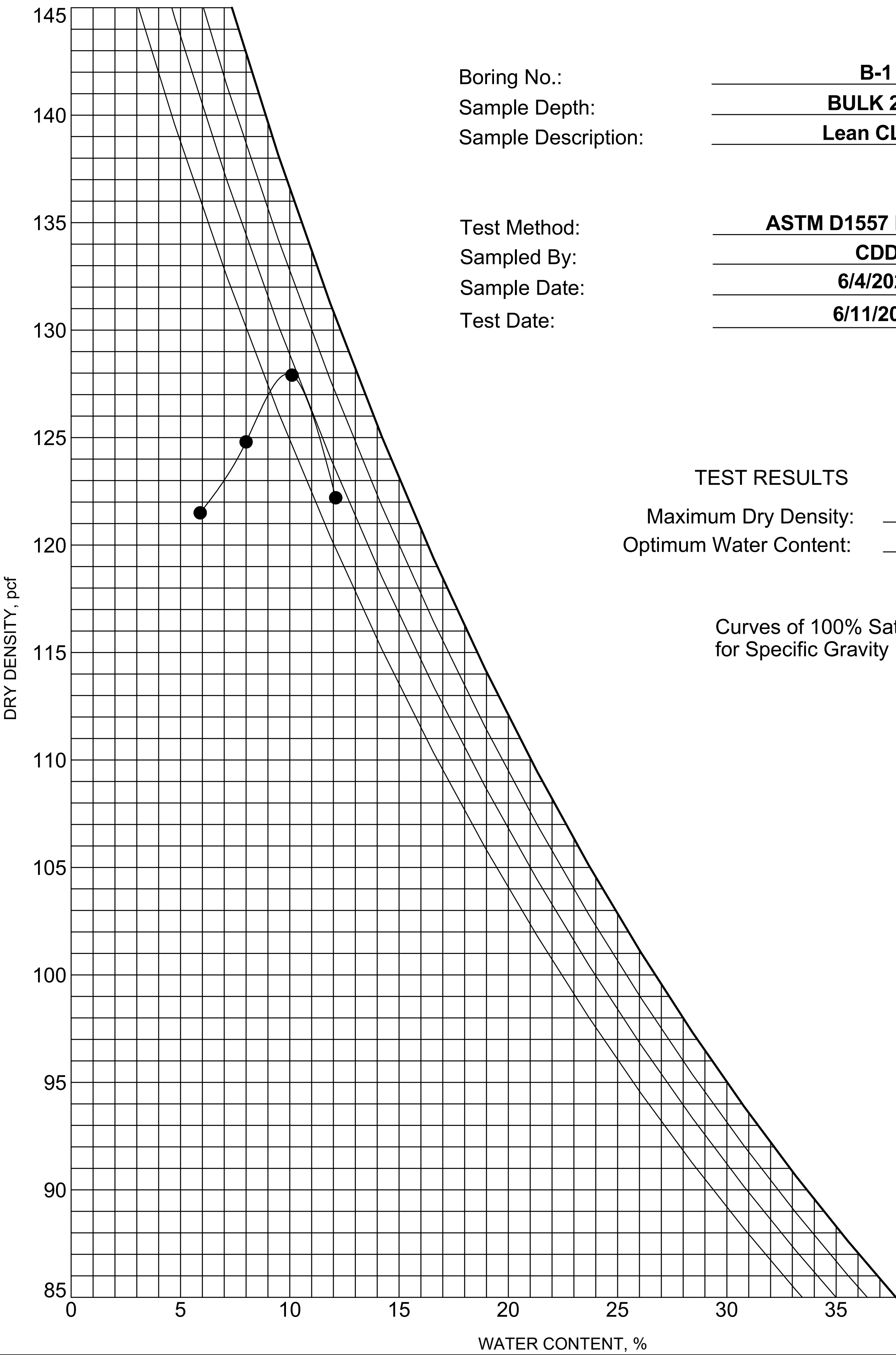
CONSOL STRAIN 210377.1 - POLB FIRE STATION NO. 9.GPJ TWINING LABS.GDT 6/21/21

TWINING

CONSOLIDATION TEST

Fire Station No. 9
4101 Long Beach Boulevard
Long Beach, California

PROJECT NO.	REPORT DATE	
210377.1	June 2021	FIGURE B-6



Boring No.: **B-1**

Sample Depth: **BULK 2-5'**

Sample Description: **Lean CLAY**

Test Method: **ASTM D1557 Method A**

Sampled By: **CDD**

Sample Date: **6/4/2021**

Test Date: **6/11/2021**

TEST RESULTS

Maximum Dry Density: **128.0 pcf**

Optimum Water Content: **10.0 %**

Curves of 100% Saturation
for Specific Gravity Equal to:

2.80

2.70

2.60

2.50

TWINING

MOISTURE-DENSITY RELATIONSHIP

Fire Station No. 9
4101 Long Beach Boulevard
Long Beach, California

PROJECT NO.	REPORT DATE	FIGURE B-7
210377.1	June 2021	

COMPACTION (MODIFIED BY PAUL) 210377.1 - POLB FIRE STATION NO. 9.GPJ TWINING LABS.GDT 6/21/21

ANAHEIM TEST LAB, INC.

196 Technology Drive, Unit D
Irvine, CA 92618
Phone (949)336-6544

DATE: 6/14/2021

TWINING LABS
3310 AIRPORT WAY
LONG BEACH, CA 90806

P.O. NO: Soils06092021

LAB NO: C-4916

SPECIFICATION: CTM-643/417/422

MATERIAL: Soil

Project No.: 210377.1
WO#: W01-21-12783
Project Name: Fire Station No. 9
Date sampled: 6/4/2021
Sample ID: B-1 @ 2-5'

ANALYTICAL REPORT

CORROSION SERIES

SUMMARY OF DATA

pH	MIN. RESISTIVITY per CT. 643 ohm-cm	SOLUBLE SULFATES per CT. 417 ppm	SOLUBLE CHLORIDES per CT. 422 ppm
7.5	3,600	489	81

RESPECTFULLY SUBMITTED

WES BRIDGER LAB MANAGER