B. Geotechnical Investigation

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Geohazard and Geotechnical Engineering Evaluation Report

Rise Kohyang Charter Project
3500 W. 1st Street
Los Angeles, California

<u>Prepared for:</u>
Bright Star Schools
c/o Pacific Charter School Development
600 Wilshire Boulevard, Suite 200
Los Angeles, CA 90017

September 28, 2018
Project No.: 180719.1

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September 28, 2018
Project No. 180719.1

Bright Star Schools
Attn: Elijah Sugay
c/o Pacific Charter School Development
600 Wilshire Boulevard, Suite 200
Los Angeles, CA 90017

Subject: Geotechnical Evaluation Report
Rise Kohyang Charter Project
3500 W. 1st Street
Los Angeles, CA

Dear Mr. Sugay:

In accordance with your request and authorization, we are presenting the results of our geotechnical investigation for the proposed Rise Kohyang Charter Project located at 3500 W. 1st Street in the City of Los Angeles, California. The purpose of this investigation has been to evaluate the potential geologic hazards and subsurface conditions at the site and to provide recommendations for the proposed site developments. To the extent that project design information has been provided to us, we have performed our geohazard evaluation in conformance with Chapter 18A of Title 24, Part 2, Volume 2 of the 2016 California Building Code and California Geological Survey Note 48.

Please note that the recommendations presented within the report are based on assumptions stated herein. Should conditions encountered during development differ from those assumed in our analyses, or should the proposed development change, our recommendations may need to be modified accordingly.

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,

TWINING CONSULTING, INC.

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1. INTRODUCTION

This report presents the results of our geotechnical engineering evaluation performed for the Rise Kohyang Charter Project located at 3500 W. 1st Street in the City of Los Angeles, California (Figure 1, Site Location Map). The purpose of this study has been to evaluate the subsurface conditions at the site and to provide geotechnical recommendations related to the design and construction of the proposed structures, including recommendations for foundations and earthwork. Our geotechnical investigation was performed in conformance with Chapter 18A of Title 24, Part 2, Volume 2 of the 2016 California Building Code (CBC), and California Geological Survey Note 48.

2. SITE DESCRIPTON AND PROPOSED DEVELOPMENT

2.1. Site Description

The proposed charter school project is located at 3500 W. 1st Street in Los Angeles, California as shown in Figure 1, Site Location Map. The property is roughly a trapezoidal lot with approximately 280 feet along W. 1st Street, and 230 feet along S. Madison Avenue. The site is currently occupied by a church with five (5), 1- to 2-story buildings and surface parking lot. The site is bounded on the north by W. 1st Street, on the east by S. Madison Avenue, on the west by a surface parking lot, and on the south by residential houses and parking lot. The property and surrounding vicinity area shown in Figure 2, Site Geologic and Boring Location Map.

The site exhibits a gentle slope with approximately 18 feet relief. The high point is located at west property line at an elevation of +/- 277 feet above mean sea level (MSL). The low point is located at northeast property line at an elevation of +/- 258 feet above mean sea level (MSL). Drainage across the site is by uncontrolled sheet flow to the adjacent streets and drainage course, as well as by infiltration within unpaved areas.

The approximate site coordinates are latitude 34.0732°N and longitude 118.2869°W, and the site is located on the Hollywood, California 7½-Minute Quadrangle (United States Geological Survey, 1981).

2.2. Proposed Project

Based on the preliminary information provided by the project architect, the proposed project consists of removing the existing buildings and constructing approximately 80,000 sq. ft. multi-story building with a basement garage at the subject site. The basement will be located on the eastern portion of property. The design structural loads have not yet been provided to us as of the issuance of this report. We have assumed anticipated column loading to be approximately 500 kips (dead + live loads) and anticipated wall loading to be approximately 12 kips per foot, based on our experience with similar projects. Should the actual design loads differ significantly from these values, this office should be contacted to provide revised recommendations.

3. SCOPE OF WORK

To prepare this report, we have performed the following tasks:

3.1. Review of Background Information

We reviewed readily available background data including in-house geophysical data, geologic maps, topographic maps, and aerial photographs relevant to the subject site in preparation of this report. The list of literature reviewed is presented in the "Selected References" section of this report.

3.2. Field Exploration

The field exploration consisted of eight (8) exploratory, hollow stem auger borings and two (2), exploratory hand-dug test pits conducted at the site on August 30 and 31, and September 6, 2018. Two (2) of the borings were used to perform percolation testing. The soil borings were advanced to approximate depths ranging between 16.5 and 21.5 feet below the existing grades. The drilling operation was performed using an 8-inch diameter, truck-mounted, hollow-stem auger drill rig.

The approximate locations of the exploratory borings and test pits are shown in Figure 2, Site Geologic and Boring Location Map. Detailed exploration information of the soil borings is presented in Appendix A, Field Exploration.

3.3. Percolation Testing

Two percolation tests were conducted on August 30, 2018 to evaluate the feasibility of implementing an infiltration system for the proposed development and to evaluate design infiltration rates in accordance with County of Los Angeles guidelines. The details of our percolation testing procedures, field results, calculations, conclusions and recommendations are presented in Appendix C, Percolation Testing.

3.4. Geotechnical Laboratory Testing

Laboratory tests were performed on selected samples obtained from the borings to aid in soil classification and to evaluate the engineering properties of the foundation soils. Laboratory tests included in-situ moisture and density, #200 Sieve Wash, expansion index, direct shear, consolidation, and soil corrosivity. The detailed laboratory test results from the exploration are presented in Appendix B – Laboratory Testing.

3.5. Engineering Analyses and Report Preparation

We compiled and analyzed the data collected from our site reconnaissance, subsurface evaluation, and laboratory testing, and prepared this report to present our conclusions and recommendations, including:

- Evaluation of general subsurface conditions and description of types, distribution, and engineering characteristics of subsurface materials;
- Evaluation of geologic hazards, including site seismicity, liquefaction and seismic settlement potential, and preliminary recommendations for appropriate mitigation measures;
- Evaluation of site-specific seismic design parameters in accordance with 2016 California Building Code;

- Evaluation of current and historical groundwater conditions at the site and potential impact on the existing structures;
- Evaluation of project feasibility and suitability of on-site soils for foundation support;
- Evaluation of foundation design parameters including soil bearing capacity, lateral resistance, friction coefficient, and seismic considerations; and
- Evaluation of the potential for on-site materials to corrode buried concrete and metals,
- Recommendations for pavement structural sections, and
- Recommendations for stormwater quality control measures.

4. SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1. Regional Geology

According to the Lamar, D. L., 1970, the project site is underlain by bedrock of the Puente Formation (map symbol: Tp) consisting of interbedded siltstone, shale and occasional sandstone was encountered in all of our exploratory excavations at the site. Portions of the map are reproduced as Figure 3, Regional Geologic Map.

4.2. Subsurface Earth Materials

Geologic units encountered during our subsurface evaluation comprise undocumented fill overlying sedimentary bedrock. The following sections provide generalized descriptions of the materials encountered. It should be noted that the thickness of undocumented fill may vary across the site. Detailed information of soils borings is presented in Appendix A, Field Exploration.

Based on our field investigation and review of regional topographic maps, the site appears to have been originally graded circa pre-1928 by placing artificial fill with the western margin of a north-south trending ancestral canyon. The north-south trending canyon appears to be located along the eastern portion of the property (present location of Madison Avenue). Up to 15 feet of uncertified artificial fill was encountered in our exploratory borings on the east and northeast sides of the property. The depth of artificial fill should be anticipated to vary across the site.

Generalized descriptions of the subsurface materials encountered in the exploratory excavations and test pits at the site are presented in the following paragraphs. Detailed descriptions of the earth materials encountered in the exploratory borings and test pits are presented in Appendix A, Field Exploration. Schematic geologic cross sections illustrating the subsurface conditions at the site are presented in Figure 4, Geologic Cross Sections A-A’ and B-B’.

4.2.1.Fill

Fill, most likely associated with landscaping activities, with previous grading, and with paving of the site, was encountered in the exploratory borings. As encountered in our excavations, the thickness of fill extends approximately 1- to 18-feet below the existing grade. The deep fill appears to be located in the eastern and northeastern portions of the property and appears to be associated with the western margin of a fill north-south trending ancestral canyon. The fill generally consists of brown, damp, loose to medium dense, clayey sand and lean clay. A layer of ¾ inch gravel was discovered at approximately 10 to 15 feet below ground surface at Borings B-1 and B-8. Artificial fill thickness should be anticipated to vary across the site.

4.2.2. Bedrock of Puente Formation (Tp)

Interbedded siltstone, shale and occasional sandstone bedrock was encountered underlying the artificial fill in each of the exploratory borings and test pits. These materials extend from approximately 1 foot below ground surface to the total depth of exploration. The bedrock generally consist of interlayered orange brown, tan brown and gray, firm to hard, dry to slightly moist, poorly to well bedded,, moderately well cemented and slightly to moderately fractured sedimentary bedrock.

4.3. Bedding Plane Orientation

Based on the downhole logging of our exploratory test pits, the predominant orientation of the sedimentary bedrock is a northwest to east-west strike and a southwest to south dip ranging from about 6 to 23 degrees. The bedding plane orientations are consistent with that illustrated on the regional geologic maps of the site's vicinity. West- and south-facing temporary excavations are anticipated to present a component of unsupported bedding (“daylighted bedding construction”), which will require mitigation during construction. Recommendations for excavations exposing favorably-oriented and daylighted bedding conditions are provided in the “Temporary Excavations” section of this report.

4.4. Groundwater

Groundwater was not encountered in our soil borings and test pits to a maximum depth of 21.5 feet below ground surface. Based on the Seismic Hazard Report the historical high depth to groundwater is not well defined at the project site (California Department of Conservation, Division of Mines and Geology, 1998).

It is our opinion that constant groundwater is not expected to be encountered during construction of the proposed site development. However localized perch groundwater should be anticipated. 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.

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5. GEOLOGIC HAZARDS AND SEISMIC DESIGN CONSIDERATIONS

The site is 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 improvements. The hazards associated with seismic activity near the site are discussed in the following sections.

5.1. Surface Fault Rupture and Active Faulting

The subject site is not located within a State of California Earthquake Fault Zone (formerly known as a Special Studies Zone) (California Geological Survey 2018). It is our opinion that the likelihood of fault rupture occurring at the site during the design life of the proposed improvements is low.

Active faults are defined as those that have experienced surface displacement within Holocene time (approximately the last 11,000 years). The nearest known fault corresponds to the Upper Elysian Park fault system located approximately 2.65 km northeast of the site. This system has the potential to be the dominant source of strong ground motion. Table 1 lists selected known active faults within a search radius of 100 km, approximate fault-to-site distances, maximum moment magnitude (M_{max}), and fault type as published by the 2008 USGS National Seismic Hazards Maps website (USGS, 2008). The approximate site location relative to the major faults in the southern California region is presented in Figure 5, Fault Location Map.

Table 1 – Principal Active Faults

Fault	Approximate Fault-to-Site Distance[1] km	Maximum Moment Magnitude [2] (M_{max})
Elysian Park (upper)	2.65	6.7
Santa Monica (connected alt. 2)	3.62	7.4
Hollywood	4.45	6.7
Puente Hills (LA)	6.24	7.0
Raymond	7.92	6.8
Newport-Inglewood (connected alt. 2)	9.96	7.5
Newport-Inglewood (connected alt. 1)	10.07	7.2
Verdugo	10.52	6.9
Santa Monica (connected alt. 1)	11.61	7.3
Sierra Madre	17.70	7.3
Sierra Madre (San Fernando)	22.54	6.7
Malibu Coast (alt. 1)	22.62	6.7
Malibu Coast (alt. 2)	22.62	7.0
Puente Hills (Santa Fe Springs)	22.93	6.7
Elsinore (connected)	23.63	7.8
Anacapa-Dume (alt. 2)	25.25	7.2
Palos Verdes	26.89	7.3
San Gabriel	27.15	7.3
Clamshell-Sawpit	28.90	6.7
Puente Hills (Coyote Hills)	30.03	6.9
Santa Susana (alt. 1)	33.77	6.9

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San Jose	37.77	6.7
Anacapa-Dume (alt. 1)	39.07	7.2
Simi-Santa Rosa	45.64	6.9
Chino (alt. 2)	50.10	6.8
Chino (alt. 1)	50.16	6.7
Cucamonga	50.74	6.7
San Joaquin Hills	53.15	7.1
Oak Ridge (connected)	54.00	7.4
San Andreas (connected)	54.62	8.0
San Cayetano	59.52	7.2
Newport-Inglewood (offshore segment)	63.57	7.0
Elsinore (connected)	69.81	7.7
San Jacinto (connected)	70.82	7.8
S. San Andres (connected)	73.09	8.0
Cleghorn	80.20	6.8
Santa Ynez (connected)	80.28	7.4
Channel Islands Thrust	90.36	7.3
Oak Ridge (offshore)	93.14	7.0
Coronado Bank	94.41	7.4
Garlock (connected)	99.22	7.69

Notes: [1] United States Geological Survey (2008)
[2] Ellsworth Relation, United States Geological Survey (2008)

5.2. Historical Seismicity

The epicentral locations of selected historic earthquakes registered during the 1800 through 1999 period in southern California have been plotted by the California Division of Mines and Geology (Toppozada and others, 2000). A reproduction of this map in the vicinity of the project site is presented as Figure 6, Historical Seismicity, 1800-1999.

Within historical times, strong shaking from earthquakes generated along several active faults in the region has affected the site. The most prominent of these earthquakes, based on magnitude and proximity to the site, are listed in Table 2 (Southern California Earthquake Center, 2014). A short description of each of these events is provided below. As with all significant earthquakes in southern California, these temblors were followed in close proximity temporally by numerous aftershocks, some of which were sufficiently large to cause additional strong shaking at the site.

A short description of the historical earthquakes affecting the site is provided below. It should be noted that the available records do not indicate that any structural damage or ground failure occurred at the site as a consequence of any of these events.

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Table 2 - Historical Earthquakes Affecting the Site

Date	Earthquake Name	Fault(s)	M_W or M_L
December 8, 1812	Wrightwood	San Andreas	7.5
March 11, 1933	Long Beach	Newport-Inglewood	6.4
February 9, 1971	San Fernando	San Fernando	6.5
October 1, 1987	Whittier Narrows	Puente Hills thrust	5.9
June 28, 1991	Sierra Madre	Clamshell-Sawpit Canyon	5.8
January 17, 1994	Northridge	Northridge thrust	6.7

5.2.1.December 8, 1812

The so-called Wrightwood earthquake of estimated M=7.5 occurred in the morning of December 8, 1812. Recent research indicates that the earthquake probably occurred along the San Andreas fault, possibly generating as much as 106 miles of surface fault rupture between Tejon Pass and Cajon Pass. Strong shaking resulted in the deaths of 40 Native American worshipers from the collapse of the mission church in San Juan Capistrano. Additional damage was reported at Mission San Gabriel, although this damage may have been caused by another earthquake on December 21, 1812.

5.2.2.March 11, 1933

The so-called Long Beach earthquake actually was centered near the City of Newport Beach. The M=6.4 temblor along the Newport-Inglewood fault zone resulted in 120 deaths and over $50 million in property damage. Particularly hard hit were school buildings constructed of unreinforced masonry in the City of Long Beach. Legislative activity following this event included passage of the Field Act, which required that earthquake forces to be taken into account in the structural design of public school facilities.

5.2.3.February 9, 1971

The San Fernando earthquake struck in the morning of February 9, 1971. The M_W=6.5 temblor ruptured the ground surface for approximately 12 miles along the San Fernando fault zone in the San Fernando-Sylmar area. Sixty-five deaths and over $500 million in property damage were recorded, with some of the greatest damage occurring at the Veterans Administration Hospital and the Olive View Community Hospital in Sylmar. Legislation following this event included implementation of the Alquist-Priolo Special Studies Zones Act, which directed the State Geologist to delineate zones in which the hazard of surface rupture from active faults would be mitigated.

5.2.4.October 1, 1987

An M_L=5.9 occurred on a previously-unknown blind thrust fault in the Whittier Narrows area of eastern Los Angeles County during the morning of October 1, 1987. The earthquake caused eight fatalities and caused approximately $358 million in property damage, primarily to unreinforced masonry structures in "Uptown" Whittier, downtown Alhambra, and the Old Town section of Pasadena. The campus of California State University, Los Angeles suffered approximately $20 million in damage.

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5.2.5. June 28, 1991

In the morning of June 28, 1991 an M_L=5.8 earthquake occurred along the Clamshell-Sawpit Canyon fault, an offshoot of the Sierra Madre fault zone. Damage totaling approximately $40 million occurred in the San Gabriel Valley, primarily affecting buildings constructed of unreinforced masonry. Two deaths were attributed to this event – one from falling debris in Arcadia, and one from a heart attack in Glendale.

5.2.6. January 17, 1994

Much of the population of the San Fernando Valley and adjacent areas of southern California was awakened in the early morning of January 17, 1994 by a M_W=6.7 earthquake centered in the Northridge district of Los Angeles. The temblor occurred on a previously-unknown blind thrust fault. Structural damage in the San Fernando Valley and some portions of the northern Los Angeles Basin was extensive, including collapsed apartment buildings, collapsed freeway overpasses, and major damage to office buildings and parking structures. The event killed 57 people, including 16 who perished in the collapse of the Northridge Meadows apartment complex.

5.3. Liquefaction and Seismic Settlement Potential

Liquefaction occurs when the pore pressures generated within a soil mass approach the effective overburden pressure. Liquefaction of soils may be caused by cyclic loading such as that imposed by ground shaking during earthquakes. The increase in pore pressure results in a loss of strength, and the soil then can undergo both horizontal and vertical movements, depending on the site conditions. Other phenomena associated with soil liquefaction include sand boils, ground oscillation, and loss of foundation bearing capacity. 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.

Based on our review of the State of California Official Map of Seismic Hazard Zones for the Hollywood Quadrangle (California Department of Conservation, Division of Mines and Geology, 1998), the site is not located within a zone of required investigation for Liquefaction. A portion of the seismic hazard zones map is reproduced as Figure 7, Seismic Hazard Zones Map.

Due to the presence of bedrock at a shallow depth, it is our opinion that the site is not susceptible to soil liquefaction. Seismically-induced settlement is considered negligible.

5.4. Landslides

Based on our review of the referenced geologic maps, literature, topographic maps, aerial photographs, and our subsurface evaluation, no landslides or related features underlie or are adjacent to the subject site. Due to the relatively level nature of the site and surrounding areas, the potential for landslides at the project site is considered negligible.

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. Based on our review of online FEMA flood mapping, the site is located within Zone X, which is described as "Areas determined to be outside the 0.2% annual chance floodplain."

5.6. Tsunami/Seiche

Tsunamis are waves generated by massive landslides near or under sea water. The site is not located within the mapped tsunami inundation areas defined by Tsunami Inundation Maps for Emergency Planning (California Emergency Management Agency, 2009). On this basis, it is our judgment that the potential for the site to be adversely impacted by earthquake-induced tsunamis is low.

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 negligible due to the lack of any significant enclosed bodies of water located in the immediate vicinity of the site.

5.7. CBC Seismic Design Parameters

In accordance with 2016 CBC and ASCE 7-10 (ASCE, 2010) standards, the seismic design parameters for the site are presented in Table 3.

Table 3 – 2016 California Building Code Design Parameters

Design Parameters	Value
Site Class	C
Mapped Spectral Acceleration Parameter at Period of 0.2-Second, S_s	2.503g
Mapped Spectral Acceleration Parameter at Period 1-Second, S_1	0.891g
Site Coefficient, F_a	1.0
Site Coefficient, F_v	1.3
Adjusted MCE_R[1] Spectral Response Acceleration Parameter at Short Period, S_{MS}	2.503g
1-Second Period Adjusted MCE_R[1] Spectral Response Acceleration Parameter, S_{M1}	1.158g
Short Period Design Spectral Response Acceleration Parameter, S_{DS}	1.669g
1-Second Period Design Spectral Response Acceleration Parameter, S_{D1}	0.772g
Peak Ground Acceleration, PGA_M[2]	0.946g
Seismic Design Category[3]	E

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

Our recommendations for design earthquake magnitude parameters have been developed in accordance with the USGS Earthquake Hazards Unified Hazard Tool webpage https://earthquake.usgs.gov/hazards/interactive/ for the 2 percent in 50 years chance of exceedance earthquake event. Based on the calculated results, the earthquake magnitude, M_w=7.0 should be considered in seismic design.

5.8. Site Specific Ground Motion Hazard Analysis

The site-specific ground motion hazard analysis was performed in accordance with Section 21.2 of ASCE 7-10 (American Society of Civil Engineers, 2010) based on a 2% probability of exceedance in 50 years. Probabilistic and deterministic maximum considered earthquake (MCE) response accelerations were evaluated in order to develop the site-specific design response spectrum.

The derivation of the site-specific design response spectra, including the probabilistic and deterministic seismic hazard analyses, are presented in Figure 8, Site-Specific Design Response Spectrum. The detailed analysis description and results are presented below.

Our analysis was performed using the computer program EZ-FRISK by Risk Engineering (v. 7.65). Our input parameters for the site-specific design response spectrum are summarized in Table 4.

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.

The design spectral response acceleration at each period was calculated as two-thirds of the site-specific MCE_R spectral response acceleration, but taken as not less than 80 percent of the spectral response acceleration evaluated in accordance with Section 11.4.5 of ASCE 7-10.

Table 4 – Inputs for Site-Specific Design Response Spectrum Analysis

Input Parameter	Value
Latitude	34.0732
Longitude	-118.2869
Shear Wave Velocity, V_{30}	360 m/s
Depth to V_s =1000 m/s	100 m
Fault Search Radius	100 km
Fault Database	2008 CGS Statewide Fault Model
NGA Attenuation Relations	Boore and Atkinson (2008) Campbell and Bozorgnia (2008) Chiou and Youngs (2008)
Maximum Rotated Component Relations	Huang et al. (2008)

Applicable response spectra data are presented in Table 5 and on Figure 9, Site-Specific Design Response Spectrum.

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Table 5 - Site-Specific Design Response Spectrum Data

Period (sec)	2% in 50yr Probabilistic Spectral Acceleration (g)	Risk Coefficient C_R	Probabilistic MCE_R Spectral Acceleration (g)	84th Percentile Deterministic Spectral Acceleration (g)	Deterministic Lower Limit (g)	Site Specific MCE_R Spectral Acceleration (g)	80% CBC Map-based General Response Spectrum (g)	Site Specific Design Spectral Acceleration (g)
0.01	1.107	0.950	1.052	1.363	0.687	1.107	0.621	0.738
0.03	1.250	0.950	1.188	1.450	0.860	1.250	0.794	0.833
0.05	1.354	0.950	1.286	1.698	1.033	1.354	0.967	0.967
0.10	1.952	0.950	1.854	2.302	1.465	1.952	1.335	1.335
0.20	2.439	0.950	2.317	2.928	1.500	2.439	1.335	1.626
0.30	2.386	0.951	2.269	2.955	1.500	2.386	1.335	1.591
0.40	2.300	0.952	2.189	2.911	1.500	2.300	1.335	1.533
0.50	2.194	0.953	2.090	2.799	1.500	2.194	1.236	1.463
0.75	1.700	0.955	1.623	2.216	1.040	1.700	0.824	1.133
1.00	1.365	0.957	1.306	1.799	0.780	1.365	0.618	0.910
2.00	0.620	0.957	0.594	0.831	0.390	0.620	0.309	0.414
3.00	0.387	0.957	0.370	0.554	0.260	0.387	0.206	0.258
4.00	0.283	0.957	0.271	0.420	0.195	0.283	0.154	0.189

The site-specific design response parameters are provided in Table 6. These parameters were evaluated from Design Response Spectra presented in table above following guidelines of ASCE 7-10 Section 21.4.

Table 6 – Site-Specific Seismic Design Parameters

Site-Specific Seismic Design Parameters	Design Values (g)
Spectral Response Acceleration 0.2-second period, S_{MS}	2.439
Spectral Response Acceleration 1-second period, S_{M1}	1.365
Design Spectral Response Acceleration for short period, S_{DS}	1.626
Design Spectral Response Acceleration for 1-second period, S_{D1}	0.910

6. GEOTECHNICAL ENGINEERING RECOMMENDATIONS

6.1. General Considerations

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

The eastern portion of site is underlain by undocumented fill up to 17 feet in thickness. Mitigation of undocumented fill is required. Cast-In-Drilled-Hole (CIDH) piles bearing into bedrock are recommended for the proposed structures located within this area.

If the new structures are located outside the deep undocumented fill, shallow foundation bearing into bedrock can be used.

Our geotechnical engineering analyses performed for this report were based on the earth materials encountered during the subsurface exploration for the site. If the design substantially changes, then our geotechnical engineering recommendations would be subject to revision based on our evaluation of the changes. The following sections present our conclusions and recommendations pertaining to the engineering design for this project.

6.2. Expansive Soil Evaluation

Expansive soils are characterized by their ability to undergo significant volume changes (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from rainfall, landscape irrigation, utility leakage, roof drainage, perched groundwater, drought, or other factors, and may cause unacceptable settlement or heave of structures, concrete slabs supported on-grade, or pavements supported over these materials. Depending on the extent and location below finished subgrade, these soils could have a detrimental effect on the proposed construction.

Based on our field soil classification, the near surface silty soil is considered to have a "Moderate" expansion potential. We have incorporated the mitigation measures into the geotechnical recommendations presented in this report.

6.3. Corrosive Soils

The potential for the near-surface on-site materials to corrode buried steel and concrete improvements was evaluated. Laboratory testing was performed on one representative sample of on-site soils 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.

In accordance with the County of Los Angeles (2014) criteria, corrosive soil is defined as the soil has minimum 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.

6.3.1. Reinforced Concrete

Laboratory tests indicate that the potential for sulfate attack on concrete in contact with the on-site soils is negligible in accordance with ACI 318, Table 4.3.1. As a minimum, we recommend that Type I or II cement and a water-cement ratio of no greater than 0.5 be used on the project.

Test results also indicate that the potential for chloride attack of reinforcing steel in concrete structures and pipes in contact with soil is negligible.

6.3.2. Metallic

Laboratory resistivity testing indicates that the on-site near-surface soils are considerred corrosive to buried ferrous metals. A corrosion specialist may be consulted regarding suitable types of piping and appropriate protection for underground metal conduits, if needed.

6.4. Methane Zone

Based on our review of the City of Los Angeles Methane and Methane Buffer Zones map, the subject property is located within a Methane Zone. A qualified methane specialist should be consulted to evaluate the potential methane hazard and mitigation measures.

6.5. Site Preparation and Earthwork

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.

6.5.1. 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 generally 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 selectively removed and disposed offsite. The debris and unsuitable material generated during clearing and grubbing should be removed from areas to be graded and disposed at a legal dump site away from the project area.

6.5.2. Overexcavation

All proposed buildings should be supported by bedrock using CIDH piles and footings. New structures located within the area with undocumented fill deeper than 3 feet should be supported CIDH piles. The concrete slab within deep fill area should be designed as structural slab that transfers load to the piles.

For at-grade structures, such as concrete sidewalk, paving, and hardscape, due to presence of undocumented fill, the subgrade should over-excavated to a depth of at least 12 inches below the pavement section, or to depth exposing bedrock, whichever is shallower, and then recompacted in accordance with Section 6.5.4 of this report.

The extent and depths of removal should be evaluated by Twining's representative in the field based on the materials exposed. Additional removals may be recommended if loose or soft soils are exposed during grading.

6.5.3.Materials for Fill

On-site soils with an organic content of less than 3 percent by volume (or 1 percent by weight) are suitable for use as fill. Soil material to be used as fill should not contain contaminated materials, rocks, or lumps over 4 inches in largest dimension, and not more than 40 percent larger than ¾ inch. Utility trench backfill material 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.

Any imported fill material should consist of granular soil having a "very low" expansion potential (that is, 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). Materials to be used as fill should be evaluated by a Twining representative prior to importing or filling.

6.5.4. Compacted Fill

Prior to placement of compacted fill, the exposed excavation bottoms should be observed by Twining. Unless otherwise recommended, the exposed ground surface should then be scarified to a depth of approximately 6 inches and watered or dried, as needed, to achieve generally consistent moisture contents at or near the optimum moisture content. The scarified materials should then be compacted to 90 percent relative compaction in accordance with the latest version of ASTM Test Method D1557.

Fill materials should be moisture conditioned to approximately 2% above optimum moisture content prior to placement. The optimum moisture content will vary with material type and other factors. Moisture conditioning of fill soils should be generally consistent within the soil mass. Continue to place the compacted fill in horizontal lifts of approximately 6 to 8 inches in loose thickness. Prior to compaction, each lift should be watered or dried as needed, mixed, and then compacted by mechanical methods, using multiple wheel pneumatic tired rollers, sheepsfoot rollers, or other appropriate compacting rollers, to a relative compaction of 90 percent as evaluated by the latest version of ASTM D1557. Successive lifts should be treated in a like manner until the desired finish grades are achieved. Within pavement areas, the upper 12-inches of subgrade soil should be compacted to 95 percent relative compaction evaluated by ASTM D1557.

The evaluation of compaction by Twining should not be considered to preclude any requirements for observation or approval by governing agencies. It is the contractor's responsibility to notify Twining and the appropriate governing agency when project areas are ready for observation, and to provide reasonable time for that review.

6.5.5. Utility Trench Backfill

Trench excavations to receive backfill shall be free of trash, debris or other unsatisfactory materials at the time of backfill placement. The utility should be bedded with clean sand to at least one foot over the crown. The bedding sand should have a sand equivalent (SE) of 30 or greater. The remainder of trench backfill may be onsite soils compacted to 90 percent of the laboratory maximum dry density as per ASTM Standard D1557.

6.5.6.Temporary Excavations

Temporary excavations for the demolishing, earthwork, retaining walls, footing and utility trench are expected. Please note that west- and south-facing excavations are anticipated to expose artificial fill over a component of unsupported bedding ("daylighted bedding condition"). North- and east-facing excavations are anticipated to expose artificial fill over favorably oriented bedrock.

We anticipate that unsurcharged excavations with vertical side slopes less than 4 feet high will generally be stable; however, some sloughing of loose materials encountered at the site should be expected.

For west- and south-facing excavations exposing unsupported bedding and excavations that undermine the existing building or street, temporary shoring is recommended. Temporary shoring design recommendations are presented in Section 6.11 of this report.

For east- and north-facing excavations, where the space is available, temporary, unsurcharged excavation sides over 5 feet in height should be sloped no steeper than an inclination of 1H:1V (horizontal:vertical). Where sloped excavations are created, the tops of the slopes should be barricaded so that vehicles and storage loads do not encroach within 10 feet of the top of the excavated slopes. A greater setback may be necessary when considering adverse bedrock orientation and 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.

Where space for sloped excavations is not available but exposing favorable bedding, temporary shoring and slot cut may be utilized. Temporary shoring design recommendations are presented in Section 6.11 of this report. Slot cuts may be utilized for temporary excavations that are less than 6 feet in height in the areas of favorably oriented bedrock. The slots should be no wider than 8 feet and should be excavated in an A-B-C sequence so that there are at least 16 feet spacing between any two excavated slots. The excavated slots should not be left open overnight and should be backfilled on the same day it was excavated before the next set of slots are excavated.

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.

6.5.7.Rippability

The fill and bedrock materials should be generally excavatable with heavy-duty earthwork equipment in good working condition. However localized cemented bedrock may require heavy ripping or special handling. Directional ripping and downsizing breakers may be required.

6.5.8. Shrinkage/Bulking Due to Compaction

Based on our review of the in-situ soil density data, preliminary volumetric shrinkage on the order of 10 to 15 percent as a result of compaction of onsite soil may be assumed.

6.5.9. Excavation Bottom Stability

In general, we anticipate that excavation bottoms of the excavations will be stable and should provide suitable support for the proposed improvements. Unstable bottom conditions may be mitigated by over-excavation of the bottom to suitable depths, and/or replacement with a minimum 1 foot-thick aggregate base, and/or other mitigation options based on the field evaluation. Recommendations for stabilizing excavation bottoms should be based on evaluation in the field by the geotechnical consultant at the time of construction.

6.6. Spread Footings Recommendations

A shallow foundation system may be used for support of the new structures located outside the deep undocumented fill area, provided that all the footings are placed on competent bedrock. The recommended geotechnical foundation design parameters are presented in Table 2 below.

Table 7 – Geotechnical Foundation Design Parameters

Minimum Footing Dimensions	• Continuous footing: At least 12 inches in width, and at least 18 inches in depth. • Square footing: At least 24 inches in width and at least 18 inches in depth.
Allowable Net Bearing Pressure	• Footing should be supported on bedrock. • For building foundations with the minimum dimensions shown above, a net bearing pressure of 3,000 pounds per square foot (psf) can be used. • Bearing capacity can increase 350 psf for each additional foot of width, and 500 psf for each additional foot of depth to a maximum allowable capacity of 6,500 psf. • The allowable bearing values may be increased by one-third for transient live loads from wind or earthquake.
Estimated Static Settlement	• Less than 0.5 inches total settlement with differential settlement estimated to be less than 0.25 inch over 50 feet. • The static settlement of the foundation system is expected to occur on initial application of loading.
Allowable Coefficient of Friction Below Footings	0.35
Allowable Lateral Passive Resistance	400 pcf (equivalent fluid pressure)

The allowable passive resistance values may be increased by one-third when considering wind or seismic loading.

6.7. Cast-In-Drilled-Hole (CIDH) Piles

6.7.1. Design Pile Capacities

Where the proposed structures located within the deep fill area, Cast-In-Drilled-Hole (CIDH) concrete piles utilizing the frictional resistance derived from the soils should be used to support structures, where structural load demand is high or at locations require deep foundations. The CIDH piles should be at least 18 inches in diameter and at least 8 feet bearing into bedrock. A capacity curve for 24”- and 30”-diameter piles has been attached as Figure 9 – CIDH Pile Axial Capacity. The uplift pile capacity can be calculated as one-half of compressive capacity for design. Pile spacing less than 3 pile diameters should consider group effect in design. Detailed recommendations for group effect will be provided upon request, where needed.

The lateral passive resistance of the soil in terms of equivalent fluid pressure (EFP) should be used to determine the lateral capacity of the piles. A passive pressure of 400 psf per foot of depth should be used for design. The allowable bearing values and passive lateral pressure parameters may be increased by one-third for short term loads such as wind or seismic forces.

6.7.2. General Construction Guidelines for CIDH Piles

Excavation of CIDH piers: The contractor should submit the proposed excavation method to the project geotechnical engineer for review of compatibility with the design assumptions. The pile excavation bottoms should be cleaned, and any loose material and debris that falls into the excavations should be removed prior to placement of reinforcing steel and concrete. Excavations should be performed under the observation of the geotechnical engineer.

The drilling for piles should not be performed adjacent to recently excavated or recently poured piles until the concrete in the completed piles has been allowed to set for several hours. The minimum recommended spacing between adjacent pours may be taken as 6 times the pile diameters. Piles in groups should be drilled and poured in an alternating sequence to minimize the potential for fresh concrete flowing into adjacent open pile excavations.

Concrete Placement: The concrete for the piers should be placed using a downhole tremie, or similar provision such that the falling concrete does not strike the sides of the shaft. Concrete should be placed in newly excavated piers as soon as practical. Under no circumstances should the pier excavation be allowed to remain open for more than 12 hours. The concrete must be capable of propagating between the reinforcing bars to come in contact with the soil and to avoid arching during extraction of the casing. A minimum slump of 5 inches is recommended. A head of 5 feet of concrete above the bottom of casing (if used) must be maintained during casing extraction. The presence of water at the bottom of pier excavations will require downhole tremie placement such that the tremie pipe is placed below the water level and forces the water up and out of the hole. Seepage or ground water in the excavations should not be vibrated or otherwise incorporated into the foundation concrete.

Tolerances: Quality of construction is of primary importance in the construction of CIDH piers. The timely placing of concrete and the installation within specified tolerances must be respected. The pier must remain within 2 percent of vertical to develop the allowable capacities provided in this report.

Observation: Drilling of pile shafts should be observed by the Geotechnical Consultant to confirm that piles are extended to the proper depth and that material encountered is similar to that encountered in the borings drilled for this study. Pile lengths should be tabulated in the foundation plans based upon the embedment below the bottom of the pile cap or other point of reference that can be established in the field during construction.

Full-time observation of the pile construction by the geotechnical engineer is required. The observation work should provide documentation of the pier construction. We suggest that, before construction, the conditions of nearby existing structures be documented. In addition, instrumentation of certain structures may be warranted during and after the construction operations to monitor movement.

6.8. Concrete Slabs

Slabs within pile-supported building should be designed as structural slabs supported by piles. Slabs outside deep undocumented fill area should be supported on bedrock or compacted fill. For design of concrete slabs, a modulus of subgrade reaction (*k*) of 150 pounds per cubic inch (pci) may be used. For slabs not supporting heavy loads, we recommend that the concrete should have a thickness of at least 4 inches. Floor slabs reinforcement and control joints should be designed and constructed in accordance with recommendations from the structural engineer or architect.

6.9. Subgrade Preparation for Concrete Slabs

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 granular material should be dry to moist, and should not be wetted or saturated prior to the placement of concrete. The concrete slab should be allowed to cure properly prior to placing vinyl or other moisture-sensitive floor covering. Table 4 provides recommendations for various levels of protection against vapor transmission through concrete floor slabs placed over a properly prepared subgrade.

Table 8 – Options for Subgrade Preparation below Concrete Floor Slabs

Primary Objective	Recommendation
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. The on-site sandy soils appear to meet these criteria.

[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., 2015).

[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.

The recommendations presented above 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.

6.10. Basement and Retaining Walls

6.10.1. Static Lateral Earth Pressure

The values presented below assume that the supported grade is level and that surcharge loads are not applied. The recommended design lateral earth pressure is calculated assuming that a drainage system will be installed behind the basement walls and that external hydrostatic pressure will not develop behind the walls.

For walls that are free to rotate at the top (such as cantilevered walls) and have adequate drainage, may be designed for the “active” earth pressure using an EFP of 35 pcf. Walls that are supporting earth that has adequate drainage, and are restrained against rotation at the top (such as by a floor deck), may be designed for the “at-rest” earth pressure using an equivalent fluid pressure (EFP) of 60 pcf.

Vertical surcharge loads within a 1:1 projection from the bottom of the wall distributed over retained soils should be considered as additional uniform horizontal pressure acting on the wall. The additional horizontal pressure acting on the wall can be estimated as approximately 30% and 50% of the vertical surcharge pressure for the “active” and “at-rest” conditions, respectively. All permanent surcharge loading conditions should be evaluated on a case-by-case basis by the geotechnical engineer.

6.10.2. Seismic Lateral Earth Pressure

Retaining walls greater than 6 feet in height should be designed for seismic earth pressures. For structural design purpose, the “seismic earth pressure” can be considered as the sum of “static earth pressure” and “incremental seismic pressure”. We recommend a “seismic earth pressure” in terms of an EFP of 65 pcf be used for both cantilever and restrained wall design. A triangular pressure distribution can be used for design, and the resultant force can be assumed to be a 1/3 of the height of the wall from the wall base. The “incremental seismic pressure” can be calculated as the difference of “seismic earth pressure” and “static earth pressure.”

6.10.3. Backfill and Drainage of Walls

Retaining walls should be adequately drained. Adequate backfill drainage is essential in order to provide a free-drained backfill condition and to limit hydrostatic buildup behind walls. The walls should be appropriately waterproofed.

Subdrain behind retaining walls should consist of a 4-inch-diameter perforated PVC pipe (holes facing down) encased in at least 12 inches of ¾-inch gravel wrapped in non-woven filter fabric (Mirafi 140NL or equivalent) placed continuously along the back of wall. The subdrain should discharge through a solid pipe to an appropriate outlet or sump/pump system.

Where retaining walls are constructed against temporary shoring, subdrain may be provided by a geosynthetic drainage composite such as TerraDrain, MiraDrain, or equivalent, attached to the outside perimeter of the wall. A minimum 1-cubic-foot rock pockets should be installed at 8 feet on center penetrating wood lagging between soldier beams. The drain pocket should discharge through a solid pipe or weep hole to an appropriate outlet, using a sump/pump system.

The backfill material above the subdrain may use on-site soil or imported granular non-expansive material approved by the project geotechnical engineer.

6.11. Temporary Shoring

It is anticipated that temporary shoring will be required along the perimeter of the proposed basement. Based on the assumed finished floor elevation and anticipated foundation excavations, shored walls may be on the order of 10 to 15 feet high.

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 may need to be accomplished with the aid of tied-back 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 for conformance with the design intent and recommendations. Further, the shoring system should satisfy applicable requirements of CalOSHA.

6.11.1. Lateral Pressures

For design of cantilevered shoring, 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 35 pcf.

Tied-back or braced shoring should be designed to resist a trapezoidal distribution of lateral earth pressure as shown below in Diagram 1. The recommended pressure distribution, for the case where the grade is level behind the shoring, the maximum pressure equal to 22H in psf, where H is the height of the shored wall in feet.

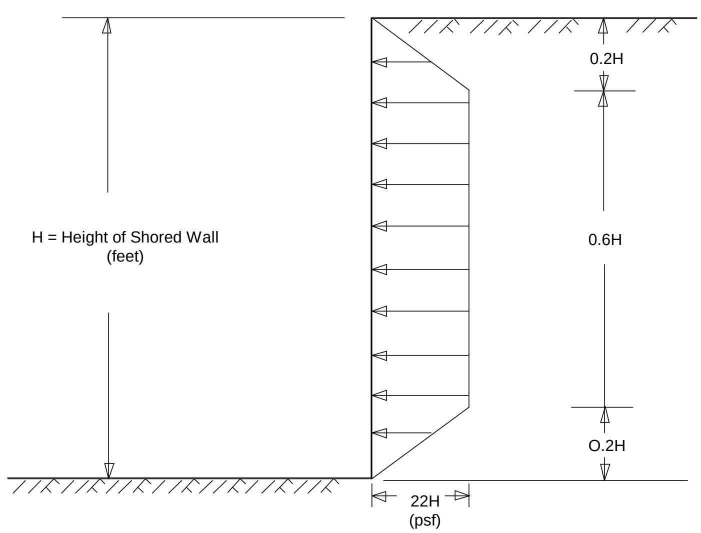

Diagram 1 – Earth Pressure Distribution for Tie-back or Braced Shoring Wall

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 0.30. 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 240 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 should be designed in accordance with the geotechnical parameters presented in Table 3. Soldier piles should be spaced no closer than 2.5D on center, where D is the diameter of the drilled shaft for the soldier piles.

Table 9 – Geotechnical Design Parameters for Soldier Piles

The lateral resistance of an isolated soldier pile drilled or driven into the on-site soils can be calculated using unfactored lateral passive resistance equivalent fluid pressure (EFP)	350 pcf
Increase (multiplier) of the ultimate lateral passive resistance due to arching (this value is applicable for soldier piles that are spaced no closer than 2.5 diameters on center)	2.0

The downward component of a tie-back 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 coefficient of friction between the soldier piles and the retained earth may be taken as 0.35 times the horizontal component of anchor load. The allowable downward capacity of a soldier pile below the excavated level may be estimated using an average allowable unit skin friction of 400 psf per foot of embedment below the excavation bottom. This allowable unit skin friction incorporates a factor of safety of 2.0. The upper 1.5D should be neglected when calculating the axial capacity below the excavated level.

6.12. Flexible Pavement Design

Our pavement structural design is in accordance with Chapter 600 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.

We used an assumed R-value of 26 for the subgrade and assumed TI values for our asphalt pavement structural calculations. On this basis, Table 9 provides recommended minimum thicknesses for hot mix asphalt (HMA) and aggregate base sections for different traffic indices.

Table 10 – Recommended Minimum HMA and Base Section Thicknesses

Location	Light Vehicle Parking	Firelane / Truck Drive Way
Traffic Index	**5.0**	**6.0**
HMA Thickness (in)	4.0	4.0
Aggregate Base Thickness (in)	4.0	8.0

Aggregate base should be compacted to 95 percent relative compaction in accordance with the ASTM Test Method D1557.

6.13. Rigid Pavement Design

Table 10 provides minimum thicknesses for Portland Cement Concrete (PCC) pavement sections constructed on top of properly prepared subgrade and aggregate base section compacted to 95 percent of the maximum dry density in accordance with ASTM D1557.

Table 11 – Recommended Minimum PCC Section Thicknesses

Location	Light Vehicular Parking	Firelane / Truck Drive Way
Traffic Index	**5.0**	**6.0**
PCC Thickness (in)	6.0	7.0
Aggregate Base Thickness (in)	4.0	4.0

The above pavement section is based on a minimum 28-day Modulus of Rupture (M-R) of 550 psi and a compressive strength of 3,000 psi. Transverse contraction joints should not be spaced more than 15 feet and should be cut to a depth of ¼ the thickness of the slab. Longitudinal joints should not be spaced more than 15 feet apart, however, are not necessary in the pavement adjacent to the curb and gutter section. 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. The subgrade surface should be scarified to a depth of approximately 6 inches and watered or dried, as needed, to achieve generally consistent moisture contents at or near the optimum moisture content. The scarified materials should then be compacted to 95 percent relative compaction in accordance with the ASTM Test Method D1557.

6.14. Surface Drainage Control

The control of surface water is essential to the satisfactory performance of the site improvements. Surface water should be controlled so that conditions of uniform moisture are maintained beneath the structure, 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 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 drain pipes.

- Planters should not be located adjacent to the structure wherever possible. If planters are to be located adjacent to the structure, 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 the 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. 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.

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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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9. SELECTED REFERENCES

American Concrete Institute, 2011, Building Code Requirements for Structural Concrete (ACI 318 11) and Commentary (ACI 318R-11).

American Society of Civil Engineers, 2010, Minimum Design Loads for Buildings and Other Structures: ASCE Standard ASCE/SEI 7-10, 608 pp.

California Department of Conservation, California Geological Survey, 2007, Fault-Rupture Hazard Zones in California: Special Publication 42, 54 pp.

California Department of Conservation, California Geological Survey, 2008, Guidelines for Evaluation and Mitigation of Seismic Hazards in California: Special Publication 117A, 98 pp.

California Department of Conservation, Division of Mines and Geology, 1998, Seismic Hazard Zone Report for the Hollywood 7.5-Minute Quadrangle, Los Angeles County, California: Seismic Hazard Zone Report 034, 47 pp. plus 3 plates.

California Department of Conservation, California Geological Survey, 1999, State of California Seismic Hazard Zones, Hollywood Quadrangle, Official Map, dated March 25, scale 1:24,000.

California Department of Transportation, 2012, Corrosion Guidelines, Version 2.0, November.

County of Los Angeles, 2014, "County of Los Angeles Department of Public Works - Geotechnical and Materials Engineering Division, Administrative Manual, Guidelines for Design, Investigation, and Reporting, Low Impact Development Stormwater Infiltration" GS200.1, dated December 31, 2014.

County of Los Angeles, 2014, "Low Impact Development Standards Manual" dated February, 2014.

Dibblee, T.W. and Ehrenspeck, H.E., ed., 1991, Geologic map of the Hollywood and Burbank (south ½) quadrangles, Los Angeles, California: Dibblee Geological Foundation, Dibblee Foundation Map DF-30, scale 1:24,000

Federal Emergency Management Agency (FEMA), 2008, Flood Insurance Rate Map (FIRM), Los Angeles County, California, Panels 1610F and 1620F of 2350

Lamar, D. L., 1970, Geology of the Elysian Park-Repetto Hills Area, Los Angeles County, California, California Division of Mines and Geology, Special Report 101, 45pp.

Public Works Standards, Inc., 2015, Standard Specification for Public Works Construction "Greenbook", Vista, California

United States Geological Survey, 1981, Hollywood Quadrangle: 7.5 Minute Series (Topographic), scale 1:24,000.

18011 SKY PARK CIRCLE
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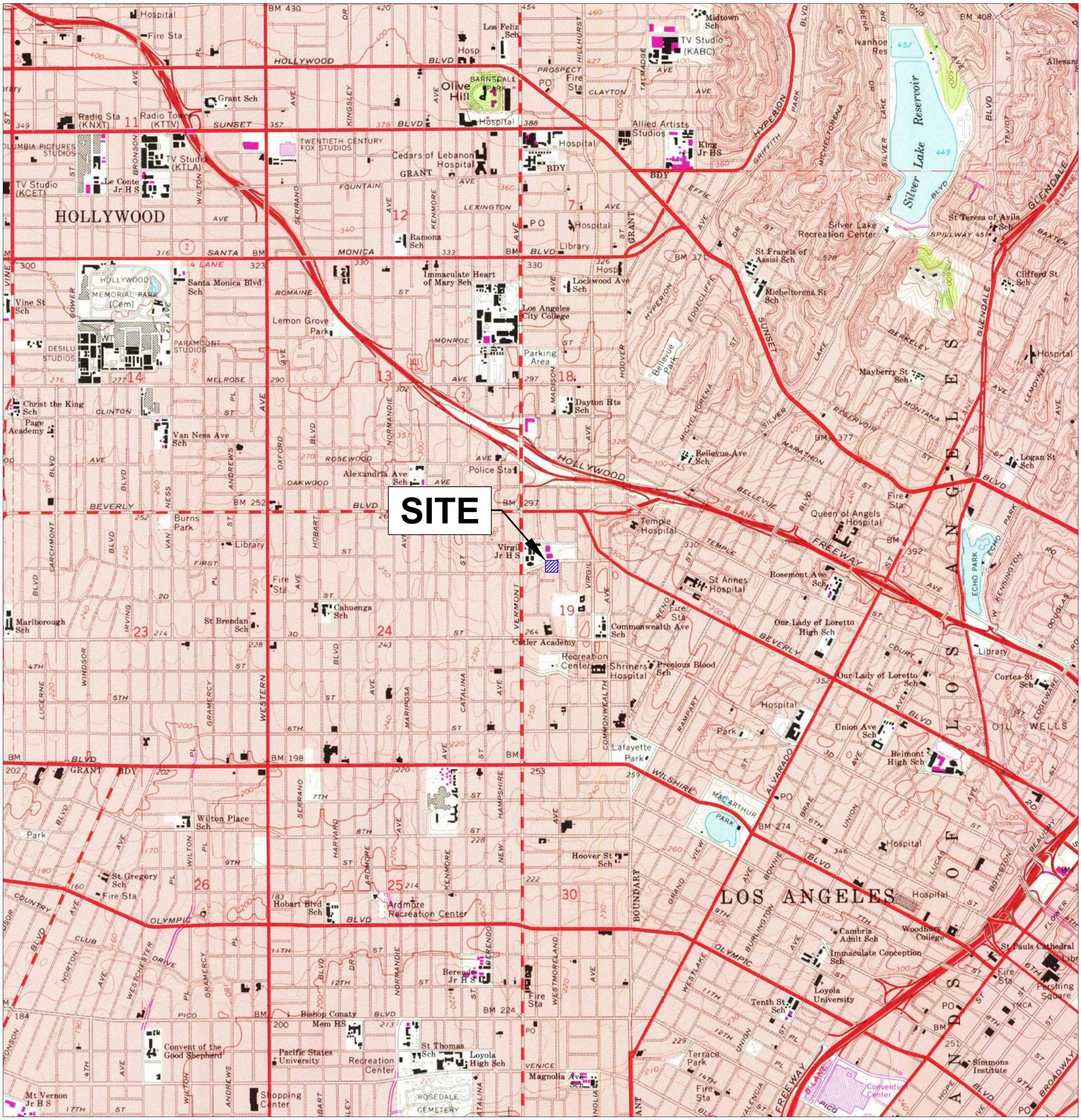
FIGURES



N

AREA ENLARGED IN FIGURE 2

0 3000' 6000'

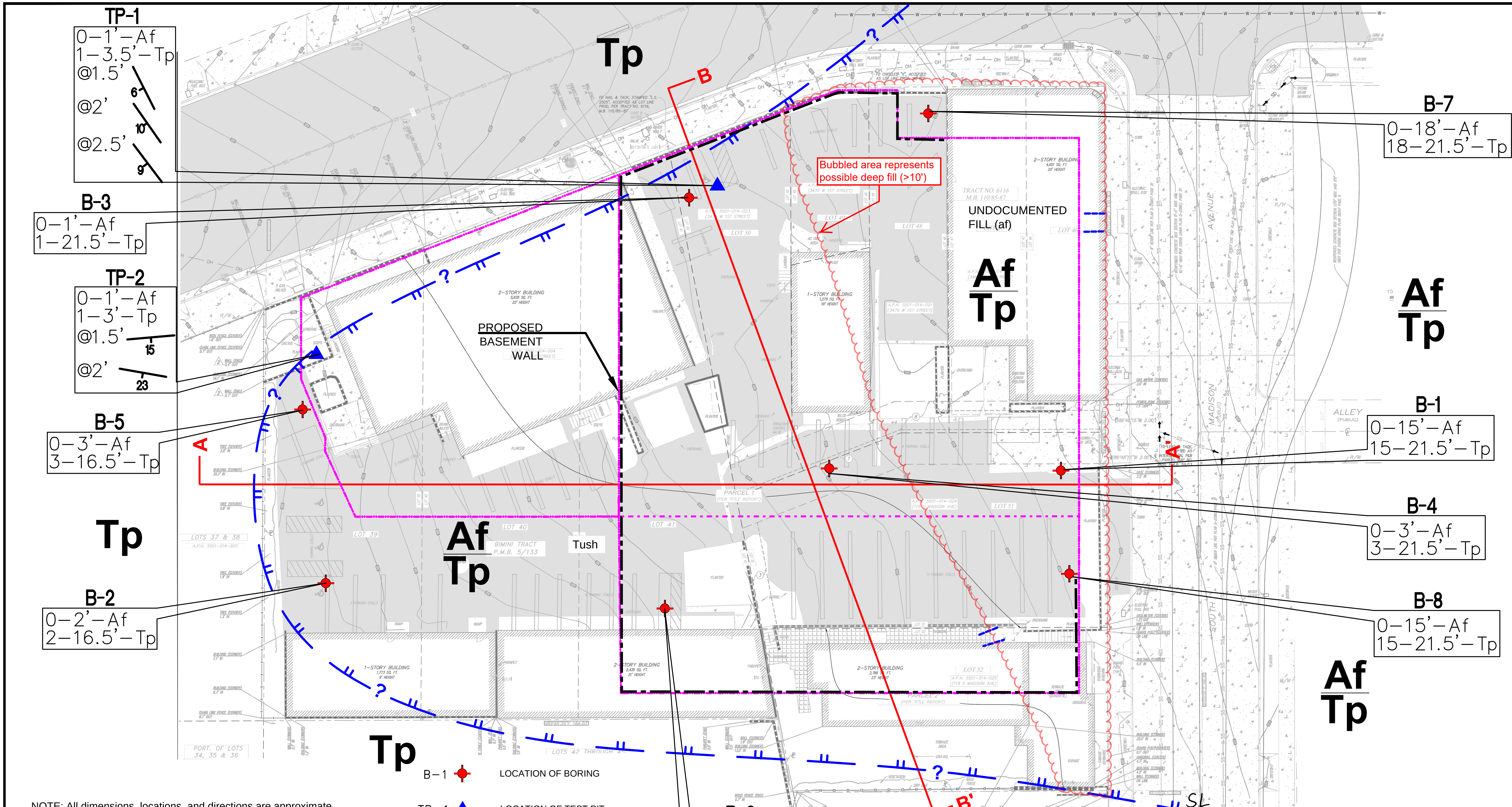
REFERENCE: United States Geological Survey (1981)

TWINING

SITE LOCATION MAP

Rise Kohyang Charter School
3500 W. 1st Street
Los Angeles, California

PROJECT NO.	REPORT DATE	FIGURE 1
180719.1	September 2018	



NOTE: All dimensions, locations, and directions are approximate.

B–1 LOCATION OF BORING

TP–1 LOCATION OF TEST PIT

A A' GEOLOGIC CROSS SECTION

APPROXIMATE LIMITS OF ARTIFICIAL FILL

PROPOSED BUILDING OUTLINE

STRIKE AND DIP OF BEDDING

Af Artificial Fill

Qa Alluvium

Tp Puente Formation

SCALE IN FEET

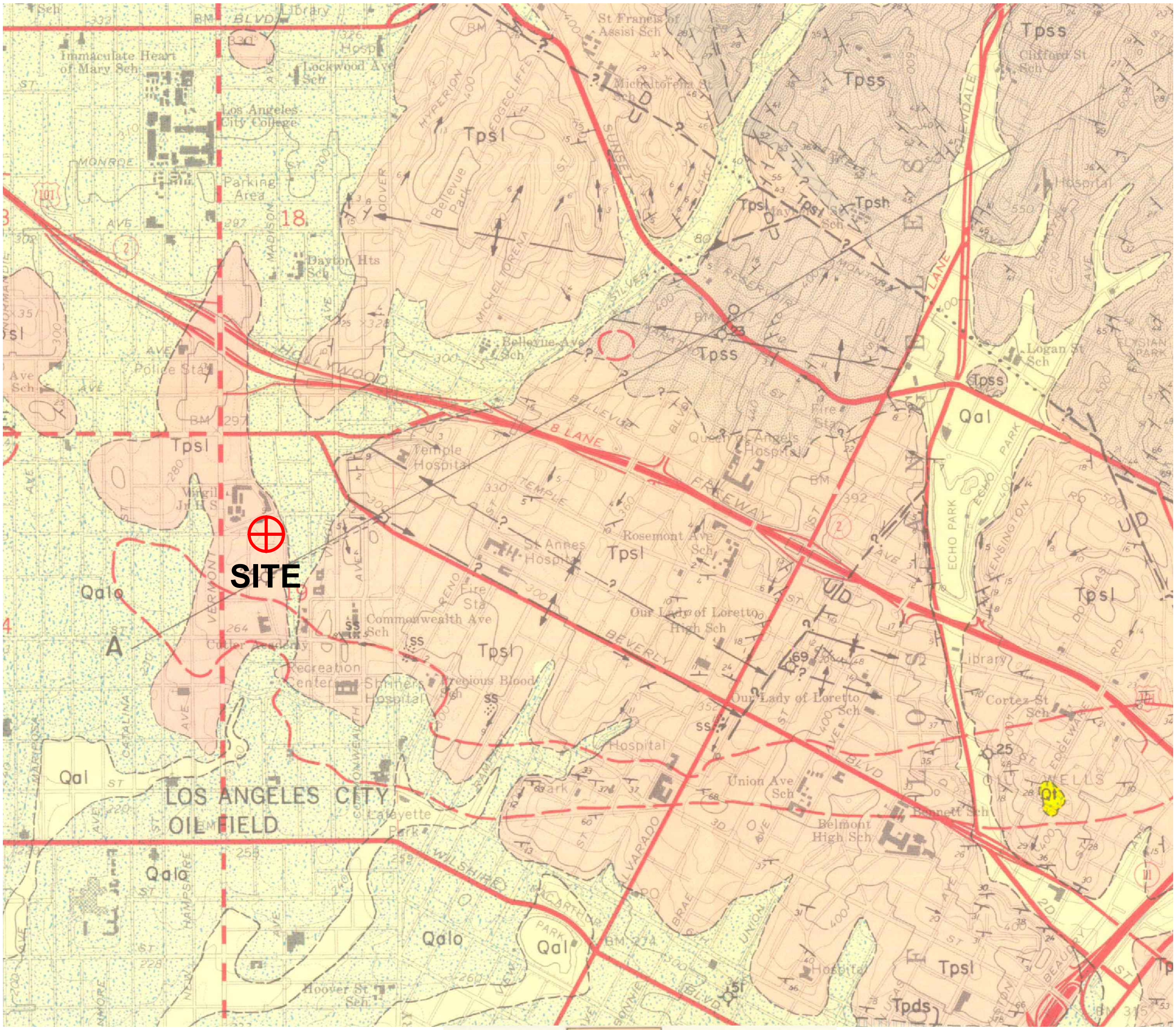
0 30 60

TWINING

SITE GEOLOGIC AND BORING LOCATION MAP

Rise Kohyang Charter School
3500 W 1st Street
Los Angeles, California

PROJECT NO.	REPORT DATE	FIGURE 2
180719.1	September 2018	



Qal

Alluvium
Silt, sand and gravel.

Qalo

Old alluvium
Silt, sand and gravel forming dissected alluvial plain and alluvial terrace deposits.

Tpsl
Tpsh
Tpds
Tpss
Tpun

Puente Formation

Tpsl: siltstone, well bedded, light brown and light gray; Tpsh: shale, well bedded, light gray, siliceous; Tpds: diatomaceous shale, punky, dull white; Tpss: sandstone, well bedded, medium- to coarse-grained, light brown to gray; Tpun: undifferentiated siltstone, shale, sandstone, and conglomerate.

N

SCALE IN FEET
0 2000 4000

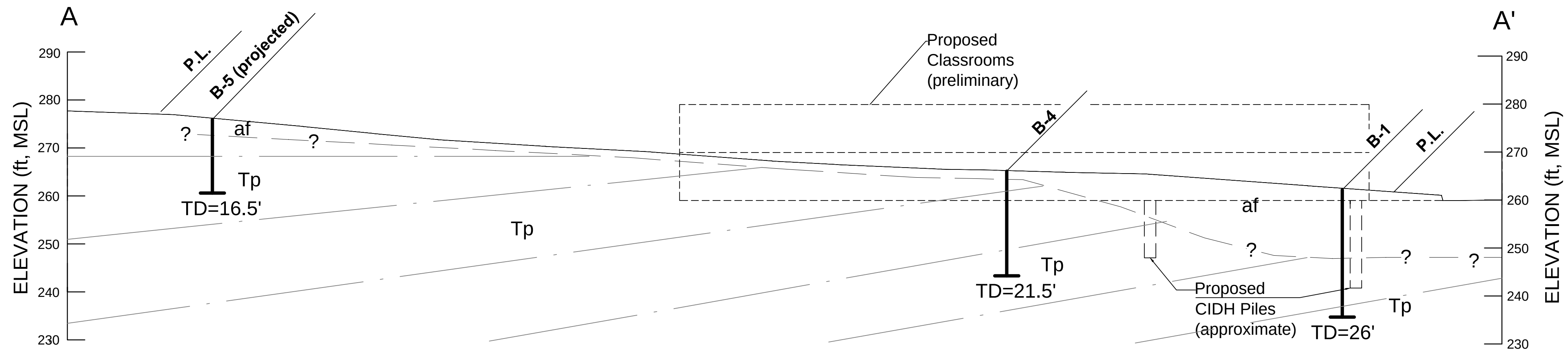
REFERENCE: LAMAR (1970)

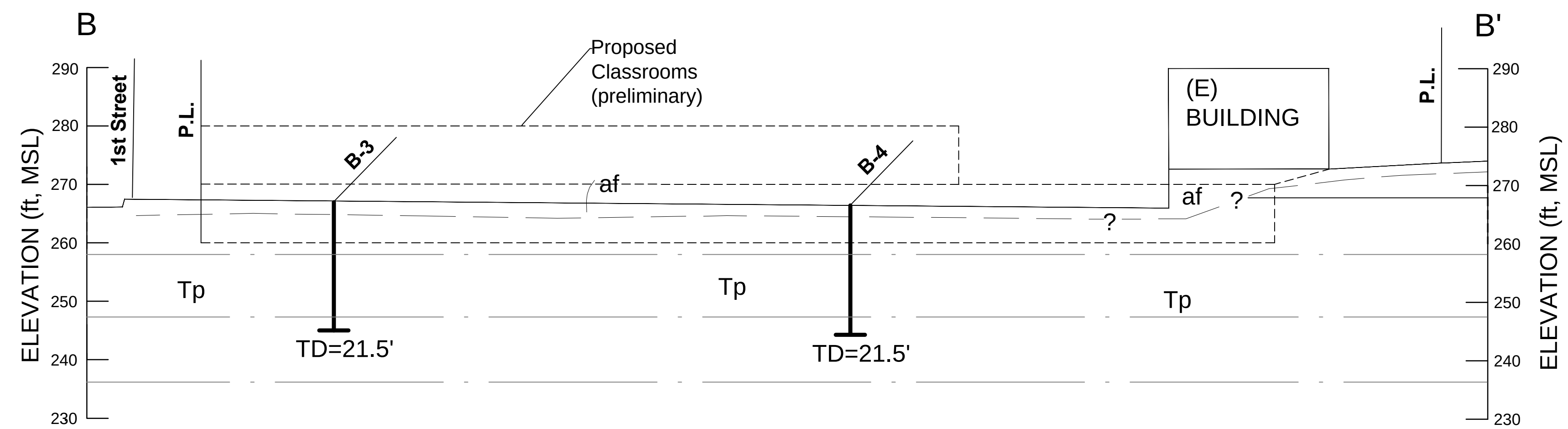
TWINING

REGIONAL GEOLOGIC MAP

Rise Kohyang Charter School
3500 W. 1st Street
Los Angeles, California

PROJECT NO.	REPORT DATE	FIGURE 3
180719.1	September 2018	





SCALE IN FEET
0 30 60

NOTE: All dimensions, locations, and directions are approximate.

LEGEND

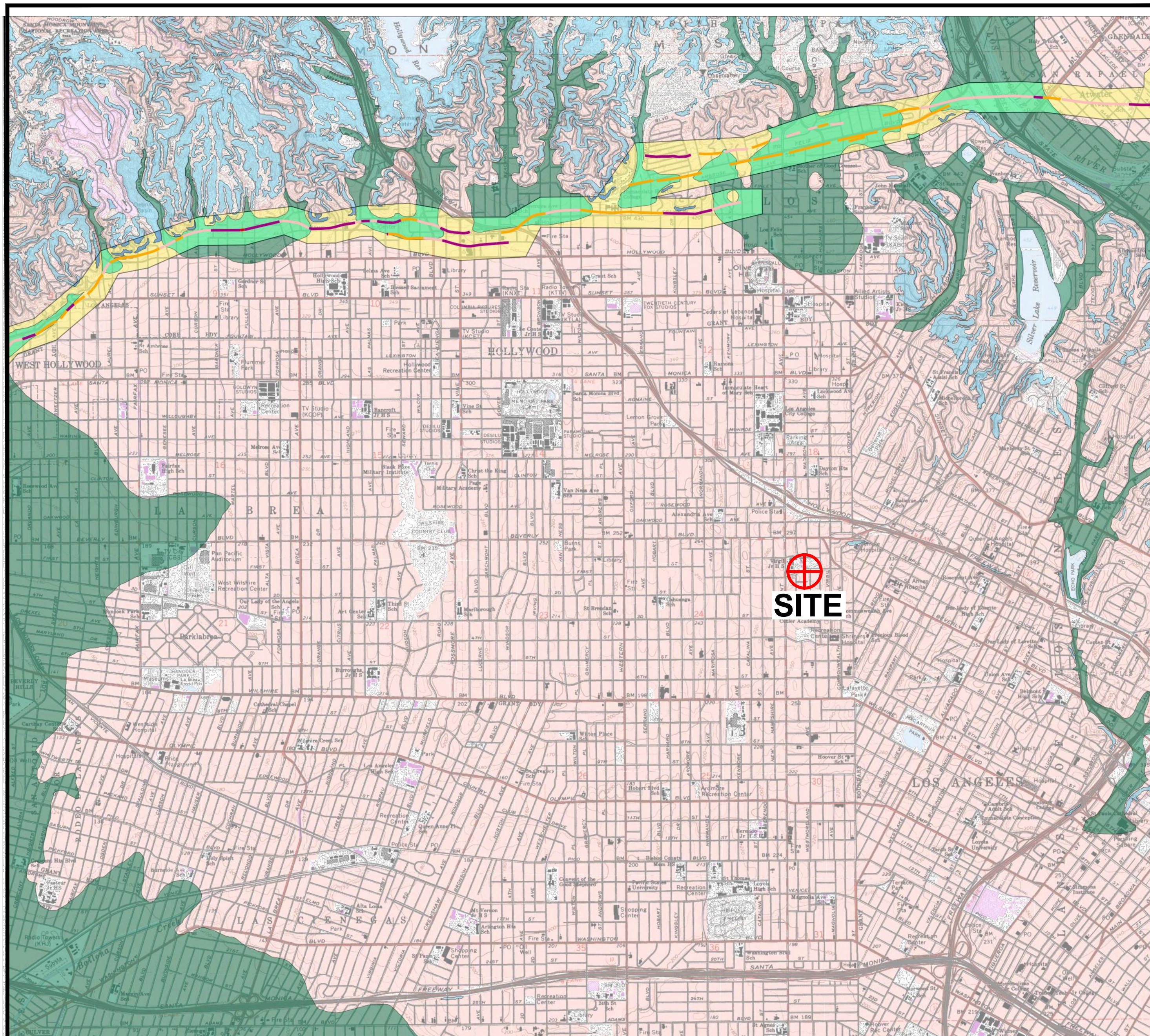
	EXISTING GRADE
B-3 TD= 21.5'	APPROXIMATE LOCATION OF EXPLORATORY BORING OR CPT TD = TERMINATION DEPTH IN FEET
	GEOLOGIC CONTACT (DASHED WHERE APPROXIMATELY LOCATED)
	APPROXIMATE GROUNDWATER TABLE
Af	ARTIFICIAL FILL
Tp	PUENTE FORMATION

TWINING

GEOLOGIC CROSS SECTION A-A' AND B-B'

Rise Kohyang Charter School
3500 W 1st Street
Los Angeles, California

PROJECT NO.	REPORT DATE	FIGURE 4
180719.1	September 2018	



MAP EXPLANATION

ALQUIST-PRIOLO 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.

Active Fault Traces
1906 C
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.

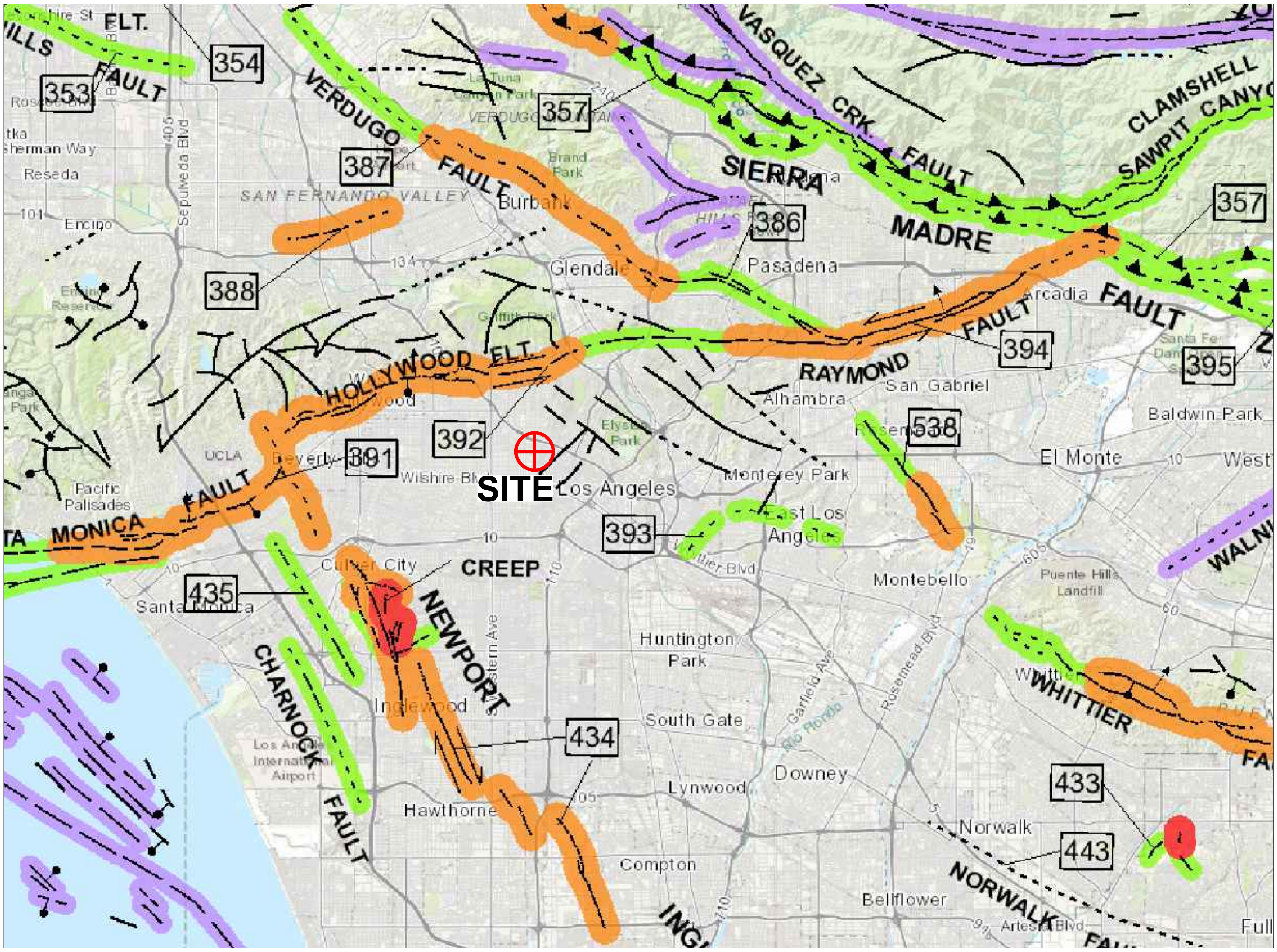
Earthquake-Induced Landslide Zones
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would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

N

0 5000 10000

TWINING

LIQUEFACTION POTENTIAL MAP		
Rise Kohyang Charter School 3500 W. 1st Street Los Angeles, California		
PROJECT NO. 180719.1	REPORT DATE September 2018	FIGURE 5



FAULT ALONG WHICH HISTORIC DISPLACEMENT HAS OCCURRED

HOLOCENE FAULT DISPLACEMENT

LATE QUATERNARY FAULT DISPLACEMENT

QUATERNARY FAULT DISPLACEMENT

PRE-QUATERNARY FAULT DISPLACEMENT

N

SCALE IN MILES

0 5 10

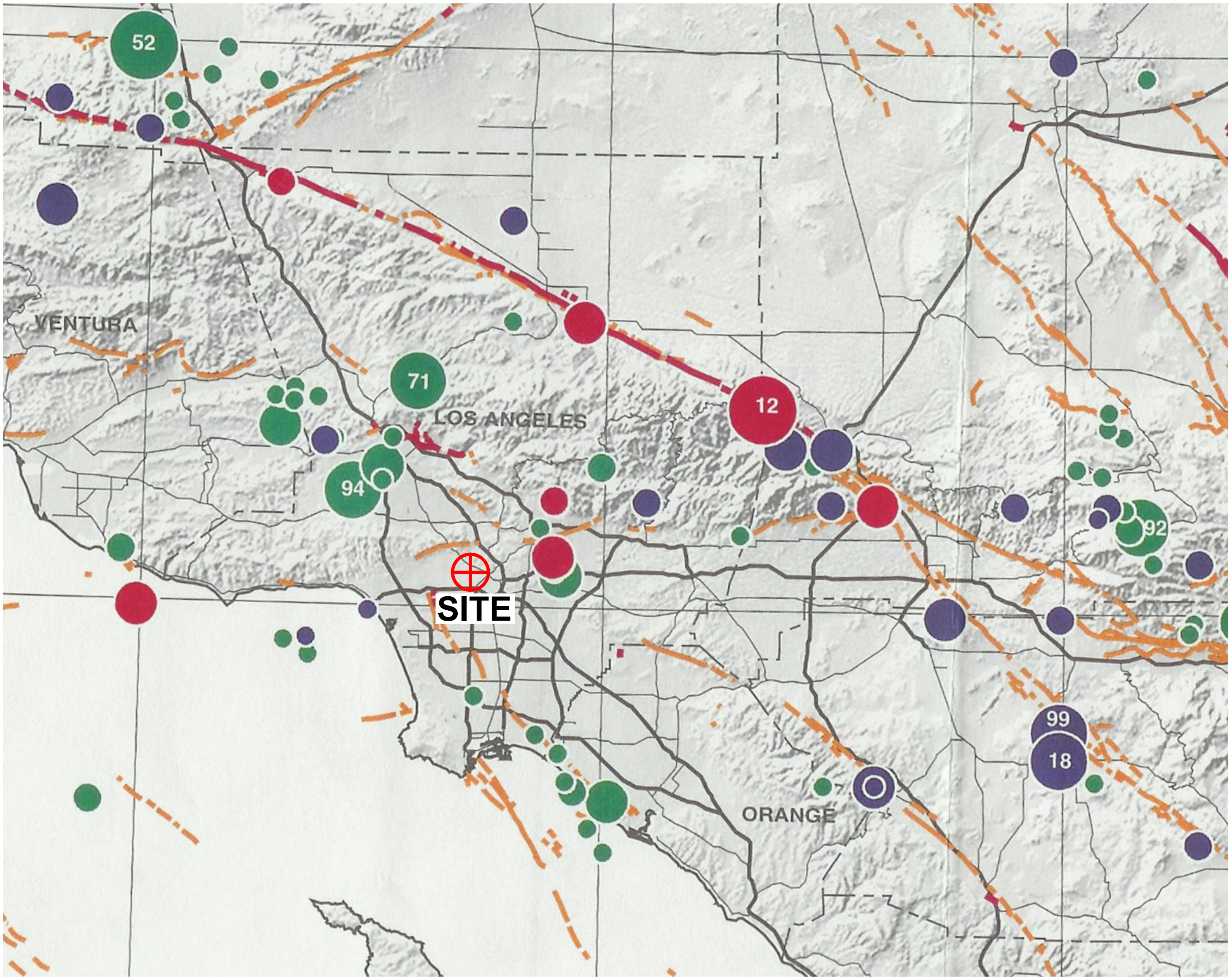
REFERENCE: JENNINGS AND BRYANT (2010)

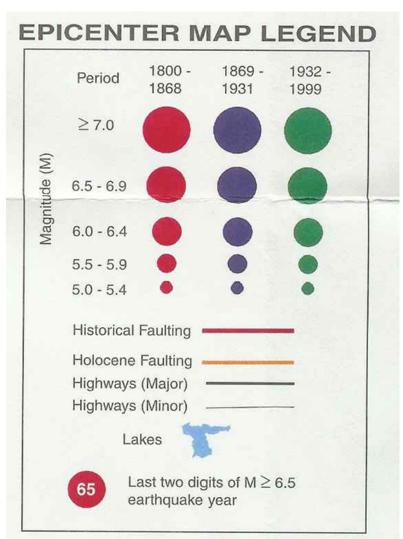
TWINING

REGIONAL FAULT LOCATION MAP

Rise Kohyang Charter School
3500 W. 1st Street
Los Angeles, California

PROJECT NO.	REPORT DATE	FIGURE 6
180719.1	September 2018	





N

SCALE IN MILES

0 20 40

REFERENCE: TOPPOZADA ET AL. (2000)

TWINING

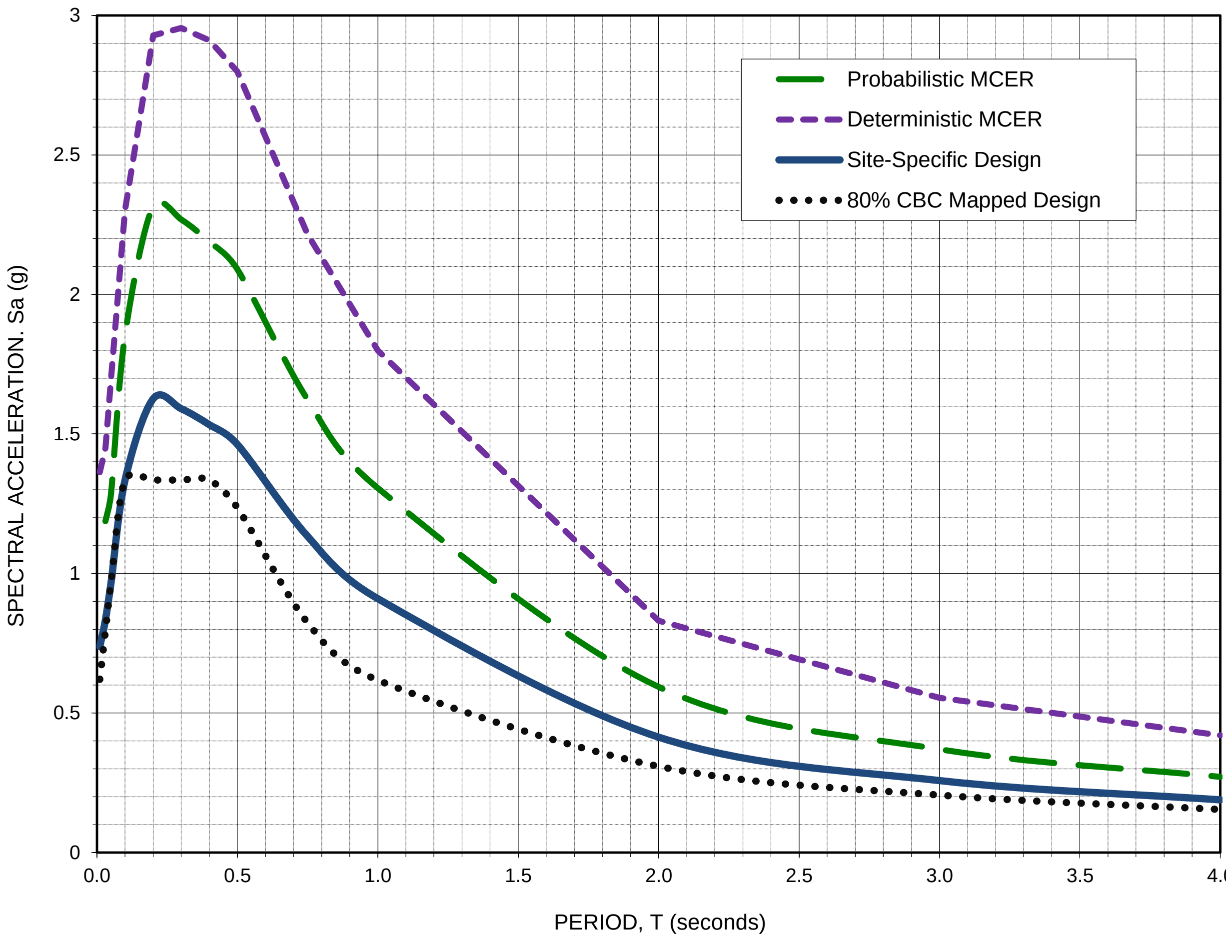
HISTORICAL SEISMICITY, 1800-1999

Rise Kohyang Charter School
3500 W. 1st Street
Los Angeles, California

PROJECT NO.	REPORT DATE	FIGURE 7
180719.1	September 2018	

PERIOD (seconds)	SITE-SPECIFIC DESIGN RESPONSE SPECTRUM Sa, (g)
0.010	0.738
0.030	0.833
0.050	0.967
0.100	1.335
0.200	1.626
0.300	1.591
0.400	1.533

PERIOD (seconds)	SITE-SPECIFIC DESIGN RESPONSE SPECTRUM Sa, (g)
0.500	1.463
0.750	1.133
1.000	0.910
2.000	0.414
3.000	0.258
4.000	0.189

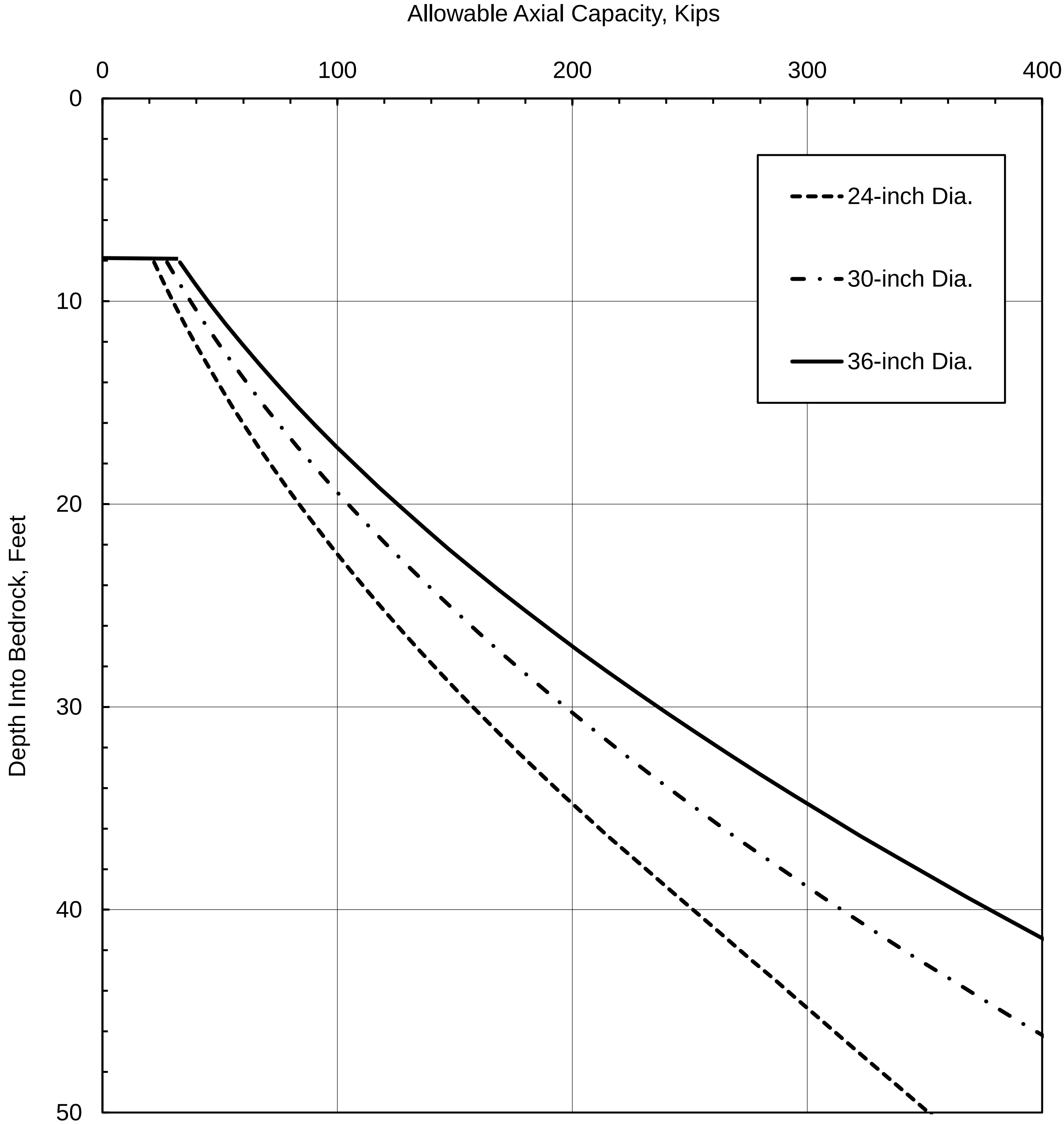


TWINING

SITE-SPECIFIC DESIGN RESPONSE SPECTRUM

Rise Kohyang Charter School
3500 W. 1st Street
Los Angeles, CA

PROJECT NO.	DATE	FIGURE 8
180719.1	September 2018	



TWINING

AXIAL PILE CAPACITY		
Rise Kohyang Charter School 3500 W. 1st Street Los Angeles, CA		
PROJECT NO. 180719.1	DATE September 2018	FIGURE 9

Appendix A
Field Exploration

TWINING CONSULTING

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Appendix A
Field Exploration

General

The subsurface exploration program consisting of drilling and logging eight exploratory borings on August 30 and 31, 2018. The borings were conducted using an 8-inch diameter hollow-stem auger drill and were conducted by 2R Drilling of Chino Hills, California. The soil borings were advanced to depths of approximately 16½ to 21½ feet below existing grade. Additionally, two test pits were excavated by the project geologist to depths of 3 to 3½ feet below existing grade. The test pits were excavated on September 6, 2018.

The approximate locations of the borings and test pits are shown in Figure 2, Site Plan and Boring Location Map.

Drilling and Sampling

The boring logs performed by Twining are presented as Figures A-2 through A-9. The log of the test pits is presented as Figure A-10. An explanation of these logs is presented as Figure A-1. The boring logs describe the earth materials encountered, samples obtained, and show the field and laboratory tests performed. The log also shows the boring number and drilling date. The borings were logged by an engineer using the Unified Soil Classification System. The boundaries between soil types shown on the logs area approximate because the transition between different soil layers may be gradual. Bulk samples of representative earth materials were obtained from the borings. After completion of the drilling, both boreholes were backfilled with cuttings. The boreholes and test pits were backfilled with cuttings and the surface patched with cold patch asphalt concrete.

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

Bright Star Charter
3500 W 1st Street
Los Angeles, California

PROJECT NO.	REPORT DATE	
180719.1	September 2018	FIGURE A-1

STANDARD LOG EXPLANATION 180719.1 - 1ST STREET CHARTER SCHOOL.GPJ TWINING LABS.GDT 9/13/18

DATE DRILLED	8/30/18	LOGGED BY	DHC	**BORING NO.**	**B-1**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	N/E
DRILLING METHOD	8" HSA	DRILLER	2R	SURFACE ELEVATION (ft.)	262 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
									6" AC pavement with no base
								SC	**FILL:** Clayey SAND; light brown; moist
257	5			11				SC	-- same; medium dense
252	10			10				GP	Poorly graded GRAVEL; grey; dry; open graded gravel; sample collected in SPT bag
247	15			33					**BEDROCK:** Puente Formation (Tp); siltstone; moderately hard; greenish grey; moist; slight hydrocarbon odor
242	20			37/50 for 5"	25.2	96.4			-- same; very hard; greenishy to reddish grey
									Total Depth = 21.5 feet Backfilled on 8/30/2018 No groundwater encountered. Borehole filled with cuttings at completion. Surface patched with asphalt patch.
237	25								
232	30								
227	35								

TWINING

LOG OF BORING

Rise Kohyang Charter School
3500 W 1st Street
Los Angeles, California

PROJECT NO.	REPORT DATE	
180719.1	September 2018	FIGURE A - 2

DATE DRILLED	8/31/18	LOGGED BY	DHC	**BORING NO.**	**B-2**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	N/E
DRILLING METHOD	8" HSA	DRILLER	2R	SURFACE ELEVATION (ft.)	276 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
									4" AC pavement over 3" base
								SC	**FILL:** Sandy CLAY; yellowish brown; moist
									BEDROCK: Puente Formation (Tp); siltstone; grey to orange; slightly moist
271	5		X	82	20.0	96.1			-- same; very hard; sampler hit large rock
266	10			40					-- same; very hard
261	15		X	77	22.4	101.9			-- same
									Total Depth = 16.5 feet Backfilled on 8/31/2018 No groundwater encountered. Borehole filled with cuttings at completion. Surface patched with asphalt patch.
256	20								
251	25								
246	30								
241	35								

TWINING

LOG OF BORING

Rise Kohyang Charter School
3500 W 1st Street
Los Angeles, California

PROJECT NO.	REPORT DATE	FIGURE A - 3
180719.1	September 2018	

DATE DRILLED	8/30/18	LOGGED BY	DHC	BORING NO.	B-3
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	N/E
DRILLING METHOD	8" HSA	DRILLER	2R	SURFACE ELEVATION (ft.)	267 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
									4" AC pavement with no base
								SC	**FILL:** Sandy CLAY; yellowish brown; moist
									BEDROCK: Puente Formation (Tp); siltstone; yellow to brown to grey to orange; moist
262	5			67	21.9	99.4			-- same; very hard
257	10			33					-- same; hard
252	15			20/50 for 6"	29.9	87.5			-- same; very hard
247	20			40					-- same; very hard
									Total Depth = 21.5 feet Backfilled on 8/30/2018 No groundwater encountered. Borehole filled with cuttings at completion. Surface patched with asphalt patch.
242	25								
237	30								
232	35								

TWINING

LOG OF BORING

Rise Kohyang Charter School
3500 W 1st Street
Los Angeles, California

PROJECT NO.	REPORT DATE	
180719.1	September 2018	FIGURE A - 4

DATE DRILLED	8/31/18	LOGGED BY	DHC	**BORING NO.**	**B-4**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	N/E
DRILLING METHOD	8" HSA	DRILLER	2R	SURFACE ELEVATION (ft.)	266 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
								8" AC pavement with no base
							CL	**FILL:** Sandy Lean CLAY; olive to yellowish brown; moist
								BEDROCK: Puente Formation (Tp); claystone; moist
261	5	Driven	35	30.6	87.4			-- same; firm
256	10	Driven	17					-- same; greyish brown
251	15	Driven	47	26.7	92.8			Siltstone; hard; grey to orange; moist
246	20	Driven	45					Claystone; moderately hard; dark grey; very moist
								Total Depth = 21.5 feet Backfilled on 8/31/2018 No groundwater encountered. Borehole filled with cuttings at completion. Surface patched with asphalt patch.
241	25							
236	30							
231	35							

BORING LOG 180719.1 - 1ST STREET CHARTER SCHOOL.GPJ TWINING LABS.GDT 10/1/18

TWINING

LOG OF BORING

Rise Kohyang Charter School
3500 W 1st Street
Los Angeles, California

PROJECT NO.	REPORT DATE	
180719.1	September 2018	FIGURE A - 5

DATE DRILLED	8/31/18	LOGGED BY	DHC	BORING NO.	B-5
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	N/E
DRILLING METHOD	8" HSA	DRILLER	2R	SURFACE ELEVATION (ft.)	276 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
									4" AC pavement over 4" base
								SC	**FILL:** Clayey SAND; yellowish brown; moist
									BEDROCK: Puente Formation (Tp); siltstone; slightly moist
271	5			25					-- same; firm
266	10			69	16.1	105.0			-- same; very hard; some oxidation staining
261	15			44					-- same; hard
									Total Depth = 16.5 feet Backfilled on 8/31/2018 No groundwater encountered. Borehole filled with cuttings at completion. Surface patched with asphalt patch.
256	20								
251	25								
246	30								
241	35								

TWINING

LOG OF BORING

Rise Kohyang Charter School
3500 W 1st Street
Los Angeles, California

PROJECT NO.	REPORT DATE	
180719.1	September 2018	FIGURE A - 6

DATE DRILLED	8/31/18	LOGGED BY	DHC	**BORING NO.**	**B-6**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	N/E
DRILLING METHOD	8" HSA	DRILLER	2R	SURFACE ELEVATION (ft.)	276 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
									3" AC pavement with 3" base
								SC	**FILL:** Clayey SAND; yellowish brown; moist
									BEDROCK: Puente Formation (Tp); siltstone; grey to orange; slightly moist
271	5			34					-- same; firm
266	10			34/50 for 6"	22.0	97.2			-- same; light brown; very hard
261	15			24					-- same; light grey; hard
									Total Depth = 16.5 feet Backfilled on 8/31/2018 No groundwater encountered. Borehole filled with cuttings at completion. Surface patched with asphalt patch.
256	20								
251	25								
246	30								
241	35								

LOG OF BORING

Rise Kohyang Charter School
3500 W 1st Street
Los Angeles, California

TWINING

PROJECT NO.	REPORT DATE	
180719.1	September 2018	FIGURE A - 7

DATE DRILLED	8/30/18	LOGGED BY	DHC	**BORING NO.**	**B-7**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	N/E
DRILLING METHOD	8" HSA	DRILLER	2R	SURFACE ELEVATION (ft.)	276 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
									4" AC pavement over 8" base
								SC	**FILL:** Clayey SAND; yellow to brown to white; moist; with trace gravel
271	5			8				SC	-- same; loose
266	10			14	23.1	88.3		SC	-- same; medium dense
261	15			9				SC	-- same; loose
									BEDROCK: Puente Formation (Tp); siltstone; grey to olive to yellow; moist
256	20			33	29.6	91.2			-- firm
									Total Depth = 21.5 feet Backfilled on 8/30/2018 No groundwater encountered. Borehole filled with cuttings at completion. Surface patched with asphalt patch.
251	25								
246	30								
241	35								

TWINING

LOG OF BORING

Rise Kohyang Charter School
3500 W 1st Street
Los Angeles, California

PROJECT NO.	REPORT DATE	
180719.1	September 2018	FIGURE A - 8

DATE DRILLED	8/30/18	LOGGED BY	SL	BORING NO.	B-8
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	N/E
DRILLING METHOD	8" HSA	DRILLER	2R	SURFACE ELEVATION (ft.)	276 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
									3" AC pavement with no base
								SC	**FILL:** Clayey SAND; brown; slightly moist
271	5			37	11.9	113.5		SC	-- same; dense
266	10			9				GP	Poorly graded GRAVEL; grey; slightly moist; open graded gravel with some silt; no recovery from sampler
261	15			68	30.3	89.6			**BEDROCK:** Puente Formation (Tp); siltstone; very hard; brown to light brown; slight moist
256	20			42					-- same; dark grey; dry
									Total Depth = 21.5 feet Backfilled on 8/30/2018 No groundwater encountered. Borehole filled with cuttings at completion. Surface patched with asphalt patch.
251	25								
246	30								
241	35								

TWINING

LOG OF BORING

Rise Kohyang Charter School
3500 W 1st Street
Los Angeles, California

PROJECT NO.	REPORT DATE	
180719.1	September 2018	FIGURE A - 9

LOG OF TEST PIT TP-1 & TP-2

Project: 3500 W. 1st Street, Los Angeles, California.
Project Number:
Excavation Method: Hand Labor
Logged By: Mark Kruger
Date Excavated: 9-6-18
Scale: 1"=5'

TWINING

LEGEND
(b) = Strike & dip of bedding
T.D. = Total Depth
■ = Sample Location

LITHOLOGIC DESCRIPTION

Note: The depth of earth materials shown on the Log of Test Pit are approximate and are based on visual classification. This log of subsurface conditions applies only at the specific location and the date shown on the Log of Test Pit. Subsurface conditions may vary at other locations and times.

Artificial Fill (Af) - Gravelly clayey sandy silt, light brown to brown, dry to slightly moist, moderately firm to firm, with few to numerous small to medium sized roots, with subangular rock fragments up to 3 inches in diameter, with occasional man-made debris (brick fragments)

Bedrock - Puente Formation (Tp) - Interbedded tan brown, orange brown & gray siltstone & shale with occasional sandstone, firm to hard, dry to slightly moist, tight, poorly to well bedded, moderately well cemented, slightly to moderately fractured (random, tight), upper 1 to 2 feet is slightly to moderately weathered

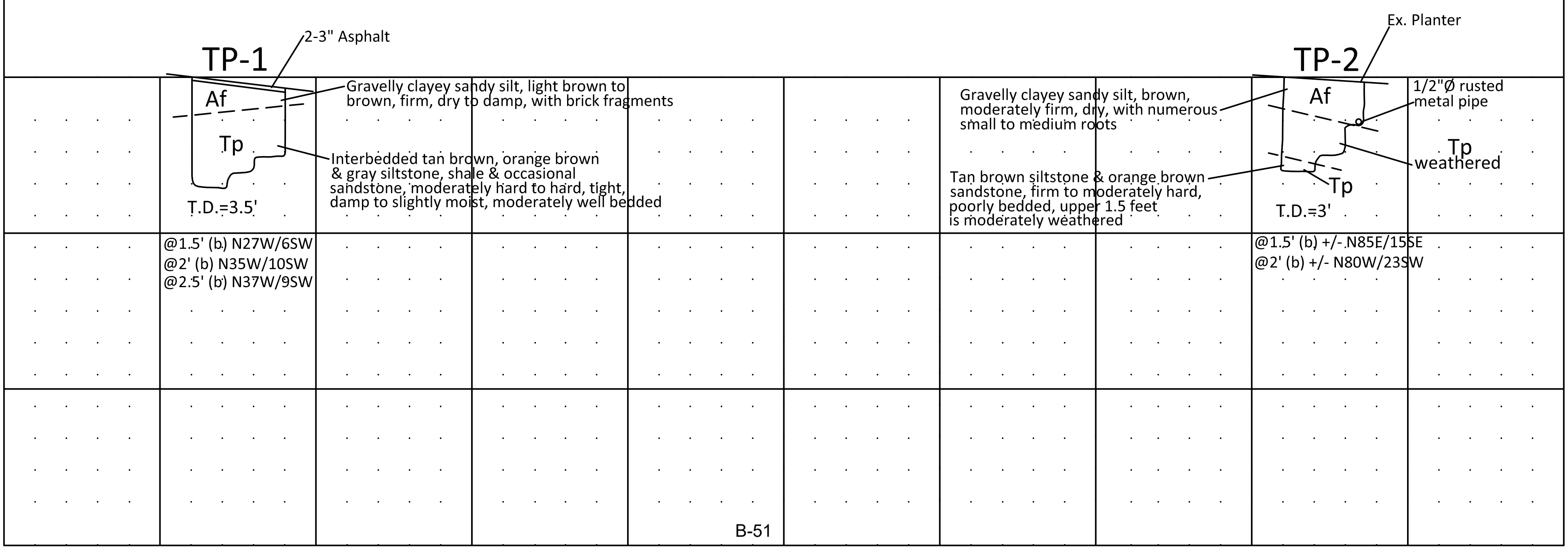

PLATE TP-1

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Appendix B
Laboratory Testing

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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 D 2937. The test results are presented on the logs of the exploratory borings in Appendix A.

Wash Sieve

The amount of fines passing the No. 200 sieve was evaluated by the wash sieve. The test procedure was in general accordance with ASTM D 1140. The results are presented in Table B-2.

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-1.

Direct Shear

Direct shear tests were performed on selected relatively undisturbed soil samples in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of the materials. The samples were inundated during shearing to represent adverse field conditions. The results are presented in Figures B-2 through B-3.

Consolidation Test

Consolidation tests were performed on a selected driven soil sample by in general accordance with the latest version of ASTM D2435. The sample was 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. The results of the test are attached to this appendix as Figures B-4 through B-5.

Expansion Index Test

The expansion index of a representative soil was evaluated in accordance with ASTM D 4829. The specimen was molded under a specified compactive energy at approximately 50 percent saturation. The prepared 1-inch thick by 4-inch diameter specimen was loaded with a surcharge of 144 pounds per square foot and was inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The result of the Expansion Index test is presented on Table B-2.

Corrosivity

Soil pH and resistivity tests were performed by Anaheim Test Lab on a representative soil sample in general accordance with the latest version of California Test Method 643. The chloride content of the selected sample was evaluated in general accordance with the latest version of California Test Method 422. The sulfate content of the selected samples was evaluated in general accordance with the latest version of California Test Method 417. The test results are presented on 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 accordance with ASTM D 2844. The results are summarized in Table B-4.

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Table B-2
No. 200 Wash Sieve Results

Boring No.	Depth (feet)	Percent Passing #200
B-3	0 - 5	76

Table B-2
Expansion Index

Boring No.	Depth (feet)	Expansion Index
B-2	0 – 5	50

Table B-3
Soil Corrosivity Test Results

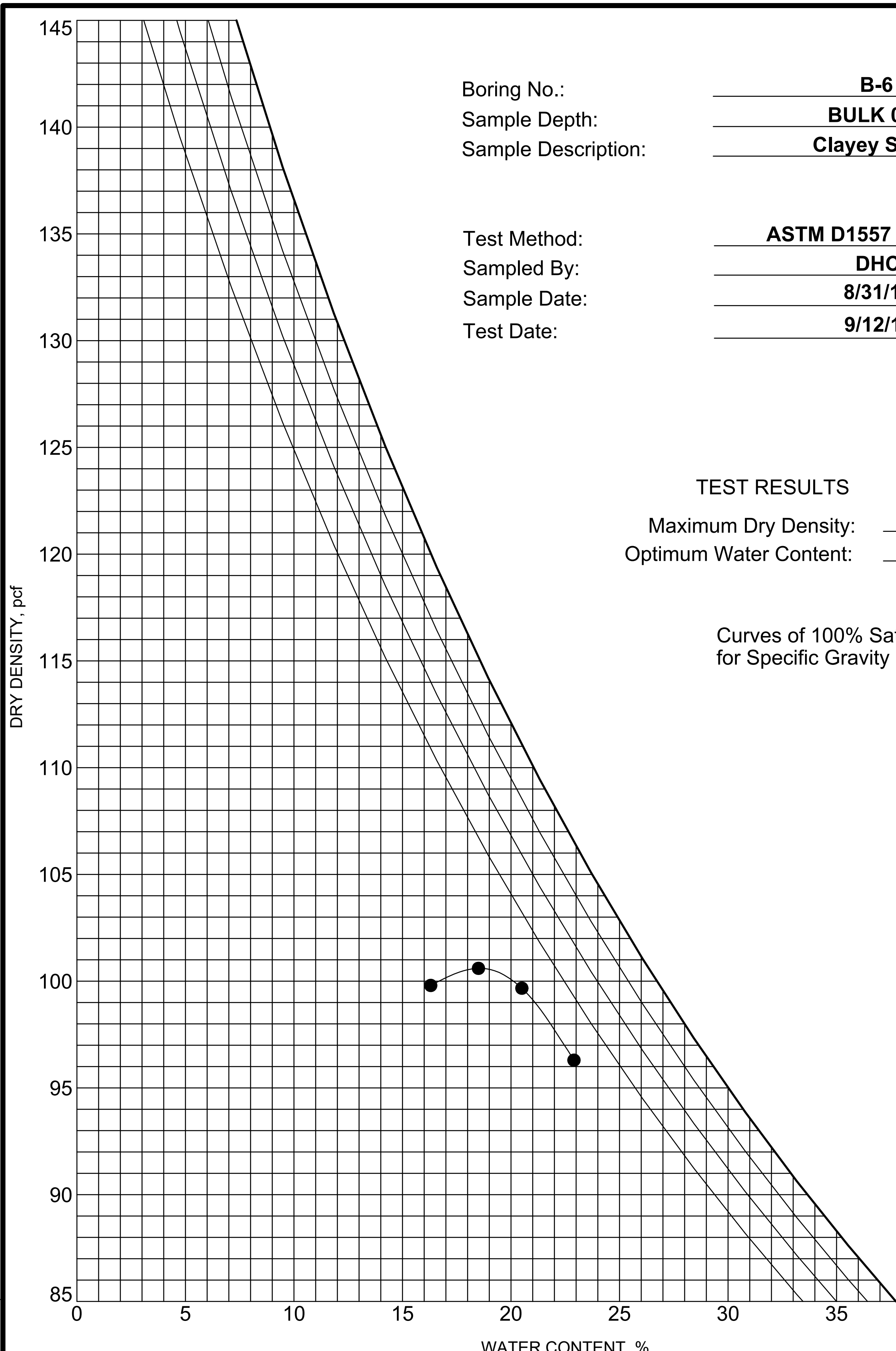
Boring No.	Depth (feet)	pH	Water Soluble Sulfate (ppm)	Water Soluble Chloride (ppm)	Minimum Resistivity (ohm-cm)
B-2	0 – 5	7.2	100	72	1,000

Table B-4
Resistance Value (R-value)

Boring No.	Depth (feet)	R-value
B-2	0 - 5	26

Boring No.:	**B-6**
Sample Depth:	**BULK 0-5'**
Sample Description:	**Clayey SAND**

Test Method:	**ASTM D1557 Method A**
Sampled By:	**DHC**
Sample Date:	**8/31/18**
Test Date:	**9/12/18**



TEST RESULTS

Maximum Dry Density: **100.5 pcf**

Optimum Water Content: **18.5 %**

Curves of 100% Saturation
for Specific Gravity Equal to:

2.80

2.70

2.60

2.50

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MOISTURE-DENSITY RELATIONSHIP

Rise Kohyang Charter School
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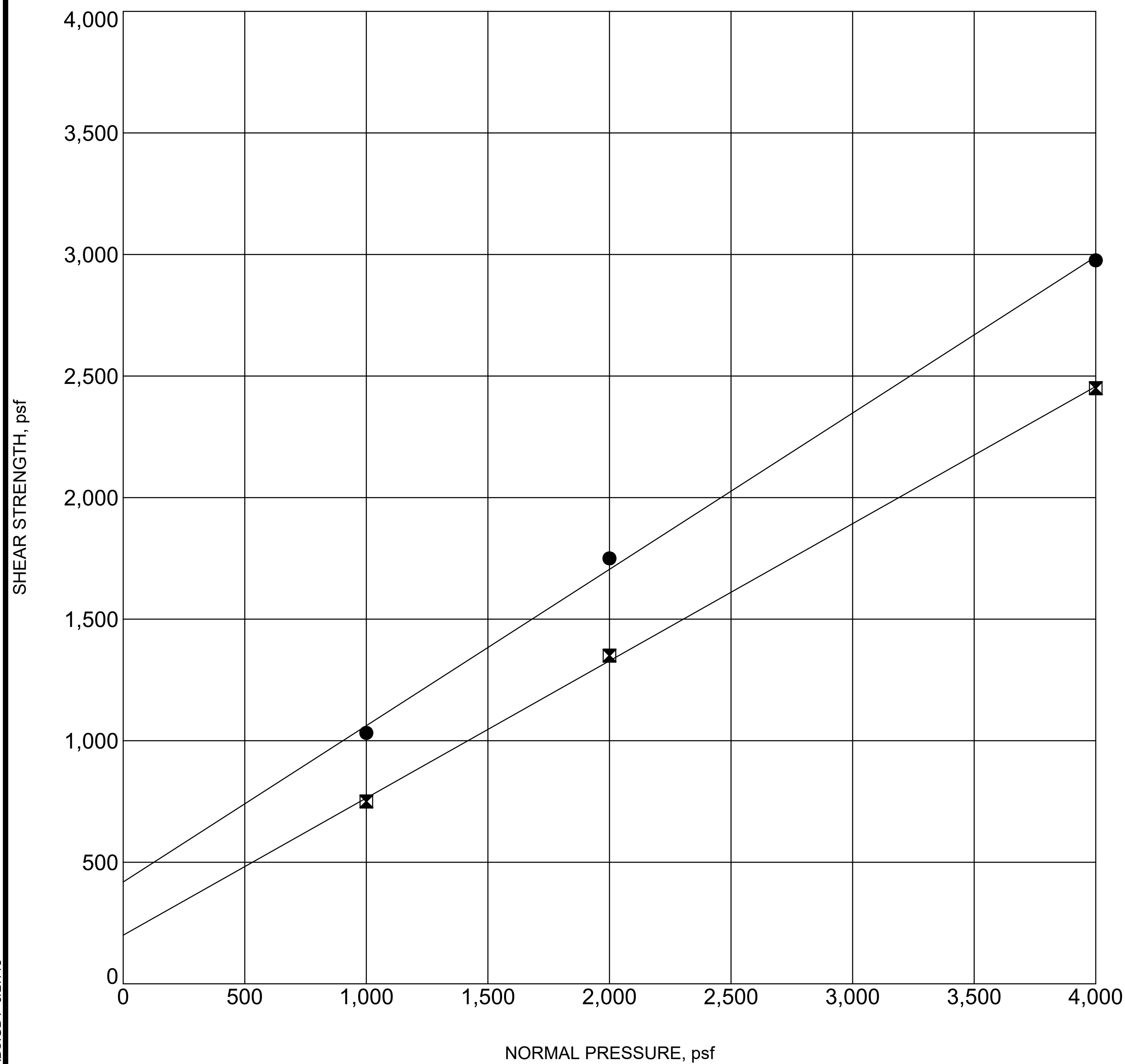
Los Angeles, California

PROJECT NO.	REPORT DATE	FIGURE B-1
180719.1	September 2018	



Boring No.: B-4
Sample Depth (ft): 5
Sample Description: Siltstone
Strain Rate (in./min): 0.005
Dry Density (pcf): 87.4

Shear Strength Parameters

	Peak	Ultimate
Cohesion, C (psf):	419	240
Friction Angle, Ø (deg):	33	32

Initial Moisture (%): 30.6
Final Moisture (%): 32.8

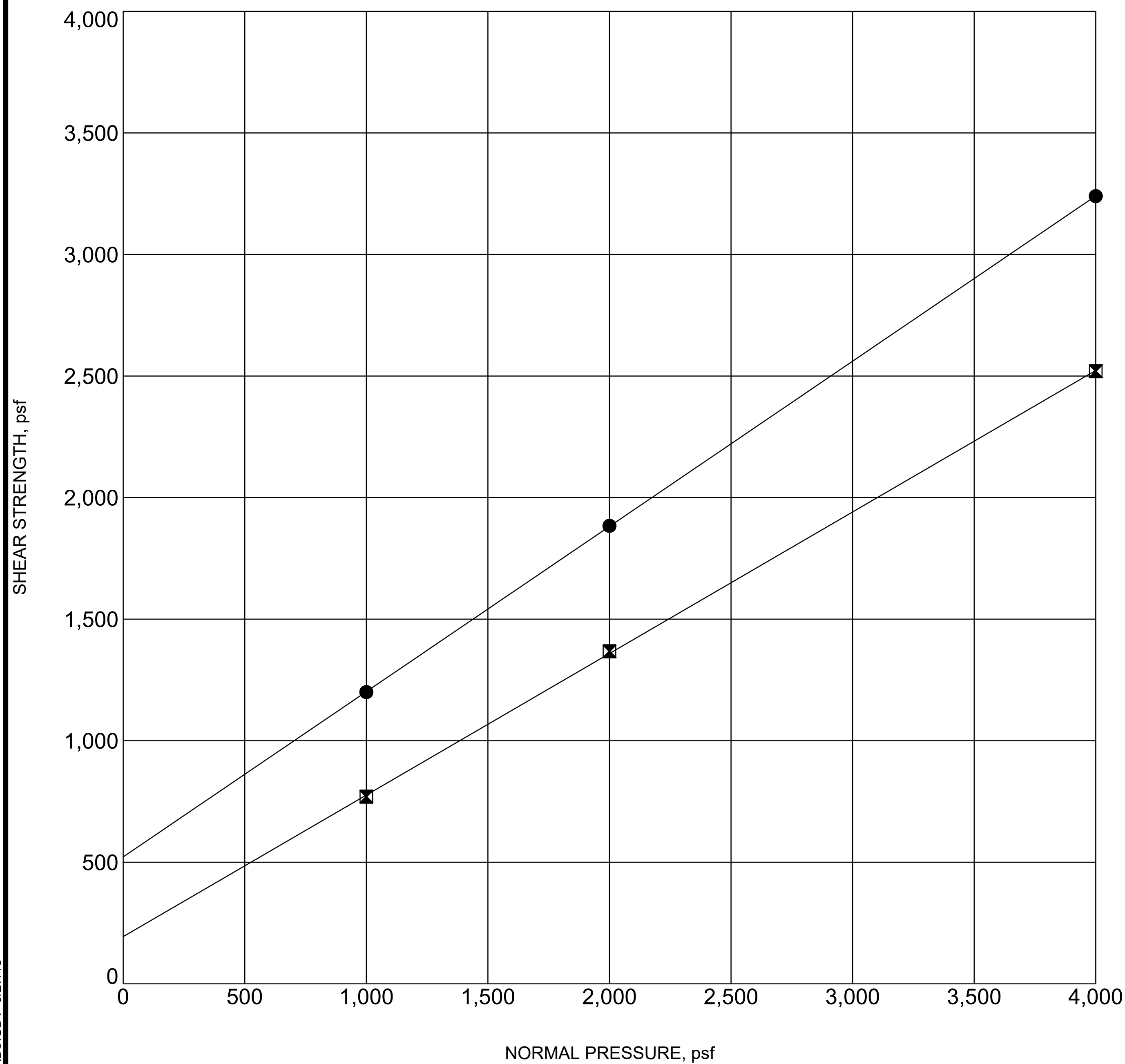
DIRECT SHEAR 180719.1 - 1ST STREET CHARTER SCHOOL.GPJ TWINING LABS.GDT 9/27/18

TWINING

DIRECT SHEAR TEST

Rise Kohyang Charter School
3500 W 1st Street
Los Angeles, California

PROJECT NO.	REPORT DATE	FIGURE B-2
180719.1	September 2018	



Boring No.: B-5
Sample Depth (ft): 10
Sample Description: Siltstone
Strain Rate (in./min): 0.005
Dry Density (pcf): 105.0

Shear Strength Parameters

	Peak	Ultimate
Cohesion, C (psf):	522	250
Friction Angle, Ø (deg):	34	34

Initial Moisture (%): 16.1
Final Moisture (%): 22.6

DIRECT SHEAR 180719.1 - 1ST STREET CHARTER SCHOOL.GPJ TWINING LABS.GDT 9/27/18

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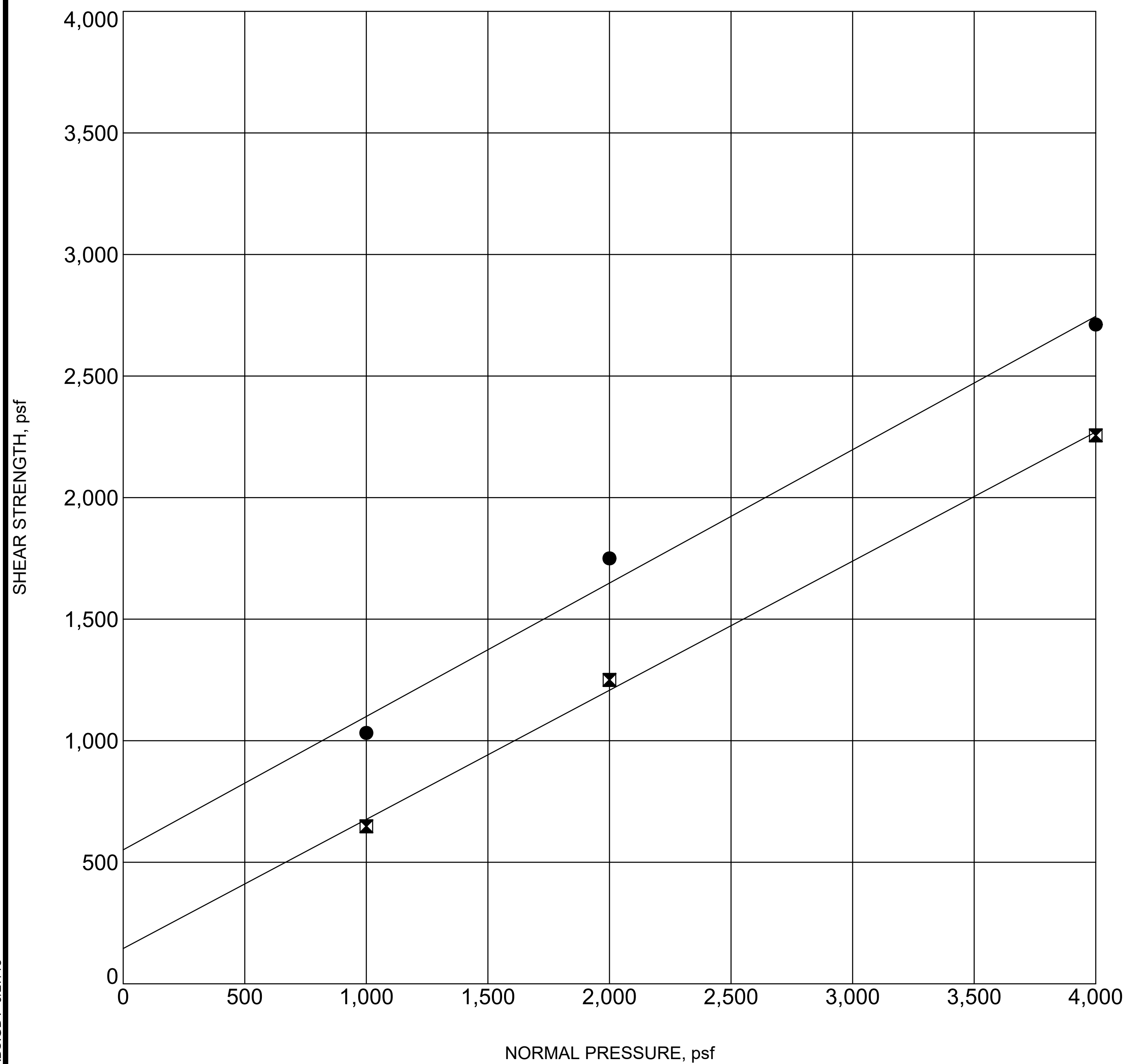
DIRECT SHEAR TEST

Rise Kohyang Charter School
3500 W 1st Street
Los Angeles, California

PROJECT NO.	REPORT DATE	FIGURE B-3
180719.1	September 2018	



Shear Strength Parameters

	Peak	Ultimate
Cohesion, C (psf):	551	200
Friction Angle, Ø (deg):	29	29

Initial Moisture (%): 29.6
Final Moisture (%): 29.8

Boring No.: B-7
Sample Depth (ft): 20
Sample Description: Siltstone
Strain Rate (in./min): 0.005
Dry Density (pcf): 91.2

DIRECT SHEAR 180719.1 - 1ST STREET CHARTER SCHOOL.GPJ TWINING LABS.GDT 9/27/18

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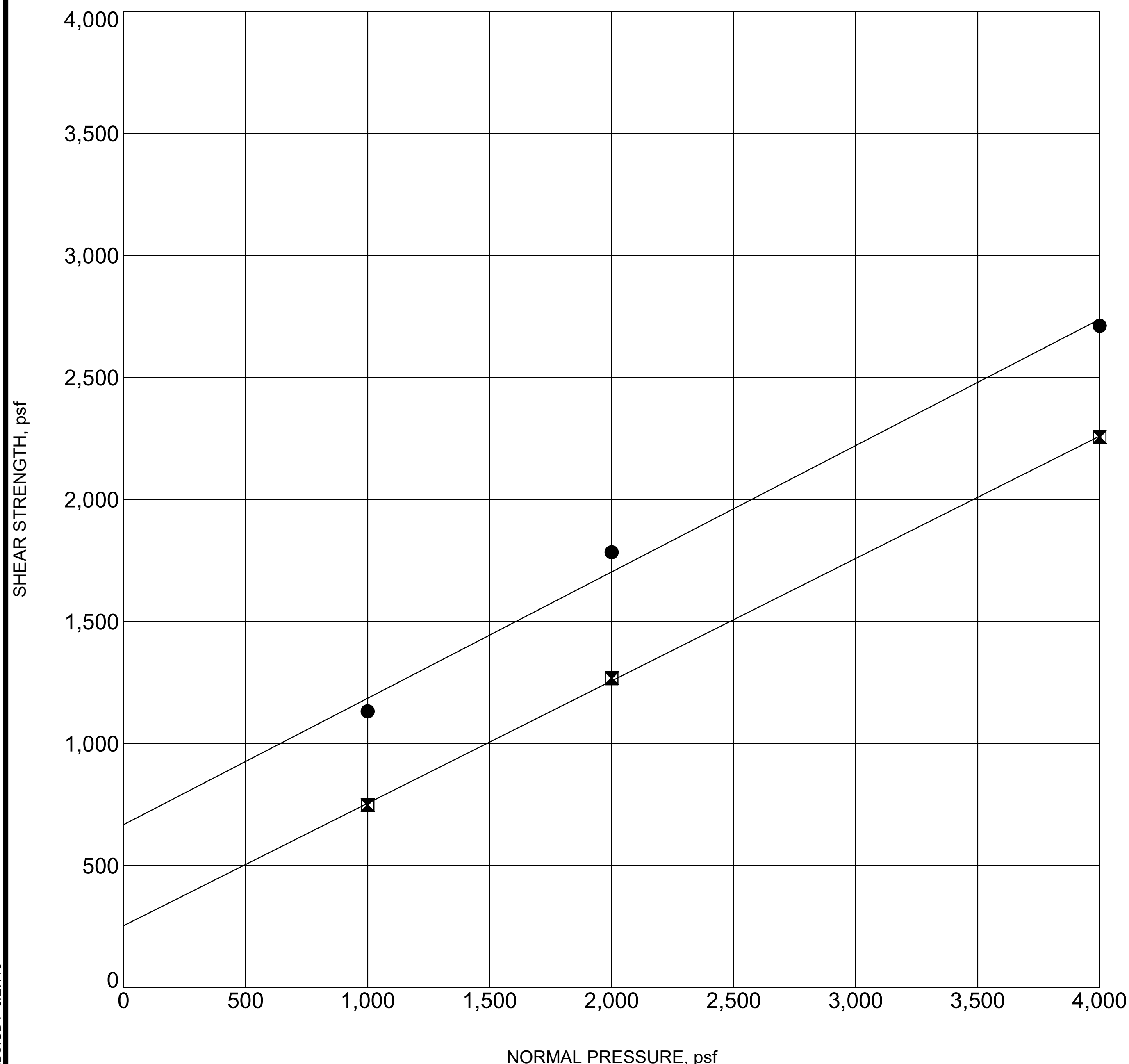
DIRECT SHEAR TEST

Rise Kohyang Charter School
3500 W 1st Street
Los Angeles, California

PROJECT NO.	REPORT DATE	
180719.1	September 2018	FIGURE B-4



Shear Strength Parameters

	Peak	Ultimate
Cohesion, C (psf):	668	250
Friction Angle, Ø (deg):	27	27

Boring No.: B-8
Sample Depth (ft): 5
Sample Description: Clayey SAND
Strain Rate (in./min): 0.005
Dry Density (pcf): 113.5

Initial Moisture (%): 11.9
Final Moisture (%): 20.5

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DIRECT SHEAR TEST

Rise Kohyang Charter School
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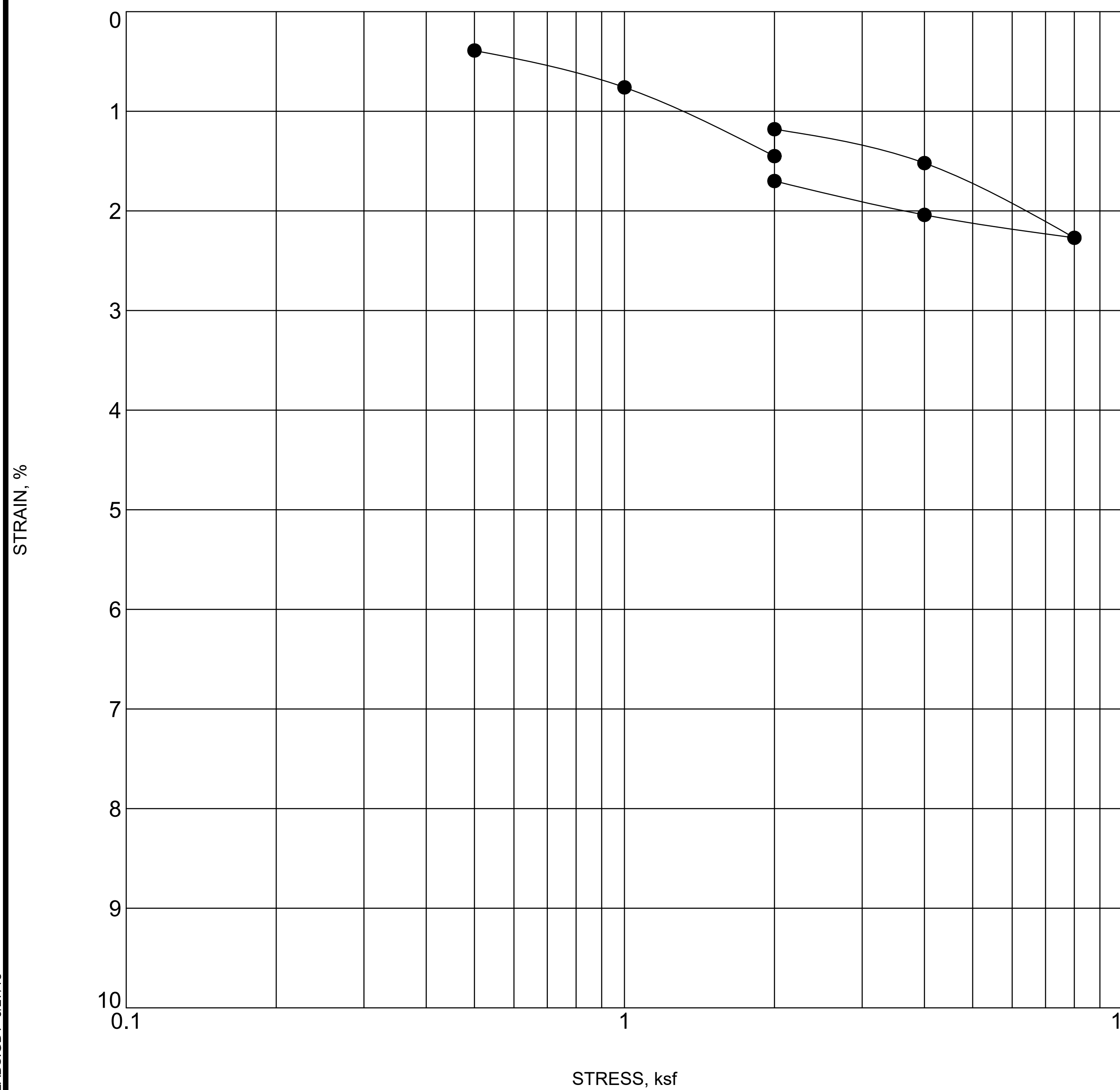
PROJECT NO.	REPORT DATE	FIGURE B-5
180719.1	September 2018	



	Sample Location	Soil Description	Dry Density (pcf)	Moisture Content (%)
●	**B-6 at 10 ft**	**Siltstone**	**93.4**	**26.9**

CONSOL STRAIN 180719.1 - 1ST STREET CHARTER SCHOOL.GPJ TWINING LABS.GDT 9/27/18

TWINING

CONSOLIDATION TEST

Rise Kohyang Charter School
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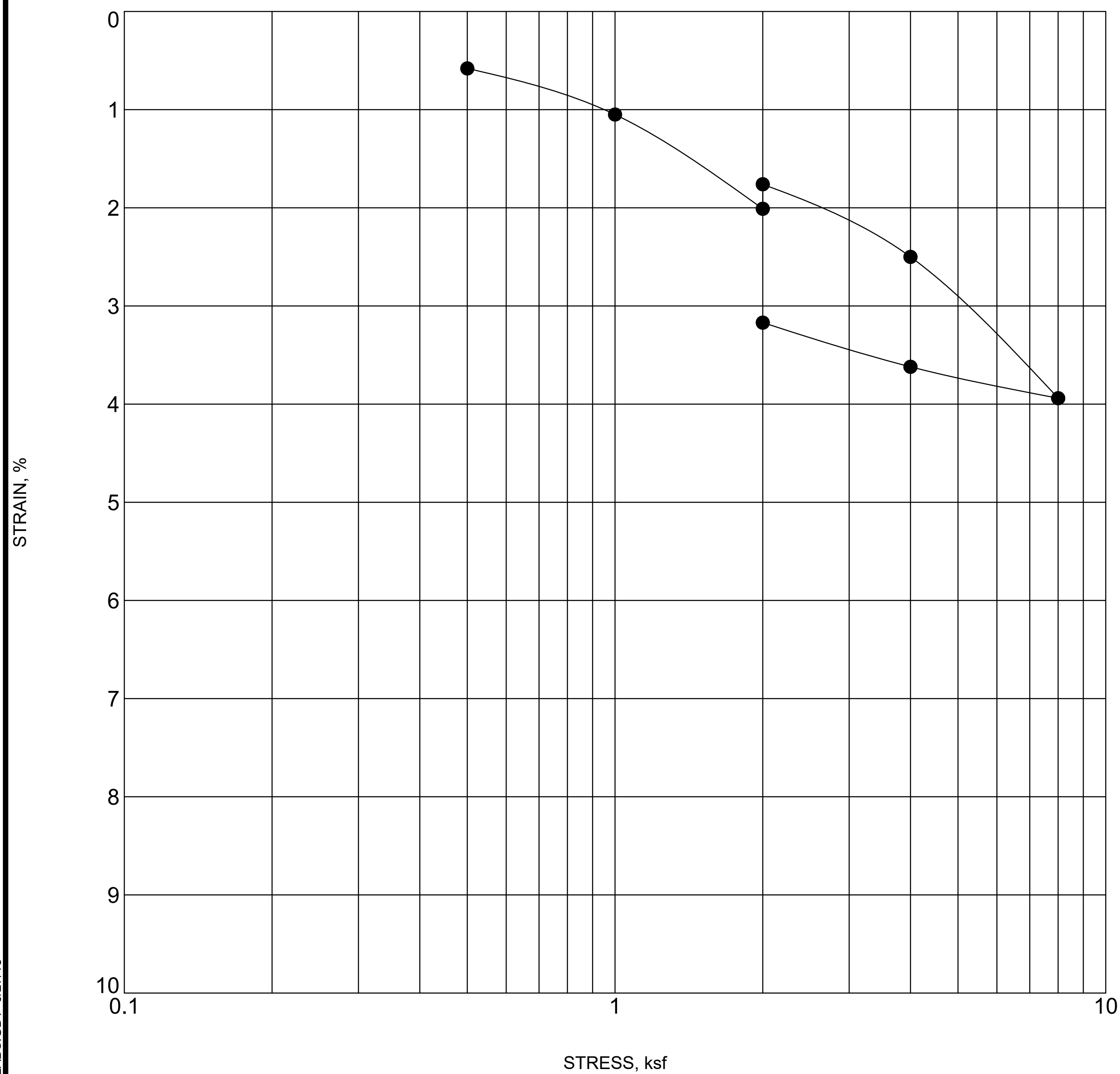
PROJECT NO.	REPORT DATE	
180719.1	September 2018	FIGURE B-6



	Sample Location	Soil Description	Dry Density (pcf)	Moisture Content (%)
●	**B-7 at 20 ft**	**Siltstone**	**90.2**	**31.0**

CONSOL STRAIN 180719.1 - 1ST STREET CHARTER SCHOOL.GPJ TWINING LABS.GDT 9/27/18

TWINING

CONSOLIDATION TEST

Rise Kohyang Charter School
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PROJECT NO.	REPORT DATE	FIGURE B-7
180719.1	September 2018	

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Appendix C
Percolation Testing

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Appendix C
Percolation Testing

One percolation boring was excavated at the project site as shown on Figure 2 – Site Geologic and Boring Location Map. The boring was advanced using an 8-inch hollow stem auger drill rig to approximately 5 feet below existing ground surface. Percolation testing was on Augus 30, 2017 in general conformance with the County of Los Angeles requirements.

The purpose of the tests was to evaluate the infiltration rates of subgrade soils. At the completion of the boring excavation, a 3-inch diameter slotted 20-foot-long PVC pipe was inserted in the borehole. The borehole was presoaked prior to testing. After the completion of presoaking, the borings were filled with water to a minimum depth of 3 feet above the bottom of excavation. Measurements of the distance from the top of the hole to the top of the water were taken every 30 minutes. The procedure was replicated for a total of 4 readings. Upon completion of the borings and testing, the boreholes were backfilled with soil from the cuttings as noted in the Log of Borings.

The infiltration rate was calculated by dividing the measured percolation rate by a reduction factor to account for discharge of water from the sides of the boring (i.e., non-vertical flow) as described in the referenced manual. The following formula was used:

Percolation Rate = (Δd / [Time Interval/60 minutes])
Reduction Factor (R_f) = ($2d_1$ / D - Δd/D) + 1
Infiltration Rate = (Percolation Rate) / (Reduction Factor)

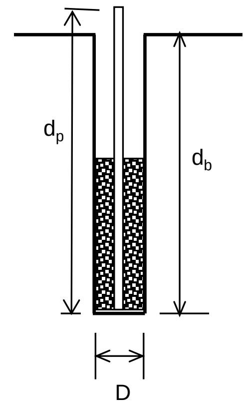
The lowest reading was used to determine the infiltration rate. A summary of test results is presented in Table C-1 and the detailed test data is attached to this appendix.

Table C-1 - Summary of Percolation Test Results

Test Location	Depth of Test Hole (ft.)	Design Infiltration Rate (in/hr)
B-7 (P-1)	21.5	0.06
B-8 (P-2)	21.5	0.04

Due to the presence of shallow bedrock and a very low infiltration rate, an infiltration BMP facility is not feasible at this site. If required, a filtration type of stormwater BMP facility, such as a bio-filtration planter, is recommended.

Project No.: 180719.1
Project Name: Rise Hohyang Charter



Boring No.: **B-7 (P-1)**
Diameter of Boring (D): 8.0 inches
Depth of Boring (d_b): 20.0 feet = 240 inches
Diameter of Perc. Pipe : 3.0 inches
Length of Pipe (d_p) : 20.0 feet = 240 inches

PRE-SOAK Number One		
Date:	8/30/2018	
Start Time:	8:10 AM	
Elapsed Time:	30.00	minutes
Water Remaining:	Yes	

PRE-SOAK Number Two		
Date:	8/30/2018	
Start Time:	8:41 AM	
Elapsed Time:	30.00	minutes
Water Remaining:	Yes	

CORRECTION FACTORS

Boring method: $CF_t = R_f = (2*d_i - \Delta d)/D + 1$
Site variability: $CF_v =$ 1.5 (1 ~ 3)
Long-term siltation: $CF_s =$ 1.5 (1 ~ 3)
Total Correction Factor: $CF = CF_t \times CF_v \times CF_s$

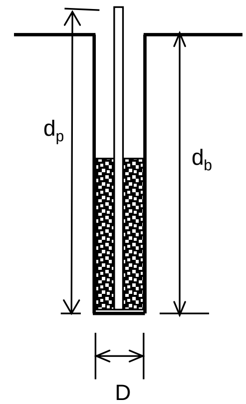
PERCOLATION TEST **Test Date:** 7/21/2017 **Test Performer:** SL **Calculated by:** SL

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)	Raw Percolation Rate $k_i = \Delta d / \Delta T$ (inch/hr)	Reduction Factor R_f	Total Correction Factor CF	Design Infiltration Rate $k = k_i / CF$ (inch/hr)
1	9:15 AM	9:45 AM	30	208.2	208.8	31.8	0.6	1.20	8.9	20.0	0.06
2	9:45 AM	10:15 AM	30	208.8	209.4	31.2	0.6	1.20	8.7	19.6	0.06
3	10:15 AM	10:45 AM	30	209.4	210.0	30.6	0.6	1.20	8.6	19.3	0.06
4	10:45 AM	11:15 AM	30	210.0	210.6	30.0	0.6	1.20	8.4	19.0	0.06

Recommended Design Infiltration Rate (inch/hr) = **0.06**

Reference: Los Angeles County (2014). Guidelines For Design, Investigation, and Reporting LID Stormwater Infiltration, GS200.1, dated 06/30/14

Project No.: 180719.1
Project Name: Rise Hohyang Charter



Boring No.: **B-8 (P-2)**
Diameter of Boring (D): 8.0 inches
Depth of Boring (d_b): 20.0 feet = 240 inches
Diameter of Perc. Pipe : 3.0 inches
Length of Pipe (d_p) : 20.0 feet = 240 inches

PRE-SOAK Number One
Date: 8/30/2018
Start Time: 10:05 AM
Elapsed Time: 30.00 minutes
Water Remaining: Yes

PRE-SOAK Number Two
Date: 8/30/2018
Start Time: 10:35 AM
Elapsed Time: 30.00 minutes
Water Remaining: Yes

CORRECTION FACTORS
Boring method: $CF_t = R_f = (2*d_i - \Delta d)/D + 1$
Site variability: CF_v = 1.5 (1 ~ 3)
Long-term siltation: CF_s = 1.5 (1 ~ 3)
Total Correction Factor: $CF = CF_t \times CF_v \times CF_s$

PERCOLATION TEST **Test Date:** 7/21/2017 **Test Performer:** SL **Calculated by:** SL

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)	Raw Percolation Rate $k_i = \Delta d / \Delta T$ (inch/hr)	Reduction Factor R_f	Total Correction Factor CF	Design Infiltration Rate $k = k_i / CF$ (inch/hr)
1	11:10 AM	11:40 AM	30	182.4	182.8	57.6	0.4	0.72	15.4	34.5	0.02
2	11:40 AM	12:10 PM	30	182.8	183.6	57.2	0.8	1.68	15.2	34.2	0.05
3	12:10 PM	12:40 PM	30	183.6	184.2	56.4	0.6	1.20	15.0	33.8	0.04
4	12:40 PM	1:10 PM	30	184.2	184.9	55.8	0.7	1.44	14.9	33.4	0.04

Recommended Design Infiltration Rate (inch/hr) = **0.04**

Reference: Los Angeles County (2014). Guidelines For Design, Investigation, and Reporting LID Stormwater Infiltration, GS200.1, dated 06/30/14