APPENDIX F

Geotechnical and Paleontological Resources Documentation

F-1 Preliminary Geotechnical Engineering Evaluation

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Geotechnical Engineering Evaluation Report

670 Mesquit Project 658 & 670 Mesquit Street Los Angeles, CA 90021

Prepared for:
RCS VE, LLC
319 Lafayette Street, Suite 133
New York, NY 10012

October 30, 2018
Project No.: 160599.1

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October 30, 2018
Project No. 160599.1

Zachary Vella
RCS VE, LLC
319 Lafayette Street, Suite 133
New York, NY 10012

Subject: Geotechnical Engineering Evaluation Report
670 Mesquit Mixed-Use Development
658 & 670 Mesquit Street
Los Angeles, California

Dear Mr. Vella,

In accordance with your request and authorization, we are presenting the results of our geotechnical investigation for the proposed 670 Mesquit project located at 670 & 658 Mesquit Street, Los Angeles, California. The purpose of this investigation has been to evaluate the subsurface conditions at the site and to provide geotechnical engineering recommendations for the proposed construction.

Based on our findings, the proposed project is geotechnically feasible, provided that the recommendations in this report are incorporated into the design and are implemented during construction of the project. This report was prepared in accordance with the requirements of the 2016 California Building Code and the City of Los Angeles Department of Building and Safety.

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 670 Mesquit project located at 658 and 670 Mesquit Street, Los Angeles, California - see 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 for design and construction of the proposed structures.

2. SITE DESCRIPTION AND PROPOSED DEVELOPMENT

The approximate site coordinates are latitude 34.035437°N and longitude 118.228765°W, and the site is located on the Los Angeles, California 7½-Minute Quadrangle (United States Geological Survey, 2015).

RCS VE LLC (the Applicant) proposes to construct a new mixed-use development (Project) totaling approximately 1,792,103 square feet of floor area on an approximately 5.45-acre property at 670 Mesquit Street in the Arts District of Downtown Los Angeles.

The Project Site flanks Mesquit Street between the former 6th Street Viaduct right-of-way on the north and the 7th Street Bridge on the south. The majority of the Project Site is on the east side of Mesquit Street; the southern portion of the Project Site also includes parcels on the west side of Mesquit Street at 7th Street. As part of the Project, Mesquit Street is proposed for vacation between 6th and 7th Streets.

Project implementation would require the removal of all existing on-site uses, including warehouses containing freezers, coolers, dry storage, and associated office space, totaling approximately 205,393 square feet of floor area. New development would include creative office space (approximately 944,050 square feet); a 236-room hotel; 308 multi-family residential housing units; an Arts District Central Market, a grocery store, and general retail uses totaling approximately 136,152 square feet; restaurants totaling approximately 89,576 square feet; studio/event/gallery space and a potential museum totaling approximately 93,617 square feet; and a gym of approximately 62,148 square feet. Buildings would range between 90 feet to 360 feet tall. The resulting floor area ratio would be approximately 7.5:1, assuming the proposed Mesquit Street vacation.

The Project would provide open space for use by Project residents, hotel guests, employees, and visitors totaling approximately 83,789 square feet. Proposed open space features include at-grade landscaped areas, pedestrian passageways and walkways, viewing platforms, and above-grade landscaped terraces and pool decks.

The Applicant also seeks to construct a Deck over the railway property if agreements can be obtained with Railway Property owners and financing and other funding becomes available. The Deck would serve as a multi-modal connection between the 7th Street Bridge and the Project Site's Northern Landscaped Area, which would provide access to the City's proposed PARC Improvements. The Deck could also provide access directly to the Los Angeles River.

The Project would include up to five levels of below-grade parking that spans the entire building footprint and would include at-grade and above-grade parking at the southern end of the Project Site. Approximately 3,800 parking spaces and 900 bicycle parking spaces are proposed. A rooftop heliport is proposed for emergency and occasional use incidental to the proposed office uses.

Construction would include approximately 527,100 cubic yards of grading (cut), all of which would be exported from the Project Site, with excavations extending to elevations of 185 feet above MSL for the

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lowest parking structure level and maximum excavations down to approximately 177 feet above MSL for elevator pits.

Project construction is anticipated to commence as early as 2020 and be completed as early as 2024, in a single phase, or as late as 2040 if built in separate phases over time. In the event construction is phased, construction of below-grade parking may also be phased.

3. SCOPE OF WORK

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

3.1. Literature Review

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. In addition, we have performed review of available grading documents at Los Angeles City Department of Building and Safety (LADBS). The list of documents reviewed is presented in the "References" section of this report.

3.2. Field Exploration

The subsurface conditions were evaluated on August 6 and 13, 2016 and on February 3, 2018 by advancing a total of eight 8-inch-diameter, hollow-stem-auger borings and three Cone Penetration Testing (CPT) soundings at various locations across the subject site. The borings were advanced to depths ranging from 40 to 75.8 feet below the existing grade, and the CPTs were advanced to a maximum depth of 37 ½ feet below the existing grade. The approximate locations of the borings and the CPTs are shown on Figure 2, Site Plan and Exploration Locations Map. Detailed exploration information of soils borings is presented in Appendix A, Field Exploration.

3.3. Percolation Testing

Percolation testing was performed on February 3, 2018 to evaluate the feasibility of infiltrating water at depths between 30 and 50 feet bgs, below the bottom of the proposed foundations. The details of our percolation testing procedures, results, calculations, conclusions and recommendations are discussed further in this report. Percolation testing data are presented in Appendix C, Percolation Testing.

3.4. Evaluation of Existing Sewer Line

Our evaluation of an existing sewer line below the Project Site was performed based on our review of the as-built sewer line plans prepared by the City of Los Angeles Department of Public Works, the alignment of the sewer line as shown on the project architectural plans, and the anticipated structural loading memo, included in Appendix F, prepared by the project structural engineer, Thornton Tomasetti (2018). We performed analyses including the evaluation of the potential for the line to deflect as a result of the proposed project excavations and the stress distribution that will be imposed on the line due to the proposed development.

Based on our review of the plans, it is our understanding that the line is 11 feet in diameter and runs beneath the southern portion of the property. The top of the sewer line is at an elevation of approximately 179 feet above MSL where it crosses the east project limit and at an elevation of approximately 178 feet above MSL where it crosses the west project limit (depths below the existing grade of approximately 72 and 73 feet, respectively). The bottom of the five-level subterranean parking is anticipated to be at an elevation of approximately 185 feet above MSL at its deepest

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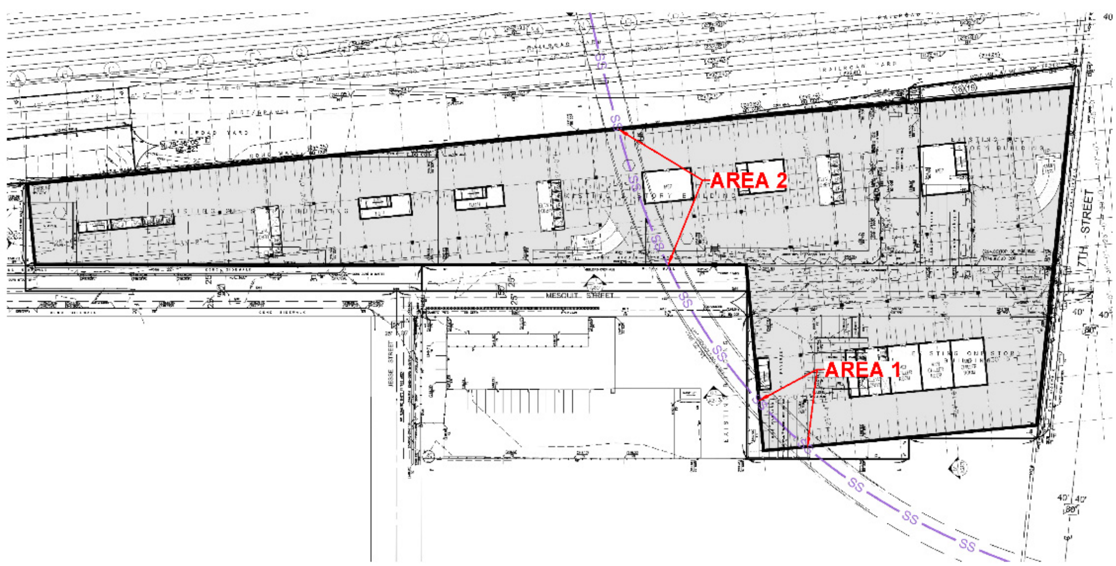
location above the pipe. Excavations for the foundations above the sewer line will extend to approximately 66 feet below the existing ground surface, with the resulting vertical distance between the top of the pipe and the bottom of the foundations to be approximately 6 feet where the line is shallowest. We note that elevator pit excavations would extend to depths between approximately 71 and 75 feet below the existing ground surface; however, elevator pits will be outside of the areas where the sewer locations and will not impact the existing sewer line. The sewer line location in relation to the proposed construction is depicted below in Diagram 1 showing the sections of the sewer line below the building footprint designated as Area 1 and Area 2.

Diagram 1 - Existing Sewer Line Within Proposed Building Footprint



The upward deflection of the sewer line due to ground stress relief resulting from the anticipated 66-foot-deep excavation was evaluated using Rocscience Settle3D software. The resulting stress relief from the anticipated excavation is approximately 7,920 psf based on a unit weight of soil of 120 pcf. We assumed a modulus of elasticity of 4,800 ksf for the soils below the excavation depth based on the cone penetration tests (CPT) and drilled borings performed at the site and our engineering judgement. Settle3D uses the elastic modulus of the soil to evaluate the strain produced by a resultant negative effective stress caused by the stress relief during the site excavations. Our analyses show a maximum upward deflection of approximately 2.6 inches at the center, and 2.1 inches at the edges of the deep excavation at Area 1. We estimated an upward deflection of approximately 4.2 inches at the center, and 3.1 inches at the edges of the deep excavation at Area 2.

Our analyses to determine the maximum stress that will be imposed at the top of the sewer line upon construction of the building is based on our review of a structural loading memo from Thornton Tomasetti. Diagram 2 below shows the loads used in our analyses, based on the provided building weights. The total building load in Area 1 is 3,650 psf, including above-grade structure, below-grade structure, and mat foundation weights. The total building load in Area 2 is 4,950 psf, including above-grade structure, below-grade structure, and mat foundation weights.

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Diagram 2 - Total Building Loads

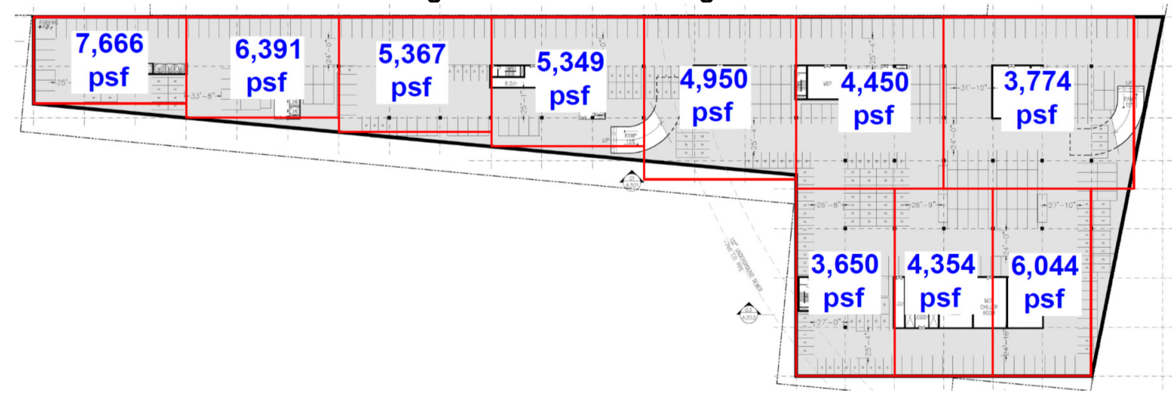

Table 1 below provides the maximum anticipated estimated stresses due to the proposed building loads for each of the areas below the building footprint and for each mat foundation type (i.e., rigid or flexible). The values provided include the weight of 6 feet of soil resting on top of the sewer line below the bottom of the mat foundations.

Table 1 – Maximum Anticipated Stress Upon Loading of Mat Per Area and Type of Mat Foundation

Type of Mat Foundation	Location	Estimated Maximum Stress (psf)
Rigid	Area 1	5,180
	Area 2	7,010
Flexible	Area 1	4,570
	Area 2	5,770

Based on the calculated loads, we anticipate that the stress acting upon the construction of the project will be less than the stress in its existing condition. The existing stress on the sewer line can be estimated as 8,640 psf, based on the soil overburden above it. Upon construction of the project, the stress on the sewer line will be ultimately reduced to a maximum of 7,010 psf for a rigid mat foundation.

Because the pressure anticipated from the building loads is of relatively similar magnitude of the relieved stress resulting from the building excavation, we anticipate that the sewer line will deflect downward with approximately the same magnitude as the estimated upward deflection. That is, we anticipate the final stress on the sewer line and the final elevation of the sewer line upon construction of the building will be approximately the same as it exists prior to any site construction.

A structural evaluation of the sewer line itself should be performed to determine if the sewer line can accommodate these anticipated deflections. To confirm the magnitude of the cited stresses and resulting deflections of the sewer line, the anticipated building loading should be reviewed upon determination of the final thickness of the mat foundation.

3.5. Evaluation of Effects on Surrounding Structures

We preliminarily evaluated the effects on surrounding structures as a result of the proposed development. The proposed excavation will be adjacent to the 7th Street Bridge to the south, the railroad tracks on the east, and existing buildings adjacent to the proposed Building 5. Provided that the following items in this section of the report are addressed, it is our opinion that the potential for affecting surrounding structures is substantially reduced.

Based on preliminary layout of the proposed basement, we anticipate that the 7th Street Bridge will impose loading on the temporary shoring at the southern edge of the Project Site and the permanent basement walls of Buildings 4 and 5. We reviewed plans that were prepared for the seismic retrofit of the bridge in 1995 (City of Los Angeles, 1995) for information regarding foundation type and depth. Based on those plans, the bridge is supported by several column supports along its span with abutments at the east (approach abutment) and west (abutment adjacent to the river) ends of the portion of the bridge on the west side of the Los Angeles River. Buildings 4 and 5 will be adjacent to and potentially be affected by nine foundations supporting columns along the bridge span and the east approach abutment. Foundations supporting the columns within the span and the approach abutment consist of concrete spread footings that are embedded on the order of 10 feet below the ground surface. This correlates to bottom of footing elevations at an elevation of approximately 220 to 230 feet above MSL. The bottom of the planned excavation for the basement of Buildings 4 and 5 will be at an elevation of approximately 186 feet above MSL. Therefore, depending on the lateral distance from the bridge to the shoring and basement wall of Buildings 4 and 5, surcharge loading on the shoring and basement wall from the bridge foundations will need to be considered. Additionally, locations of tiebacks for shoring elements should be carefully evaluated with respect to the bridge foundation elements.

In general, foundations that are situated above a 1:1 plane projected up from the bottom edge of basement mat foundation should be considered as surcharging the shoring and basement wall of Buildings 4 and 5. At this time, details of exact locations of the bridge foundations and the loading on those foundations are not available. However, when this information is available, Twining can provide an estimate of surcharge pressures on the shoring and basement walls.

Existing offsite buildings along the west side of the excavation for proposed Building 5, including 688 S. Santa Fe Avenue (3-story masonry building housing residential units) and the building immediately north of Building 5, should be considered as surcharging the shoring and basement wall of Building 5.

The railroad tracks on the east side of the project site are within the zone that impose surcharge pressure on the basement wall of Building 4 (i.e., a 1:1 plane projected up from the bottom edge of the mat foundation). Therefore, surcharge from passing trains should be considered in the design of the shoring and the basement wall of the Building 4.

To reduce the potential effects on surrounding improvements and structures, in general, temporary shoring should be designed such that no more than 1 inch of deflection at the top of the shoring is allowed adjacent to the existing building foundations and the railroad tracks. If less deflection at the top of shoring is necessary, the values for lateral earth pressures on shoring presented later in this report may be increased.

3.6. Geotechnical Laboratory Testing

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

- In-situ moisture and density;
- #200 sieve wash;
- Maximum dry density-optimum moisture content;
- Direct shear;
- Consolidation;
- R-value; and
- Corrosivity.

3.7. 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 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 and site development;
- 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;
- Evaluation of lateral earth pressures for retaining walls and recommendations for retaining wall backfill;
- Recommendations for slab-on-grade and concrete flatwork;
- Preliminary evaluation of the potential for the on-site materials to corrode buried concrete and metals; and
- Recommendations for design and construction of asphalt-concrete pavements.

4. SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1. Regional Geologic Setting

The project site is located in the northern portion of the Central Block of the Los Angeles Basin (Yerkes, et al., 1965). The Los Angeles Basin, in turn, is situated at the northwestern tip of the Peninsular Ranges Geomorphic Province. The Peninsular Ranges Geomorphic Province extends along the Pacific Ocean coast from the tip of Baja California northward to the Transverse Ranges Geomorphic Province, with the boundary occurring along the Malibu Coast – Santa Monica – Hollywood – Raymond – Sierra Madre – Cucamonga fault complex. The Peninsular Ranges Geomorphic Province is characterized by generally northwest-trending structural features including

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mountain ranges, basins, and major faults, including the Newport-Inglewood and Whittier fault zones in relatively close proximity to the site.

The Central Block is bordered by the Santa Monica – Hollywood fault complex on the north, the Newport-Inglewood and Whittier-Elsinore fault zones on the southwest and northeast, respectively, and by the uplands of Newport Mesa, San Joaquin Hills, and Santa Ana Mountains on the south and southeast. The structure of the block is dominated by a northwest-trending, doubly-plunging trough into which have been deposited Late Cretaceous through Holocene sediments extending to depths in excess of 31,000 feet below sea level at the basin's deepest point (Yerkes et al., 1965).

According to regional geologic mapping published by the Dibblee Geological Foundation, the site is underlain by geologic unit Qa - Alluvium, consisting of unconsolidated floodplain deposits of silt, sand, and gravel deposited by the Los Angeles River (Dibblee, 1989). A portion of this geologic map is reproduced as Figure 3, Regional Geologic Map.

The Union Station Oil Field is located approximately ½ mile northwest of the site. Petroleum has been produced from scattered wells located in relatively close proximity to the site, although records of the California Division of Oil, Gas, and Geothermal Resources do not indicate the presence of producing or abandoned petroleum wells on the project site (Munger Oil Information Service, Inc., 2003).

4.2. Subsurface Earth Materials

Earth materials encountered during our subsurface investigation consist of a layer of undocumented fill overlying alluvium. In general, the undocumented fill consists of silty sand containing isolated construction debris extending to a depth of approximately 5 to 6 feet below the ground surface, as encountered in our exploratory excavations. The fill is anticipated to be present across the entire project site. The alluvial deposits consist predominantly of sand with gravel, which extended to the total depth of each exploratory excavation. Additional detail regarding the subsurface materials encountered at the site is presented in Appendix A, Field Exploration.

4.3. Groundwater

The deepest exploratory boring at the site was advanced to a depth of approximately 75.8 feet below the existing grade. Water seepage was encountered in our deepest boring performed at the site at approximately 75 feet below the existing grade corresponding to an elevation of approximately 172 feet above MSL.

Based on our review of a groundwater monitoring report prepared by Kleinfelder (2018) for a site located at 590 South Santa Fe Avenue, Los Angeles, California, groundwater was recorded in five groundwater monitoring wells on that property at depths ranging between 65 and 70 feet below the ground surface in February 2018. The groundwater wells are located just north of the Santa Fe Avenue and Willow Street intersection, approximately 900 feet northwest from the northwest corner of the proposed Building 1. The wells are situated at surface elevations ranging between 255 and 261 feet above MSL, and recorded groundwater elevations range between 188 and 190 feet above MSL (Kleinfelder, 2018).

Based on our review of the Subsurface Methane Report prepared by Wood (2018), groundwater was encountered at a depth of 65 feet below the existing ground surface at two locations near the south end of the site. Based on approximate surface elevations of those two locations, the elevation of the groundwater ranges from approximately 183 to 185 feet MSL.

Based on our review of a groundwater monitoring report prepared by Arcadis (2016) at the same nearby site (590 South Santa Fe Avenue), groundwater data was collected in October 2013 for a well located approximately 500 feet northwest of the northwest corner of Building 1. The well is situated at a surface elevation of approximately 250 feet above MSL, and groundwater was recorded at a depth of approximately 66 feet below the existing ground surface, which is an elevation of approximately 184 feet above MSL (Arcadis, 2016).

According to mapping published by the California Department of Conservation, Division of Mines and Geology (1998), the historical high groundwater level is reportedly at depths greater than 125 to 150 feet below the ground surface. A portion of the map is presented as Figure 8, Historical High Groundwater Map.

Considering the information available, it is our opinion that the depth to groundwater at the site be considered at an elevation of 190 feet above MSL (approximately 57 to 61 feet below the existing surface at the project site) as recorded on the adjacent property north of the site (Kleinfelder, 2018). We note that groundwater was not observed in our borings with the exception of seepage at an elevation of 172 feet above MSL, which is approximately 18 feet higher than that observed on the adjacent property. 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 (such as heavy irrigation and groundwater injection).

5. GEOLOGIC HAZARDS AND SEISMIC DESIGN CONSIDERATIONS

5.1. Surface Fault Rupture

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

5.2. Active Faulting

Active faults are defined as those that have experienced surface displacement within Holocene time (approximately the last 11,000 years). The nearest known active fault corresponds to the Elysian Park (Upper) fault system located approximately 2.21 miles from the site. This system has the potential to be the dominant source of strong ground motion. Appendix D provides a list of selected known active faults within a search radius of 80 km (50 miles), approximate fault-to-site distances, maximum magnitude (Mmax), and fault type as published by the 2009 USGS National Seismic Hazard Maps website (USGS, 2008b). Figure 7 attached to this report presents a map depicting the project site in relation to the nearest faults. Based on our review of the faulting in the vicinity of the project site, it is our opinion that there is a low potential for ground rupture due to seismic faulting at the project site.

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 Los Angeles Quadrangle (California Department of Conservation, Division of Mines and Geology, 1999), the site is not located within a zone of required investigation for liquefaction. Based on lack of shallow groundwater, relatively dense soils at the site and relatively uniform soil stratum across the site, it is our professional opinion that the liquefaction potential at the site is very low.

Seismically-induced dry sand settlement is the ground settlement due to densification of loose, dry cohesionless soils during strong earthquake shaking. Based on the dense nature of the onsite soil and relative uniform soil profile encountered across the site, it is our professional opinion that seismically-induced dry-sand settlement is 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 the current FEMA flood map for the site, effective as of September 2008, the site is outside the 0.2% annual chance (500-year) floodplain.

5.6. Tsunamis and Seiches

Tsunamis are waves generated by massive landslides near or under sea water. The site is not located on any State of California – County of Los Angeles Tsunami Inundation Map for Emergency Planning. The potential for the site to be adversely impacted by earthquake-induced tsunamis is considered to be negligible because the site is located approximately 14 miles inland from the Pacific Ocean shore, at an elevation exceeding the maximum height of potential tsunami inundation.

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

5.7. Methane

We have reviewed the City of Los Angeles Methane and Methane Buffer Zones map. Based on our review, it appears that the subject property is not located within a Methane Zone, however, it is in a Methane Buffer Zone. A qualified methane specialist should be consulted to determine the potential of methane gas to impact the site.

5.8. Deaggregated Seismic Source Parameters

Our recommendations for design earthquake magnitude parameters and ground shaking analyses have been developed in accordance with the USGS Unified Hazard Tool webpage

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(https://earthquake.usgs.gov/hazards/interactive/) for the 2 percent in 50 year chance of exceedance earthquake event. Based on the calculated results, the earthquake magnitude M_w=6.91 should be considered for the seismic design.

5.9. CBC Seismic Design Parameters

Our recommendations for seismic design parameters have been developed in accordance with 2016 CBC and ASCE 7-10 (ASCE, 2010) standards. The applicable site class is D based on the results of our field investigation. Table 2 presents the seismic design parameters for the site in accordance with 2016 CBC and mapped spectral acceleration parameters (United States Geological Survey, 2011).

Table 2 – 2016 California Building Code Design Parameters

Design Parameters	Value
Site Class	D
Mapped Spectral Acceleration Parameter at Period of 0.2-Second, S_s	2.329 g
Mapped Spectral Acceleration Parameter at Period 1-Second, S_1	0.815 g
Site Coefficient, F_a	1.0
Site Coefficient, F_v	1.5
Adjusted MCE_R[1] Spectral Response Acceleration Parameter at Short Period, S_{MS}	2.329 g
1-Second Period Adjusted MCE_R[1] Spectral Response Acceleration Parameter, S_{M1}	1.222 g
Short Period Design Spectral Response Acceleration Parameter, S_{DS}	1.552 g
1-Second Period Design Spectral Response Acceleration Parameter, S_{D1}	0.815 g
Peak Ground Acceleration, PGA_M[2]	0.875 g
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

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.

Foundations for the subterranean parking structure should bear on competent native soils as encountered in our field exploration at the level of the proposed excavation. The exposed subgrade for the building mat foundations below the subterranean parking levels should then be scarified to a depth of at least 8 inches and recompacted in accordance with Section 6.2.4 of this report.

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. 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.

6.2.1. 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.

6.2.2. Overexcavation

In general, the upper 6 feet of the subsurface soils are undocumented fill consisting of silty sand materials that are not suitable for foundation support for the proposed development. Deeper fills and debris may also be encountered after the demolition of the existing buildings. We anticipate that the existing fill material and debris will be removed as part of the excavation for the five-level subterranean parking structure. We recommend that the bottom of the mat foundations bear on competent native soil that is scarified to a depth of at least 8 inches and recompacted to 95 percent of the maximum dry density as determined in accordance with ASTM D1557.

Pavement and/or sidewalk areas should be over-excavated to a depth of at least 2 feet below the pavement section, as measured from the bottom of the aggregate base layer. Deeper removals may be required in areas where soft, saturated, or unsuitable materials are encountered.

The extent and depths of removal should be evaluated in the field based on the materials exposed by a geotechnical engineer, or a City of Los Angeles Registered Deputy Grading Inspector representing the geotechnical engineer in the field. Additional removals may be recommended if loose or soft soils are exposed during grading.

6.2.3. Materials for Fill

Soils generated from excavations at the site 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. We anticipate that the majority of material excavated during construction of the subterranean levels will be suitable for use as fill.

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.

6.2.4. Engineered Fill

Prior to placement of engineered fill, the contractor should request an evaluation of the exposed excavation bottom by Twining. Unless otherwise recommended, the exposed ground surface should then be scarified to a depth of at least 8 inches and watered or dried, as needed, to achieve generally consistent moisture contents approximate 2 percent above the optimum moisture content. The scarified materials should then be compacted to 95 percent relative compaction in accordance with the latest version of ASTM Test Method D1557.

Engineered fill should be placed in horizontal lifts of approximately 6 to 8 inches in loose thickness. Prior to compaction, each lift should be watered or dried as needed to achieve near optimum moisture condition, mixed, and then compacted by mechanical methods, using sheepsfoot rollers, multiple-wheel pneumatic-tired rollers, or other appropriate compacting rollers, to a relative compaction of 95 percent as evaluated by ASTM D1557. Successive lifts should be treated in a like manner until the desired finished 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.

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.

6.2.5. Excavation Bottom Stability

In general, we anticipate that the bottoms of the excavations will be stable and provide suitable support to the proposed developments, except in portions of the building where the elevator pits potentially extend below the groundwater. Although not encountered in our borings at the site, information from the adjacent site to the north of the project site indicates that groundwater may be as high as 57 feet below the existing ground surface (based on a highest recorded groundwater elevation of 190 feet above MSL, Kleinfelder, 2018); should this condition exist below the site, we anticipate that the bottom of the excavation may require mitigation being 5 feet below the highest anticipated ground water level. We note that the elevator pit bottoms are anticipated to be located at a maximum depth of approximately 75 feet below the ground surface, and may also require mitigation in accordance with this section.

Unstable bottom conditions, as evidenced by yielding under construction equipment loading or elevated moisture conditions, may be significantly reduced by overexcavation and replacement with a minimum 1-foot-thick aggregate base. Other options such as the incorporation of geogrid material, may be recommended based on the field evaluation. Recommendations for stabilizing excavation bottoms should be based on evaluation in the field by geotechnical engineer at the time of construction.

6.2.6. Construction Dewatering

Based on observations of groundwater seepage at depth of approximately 65 to 75 feet below the existing grade and the maximum height of groundwater from the adjacent property (approximately 57 feet below the existing ground surface), and the anticipated maximum 66-foot-deep excavations for the majority of the proposed site buildings, dewatering measures may be necessary during excavation operations. For constructability considerations, we recommend that additional groundwater monitoring wells be installed at the site to better define the depth to groundwater at the time of construction.

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

6.2.7. Rippability

Based on our subsurface exploration of the site, the fill and alluvium should be generally excavatable with heavy-duty earthwork equipment in good working condition. However, some large-size cobbles may be encountered during excavation or drilling due to the nature of the alluvial material. Difficult drilling conditions should be anticipated at this site due to the likely presence of cobbles.

6.2.8. Caving Potential

In general, the surficial soils consist of dense sandy soils. Although caving was not encountered during drilling for our subsurface investigation, we anticipate that caving may occur during drilling for shoring soldier piles and tiebacks, particularly below the groundwater elevation. In the event of soil caving, it may be necessary to use casing and/or drilling mud to permit the installation of the soldier piles. Alternatively, a continuous flight, hollow-stem auger system can be used so that concrete can be pumped into the drill system while it is extracted from the borehole. Drilled holes for soldier piles should not be left open overnight. Concrete for piles should be placed immediately after the drilling of the hole is complete. The concrete should be pumped to the bottom of the drilled shaft using a tremie through the continuous flight, hollow-stem auger, if utilized. Once concrete pumping is initiated, the bottom of the tremie should remain below the surface of the concrete to prevent contamination of the concrete by soil inclusions. If steel casing is used, the casing should be removed as the concrete is placed, such that there is at least 10 feet of concrete head during removal.

6.2.9. Expansive Soil

Expansive soils are characterized by their ability to undergo significant volume changes (shrink or swell) due to variations in moisture content. According to our observations, the soils encountered near the ground surface and at the anticipated ground surface depths exhibit low expansion potential; therefore, recommendations to prevent expansive soil conditions are not warranted.

6.3. Foundation Recommendation

6.3.1. Mat Foundation

A mat foundation system may be used for support of the proposed buildings, provided that the mat is placed on competent native material prepared as described in the "Earthwork and Site Preparation" section of this report. The mat foundation should be designed by the structural engineer and should conform to the 2016 California Building Code.

The preliminary structural loads for the proposed high-rise building were provided to us by the office of Thornton Tomasetti.

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The following information summarizes the loading conditions and assumptions made during our analyses to estimate the settlement for the entire footprint of the building.

- The preliminary building loads along with their areas are shown in Diagram 2 in Section 3.4. These loads include the weight of the mat foundation.

- We anticipate that the bottom of the proposed up to five-level subterranean structure will be at an elevation of approximately 185 feet above MSL (which ranges between approximately 61 and 68 feet below the existing ground surface), and the bottoms of elevator pits will extend to an elevation of approximately 177 feet above MSL (which ranges between approximately 71 and 75 feet below the existing ground surface). Our assumption is based on our review of the provided parking depth summary memo prepared on September 13, 2018 by KPFF.

- We have performed our analyses for both a flexible and rigid mat foundation.

- Based on the information provided, we estimate the total settlement for a rigid and flexible mat foundation to be 3.51 and 2.88 inches, respectively. The calculated settlement values for both a rigid and flexible mat foundation are shown on Diagrams 3 and 4. We note that the total settlement for the rigid foundation is concentrated on the edge of the mat where the heaviest load is being imposed from the building, and for the flexible mat, the total settlement is concentrated near the center of the mat where the heaviest loading is generated from the eastern and western buildings sharing a common podium. Details of the settlement estimates are provided in Appendix D.

Diagram 3 - Rigid Mat Foundation – Summary of Calculated Settlement (Inches)

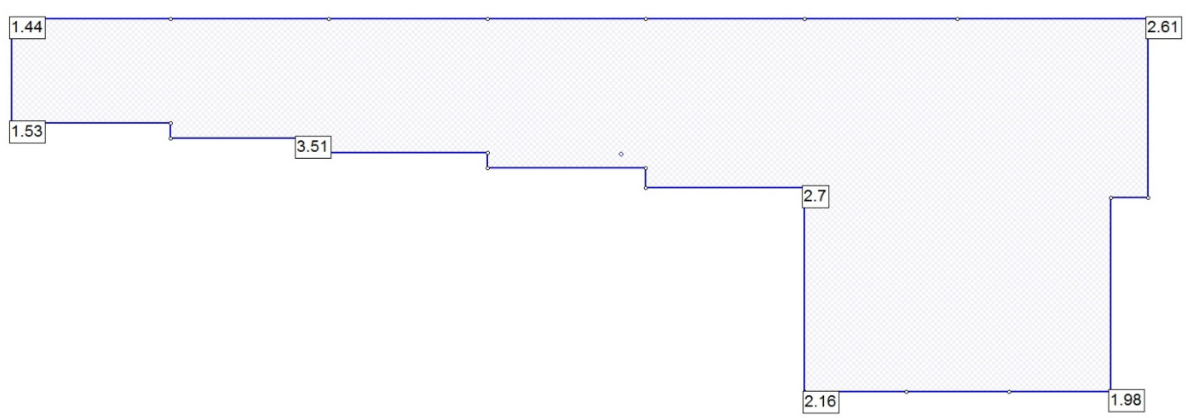

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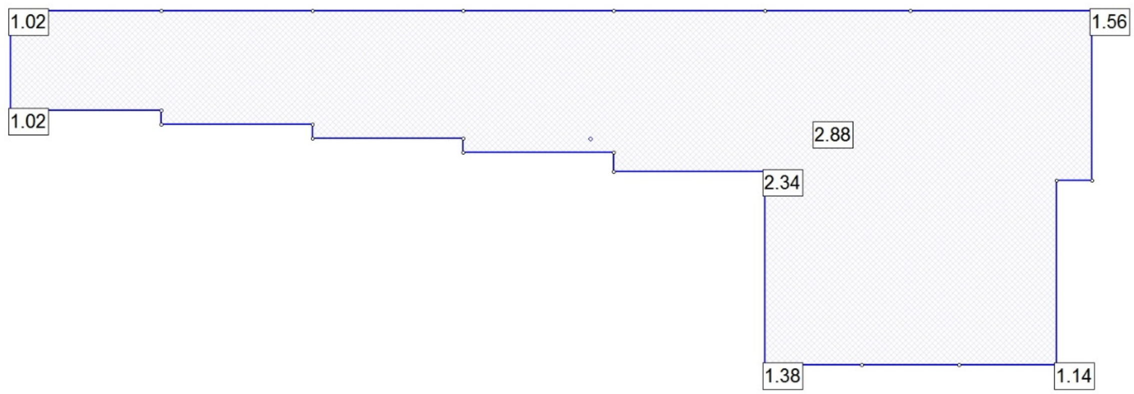
Diagram 4 - Flexible Mat Foundation – Summary of Calculated Settlement (Inches)



An allowable coefficient of friction of 0.4 can be used for design. The passive resistance can be computed using an allowable equivalent fluid pressure (EFP) or 350 pcf. The total allowable lateral resistance can be taken as the sum of the friction resistance and passive resistance, provided that the passive resistance does not exceed two-thirds of the total allowable resistance. The passive resistance values may be increased by one-third when considering wind or seismic loading.

6.4. Basement and Retaining Walls

6.4.1. 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.

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 equivalent to a fluid weighing 65 pcf. Where adequate drainage is not provided behind walls, further evaluation should be conducted by a geotechnical engineer.

For walls that are free to rotate at the top (such as cantilevered walls) and have adequate drainage, the lateral earth pressure may be designed for the “active” EFP of 35 pcf. Where adequate drainage is not provided behind walls, further evaluation should be conducted by a geotechnical engineer.

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 55% of the magnitude 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.

6.4.2. Seismic Lateral Earth Pressure

Retaining walls greater than 6 feet in height should be designed for seismic lateral earth pressures. The seismic pressure distribution may be considered to be a triangle with the maximum pressure at the bottom. The following combination of static and incremental seismic pressures shown in Diagram 5 may be used for seismic design for both cantilever and restrained walls.

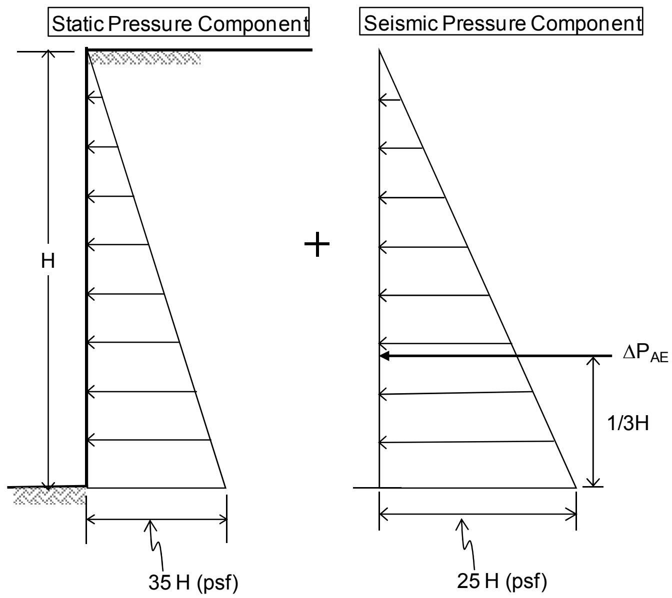


Diagram 5 - Seismic Earth Pressure Distribution of Retaining Walls

6.4.3. Backfill and Drainage of Walls

The backfill material behind walls should consist of granular non-expansive material and be approved by the project geotechnical engineer. Based on the soil materials encountered during our exploration, the majority of on-site soils will meet this requirement. 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. Drainage behind the basement walls may be provided by a geosynthetic drainage composite such as TerraDrain, MiraDrain, or equivalent, attached to the outside perimeter of the wall. The drain should be placed continuously along the back of the wall and connected to a 4-inch-diameter perforated pipe. The pipe should be sloped at least 1% and should be surrounded by 1 cubic foot per foot of ¾-inch crushed rock wrapped in suitable non-woven filter fabric (Mirafi 140NL or equivalent). The crushed rock should meet the requirements defined in Section 200-1.2 of the latest edition of The “Greenbook” Standard Specifications for Public Works Construction (Public Works Standards, 2015). The drain should discharge through a solid pipe to an appropriate outlet, using a sump/pump system.

6.4.4. Elevator Pits

We understand that the excavations for the elevator pits will extend to an elevation of 177 feet MSL (depths ranging between approximately 71 and 75 feet below the existing ground surface). The groundwater table could be as high as 190 feet MSL (57 to 61 feet below the ground surface) based on information from wells nearby the site as previously discussed in this report. On this basis, we recommend that the elevator pits be designed for hydrostatic uplift pressures acting below the bottom of and on the side walls of the elevator pits. Hydrostatic pressure should be considered for a water level that is at an elevation of 190 feet above MSL.

6.5. Concrete Slabs/Mat Foundation

Floor slabs and mat foundation slabs should be supported on native material found at the basement level and prepared in accordance with the recommendations of this report. For design of concrete floor slabs/mat slab, a modulus of subgrade reaction (*k*) of 300 pounds per cubic inch (pci) may be used for slabs/ mat foundation subgrade prepared in accordance with this report.

Floor and mat foundation slabs should be designed and reinforced in accordance with the structural engineer's recommendations. 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 3 provides recommendations for various levels of protection against vapor transmission through concrete floor slabs that are anticipated to receive carpet, tile or other moisture sensitive coverings placed over a properly prepared subgrade. The use of a moisture barrier should be determined by the project architect. Care should be taken not to puncture the plastic membrane during placement of the membrane itself and the overlying silty sand.

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Table 3 - Options for Subgrade Preparation Below Concrete Slabs

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

Notes:

[1] The silty sand should have a gradation between approximately 15 and 40 percent passing the No. 200 sieve and a plasticity index of less than 4. 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., 2012).

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

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.

6.6. Drainage Control

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

- Ponding and areas of low flow gradients should be avoided.
- If bare soil within 5 feet of the structure is not avoidable, then a gradient of 5 percent or more should be provided sloping away from the improvement. Corresponding paved surfaces should be provided with a gradient of at least 1 percent.
- The remainder of the unpaved areas should be provided with a drainage gradient of at least 2 percent.
- Positive drainage devices, such as graded swales, paved ditches, and/or catch basins should be employed to accumulate and to convey water to appropriate discharge points.
- Concrete walks and flatwork should not obstruct the free flow of surface water.
- Brick flatwork should be sealed by mortar or be placed over an impermeable membrane.
- Area drains should be recessed below grade to allow free flow of water into the basin.
- Enclosed raised planters should be sealed at the bottom and provided with an ample flow gradient to a drainage device. Recessed planters and landscaped areas should be provided with area inlet and subsurface drain pipes.
- Planters should not be located adjacent to the structures wherever possible. If planters are to be located adjacent to the structures, the planters should be positively sealed, should incorporate a subdrain, and should be provided with free discharge capacity to a drainage device.
- Planting areas at grade should be provided with positive drainage. Wherever possible, the grade of exposed soil areas should be established above adjacent paved grades. Drainage devices and curbing should be provided to prevent runoff from adjacent pavement or walks into planted areas.
- Gutter and downspout systems should be provided to capture discharge from roof areas. The accumulated roof water should be conveyed to off-site disposal areas by a pipe or concrete swale system.
- Landscape watering should be performed judiciously to preclude either soaking or desiccation of soils. The watering should be such that it just sustains plant growth without excessive watering. Sprinkler systems should be checked periodically to detect leakage and they should be turned off during the rainy season.

6.7. Temporary Excavations

Temporary excavations for the demolishing, earthwork, footing and utility trench are expected. We anticipate that unsurcharged excavations with vertical side slopes less than 3 feet high will generally be stable; however, sloughing of cohesionless sandy materials encountered at the site should be expected.

Where the space is available, temporary, unsurcharged excavation sides over 3 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

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necessary when considering heavy vehicles, such as concrete trucks and cranes. Twining should be advised of such heavy vehicle loadings so that specific setback requirements can be established. If the temporary construction slopes are to be maintained during the rainy season, berms are recommended to be graded along the tops of the slopes in order to prevent runoff water from entering the excavation and eroding the slope faces. Where space for sloped excavations is not available, temporary shoring may be utilized. Geotechnical recommendations for the design and construction of temporary shoring are presented in the "Temporary Shoring" section of this report. Personnel from Twining should observe the excavation 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.

Excavations shall not undermine the existing adjacent building footings. Where space for sloped excavations is not available, temporary shoring may be utilized.

6.8. Temporary Shoring

Temporary shoring is anticipated to be placed along the perimeter of the proposed basement parking garage. Based on the assumed finished floor elevation and anticipated foundation excavations, shored walls may be on the order of 60 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.

6.8.1. 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. 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 – see Diagram 2 – Earth Pressure Distribution for Tie-back or Braced Shoring Wall below.

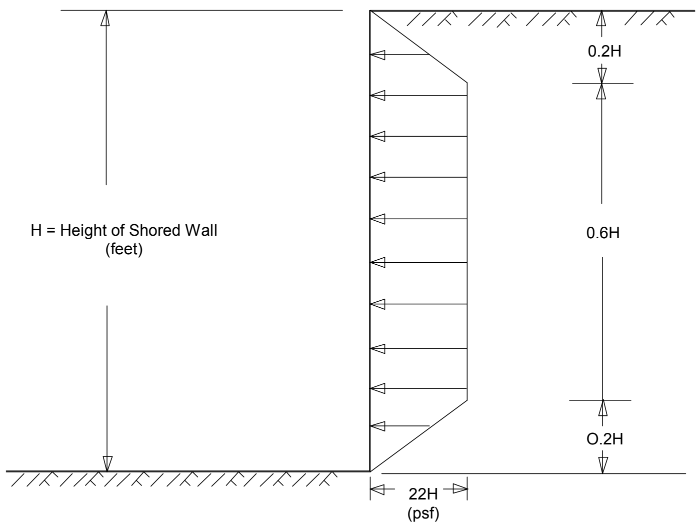

Diagram 6 – Earth Pressure Distribution for Tie-back or Braced Shoring Wall

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 250 psf vertical uniform surcharge is recommended to account for nominal construction and/or traffic loads. More detailed lateral pressure and loading information can be provided, if needed, for specific loading scenarios as recognized through the design process.

6.8.2. Soldier Pile Design

The soldier piles should be designed in accordance with the geotechnical parameters presented in Table 4. 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 4 - 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 450 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.

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

Drilling of the soldier pile shafts can be accomplished using conventional drilling equipment. However, it is possible that rock-coring equipment may be necessary to advance drill holes through the cobbles and boulders present below this site. Additionally, caving should be anticipated within the upper approximately 40 feet where layers of loose to medium dense clean sand with gravel and cobble were encountered during our drilling program. In the event of soil caving, it may be necessary to use casing and/or drilling mud to permit the installation of the soldier piles. Drilled holes for soldier piles should not be left open overnight. Concrete for piles should be placed immediately after the drilling of the hole is complete. The concrete should be pumped to the bottom of the drilled shaft using a tremie. Once concrete pumping is initiated, the bottom of the tremie should remain below the surface of the concrete to prevent contamination of the concrete by soil inclusions. If steel casing is used, the casing should be removed as the concrete is placed. The contractor should consider the use of driven piles or piles that are vibrated into place in lieu of drilled piles to address potential issues related to caving of drilled shafts.

6.8.3. Tie-Back Earth Anchor Design

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

The capacity of the anchors should be evaluated by testing of initial anchors installed. For preliminary design purposes, conventional drilled anchors (gravity grouted) may be designed for an allowable bond stress of 50 psf for every foot of overburden above the tie-back anchor. Only the resistance developed beyond the failure wedge should be used in resisting lateral loads. If the anchors are spaced at least 6 feet on center, no reduction in the capacity of the anchors need be considered due to group action.

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

6.8.4. Anchor Testing

All of the production anchors should be tested to at least 150% of the design load; the total deflection during the tests should not exceed 12 inches. The rate of creep under the 150% test should not exceed 0.1 inch over a 15 minute period for the anchor to be approved for the design loading.

After a satisfactory test, each production anchor should be locked-off at the design load. The locked-off load should be verified by rechecking the load in the anchor. If the locked-off load varies by more than 10% from the design load, the load should be reset until the anchor is locked-off within 10% of the design load.

To reduce chances of caving during tie-back testing, the portion of the anchor shafts within the failure wedge may need to be backfilled with sand before testing the anchor. This portion of the shaft should be filled tightly and be flush with the face of the excavation. The sand backfill may contain a small amount of cement to allow the sand to be placed by pumping.

6.8.5. Anchor Installation

The anchors may be installed at angles of 15 to 30 degrees below the horizontal. Although we did not encounter caving during drilling for our subsurface investigation, we anticipate that caving may occur during the drilling of tiebacks. The contractor should implement appropriate measures to stabilize the drilled hole such as the installation of steel casing for the loose cohesionless materials or the use of drilling mud. The anchors should be filled with concrete placed by pumping from the tip out. The portion of the anchor tendons within the failure wedge should be sleeved in plastic. If the anchor tendons are sleeved, it is acceptable to grout the entire length of the anchor.

6.8.6. Shoring Deflection

In general, deflection at the top of the shoring is difficult to accurately predict. However, some deflection will occur. For properly designed shoring system using the design parameters provided in this report, we anticipate deflection of the shoring to be on the order of 1 inch. To reduce the anticipated deflection, the recommended shoring design parameters can be increased and additional bracing of the excavation may be required.

6.8.7. Monitoring

Due to the proximity of the excavation to existing improvements, some means of monitoring the performance of the shoring system is recommended. Monitoring should consist of periodic surveying of lateral and vertical locations at the tops of all soldier piles. We will be pleased to discuss this further with the design consultants and the contractor when the design of the shoring system has been finalized. Also, we should review the shoring plans and calculations to evaluate whether our recommendations have been incorporated into the design.

6.9. Corrosive Soil

Laboratory testing was performed on two representative samples 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.

Based on County of Los Angeles (2013) criteria, the soil is considered corrosive when having 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.

6.9.1. Reinforced Concrete

Laboratory tests indicate that the onsite soils within the upper 5 feet are classified as having a “Moderate” sulfate exposure and “S1” sulfate exposure category per ACI 318-14, Table 19.3.1.1. On this basis, for structural features to be in direct contact within the upper 5 feet of the onsite soils, restrictions on the type of Portland cement, water to cement ratio, and the concrete compressive strength should be followed per Table 19.3.2.1.

Laboratory test results were performed on a sample at 30 feet to evaluate the potential of sulfate attack on concrete. The test result indicates that sulfate attack from onsite soils at a depth of 30 feet below the existing grade is negligible.

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

6.9.2. Metallic

Laboratory resistivity testing indicates that the on-site soils are mildly corrosive to buried ferrous metals. However, a corrosion specialist may be consulted regarding suitable types of piping and appropriate protection for underground metal conduits.

6.10. 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 assumed an R-value of 50 for the subgrade material for asphalt pavement structural calculations with assumed TI since no traffic study data is available to us. On this basis, Table 5 provides recommended minimum thicknesses for hot mix asphalt (HMA) and aggregate base sections for different traffic indices.

Table 5 – Recommended Minimum HMA and Base Section Thicknesses

Location	Light Vehicular Parking	Drive Aisle	Firelane
Traffic Index	**5.0**	**6.0**	**7.0**
HMA Thickness (in)	3.5	3.5	4.0
Aggregate Base Thickness (in)	5.0	6.0	7.0

6.11. Stormwater Quality Control Measures Recommendations

We performed field infiltration testing at depths between 30 and 50 feet below existing grades. Based on the percolation results presented in Appendix C, infiltration is acceptable within the proposed infiltration zone of 30 to 50 feet below the existing ground surface. However, we note that the groundwater elevation is potentially as high as 57 feet below the existing ground surface, which would require the termination of the bottoms of infiltration dry wells at a depth of 47 feet below the existing ground surface in order to meet the required 10-foot separation between groundwater elevation and elevation of bottom of dry well. The proposed infiltration BMP must comply with the minimum setback requirements presented in City of Los Angeles Information Bulletin/Public-Building Code 2014-118 (P/BC 2017-118).

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 of excavations 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 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. The substrata exposed during the construction may differ from that encountered in the test excavations. Continuous observation by a representative of Twining during construction allows for evaluation of the soil conditions as they are encountered, and allows the opportunity to recommend appropriate revisions where necessary.

The project engineer should be notified prior to exposure of subgrades. It is critically important that the engineer be provided with an opportunity to observe all exposed subgrades prior to burial or covering.

8. LIMITATIONS

The recommendations and opinions expressed in this report are based on information obtained from our field exploration for the site. In the event that any of our recommendations conflict with recommendations provided by other design professionals, we should be contacted to aid in resolving the discrepancy.

Due to the limited nature of our field explorations, conditions not observed and described in this report may be present on the site. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation and laboratory testing can be performed upon request. It should be understood that conditions different from those anticipated in this report may be encountered during excavation operations, for example, the presence of unsuitable soil, and that additional effort may be required to address 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 has no control.

Twining's recommendations for this site are, to a high degree, dependent upon appropriate quality control of foundation construction. Accordingly, the recommendations are made contingent upon the opportunity for Twining to observe 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 and the engineering geologist 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 design and construction of the project described herein. Any party other than the client who wishes to use this report for an adjacent or nearby project, shall notify Twining of such intended use. 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 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 any other party will release Twining from any liability resulting from the use of this report by any unauthorized party.

Twining has endeavored to perform 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 expressed or implied, is made as to the conclusions and recommendations contained in this report.

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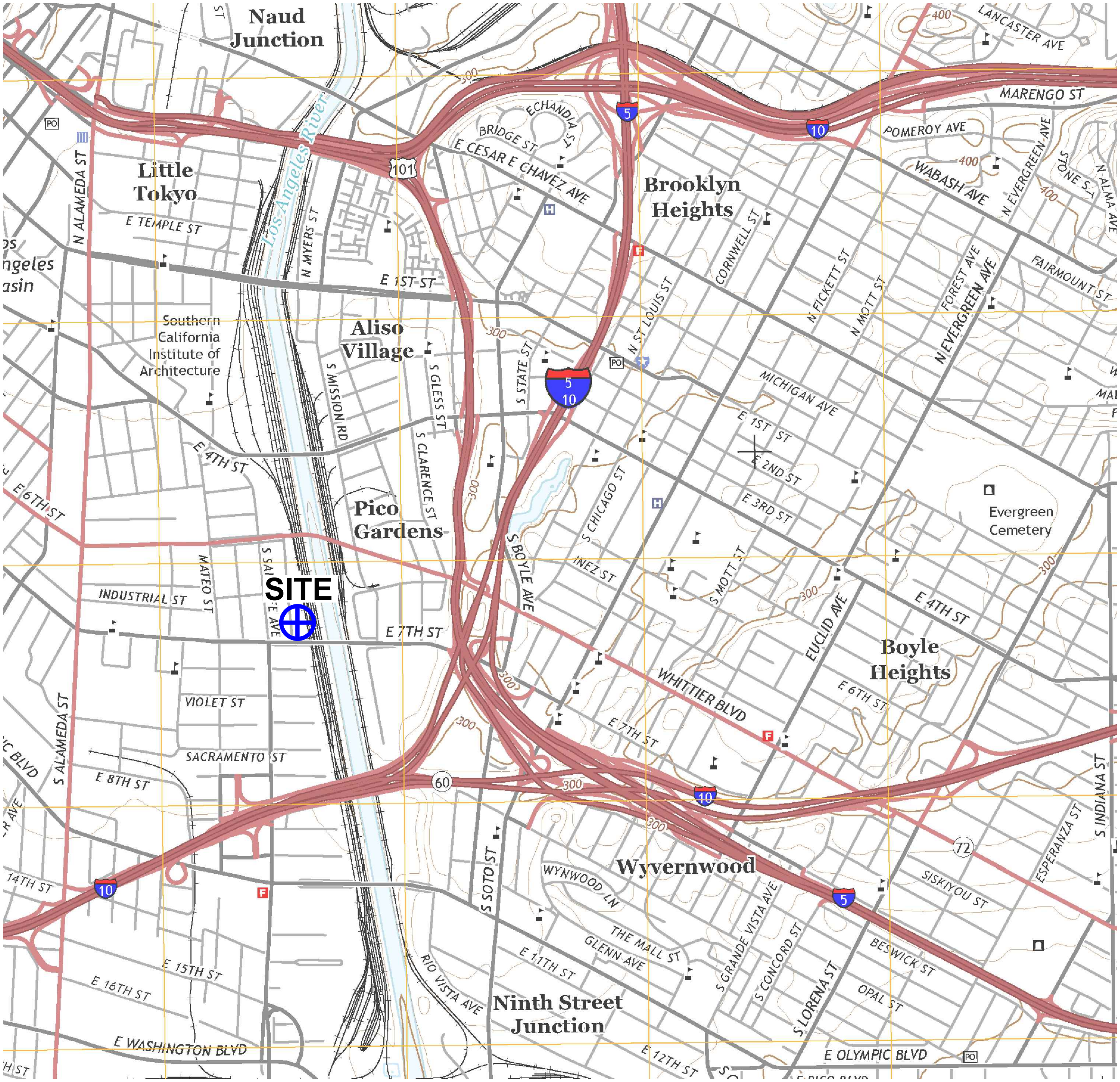
FIGURES



SITE COORDINATES
Latitude: 34.0350°N
Longitude: 118.2287°W

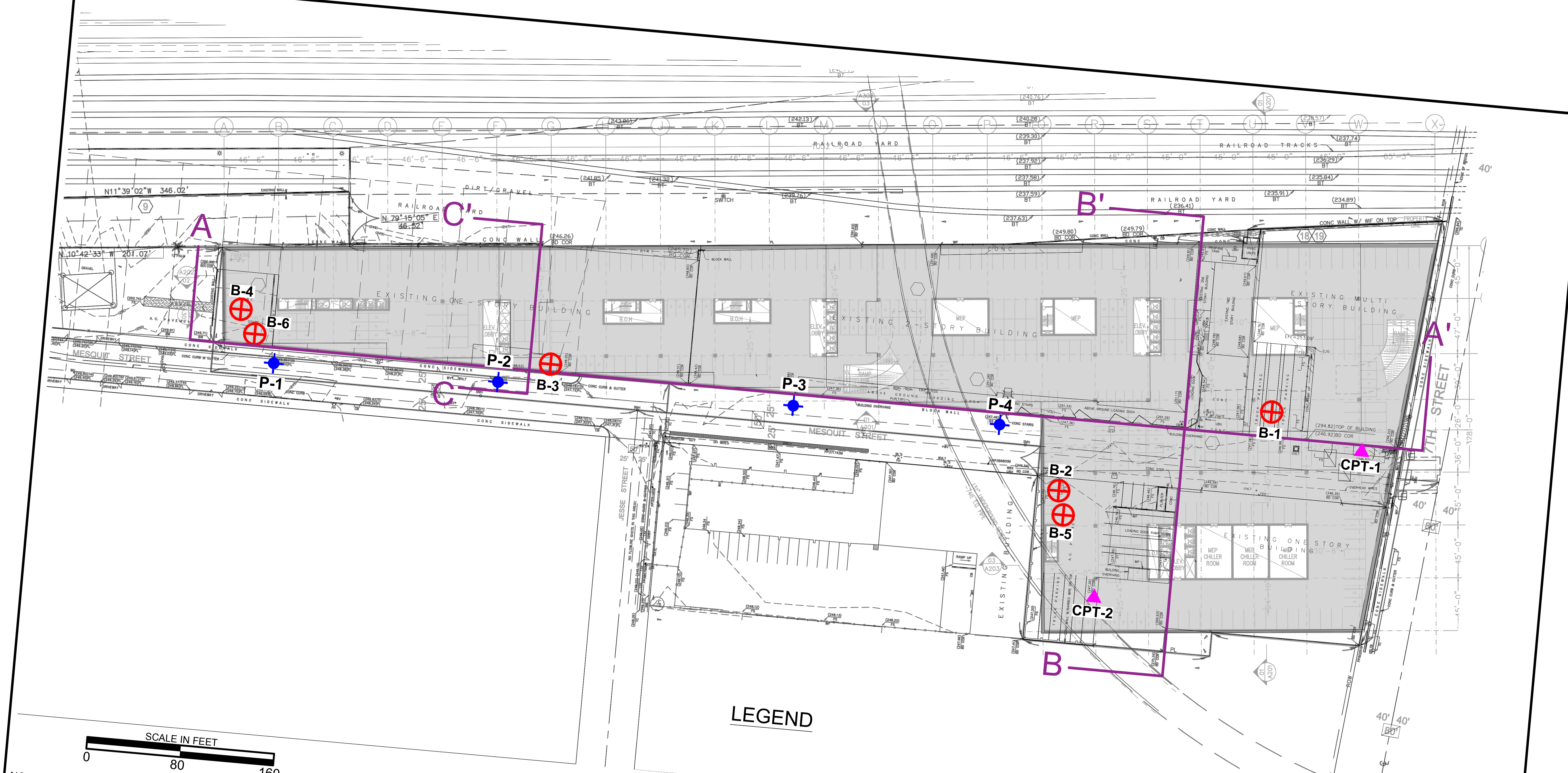
N

SCALE IN FEET
0 2,000 4,000

REFERENCE: USGS (2015)

TWINING CONSULTING

SITE LOCATION MAP		
670 Mesquit Mixed-Use Development 658 & 670 Mesquit Street Los Angeles, California		
PROJECT NO. 160599.1	REPORT DATE October 2018	FIGURE 1



SCALE IN FEET

0 80 160

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

TWINING CONSULTING

REFERENCE: KPFF (2018)

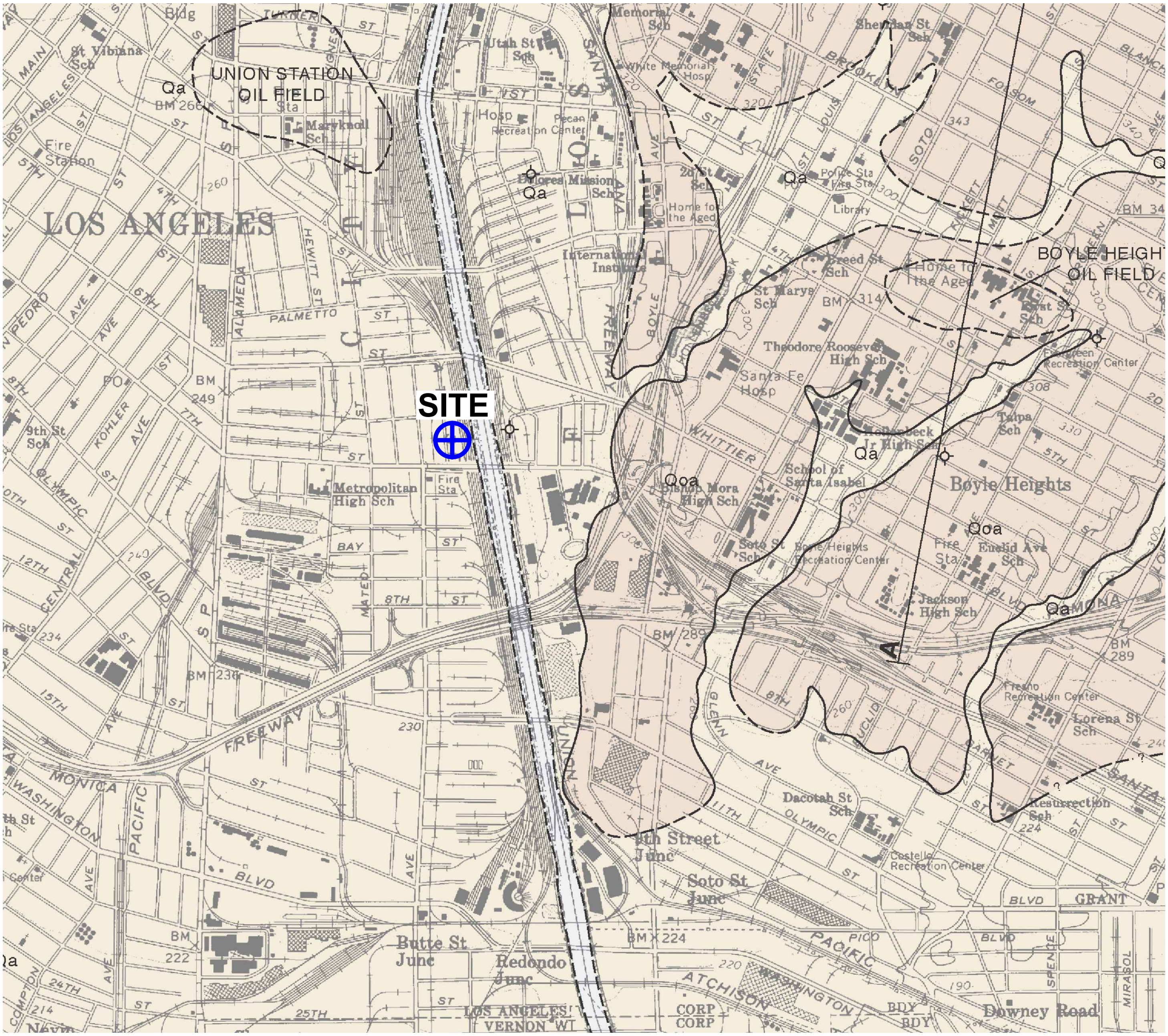
LEGEND

B-1	APPROXIMATE LOCATION OF SOIL BORING
P-1	APPROXIMATE LOCATION OF PERCOLATION TEST
CPT-1	APPROXIMATE LOCATION OF CPT
A A'	CROSS SECTION LOCATION

SITE PLAN AND EXPLORATION LOCATION MAP

670 Mesquit Mixed-Use Development
658 & 670 Mesquit Street
Los Angeles, California

PROJECT NO.	REPORT DATE	
160599.1	October 2018	FIGURE 2



LEGEND

af
Qg
Qa

SURFICIAL SEDIMENTS

af *Artificial fill*
Qg *Alluvial clay and sand of valley areas*
Qa *Alluvium: unconsolidated floodplain deposits of silt, sand and gravel*

Qoa Qof Qog

OLDER SURFICIAL SEDIMENTS

Qoa *Remnants of older weakly consolidated alluvial deposits of gravel, sand and silt*
Qof *Alluvial fan gravel and sand*
Qog *Elevated remnants of older alluvial gravel and fanglomerate deposits, weakly indurated*

N

SCALE IN FEET

0 2,000 4,000

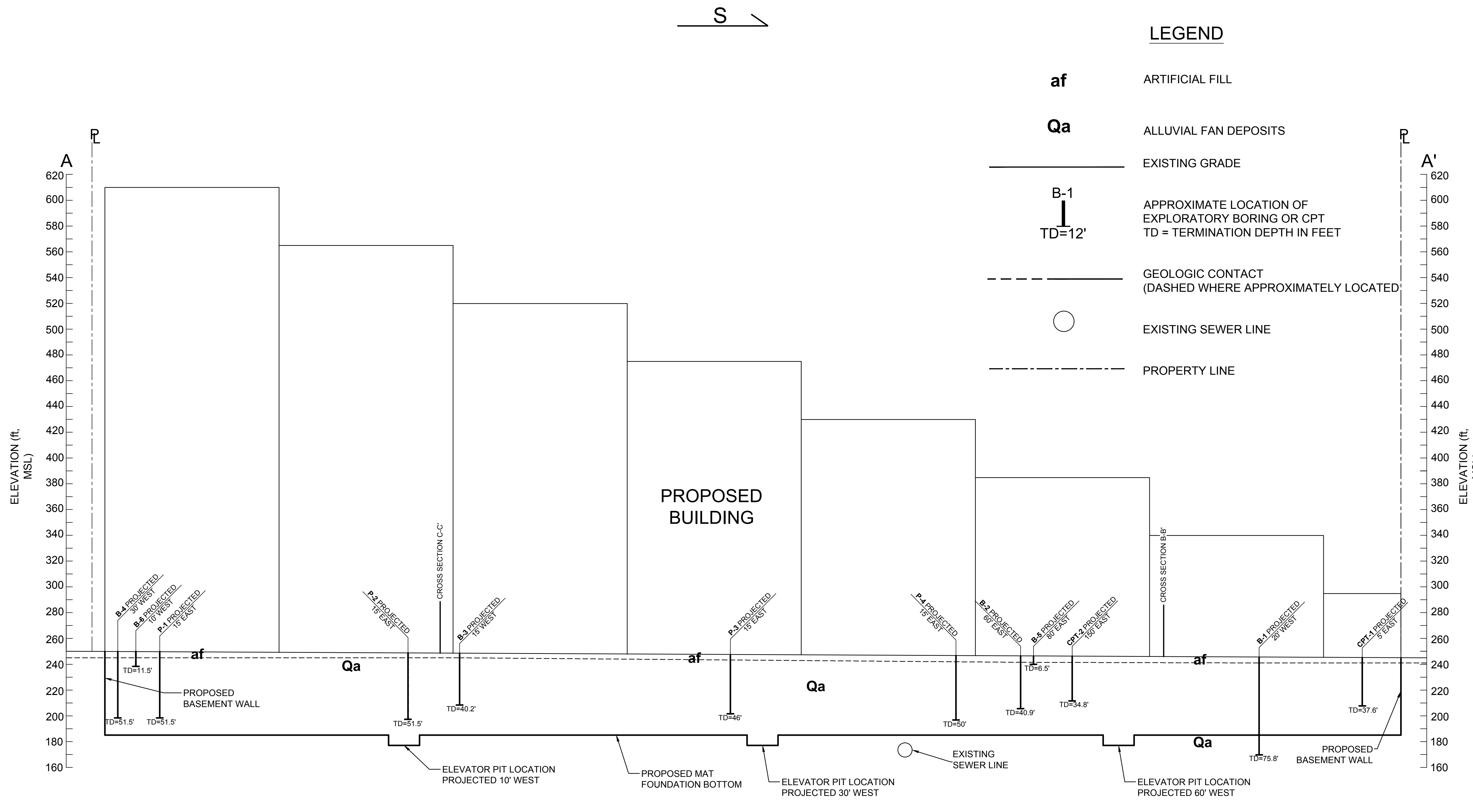
REFERENCE: DIBBLEE (1989)

TWINING CONSULTING

REGIONAL GEOLOGIC MAP

670 Mesquit Mixed-Use Development
658 & 670 Mesquit Street
Los Angeles, California

PROJECT NO.	REPORT DATE	FIGURE 3
160599.1	October 2018	



CROSS SECTION A-A'

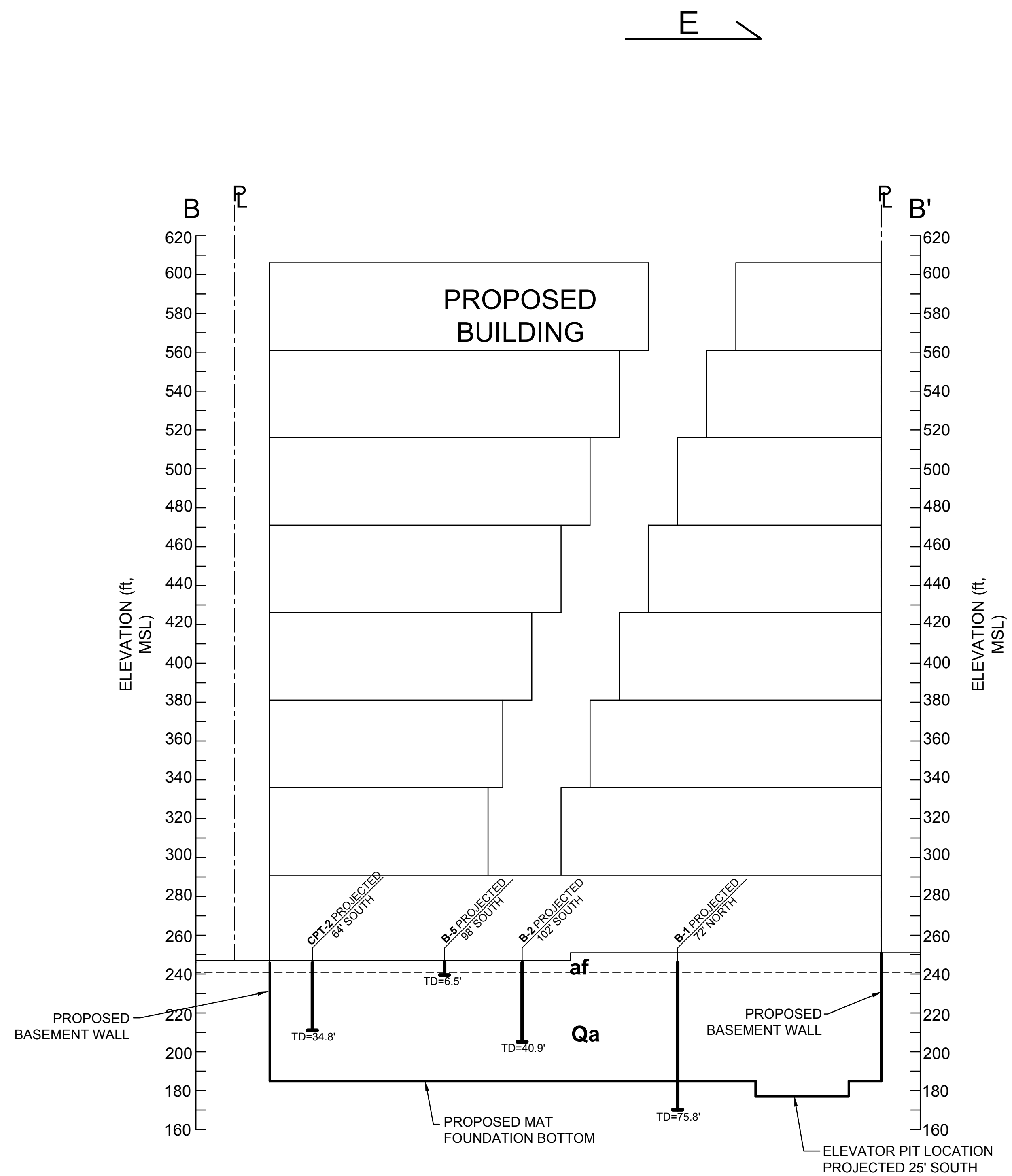
NOTE: All dimensions, locations, and directions are approximate.

TWINING CONSULTING

GEOLOGIC CROSS SECTION A-A'

670 Mesquit Mixed-Use Development
658 & 670 Mesquit Street
Los Angeles, California

PROJECT NO.	REPORT DATE	FIGURE 4A
160599.1	October 2018	



LEGEND

af	ARTIFICIAL FILL
Qa	ALLUVIAL FAN DEPOSITS
———	EXISTING GRADE
B-1 / TD=12'	APPROXIMATE LOCATION OF EXPLORATORY BORING OR CPT TD = TERMINATION DEPTH IN FEET
— — — ———	GEOLOGIC CONTACT (DASHED WHERE APPROXIMATELY LOCATED)
—·—·—·—	PROPERTY LINE

SCALE IN FEET
0 80 160

CROSS SECTION B-B'

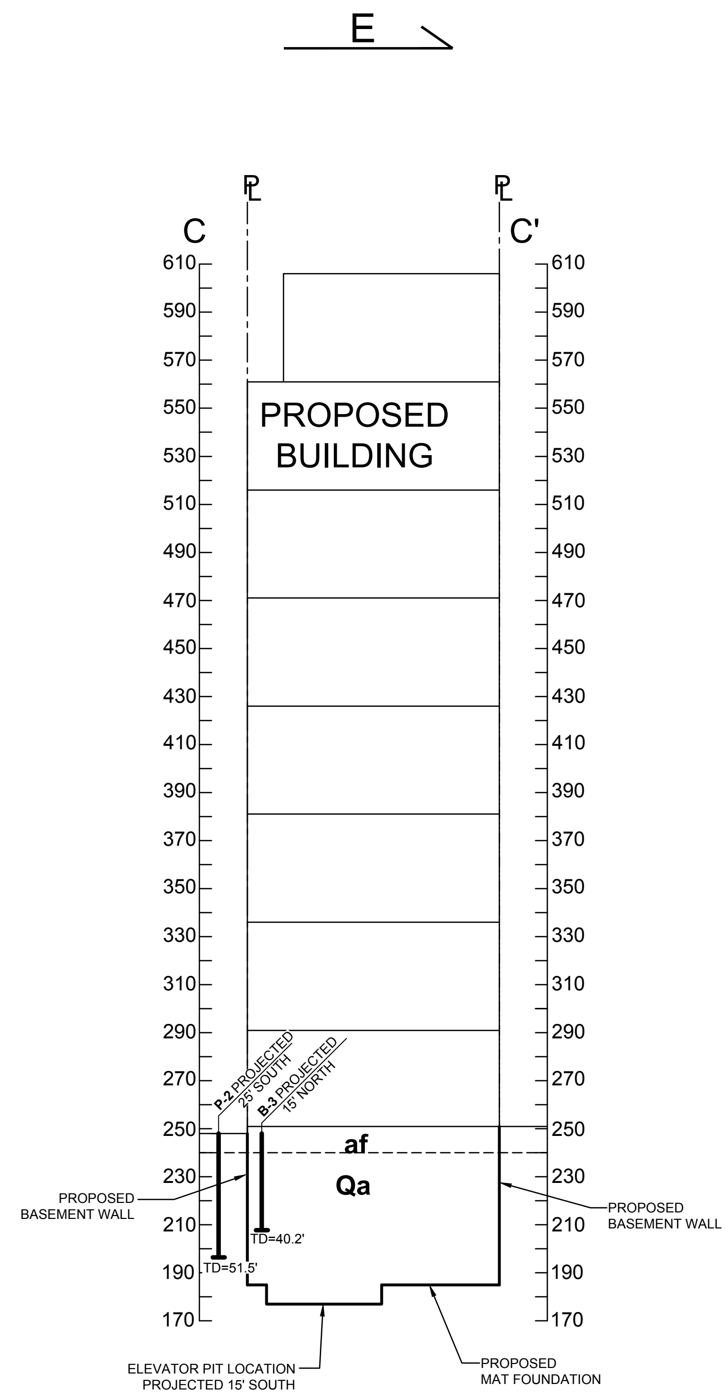
NOTE: All dimensions, locations, and directions are approximate.

TWINING CONSULTING

GEOLOGIC CROSS SECTION B-B'

670 Mesquit Mixed-Use Development
658 & 670 Mesquit Street
Los Angeles, California

PROJECT NO.	REPORT DATE	FIGURE 4B
160599.1	October 2018	



CROSS SECTION C-C'

LEGEND

af	ARTIFICIAL FILL
Qa	ALLUVIAL FAN DEPOSITS
	EXISTING GRADE
B-1 TD=12'	APPROXIMATE LOCATION OF EXPLORATORY BORING OR CPT TD = TERMINATION DEPTH IN FEET
	GEOLOGIC CONTACT (DASHED WHERE APPROXIMATELY LOCATED)
	PROPERTY LINE

SCALE IN FEET
0 80 160

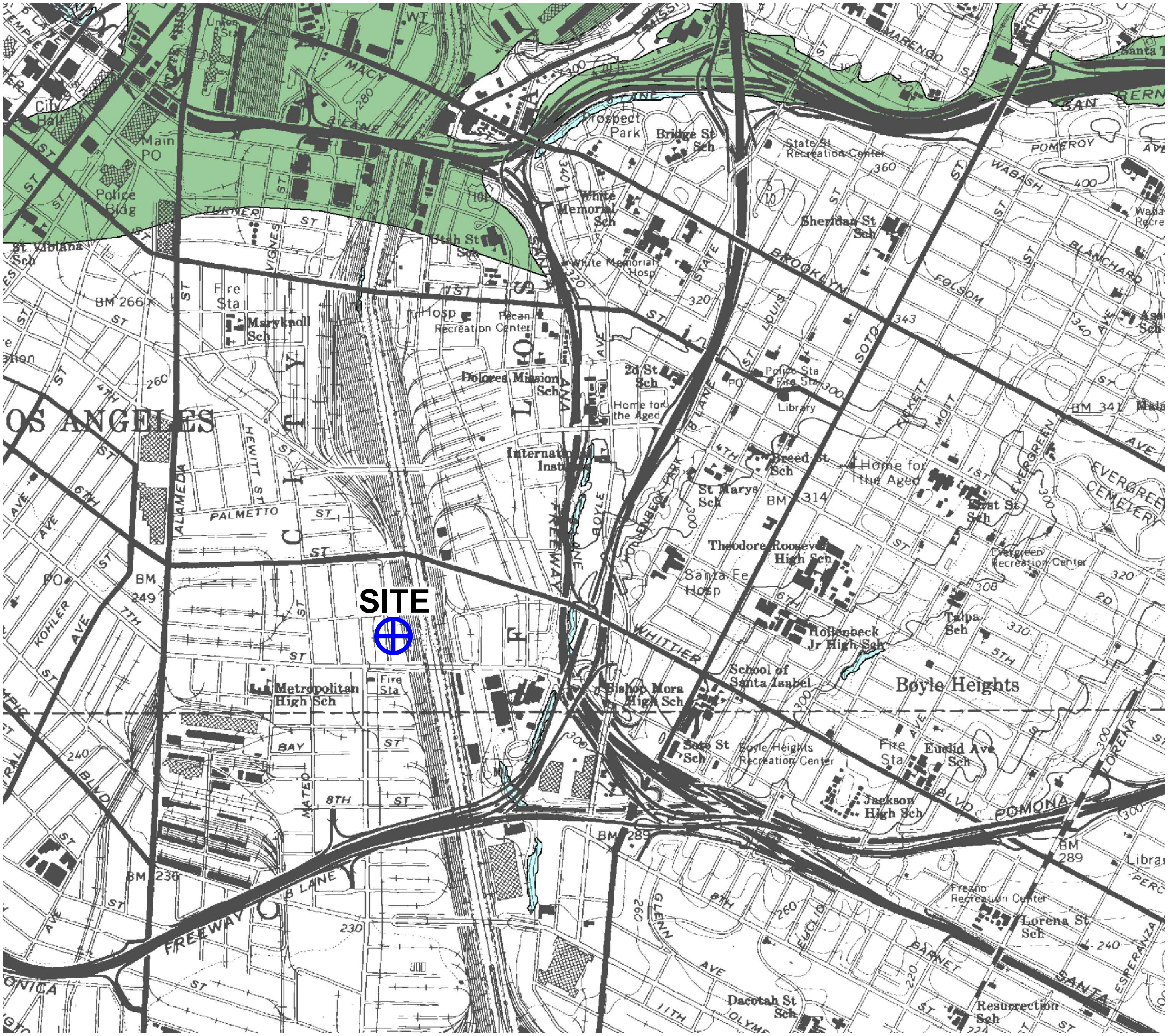
NOTE: All dimensions, locations, and directions are approximate.

TWINING CONSULTING

GEOLOGIC CROSS SECTION C-C'

670 Mesquit Mixed-Use Development
658 & 670 Mesquit Street
Los Angeles, California

PROJECT NO.	REPORT DATE	FIGURE 4C
160599.1	October 2018	



LEGEND

Liquefaction

Areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater 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 Landslides

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

SCALE IN FEET

0 2,000 4,000

REFERENCE: CALIFORNIA DEPARTMENT OF CONSERVATION, DIVISION OF MINES AND GEOLOGY (1999)

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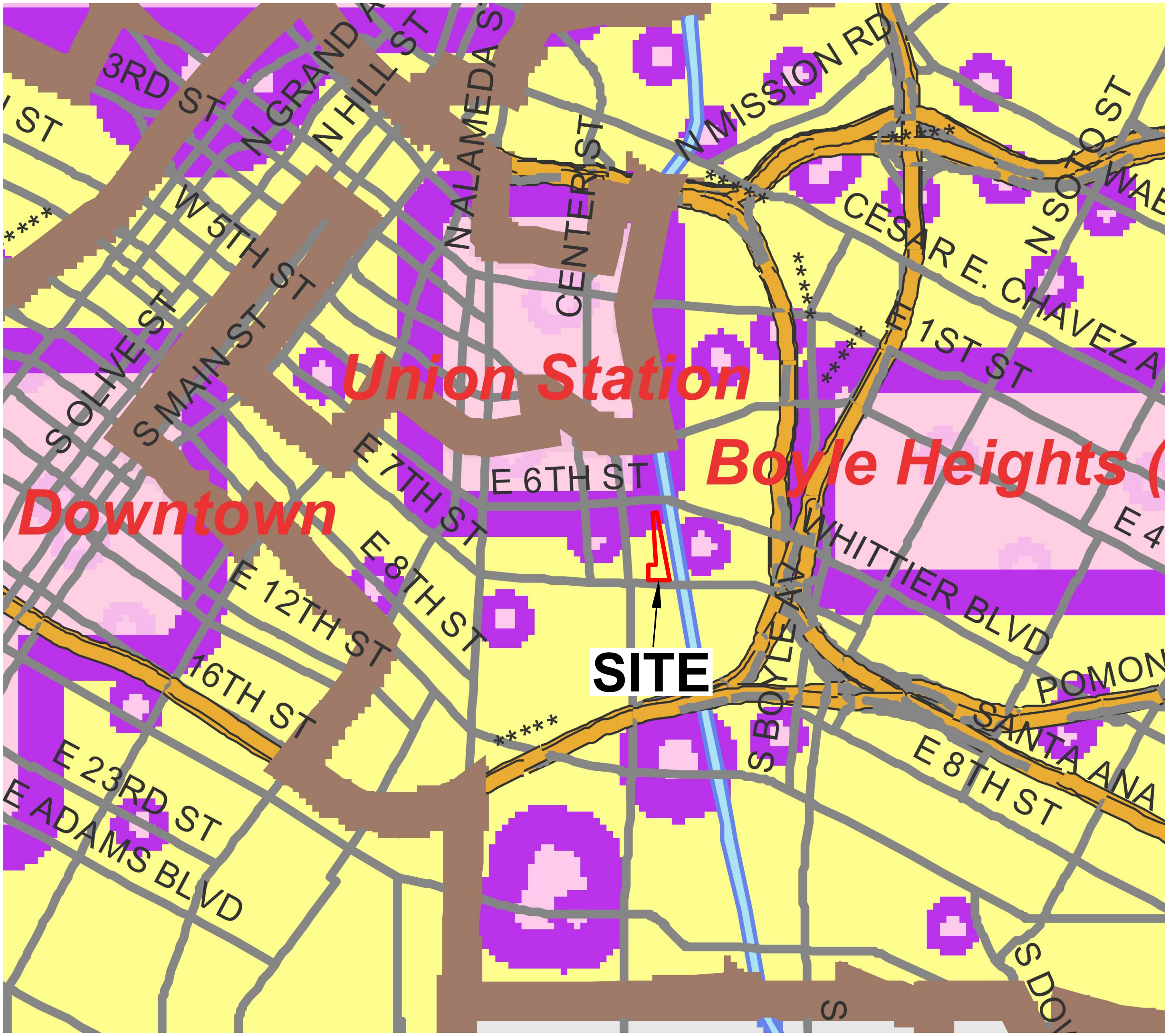
SEISMIC HAZARD ZONES MAP

670 Mesquit Mixed-Use Development
658 & 670 Mesquit Street
Los Angeles, California

PROJECT NO.	REPORT DATE	FIGURE 5
160599.1	October 2018	



LEGEND

- Methane Zone
- Methane Buffer Zone
- Council District Boundary

N

SCALE IN MILES
0 1/2 1

REFERENCE: CITY OF LOS ANGELES, BUREAU OF ENGINEERING (2002)

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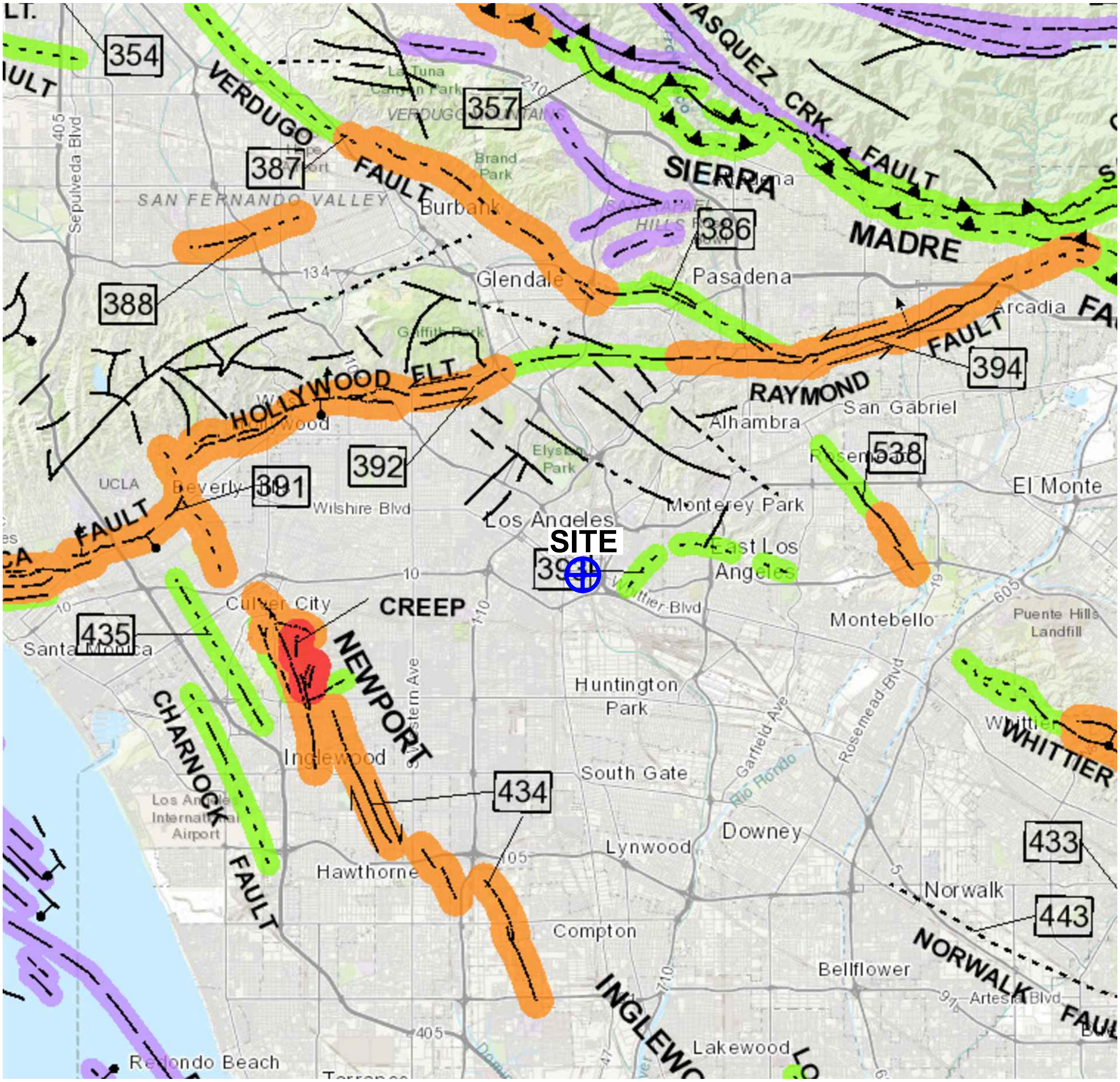
METHANE ZONE RISK MAP

670 Mesquit Mixed-Use Development
658 & 670 Mesquit Street
Los Angeles, California

PROJECT NO.	REPORT DATE	FIGURE 6
160599.1	October 2018	



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 4 8

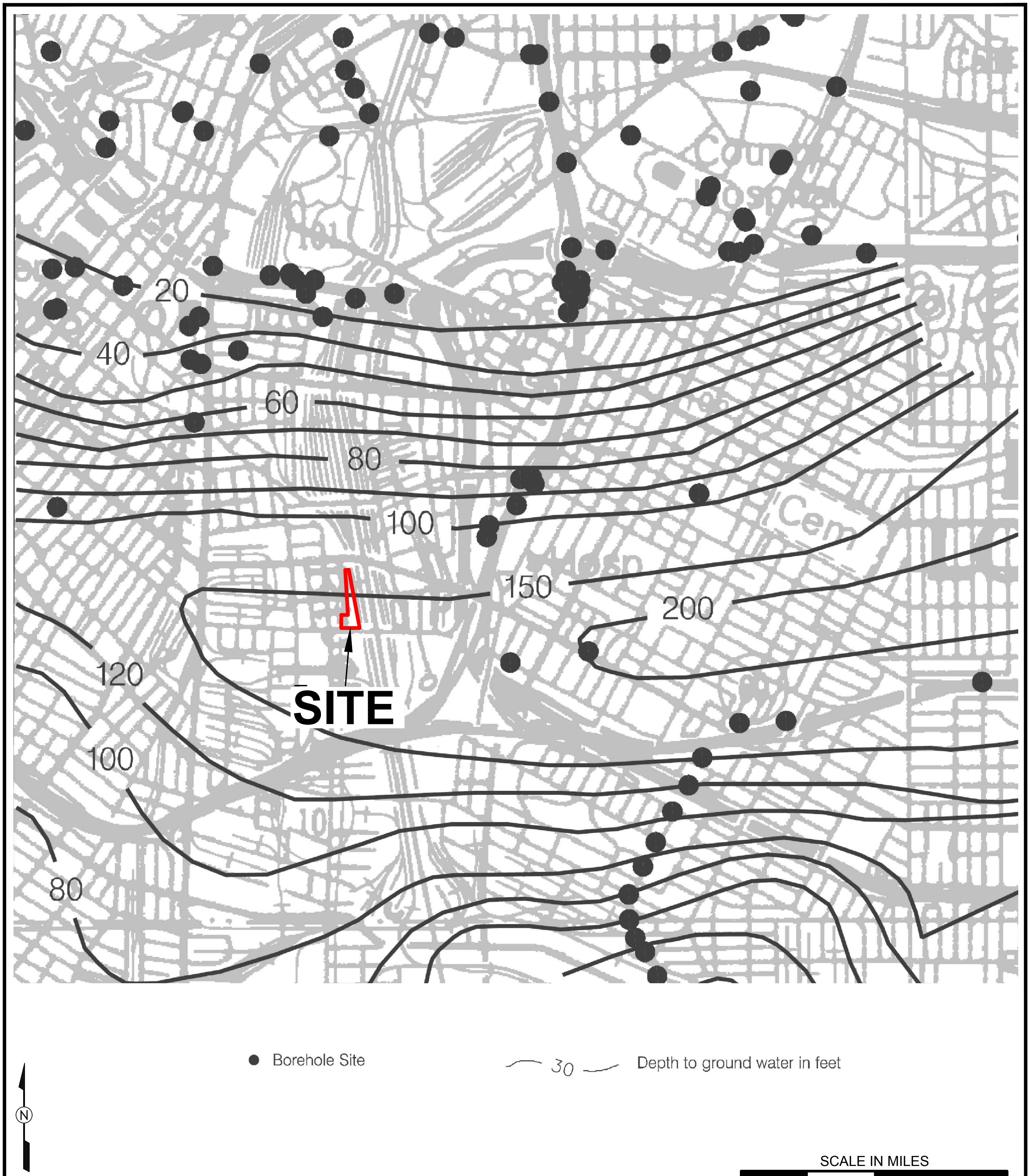
REFERENCE: JENNINGS AND BRYANT (2010)

TWINING CONSULTING

REGIONAL FAULT LOCATION MAP

670 Mesquit Mixed-Use Development
658 & 670 Mesquit Street
Los Angeles, California

PROJECT NO.	REPORT DATE	FIGURE 7
160599.1	October 2018	



REFERENCE: CALIFORNIA GEOLOGIC SURVEY (1998)

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HISTORICAL HIGH GROUNDWATER MAP

670 Mesquit Mixed-Use Development
658 & 670 Mesquit Street
Los Angeles, California

PROJECT NO.	REPORT DATE	FIGURE 8
160599.1	October 2018	

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APPENDIX A
FIELD EXPLORATION

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Appendix A
Field Exploration

General

The subsurface exploration program for the proposed project consisted of advancing a total of eight 8-inch-diameter, hollow-stem-auger borings and two Cone Penetration Testing (CPT) soundings at various locations across the subject site on August 6 and 13, 2016, and February 2, 2018. The borings were advanced to depths ranging from 40 to 75.8 feet below the existing grade, and the CPTs were advanced to a maximum depth of 37 feet below the existing grade. Two borings were advanced to depths of 5 and 10 feet below existing grade and were used for percolation testing. The drilling operation was performed by Gregg Drilling of Signal Hill, California.

Drilling and Sampling

The Boring Logs are presented as Figures A-2 through A-10 and CPT graphs are also presented in Appendix A. 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, drilling date, and the name of the logger and drilling subcontractor. The borings were logged by an engineer using the Unified Soil Classification System. The boundaries between soil types shown on the logs are approximate because the transition between different soil layers may be gradual. Drive and bulk samples of representative earth materials were obtained from the borings.

Disturbed samples were obtained using a Standard Penetration Sampler (SPT). This sampler consists of a 2-inch O.D., 1.4-inch I.D. split barrel shaft that is advanced into the soil at the bottom of the drilled hole a total of 18 inches. The number of blows required to drive the sampler the final 12 inches is presented on the boring logs. Soil samples obtained by the SPT were retained in plastic bags.

A California modified sampler was used to obtain drive samples of the soil encountered. This sampler consists of a 3-inch outside diameter (O.D.), 2.4-inch inside diameter (I.D.) split barrel shaft that was driven a total of 12-inches into the soil at the bottom of the boring by a safety hammer weighing 140 pounds at a drop height of approximately 30 inches. The soil was retained in brass rings for laboratory testing. Additional soil from each drive remaining in the cutting shoe was usually discarded after visually classifying the soil. The number of blows required to drive the sampler the final 12 inches is presented on the boring logs.

Upon completion of the borings, the boreholes were backfilled with soil from the cuttings and patched with rapid-set concrete where needed.

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

670 Mesquit Mixed-Use Development
658 & 670 Mesquit Street
Los Angeles, California

PROJECT NO.	REPORT DATE	FIGURE A-1
160599.1	April 2018	

STANDARD LOG EXPLANATION 160599.1 - 670 MESQUIT GINT.GPJ TWINING LABS.GDT 4/23/18

DATE DRILLED	8/6/2016	LOGGED BY	DH	**BORING NO.**	**B-1**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	75
DRILLING METHOD	8" HSA	DRILLER	Gregg Drilling	SURFACE ELEVATION (ft.)	247 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
									Concrete slab (approximately 12 inches)
		Bulk				#200 WASH		SM	FILL: Silty SAND, loose, light brown, moist; loose, fine to medium grained sand, some gravel, pieces of bricks
242	5	Driven	8						
								SP-SM	ALLUVIUM: Poorly graded SAND with silt, light brown, moist; fine to coarse grained sand
237	10	Driven	7						-- loose
232	15	Driven	23	2.5	110.0				-- medium dense
227	20	Driven	40			#200 WASH			-- dense, some gravel
222	25	Driven	56						
217	30	Driven	28			CORR			
212	35								

BORING LOG 160599.1 - 670 MESQUIT GINT.GPJ TWINING LABS.GDT 10/15/18

TWINING

LOG OF BORING

670 Mesquit Mixed-Use Development
658 & 670 Mesquit Street
Los Angeles, California

PROJECT NO.	REPORT DATE	
160599.1	October 2018	FIGURE A - 2

DATE DRILLED	8/6/2016	LOGGED BY	DH	**BORING NO.**	**B-1**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	75
DRILLING METHOD	8" HSA	DRILLER	Gregg Drilling	SURFACE ELEVATION (ft.)	247 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
		Driven	55	4.0	104.2	DS		SP-SM	ALLUVIUM: Poorly graded SAND with silt, light brown, moist; fine to coarse grained sand *(continued)*
207	40	Driven	50/3"						-- dense to very dense
								GP	Poorly graded gravelly SAND, light brown, moist; fine- to- coarse-grained sand
202	45	Driven	50/1"						
								SP-SM	Poorly graded SAND with silt, gray brown, moist; very dense, some gravel, medium to coarse grained sand
197	50	Driven	50/2"					GP	Poorly graded gravelly SAND, gray brown, moist; very dense, fine- to- coarse-grained sand
192	55	Driven	77	9.5	110.5			SP-SM	Poorly graded SAND with silt, gray brown, moist; very dense, some gravel, medium to coarse grained sand
187	60	Driven	57						
								GP	Poorly graded gravelly SAND, gray brown, moist; very dense, fine- to- coarse-grained sand
182	65	Driven	71					SP-SM	Poorly graded SAND with silt, light brown, moist; very dense, some gravel, medium to coarse grained sand
177	70								

TWINING

LOG OF BORING

670 Mesquit Mixed-Use Development
658 & 670 Mesquit Street
Los Angeles, California

PROJECT NO.	REPORT DATE	
160599.1	October 2018	FIGURE A - 2

DATE DRILLED	8/6/2016	LOGGED BY	DH	**BORING NO.**	**B-1**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	75
DRILLING METHOD	8" HSA	DRILLER	Gregg Drilling	SURFACE ELEVATION (ft.)	247 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
				65					SP-SM	Poorly graded SAND with silt, light brown, moist; very dense, some gravel, medium to coarse grained sand *(continued)*
172	75			50/4"						-- water seepage at 75 feet
										Total Depth = 75.8 feet Backfilled on 8/6/2016 Backfilled with soil from cuttings Groundwater possible at approximate depth of 75 feet.
167	80									
162	85									
157	90									
152	95									
147	100									
142	105									

TWINING

LOG OF BORING

670 Mesquit Mixed-Use Development
658 & 670 Mesquit Street
Los Angeles, California

PROJECT NO.	REPORT DATE	
160599.1	October 2018	FIGURE A - 2

DATE DRILLED	8/6/2016	LOGGED BY	DH	**BORING NO.**	**B-2**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	8" HSA	DRILLER	Gregg Drilling	SURFACE ELEVATION (ft.)	247 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
	0								Asphalt Concrete: 7 inches
								SM	FILL: Silty SAND with some gravel, loose, light brown, moist, fine grained sand
242	5	Bulk / Driven	8			#200 WASH		SP-SM	ALLUVIUM: Poorly graded SAND, tan to light brown, moist; loose, fine- to coarse-grained sand -- fine grained sand
237	10	Driven	17						-- medium dense, light tan, dry, medium- to coarse-grained sand
232	15	Driven	21						-- moist, some gravel, medium- to coarse-grained sand
227	20	Driven	72	2.6	116.5	DS			-- very dense, few gravel and cobbles, medium- to-coarse-grained sand
222	25	Driven	32			#200 WASH			-- dense, little gravel
217	30	Driven	85	1.6	123.5				-- some gravel/cobbles, medium- to coarse-grained sand
								GP	Poorly graded gravelly SAND, light brown, moist; very dense, medium- to coarse-grained sand
212	35								

TWINING

LOG OF BORING

670 Mesquit Mixed-Use Development
658 & 670 Mesquit Street
Los Angeles, California

PROJECT NO.	REPORT DATE	
160599.1	October 2018	FIGURE A - 3

DATE DRILLED	8/6/2016	LOGGED BY	DH	**BORING NO.**	**B-2**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	8" HSA	DRILLER	Gregg Drilling	SURFACE ELEVATION (ft.)	247 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
				49					GP	Poorly graded gravelly SAND, light brown, moist; very dense, medium- to coarse-grained sand *(continued)* -- fine to medium grained sand
207	40			50/5"						
										Total Depth = 40.9 feet Backfilled on 8/6/2016 Backfilled with soil from cuttings Groundwater not encountered
202	45									
197	50									
192	55									
187	60									
182	65									
177	70									

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LOG OF BORING

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658 & 670 Mesquit Street
Los Angeles, California

PROJECT NO.	REPORT DATE	
160599.1	October 2018	FIGURE A - 3

DATE DRILLED	8/13/16	LOGGED BY	RA	**BORING NO.**	**B-3**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	8" HSA	DRILLER	Gregg Drilling	SURFACE ELEVATION (ft.)	250 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk / Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
	0								Concrete slab (approximately 8 inches)
		Bulk				RV		SM	FILL: Silty SAND, brown, fine grained, moist; loose
245	5	Driven	9	8.3	102.2	DS		SP-SM	ALLUVIUM: Poorly graded SAND with silt, light brown, moist; loose
240	10	Driven	15			#200 WASH			
235	15	Driven	18						-- medium dense, some gravel encountered
230	20	Driven	17						
225	25	Driven	37	3.7	103.2				
220	30	Driven	71			#200 WASH			
								GP	Poorly graded gravelly SAND, light brown, moist; very dense
215	35								

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LOG OF BORING

670 Mesquit Mixed-Use Development
658 & 670 Mesquit Street
Los Angeles, California

PROJECT NO.	REPORT DATE	
160599.1	October 2018	FIGURE A - 4

DATE DRILLED	8/13/16	LOGGED BY	RA	**BORING NO.**	**B-3**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	8" HSA	DRILLER	Gregg Drilling	SURFACE ELEVATION (ft.)	250 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
			X	80/11"	3.0	111.9	CONSOL		GP	Poorly graded gravelly SAND, light brown, moist; very dense *(continued)*
210	40			50/2"						
										Total Depth = 40.2 feet Backfilled on 8/13/2016 Backfilled with soil from cuttings Groundwater not encountered
205	45									
200	50									
195	55									
190	60									
185	65									
180	70									

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LOG OF BORING

670 Mesquit Mixed-Use Development
658 & 670 Mesquit Street
Los Angeles, California

PROJECT NO.	REPORT DATE	
160599.1	October 2018	FIGURE A - 4

DATE DRILLED 8/6/2016 | LOGGED BY RA | **BORING NO. B-4**

DRIVE WEIGHT 140 lbs. | DROP 30 inches | DEPTH TO GROUNDWATER (ft.) NE

DRILLING METHOD 8" HSA | DRILLER Gregg Drilling | SURFACE ELEVATION (ft.) 251 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
									Concrete slab (approximately 8 inches)
								SM	FILL: Silty SAND, brown, moist; loose, fine-grained sand
246	5			6					
								SM	ALLUVIUM: Silty SAND, brown, moist; fine- to- coarse-grained sand
241	10			14	2.2	106.3			
								SP-SM	Poorly graded SAND with silt, light brown, moist; loose
236	15			23					-- medium dense, some gravel up to 2 inch in diameter
231	20			52					-- dense
226	25			29					
221	30			65					
216	35								

TWINING

LOG OF BORING

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PROJECT NO.	REPORT DATE	FIGURE A - 5
160599.1	October 2018	

DATE DRILLED	8/6/2016	LOGGED BY	RA	**BORING NO.**	**B-4**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	8" HSA	DRILLER	Gregg Drilling	SURFACE ELEVATION (ft.)	251 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
		Driven	50/3"				GP	Poorly graded gravelly SAND, gray brown, moist; very dense
211	40	Driven	50/4"	3.1	107.2			
206	45	Driven	93/10"					
201	50	Driven	94					
								Total Depth = 51.5 feet Backfilled on 8/13/2016 Backfilled with soil from cuttings Groundwater not encountered
196	55							
191	60							
186	65							
181	70							

BORING LOG 160599.1 - 670 MESQUIT GINT.GPJ TWINING LABS.GDT 10/15/18

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PROJECT NO.	REPORT DATE	
160599.1	October 2018	FIGURE A - 5

DATE DRILLED	8/13/16	LOGGED BY	RA	**BORING NO.**	**B-5**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	8" HSA	DRILLER	Gregg Drilling	SURFACE ELEVATION (ft.)	247 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
	0								SM	Asphalt Concrete (Approximately 4 inches) FILL: Silty SAND, brown, moist; loose, fine-grained sand
242	5			5			#200 WASH			
										Total Depth = 6.5 feet Backfilled on 8/13/2016 Backfilled with soil from cuttings Groundwater not encountered
237	10									
232	15									
227	20									
222	25									
217	30									
212	35									

TWINING

LOG OF BORING

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658 & 670 Mesquit Street
Los Angeles, California

PROJECT NO.	REPORT DATE	
160599.1	October 2018	FIGURE A - 6

DATE DRILLED	8/13/16	LOGGED BY	RA	**BORING NO.**	**B-6**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	8" HSA	DRILLER	Gregg Drilling	SURFACE ELEVATION (ft.)	251 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
										Concrete slab (approximately 8 inches)
									SM	FILL: Silty SAND, brown, moist, fine grained sand
246	5									
									SM	ALLUVIUM: Silty SAND, brown to light brown, moist; fine- to coarse-grained sand
241	10			14			#200 WASH			
									SP-SM	Poorly graded SAND with silt, light brown, moist; medium dense
										Total Depth = 11.5 feet Backfilled on 8/13/2016 Backfilled with soil from cuttings Groundwater not encountered
236	15									
231	20									
226	25									
221	30									
216	35									

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LOG OF BORING

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PROJECT NO.	REPORT DATE	
160599.1	October 2018	FIGURE A - 7

DATE DRILLED	2/3/2018	LOGGED BY	AM	**BORING NO.**	**P-1**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	8" HSA	DRILLER	2R Drilling	SURFACE ELEVATION (ft.)	247 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
	0								Asphalt Concrete: 8 inches
								SM	Silty SAND to Poorly graded SAND with silt; medium brown; moist; with some gravel up to 2"
242	5							SP	Poorly graded SAND; light brown to tan; moist; with some gravel up to 2"
237	10								
232	15								
227	20								
222	25								
217	30		X	28/50 for 6"				SP	Poorly graded SAND with gravel to gravelly SAND; very dense; brown; moist; medium to coarse grained
212	35								

LOG OF BORING

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658 & 670 Mesquit Street
Los Angeles, California

PROJECT NO.	REPORT DATE	
160599.1	October 2018	FIGURE A - 8

DATE DRILLED	2/3/2018	LOGGED BY	AM	**BORING NO.**	**P-1**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	8" HSA	DRILLER	2R Drilling	SURFACE ELEVATION (ft.)	247 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
								SP	Poorly graded SAND; light brown to tan; moist; with some gravel up to 2" *(continued)*
207	40		X	50 for 3"				SP	-- same; sample disturbed
202	45								
197	50			38/50 for 5"					
									Total Depth = 51.5 feet Backfilled on 2/3/2018 Backfilled with soil cuttings. Groundwater not encountered.
192	55								
187	60								
182	65								
177	70								

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658 & 670 Mesquit Street
Los Angeles, California

PROJECT NO.	REPORT DATE	
160599.1	October 2018	FIGURE A - 8

DATE DRILLED	2/3/2018	LOGGED BY	AM	**BORING NO.**	**P-2**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	8" HSA	DRILLER	2R Drilling	SURFACE ELEVATION (ft.)	247 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
	0							SM	Asphalt Concrete: 8 inches Silty SAND to Poorly graded SAND with silt; medium brown; moist; with some gravel up to 2"
242	5							SP	Poorly graded SAND; light brown to tan; moist; with some gravel up to 2"
237	10								
232	15								
227	20								
222	25								
217	30		X					SP	Poorly graded SAND with gravel to gravelly SAND; very dense; brown; moist; medium to coarse grained
212	35								

LOG OF BORING

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658 & 670 Mesquit Street
Los Angeles, California

PROJECT NO.	REPORT DATE	
160599.1	October 2018	FIGURE A - 9

DATE DRILLED	2/3/2018	LOGGED BY	AM	**BORING NO.**	**P-2**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	8" HSA	DRILLER	2R Drilling	SURFACE ELEVATION (ft.)	247 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
								SP	Poorly graded SAND; light brown to tan; moist; with some gravel up to 2" *(continued)*
207	40			28/50 for 5"				SP	-- same; sample disturbed
202	45								
197	50			34				SC	Silty Clayey SAND; dense; bluish gray; moist
									Total Depth = 51.5 feet Backfilled on 2/3/2018 Backfilled with soil cuttings. Groundwater not encountered.
192	55								
187	60								
182	65								
177	70								

TWINING

LOG OF BORING

670 Mesquit Mixed-Use Development
658 & 670 Mesquit Street
Los Angeles, California

PROJECT NO.	REPORT DATE	
160599.1	October 2018	FIGURE A - 9

DATE DRILLED	2/3/2018	LOGGED BY	AM	**BORING NO.**	**P-3**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	8" HSA	DRILLER	2R Drilling	SURFACE ELEVATION (ft.)	247 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
									Asphalt Concrete: 8 inches
								SP	Poorly graded SAND with silt; tan to light orange brown; moist; coarse to medium grained
242	5								
237	10								
232	15								
227	20								
222	25								-- same; with little gravel; 1/2" rounded/subrounded gravel
217	30								-- same; with larger gravel, up to 3"
212	35								

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658 & 670 Mesquit Street
Los Angeles, California

PROJECT NO.	REPORT DATE	
160599.1	October 2018	FIGURE A - 10

DATE DRILLED	2/3/2018	LOGGED BY	AM	**BORING NO.**	**P-3**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	8" HSA	DRILLER	2R Drilling	SURFACE ELEVATION (ft.)	247 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
								SP	Poorly graded SAND with silt; tan to light orange brown; moist; coarse to medium grained *(continued)*
207	40								
202	45								Drill rig throttle switch dead; unable to sample or advance boring.
									Total Depth = 46.0 feet Backfilled on 2/3/2018 Backfilled with soil cuttings. Groundwater not encountered.
197	50								
192	55								
187	60								
182	65								
177	70								

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Los Angeles, California

PROJECT NO.	REPORT DATE	
160599.1	October 2018	FIGURE A - 10

DATE DRILLED	2/3/2018	LOGGED BY	AM	**BORING NO.**	**P-4**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	8" HSA	DRILLER	2R Drilling	SURFACE ELEVATION (ft.)	247 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
									Asphalt Concrete: 3 inches
								SM	Concrete: 5 inches
									Silty SAND; medium dense; medium brown; moist; fine to medium SAND; trace fine GRAVEL
242	5								-- light brown
									-- slightly coarser
237	10								
232	15								
									-- medium to coarse SAND; increase in GRAVEL
227	20								
222	25								
217	30		X	88					Silty SAND with GRAVEL; very dense; light brown; fine to coarse SAND
212	35								

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670 Mesquit Mixed-Use Development
658 & 670 Mesquit Street
Los Angeles, California

PROJECT NO.	REPORT DATE	
160599.1	October 2018	FIGURE A - 11

DATE DRILLED	2/3/2018	LOGGED BY	AM	**BORING NO.**	**P-4**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	NE
DRILLING METHOD	8" HSA	DRILLER	2R Drilling	SURFACE ELEVATION (ft.)	247 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
								SM	Silty SAND; medium dense; medium brown; moist; fine to medium SAND; trace fine GRAVEL *(continued)*
									-- difficulty drilling
207	40		X	50/2"					-- no recovery
202	45							GP	Sandy GRAVEL; very dense; medium brown
197	50								Total Depth = 50.0 feet Backfilled on 2/3/2018 Backfilled with soil cuttings. Groundwater not encountered.
192	55								
187	60								
182	65								
177	70								

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670 Mesquit Mixed-Use Development
658 & 670 Mesquit Street
Los Angeles, California

PROJECT NO.	REPORT DATE	
160599.1	October 2018	FIGURE A - 11

GREGG

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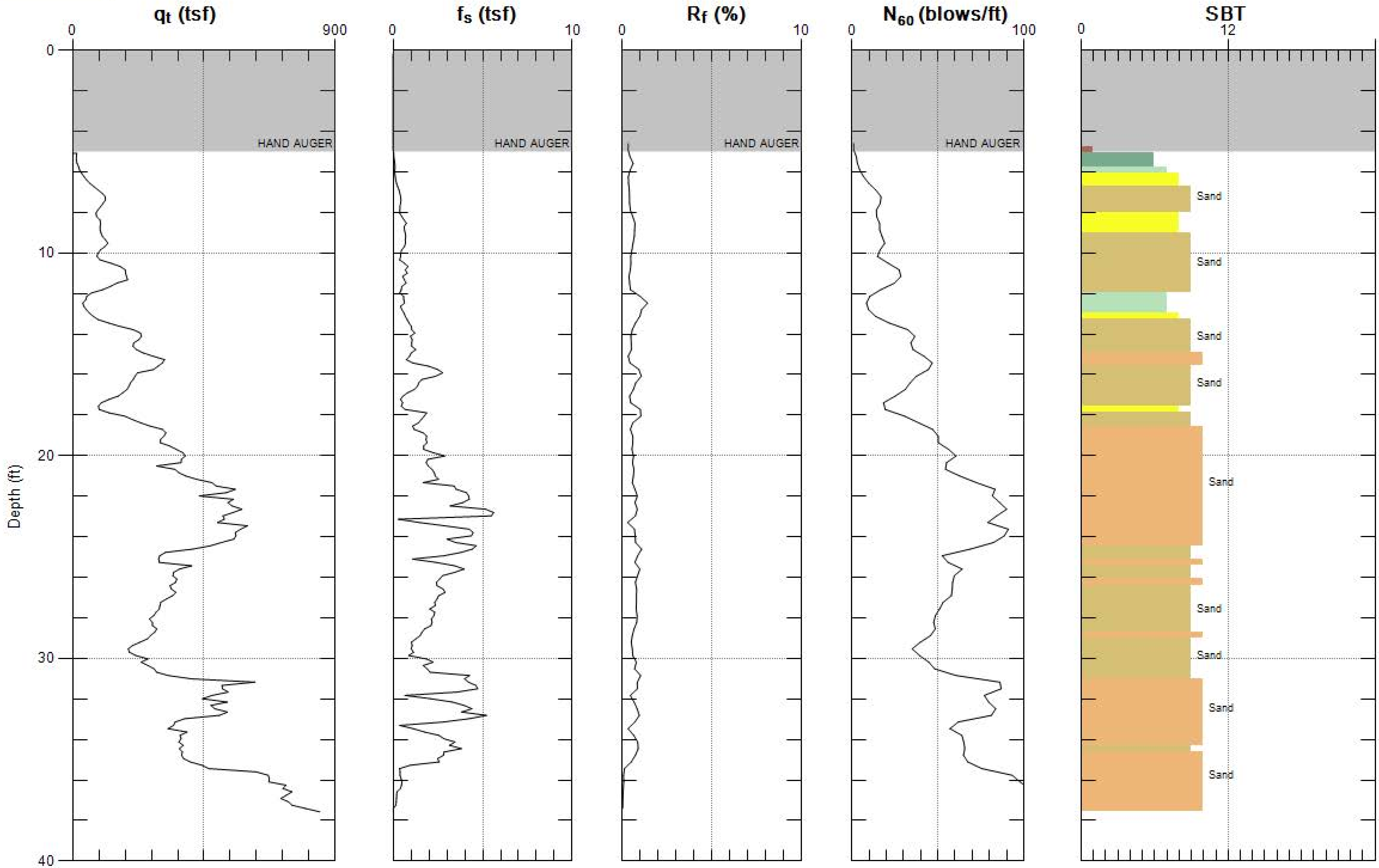
Site: 670 NESQUIT

Sounding: C-1

Engineer: P.SOLTIS

Date: 8/6/2016 12:59



Max. Depth: 37.566 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

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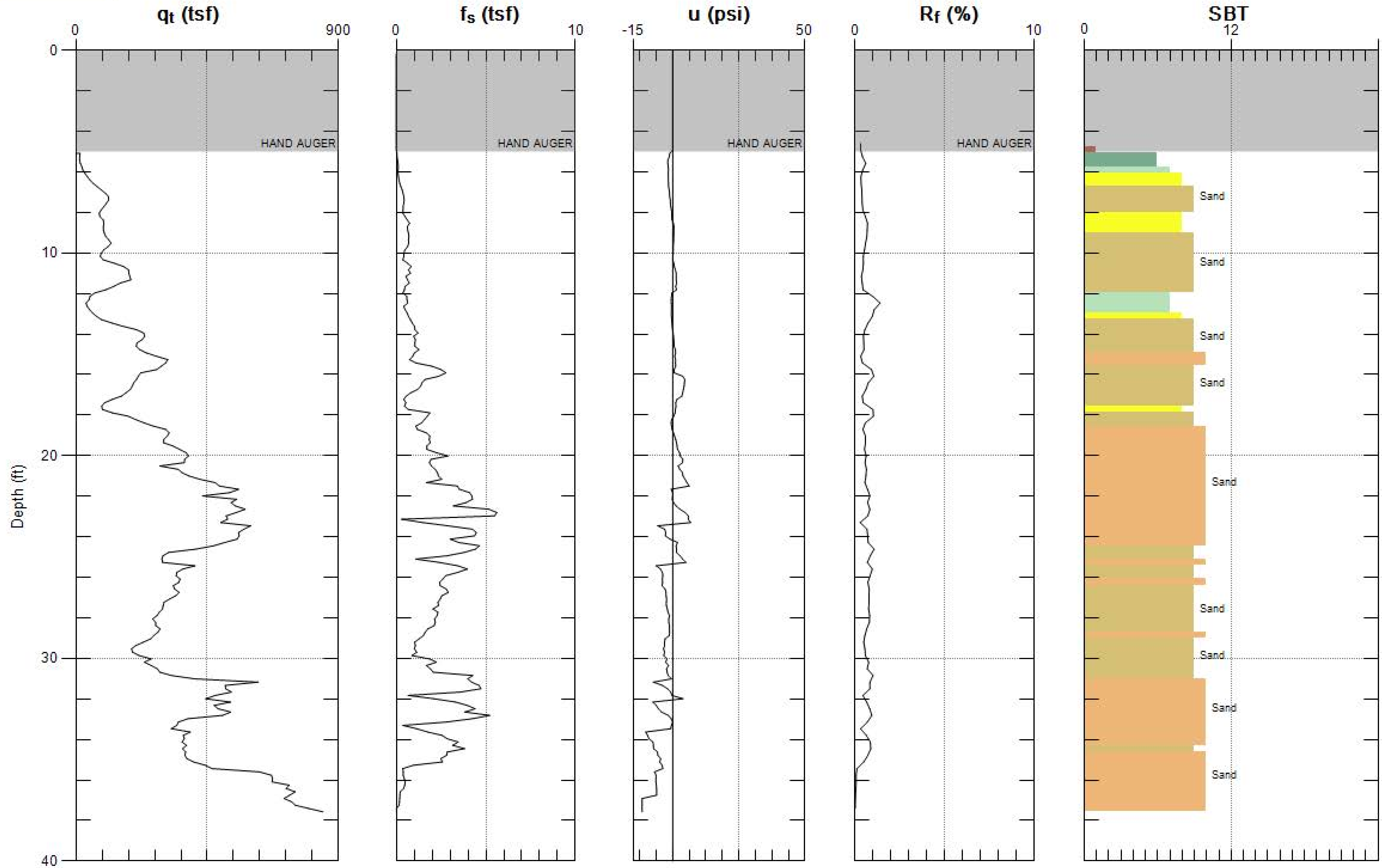
Site: 670 NESQUIT

Sounding: C-1

Engineer: P.SOLTIS

Date: 8/6/2016 12:59



Max. Depth: 37.566 (ft)
Avg. Interval: 0.328 (ft)

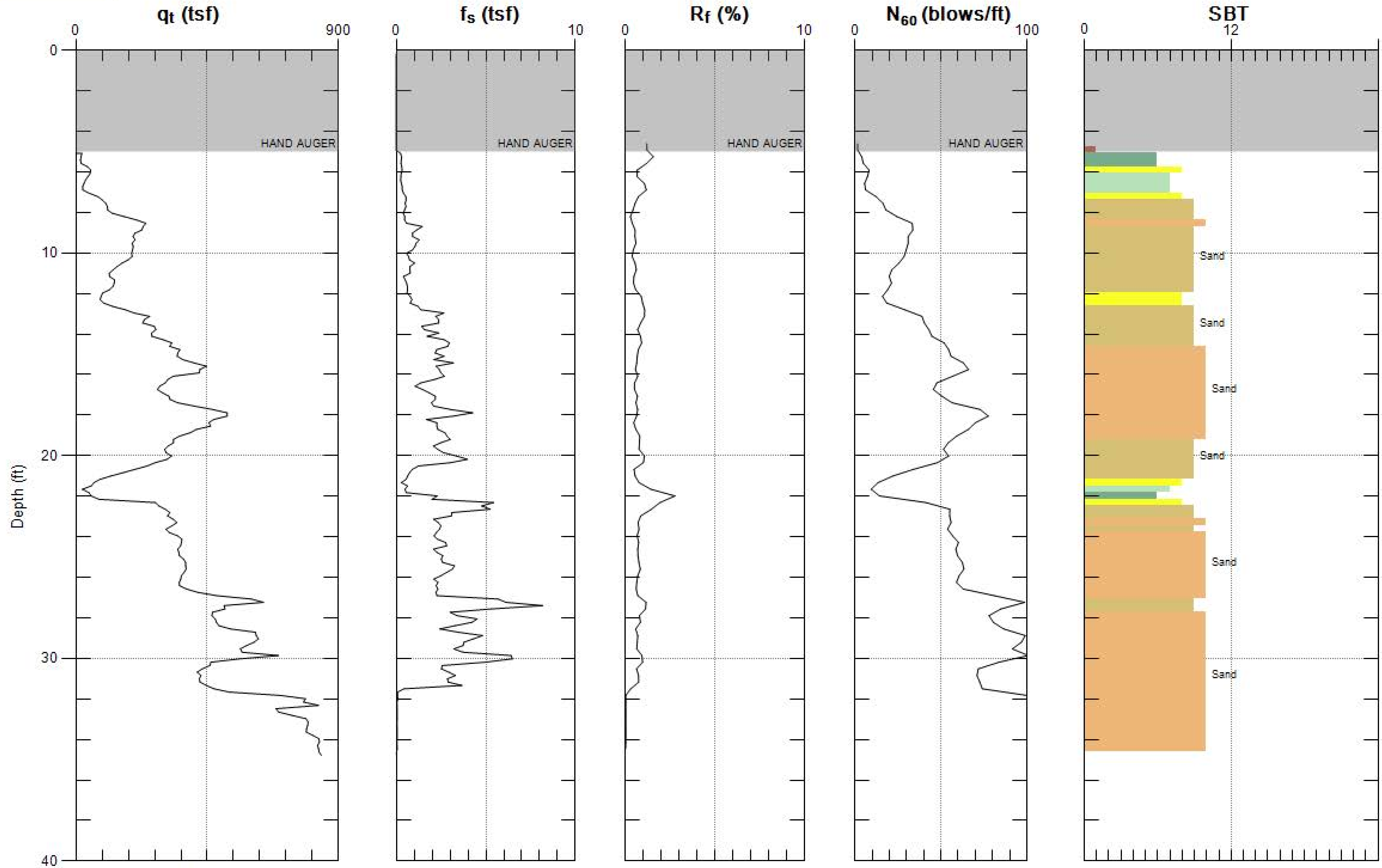
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TWINING CONSULTING

Site: 670 NESQUIT
Sounding: C-2-A

Engineer: P.SOLTIS
Date: 8/6/2016 10:12



Max. Depth: 34.777 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

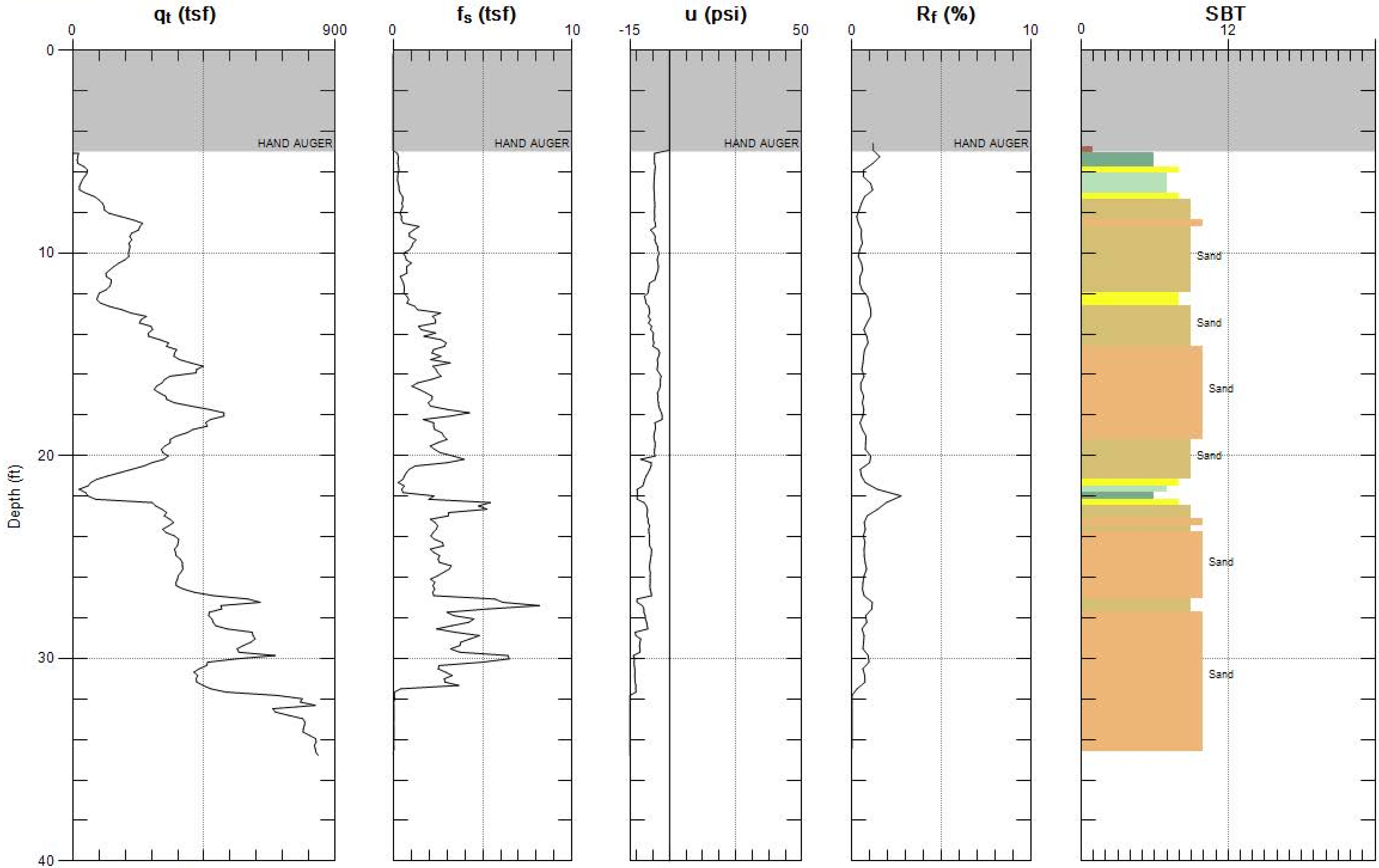
GREGG TWINING CONSULTING

Site: 670 NESQUIT

Sounding: C-2-A

Engineer: P.SOLTIS

Date: 8/6/2016 10:12



Max. Depth: 34.777 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

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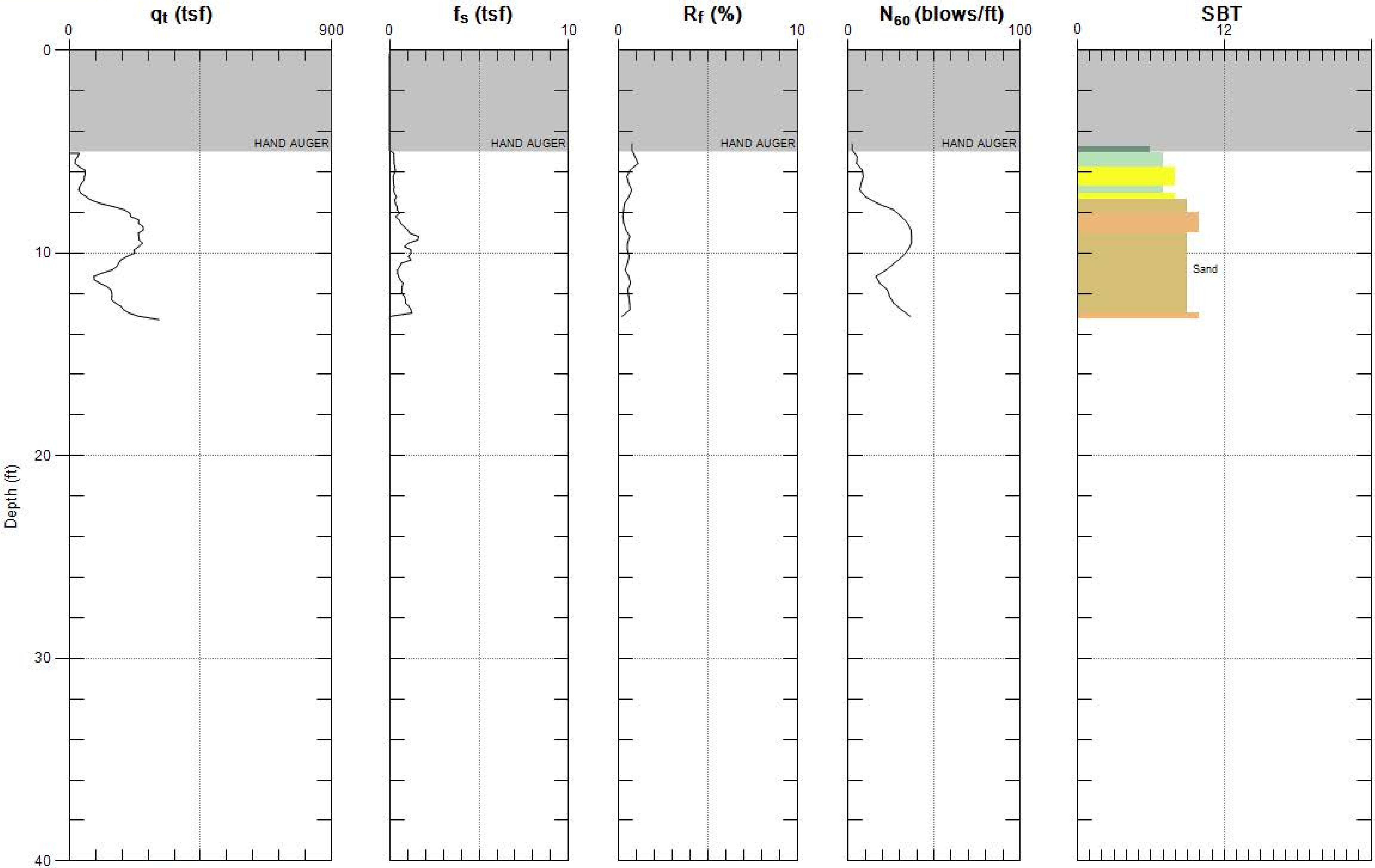
Site: 670 NESQUIT

Sounding: C-2

Engineer: P.SOLTIS

Date: 8/6/2016 08:26



Max. Depth: 13.287 (ft)

Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

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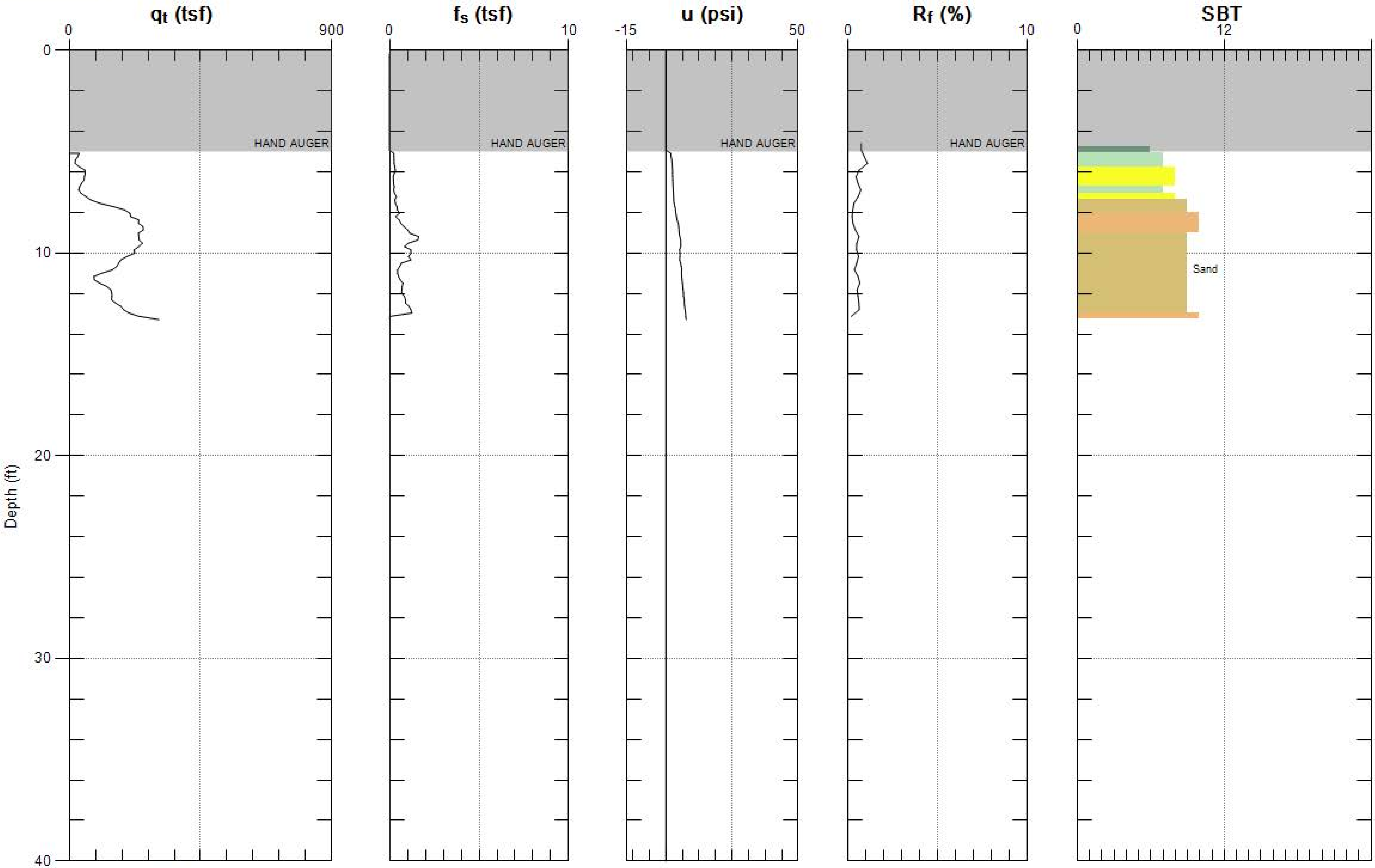
Site: 670 NESQUIT
Sounding: C-2

Engineer: P.SOLTIS
Date: 8/6/2016 08:26



Max. Depth: 13.287 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

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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-1.

Direct Shear Tests

Direct shear tests were performed on selected remolded and 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. Test results are presented on Figure B-1 to B-3.

Consolidation Test

Consolidation tests was performed on a selected driven soil sample 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 presented on Figure B-4 and B-5.

Corrosivity

Soil pH and resistivity tests were performed by Anaheim Test Laboratories 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-2.

Resistance Value (R-Value)

R-value testing was performed on a select bulk sample of the near-surface soils encountered at the site. The test was performed in general accordance with ASTM D 28444. The results are summarized in Table B-3.

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Table B-1
No. 200 Wash Sieve Results

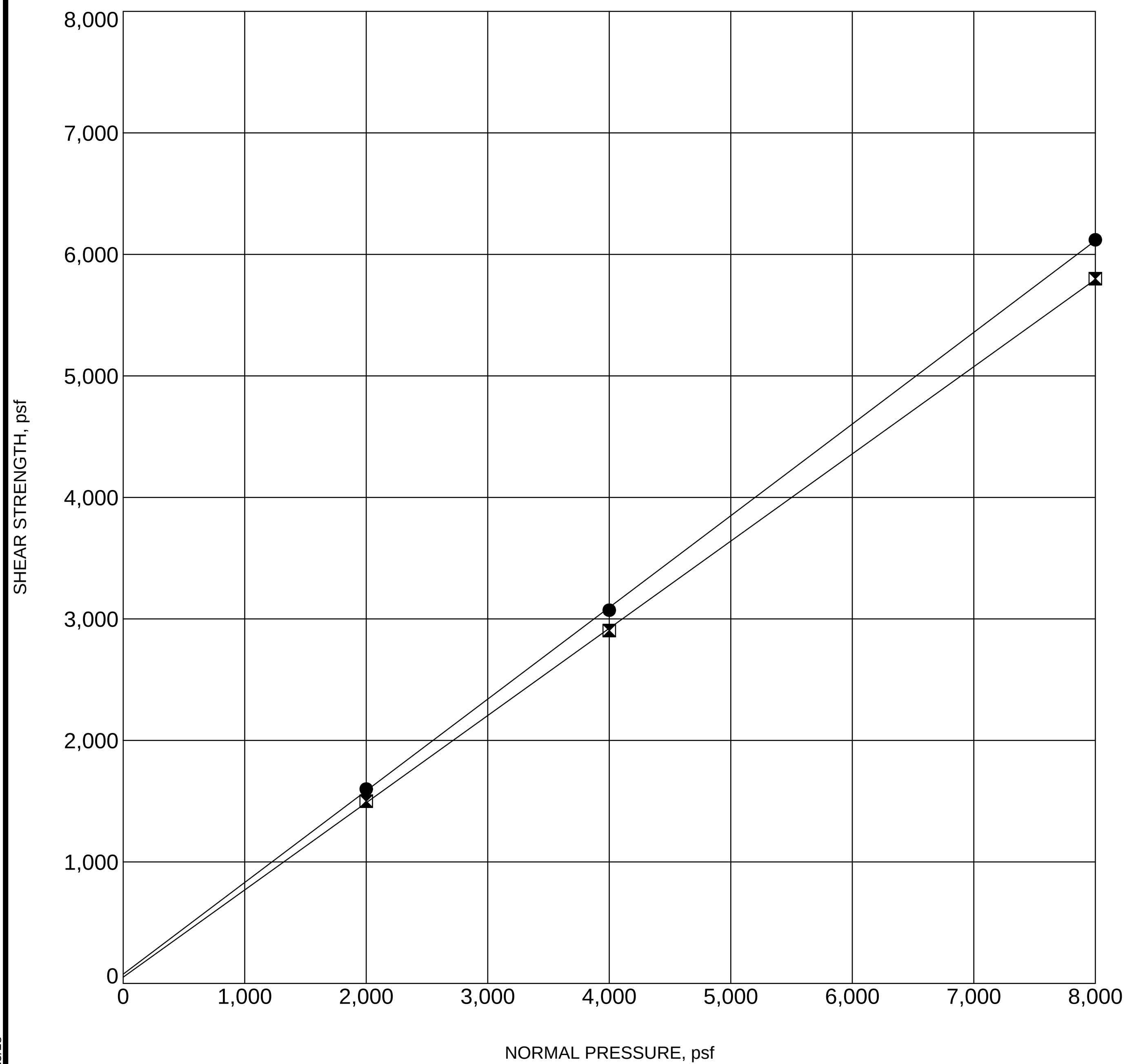
Boring No.	Depth (feet)	Percent Passing #200
B-1	0-5	33.8
B-1	20	6.7
B-2	0-5	20.6
B-2	25	5.2
B-3	10	6.5
B-3	30	12.3
B-4	15	9.6
B-4	45	9.8
P-1	5	23.4
P-2	10	15.5

Table B-2
Soil Corrosivity Test Results

Boring No.	Depth (feet)	pH	Water Soluble Sulfate (ppm)	Water Soluble Chloride (ppm)	Minimum Resistivity (ohm-cm)
B-1	30	7.6	140	22	6,300
B-4	0-5	7.3	181	17	5,600

Table B-3
Resistance Value (R-Value)

Boring No.	Depth (feet)	R-Value
B-5	0 – 5	76



Shear Strength Parameters

		Peak	Ultimate
Boring No.:	B-1		
Sample Depth (ft):	35		
Sample Description:	Poorly graded SAND with silt		
Strain Rate (in./min):	0.005		
Dry Density (pcf):	104.2		
Cohesion, C (psf):		75	50
Friction Angle, Ø (deg):		37	36
Initial Moisture (%):		4.0	
Final Moisture (%):		12.9	

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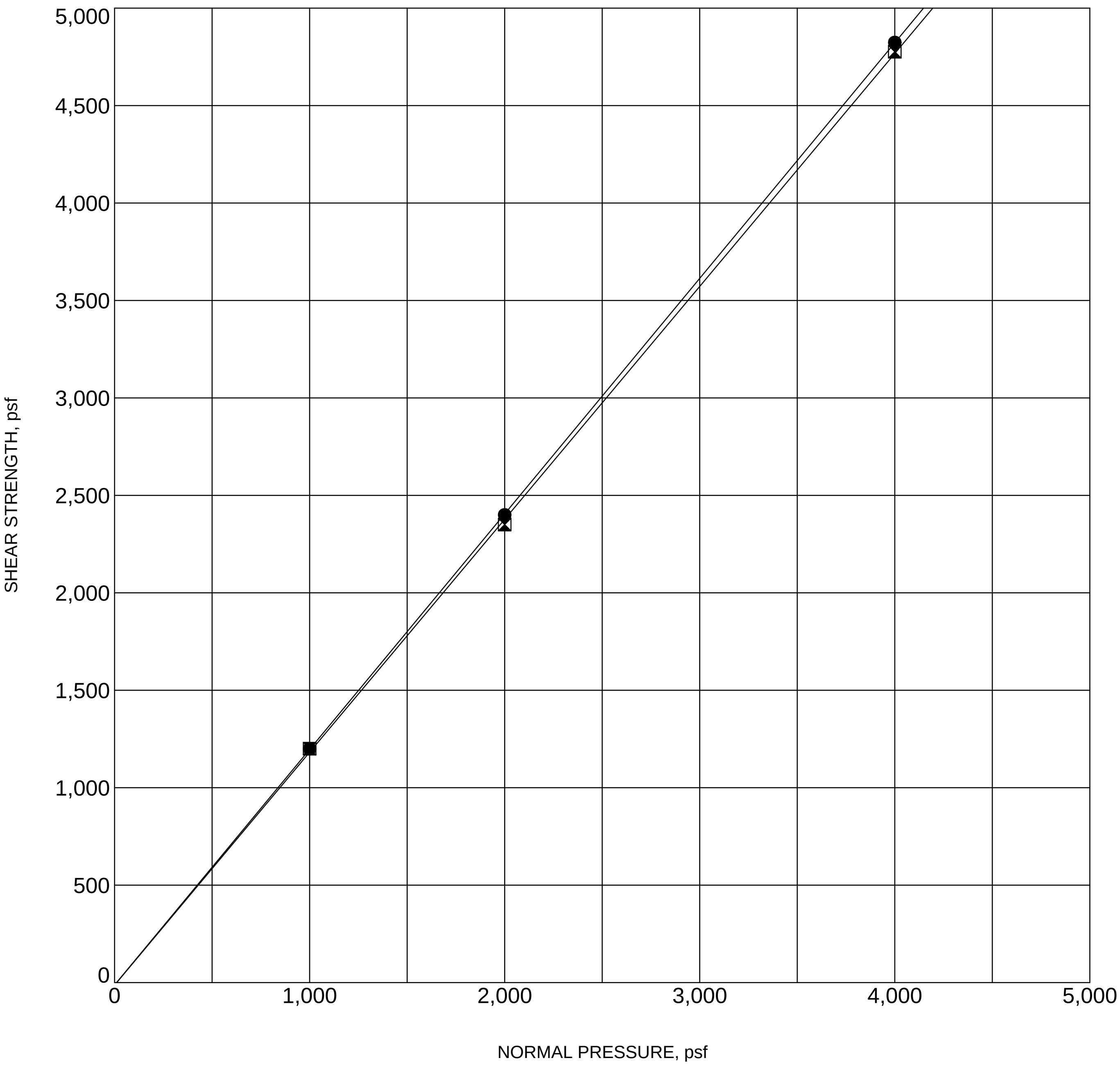
DIRECT SHEAR TEST

670 Mesquit Mixed-Use Development
658 & 670 Mesquit Street
Los Angeles, California

PROJECT NO.	REPORT DATE	FIGURE B-1
160599.1	October 2018	



Shear Strength Parameters

	Peak	Ultimate
Cohesion, C (psf):	0	0
Friction Angle, Ø (deg):	50	50

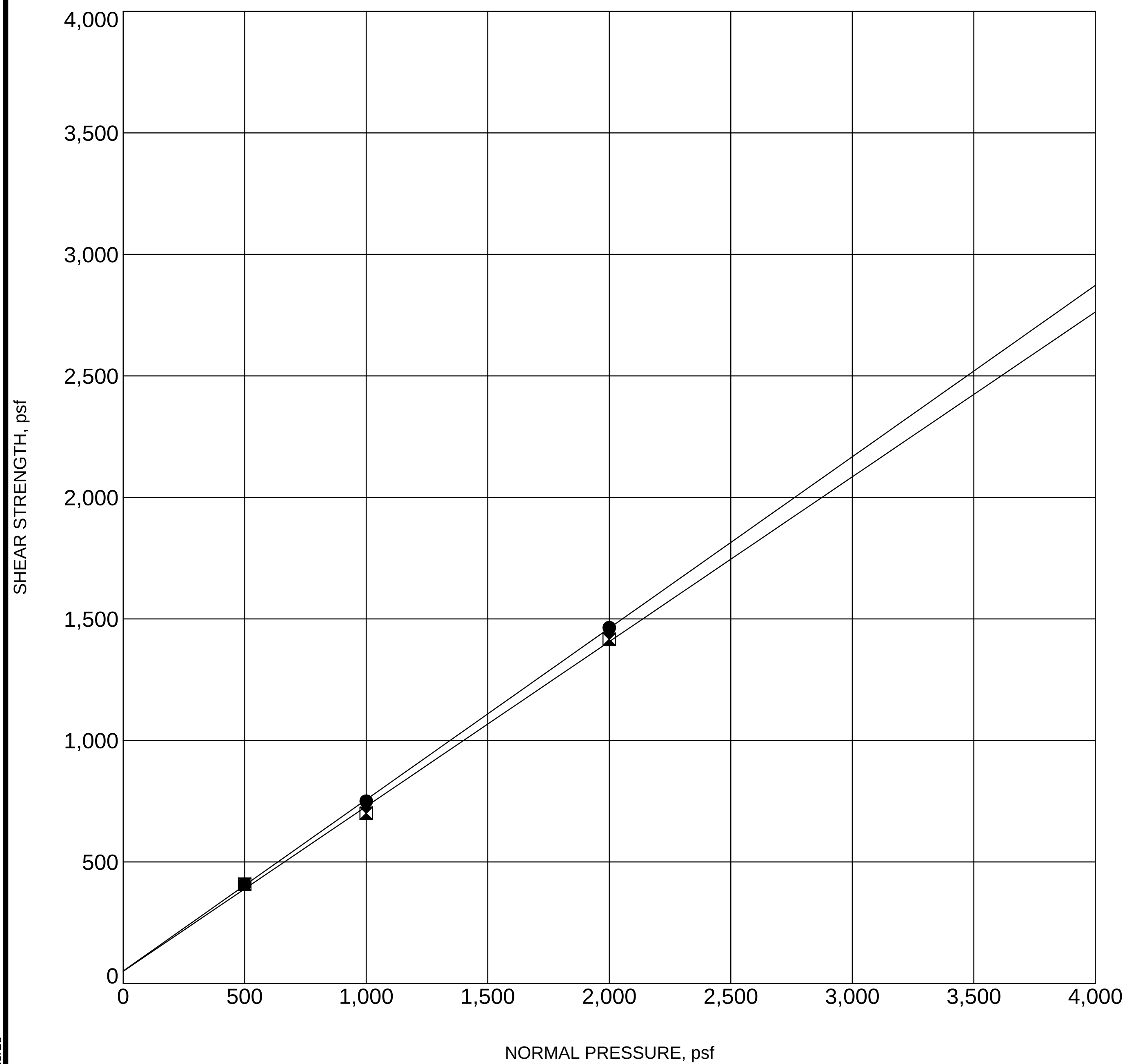
Boring No.: B-2
Sample Depth (ft): 20
Sample Description: Poorly graded SAND with silt
Strain Rate (in./min): 0.005
Dry Density (pcf): 116.5

Initial Moisture (%): 2.6
Final Moisture (%): 15.5

DIRECT SHEAR TEST

670 Mesquit Mixed-Use Development
658 & 670 Mesquit Street
Los Angeles, California

PROJECT NO.	REPORT DATE	
160599.1	October 2018	FIGURE B-2



Boring No.: B-3
Sample Depth (ft): 5
Sample Description: Silty SAND
Strain Rate (in./min): 0.005
Dry Density (pcf): 102.2

Shear Strength Parameters

	Peak	Ultimate
Cohesion, C (psf):	50	50
Friction Angle, Ø (deg):	35	34

Initial Moisture (%): 8.3
Final Moisture (%): 18.1

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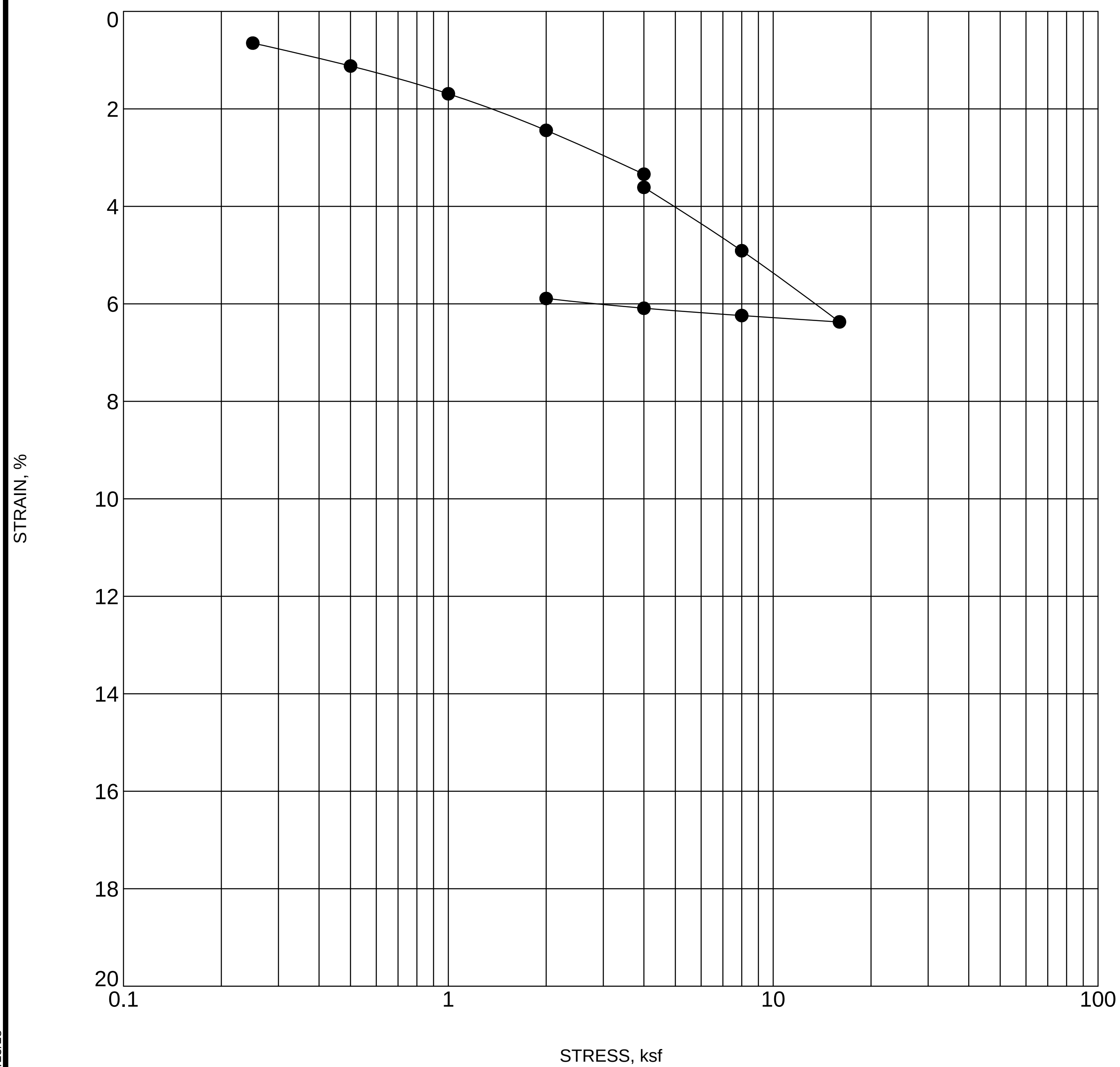
DIRECT SHEAR TEST

670 Mesquit Mixed-Use Development
658 & 670 Mesquit Street
Los Angeles, California

PROJECT NO.	REPORT DATE	FIGURE B-3
160599.1	October 2018	



	Sample Location	Soil Description	Dry Density (pcf)	Moisture Content (%)
●	**B-3 at 35 ft**	**Poorly graded SAND with silt**	**101.6**	**3.0**

CONSOLIDATION TEST

670 Mesquit Mixed-Use Development
658 & 670 Mesquit Street
Los Angeles, California

PROJECT NO.	REPORT DATE	
160599.1	October 2018	FIGURE B-4

TWINING

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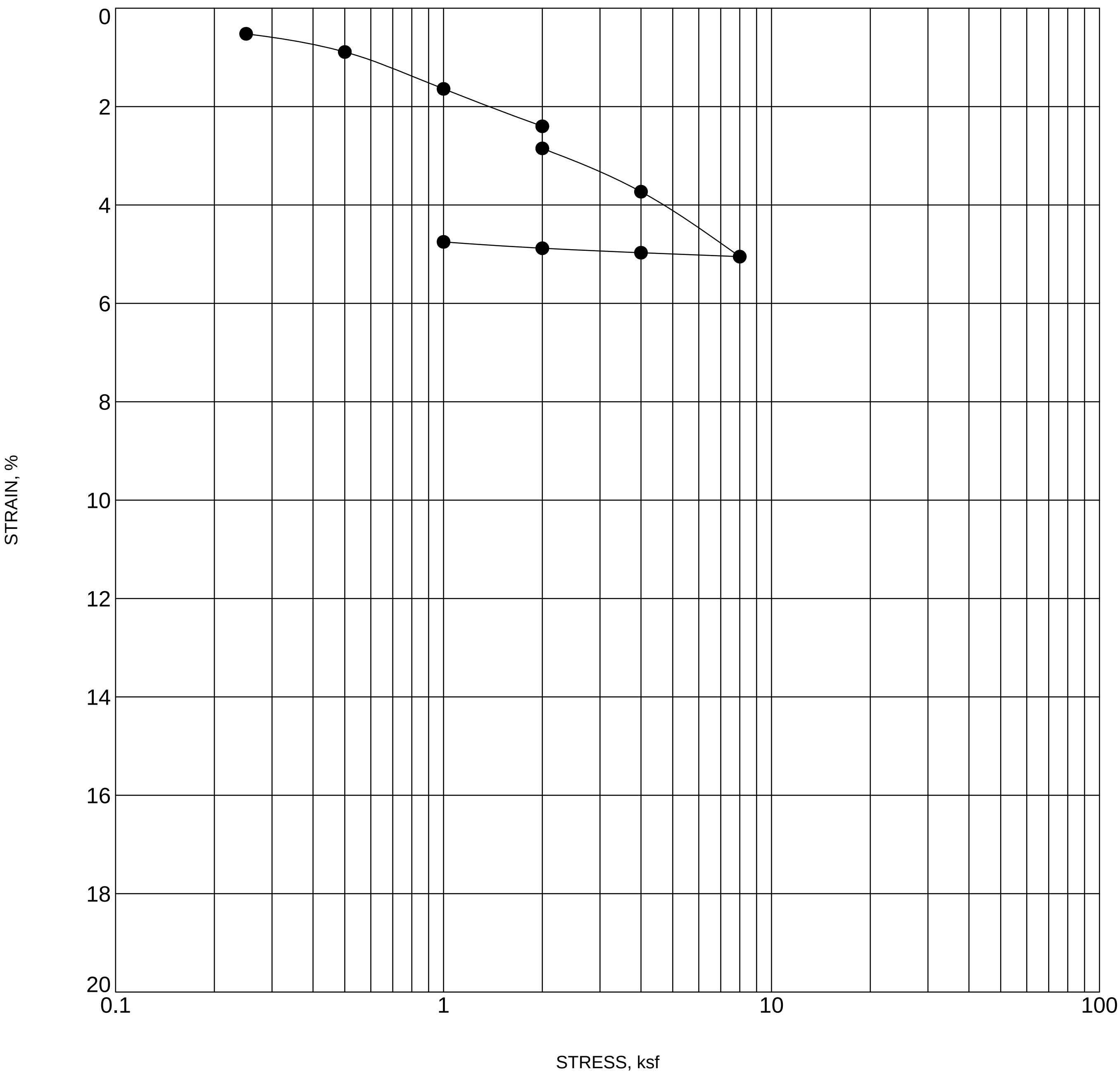

	Sample Location	Soil Description	Dry Density (pcf)	Moisture Content (%)
●	**B-4 at 10 ft**	**Poorly graded SAND with silt**	**104.4**	**2.2**

CONSOLIDATION TEST

670 Mesquit Mixed-Use Development
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PROJECT NO.	REPORT DATE	FIGURE B-5
160599.1	October 2018	

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APPENDIX C
PERCOLATION TESTING

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Appendix C
Percolation Testing

A total of four percolation test borings were excavated at the project site. The borings were advanced to approximate depths ranging between 46 and 50 feet bgs. The locations of the proposed drywells and our percolation testing locations are depicted on Figure 2 – Site Plan and Exploration Location Map.

Field Exploration

The borings were excavated according to Appendix A adjacent to the drywell locations indicated by the project civil engineer. The borings were excavated using a truck-mounted drill rig using an eight-inch diameter hollow-stem-auger. Logs of borings for the four percolation test holes are attached to this report.

Percolation Testing

Percolation testing was performed on February 3, 2018 in conformance with the County of Los Angeles GS200.1 manual. After installing pipe and filter sand, the boreholes were presoaked for two consecutive 30-minute sessions prior to testing. At the end of each presoak session, no water remained in the test hole.

After presoaking, the boreholes were filled with water to depths ranging between approximately 15-30 feet above the bottom of the excavation. Measurements were recorded at 10-minute intervals for a total of 8 readings or until percolation rates stabilized. The average drop that occurred over the last 3 readings was used to determine the percolation rate at each test location. Detailed test data is attached at the end of the report.

Conclusions and Recommendations

Based on the results of our field testing and engineering evaluation, it is our opinion that the proposed infiltration drywell is feasible from a geotechnical standpoint, provided that the recommendations in this report are incorporated into the design plans and are implemented during construction. In accordance with the City of Los Angeles (2016), we have used a factor of safety of 3 to determine our recommended design infiltration rate. The followings are our conclusions and recommendations:

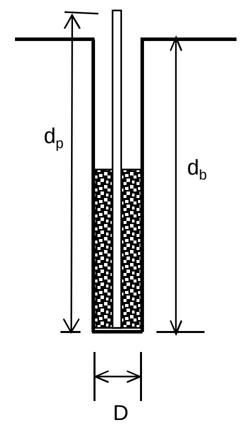
- The proposed use of drywells for infiltration of storm water at the site is feasible. Based on the consistency of site conditions encountered, it is our opinion that our recommended infiltration rate is applicable for the entirety of the site;
- The proposed infiltration zone of 30 to 47 feet below the existing ground surface is acceptable for infiltration at a recommended design infiltration rate of 5 inches/hour;
- Spacing between drywells should be at least 30 feet (center to center);
- The drywell shall be located at least 15 feet from any existing and proposed building foundations.

A summary of results is presented in Table C-1 below and the detailed data is attached.

Table C-1 - Summary of Percolation Test Results

Test Location	Depth of Test Hole (in.)	Design Infiltration Rate (in/hr)
P-1	600	**9.0**
P-2	600	**6.5**
P-3	552	**11.0**
P-4	600	**1.3**

Twining Project No.: 180100.1
Project Name: 670 Mesquit Percolation Testing



Boring No.: **P-1**
Diameter of Boring (D): 8.0 inches
Depth of Boring (d_b): 50.0 feet = 600 inches
Diameter of Perc. Pipe : 3.0 inches
Length of Pipe (d_p) : 50.0 feet = 600 inches
Depth Interval of Perforated Pipe: 30 - 50 feet

PRE-SOAK Number One	
Date:	2/3/2018
Start Time:	1:32 PM
Elapsed Time:	30 minutes
Water Remaining:	No

PRE-SOAK Number Two	
Date:	2/3/2018
Start Time:	2:10 PM
Elapsed Time:	30 minutes
Water Remaining:	No

CORRECTION FACTORS

Boring method: $CF_t = R_f = (2*d_i - \Delta d)/D + 1$
Site variability: CF_v = 1.0 (1 ~ 3)
Long-term siltation: CF_s = 3.0 (1 ~ 3)
Total Correction Factor: $CF = CF_t \times CF_v \times CF_s$

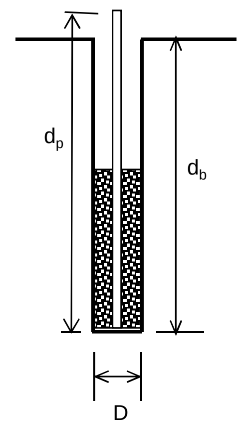
PERCOLATION TEST **Test Date:** 2/3/2018 **Test Performer:** AM **Calculated by:** AM

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	2:52 PM	3:02 PM	10	345.6	510.0	254.4	164.4	986.40	44.1	132.2	7.46
2	3:06 PM	3:16 PM	10	346.8	518.4	253.2	171.6	1029.60	42.9	128.6	8.01
3	3:22 PM	3:32 PM	10	338.4	526.8	261.6	188.4	1130.40	42.9	128.6	8.79
4	3:36 PM	3:46 PM	10	309.6	526.2	290.4	216.6	1299.60	46.5	139.6	9.31
5	3:50 PM	4:00 PM	10	325.2	526.8	274.8	201.6	1209.60	44.5	133.5	9.06
6	4:04 PM	4:14 PM	10	310.8	523.2	289.2	212.4	1274.40	46.8	140.3	9.09

Recommended Design Infiltration Rate (inch/hr) = **9.0**

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

Twining Project No.: 180100.1
Project Name: 670 Mesquit Percolation Testing



Boring No.: **P-2**
Diameter of Boring (D): 8.0 inches
Depth of Boring (d_b): 50.0 feet = 600 inches
Diameter of Perc. Pipe : 3.0 inches
Length of Pipe (d_p) : 50.0 feet = 600 inches
Depth Interval of Perforated Pipe: 30 - 50 feet

PRE-SOAK Number One
Date: 2/3/2018
Start Time: 2:12 PM
Elapsed Time: 30 minutes
Water Remaining: No

PRE-SOAK Number Two
Date: 2/3/2018
Start Time: 2:47 PM
Elapsed Time: 30 minutes
Water Remaining: No

CORRECTION FACTORS
Boring method: $CF_t = R_f = (2*d_i - \Delta d)/D + 1$
Site variability: $CF_v =$ 1.0 (1 ~ 3)
Long-term siltation: $CF_s =$ 3.0 (1 ~ 3)
Total Correction Factor: $CF = CF_t \times CF_v \times CF_s$

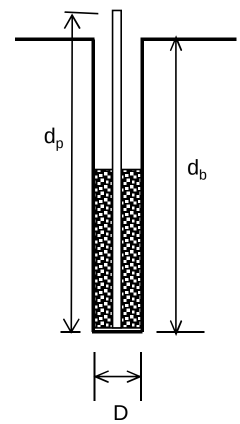
PERCOLATION TEST **Test Date:** 2/3/2018 **Test Performer:** AM **Calculated by:** AM

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	3:28 PM	3:38 PM	10	280.8	448.6	319.2	167.8	1006.56	59.8	179.5	5.61
2	3:39 PM	3:49 PM	10	277.8	458.4	322.2	180.6	1083.60	59.0	176.9	6.12
3	3:54 PM	4:04 PM	10	270.6	468.0	329.4	197.4	1184.40	58.7	176.0	6.73
4	4:08 PM	4:18 PM	10	271.2	466.8	328.8	195.6	1173.60	58.8	176.3	6.66
5	4:21 PM	4:31 PM	10	270.0	453.6	330.0	183.6	1101.60	60.6	181.7	6.06

Recommended Design Infiltration Rate (inch/hr) = **6.5**

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

Twining Project No.: 180100.1
Project Name: 670 Mesquit Percolation Testing



Boring No.: **P-3**
Diameter of Boring (D): 8.0 inches
Depth of Boring (d_b): 46.0 feet = 552 inches
Diameter of Perc. Pipe : 3.0 inches
Length of Pipe (d_p) : 46.0 feet = 552 inches
Depth Interval of Perforated Pipe: 26 - 46 feet

PRE-SOAK Number One	
Date:	2/3/2018
Start Time:	9:31 AM
Elapsed Time:	30 minutes
Water Remaining:	No

PRE-SOAK Number Two	
Date:	2/3/2018
Start Time:	10:04 AM
Elapsed Time:	30 minutes
Water Remaining:	No

CORRECTION FACTORS

Boring method: $CF_t = R_f = (2*d_i - \Delta d)/D + 1$
Site variability: $CF_v =$ 1.0 (1 ~ 3)
Long-term siltation: $CF_s =$ 3.0 (1 ~ 3)
Total Correction Factor: $CF = CF_t \times CF_v \times CF_s$

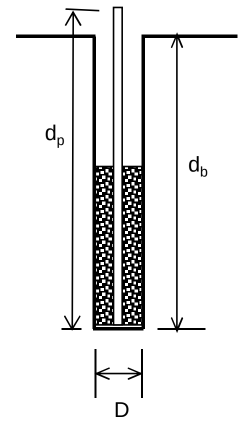
PERCOLATION TEST **Test Date:** 2/3/2018 **Test Performer:** AM **Calculated by:** AM

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	11:07 AM	11:17 AM	10	288.0	552.0	264.0	264.0	1584.00	34.0	102.0	15.53
2	11:22 AM	11:32 AM	10	276.0	501.6	276.0	225.6	1353.60	41.8	125.4	10.79
3	11:54 AM	12:04 PM	10	265.2	508.8	286.8	243.6	1461.60	42.3	126.8	11.53
4	12:12 PM	12:22 PM	10	318.0	513.0	234.0	195.0	1170.00	35.1	105.4	11.10
5	12:29 PM	12:39 PM	10	288.0	503.0	264.0	215.0	1290.24	40.1	120.4	10.72
6	1:00 PM	1:10 PM	10	262.8	511.2	289.2	248.4	1490.40	42.3	126.8	11.76
7	1:17 PM	1:27 PM	10	310.8	515.5	241.2	204.7	1228.32	35.7	107.1	11.47

Recommended Design Infiltration Rate (inch/hr) = 11.0

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

Twining Project No.: 180100.1
Project Name: 670 Mesquit Percolation Testing



Boring No.: P-4
Diameter of Boring (D): 8.0 inches
Depth of Boring (d_b): 50.0 feet = 600 inches
Diameter of Perc. Pipe : 3.0 inches
Length of Pipe (d_p) : 50.0 feet = 600 inches
Depth Interval of Perforated Pipe: 30 - 50 feet

PRE-SOAK Number One
Date: 2/3/2018
Start Time: 9:15 AM
Elapsed Time: 30 minutes
Water Remaining: No

PRE-SOAK Number Two
Date: 2/3/2018
Start Time: 9:52 AM
Elapsed Time: 30 minutes
Water Remaining: No

CORRECTION FACTORS
Boring method: $CF_t = R_f = (2*d_i - \Delta d)/D + 1$
Site variability: $CF_v =$ 1.0 (1 ~ 3)
Long-term siltation: $CF_s =$ 3.0 (1 ~ 3)
Total Correction Factor: $CF = CF_t \times CF_v \times CF_s$

PERCOLATION TEST **Test Date:** 2/3/2018 **Test Performer:** AM **Calculated by:** AM

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	10:29 AM	10:39 AM	10	370.2	416.2	229.8	46.0	275.76	52.7	158.1	1.74
2	10:43 AM	10:53 AM	10	388.8	421.0	211.2	32.2	192.96	49.8	149.3	1.29
3	11:00 AM	11:10 AM	10	372.0	425.4	228.0	53.4	320.40	51.3	154.0	2.08
4	11:12 AM	11:22 AM	10	361.2	422.4	238.8	61.2	367.20	53.1	159.2	2.31
5	11:25 AM	11:35 AM	10	392.5	426.7	207.5	34.2	205.20	48.6	145.8	1.41
6	11:50 AM	12:00 PM	10	396.8	428.2	203.2	31.3	187.92	47.9	143.6	1.31
7	12:07 PM	12:17 PM	10	390.0	422.5	210.0	32.5	195.12	49.4	148.3	1.32

Recommended Design Infiltration Rate (inch/hr) = **1.3**

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

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APPENDIX D
PRINCIPAL ACTIVE FAULTS

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Table D-1: Principal Fault Location

Fault	Approximate Fault-to-Site Distance (miles)	Maximum Moment Magnitude (Mmax)
Elysian Park (Upper)	2.19	6.5
Puente Hills (LA)	3.56	7.0
Hollywood	5.76	6.7
Raymond	5.96	6.8
Santa Monica Connected Alt 2	6.03	7.4
Newport Inglewood Connected Alt 2	7.83	7.5
Verdugo	7.87	6.9
Newport-Inglewood Alt 1	8.24	7.2
Newport Inglewood Connected Alt 1	8.24	7.5
Puente Hills (Santa Fe Springs)	10.22	6.7
Elsinore; W+GI+T+J	10.69	7.77
Elsinore; W+GI+T+J+CM	10.69	7.85
Elsinore; W+GI	10.69	7.27
Elsinore; W	10.69	7.03
Santa Monica, Alt 1	10.80	6.6
Santa Monica Connected Alt 1	10.80	7.3
Sierra Madre Connected	12.29	7.3
Sierra Madre	12.29	7.2
Puente Hills (Coyote Hills)	14.44	6.9
Clamshell-Sawpit	16.39	6.7
Palos Verdes	16.94	7.3
Palos Verdes Connected	16.94	7.7
Malibu Coast, Alt 2	17.06	7.0
Malibu Coast, Alt 1	17.06	6.7
Sierra Madre (San Fernando)	17.19	6.7
Anacapa-Dume, Alt 2	18.63	7.2
San Gabriel	19.70	7.3
San Jose	20.03	6.7
Northridge	20.78	6.9
Santa Susana, Alt 1	25.12	6.9
Anacapa-Dume, Alt 1	27.05	7.2
Chino, Alt 2	27.68	6.8
Chino, Alt 1	27.71	6.7
San Joaquin Hills	28.93	7.1
Cucamonga	29.29	6.7
Holser, Alt 1	32.09	6.8
Simi-Santa Rosa	32.59	6.9
S. San Andreas; BB+NM+SM+NSB+SSB+BG+CO	34.80	8.02
S. San Andreas; CH+CC+BB+NM+SM	34.80	7.91
S. San Andreas; CH+CC+BB+NM+SM+NSB+SSB+BG+CO	34.80	8.18
S. San Andreas; SM	34.80	7.31
S. San Andreas; CC+BB+NM+SM+NSB	34.80	7.86
S. San Andreas; CC+BB+NM+SM+NSB+SSB	34.80	7.94
S. San Andreas; CC+BB+NM+SM+NSB+SSB+BG+CO	34.80	8.11
S. San Andreas; CH+CC+BB+NM+SM+NSB	34.80	7.96
S. San Andreas; CH+CC+BB+NM+SM+NSB+SSB	34.80	8.03

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S. San Andreas; CH+CC+BB+NM+SM+NSB+SSB+BG	34.80	8.12
S. San Andreas; NM+SM	34.80	7.46
S. San Andreas; NM+SM+NSB	34.80	7.56
S. San Andreas; NM+SM+NSB+SSB	34.80	7.68
S. San Andreas; NM+SM+NSB+SSB+BG	34.80	7.83
S. San Andreas; NM+SM+NSB+SSB+BG+CO	34.80	7.93
S. San Andreas; BB+NM+SM	34.80	7.62
S. San Andreas; PK+CH+CC+BB+NM+SM	34.80	7.92
S. San Andreas; PK+CH+CC+BB+NM+SM+NSB	34.80	7.97
S. San Andreas; PK+CH+CC+BB+NM+SM+NSB+SSB	34.80	8.04
S. San Andreas; PK+CH+CC+BB+NM+SM+NSB+SSB+BG	34.80	8.12
S. San Andreas; PK+CH+CC+BB+NM+SM+NSB+SSB+BG+CO	34.80	8.18
S. San Andreas; BB+NM+SM+NSB	34.80	7.71
S. San Andreas; BB+NM+SM+NSB+SSB+BG	34.80	7.93
Newport-Inglewood (Offshore)	35.53	7.0
Oakridge Connected	37.78	7.4
Oakridge (Offshore)	37.78	7.2
Elsinore; GI+T+J+CM	39.31	7.74
San Cayetano	41.21	7.2
San Jacinto; SBV+SJV+A+CC+B	41.95	7.8
S. San Andreas; NSB+SSB+BG	43.45	7.47
Cleghorn	47.70	6.8
S. San Andreas; BB+NM	48.45	7.32
S. San Andreas; CH+CC+BB+NM+SM	48.45	7.7
S. San Andreas; CC+BB+NM	48.45	7.54
S. San Andreas; PK+CH+CC+BB+NM	48.45	7.71
S. San Andreas; NM	48.45	6.95

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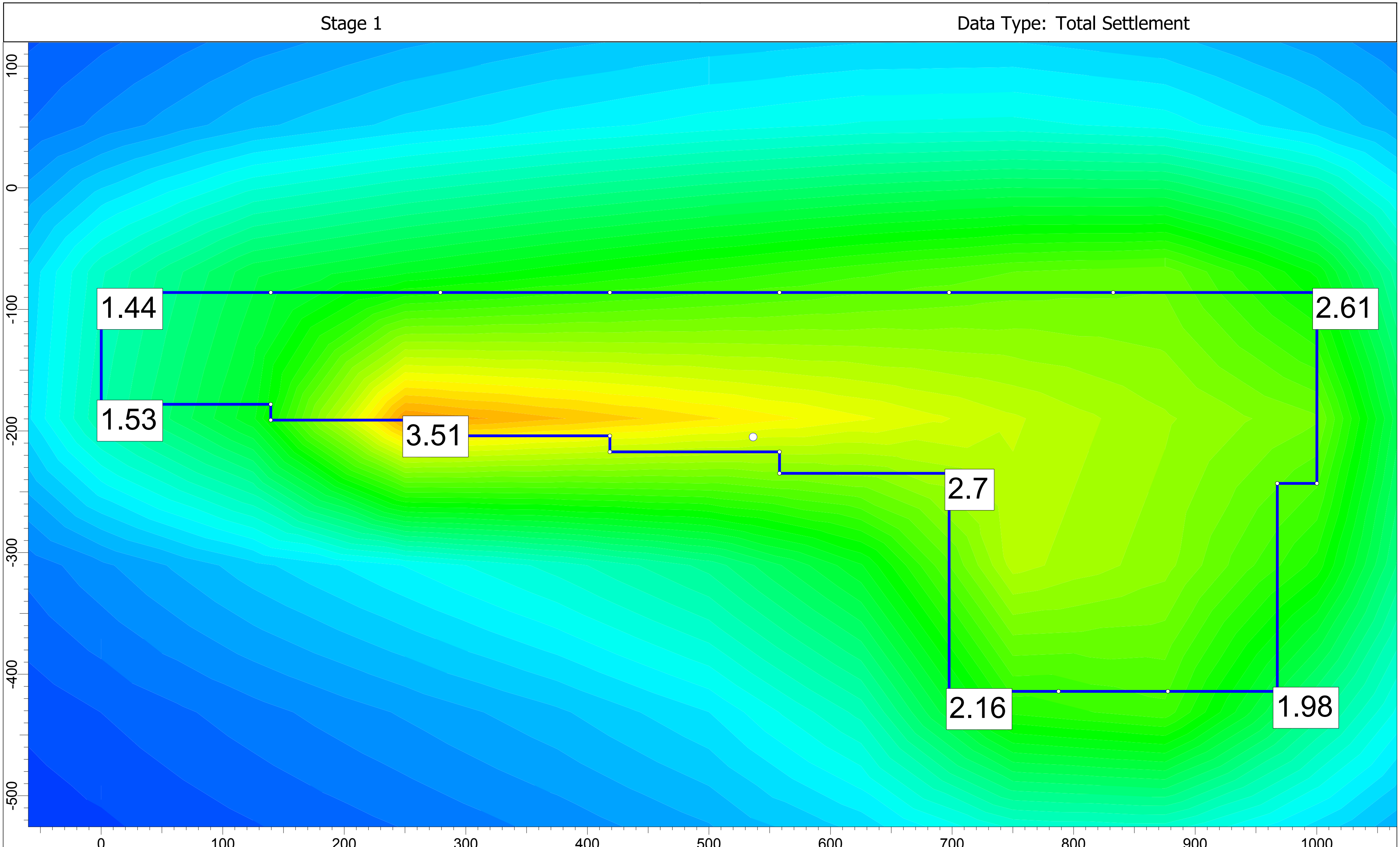
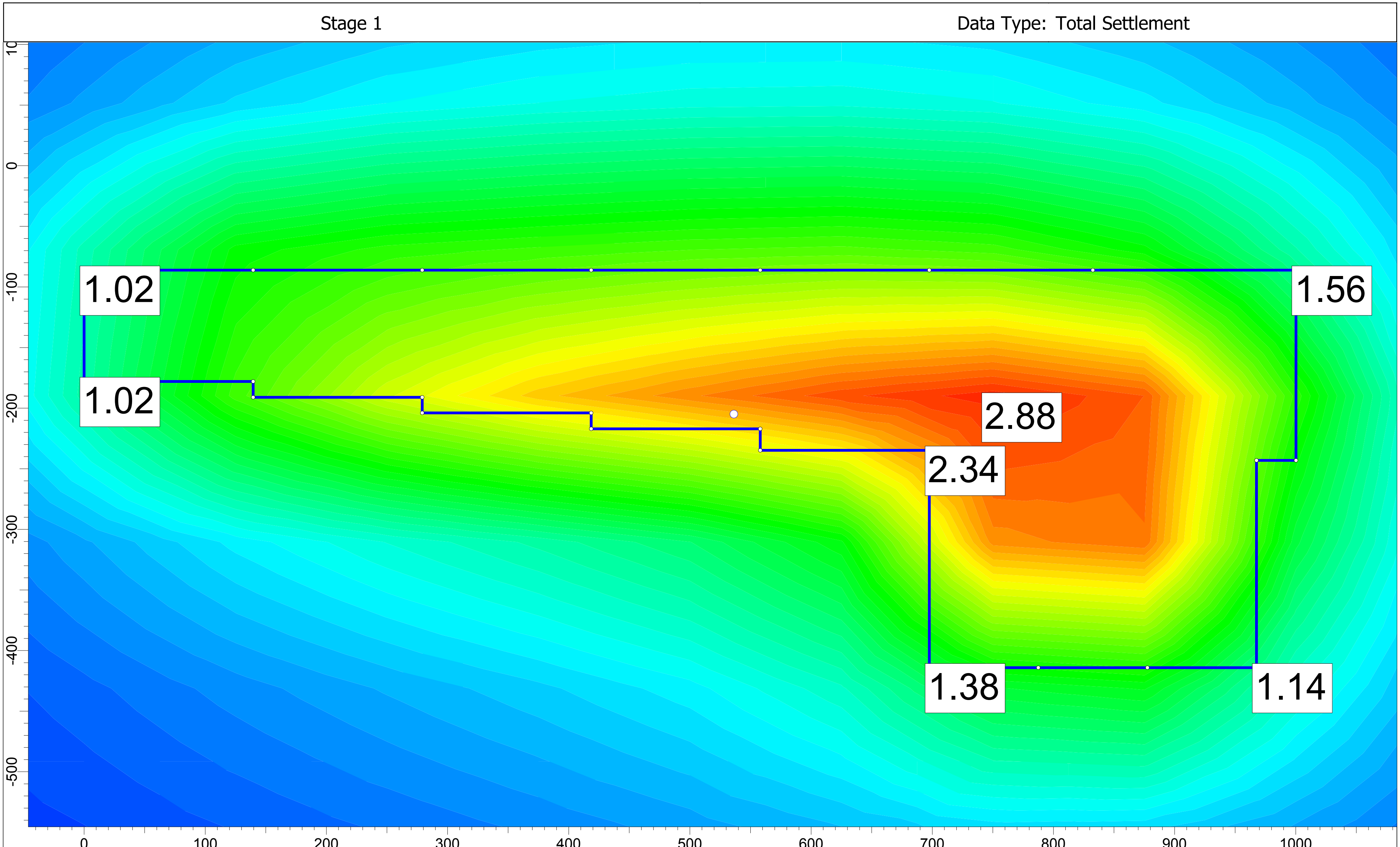
APPENDIX E
SETTLEMENT ANALYSES



Project	670 Mesquit
Analysis Description	Settlement at Bottom of Mat - Rigid
Drawn By	A. Moreno
Company	Twining, Inc.
Date	10/15/2018
Project Number	160599.1

SETTLE3D 2.018



Project	670 Mesquit Mixed-Use Development		
Analysis Description	Settlement at Bottom of Mat - Flexible		
Drawn By	A. Moreno	*Company*	Twining, Inc.
Date	10/15/2018	*Project Number*	160599.1

SETTLE3D 2.018

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APPENDIX F
STRUCTURAL LOADING MEMO

Date:	May 07, 2018, Revision 03 Oct 24, 2018
Prepared For:	Zach Vella RCS VE LLC 319 Lafayette Street, Suite 133 New York, NY 10012
Prepared By:	Ola Johansson, Gijs Libourel, Luke Lombardi
Project Name:	670 Mesquit
Project Number:	S16050.00
Memo Subject:	Loading on Sewage Line
Attachment(s):	Appendix A: Preliminary Column Axial Loads DL + LL (TT, April 6, 2018)

1.0 Loading on Sewage Line

Thornton Tomasetti (TT) developed and verified preliminary loading assumptions regarding the new building design as outlined in the architectural drawings.

Loading calculations are broken down into four components: above-grade weight of the building, below-grade weight of the building, weight of the foundation, and excavation relief pressure.

1.1 Above-Grade Weight of Building

The building weight is summarized in Appendix A. The location of the sewer pipe and loading of interest is indicated in this appendix as well. At this location, the above-grade building weight pressure is approximately 3,000 psf (pounds per square foot). This value is rounded up from the number that is given in Appendix A to account for future design developments of the actual building, given the preliminary status of the design at this time.

1.2 Below-Grade Weight of Building

The weight of L+01 and four levels of below-grade slabs (fifth level of parking supported by the foundation mat) are included as below-grade building weight and estimated to have a bearing pressure of 1,250 psf.

1.3 Weight of Foundation

Foundation mat bearing pressure is approximately 750 psf from a 5 ft thick concrete slab at 150 pcf (pounds per cubic foot).

1.4 Excavation Relief

Per calculations by Twining Consulting, the estimated pressure removed from the pipe after excavation is 7,920 psf, based on a soil unit weight of 120 pcf and an approximate average excavation depth of 66 ft.

1.5 Net Bearing Pressure on Sewer Pipe After New Construction

A total bearing pressure from the constructed building is summed and estimated to be 5,000 psf. A net pressure on the existing sewer line is found by subtracting the expected excavation relief pressure. See Figure 1. Building 3 is analyzed as the heaviest pressure above the sewer line.

The pressure from the building is expected to be less than the weight of the soil removed for excavation. This means the pressure on the pipe will be more at the current state of the pipe than upon completion of new construction.

This analysis does not consider seismic loading or other temporary loading conditions.

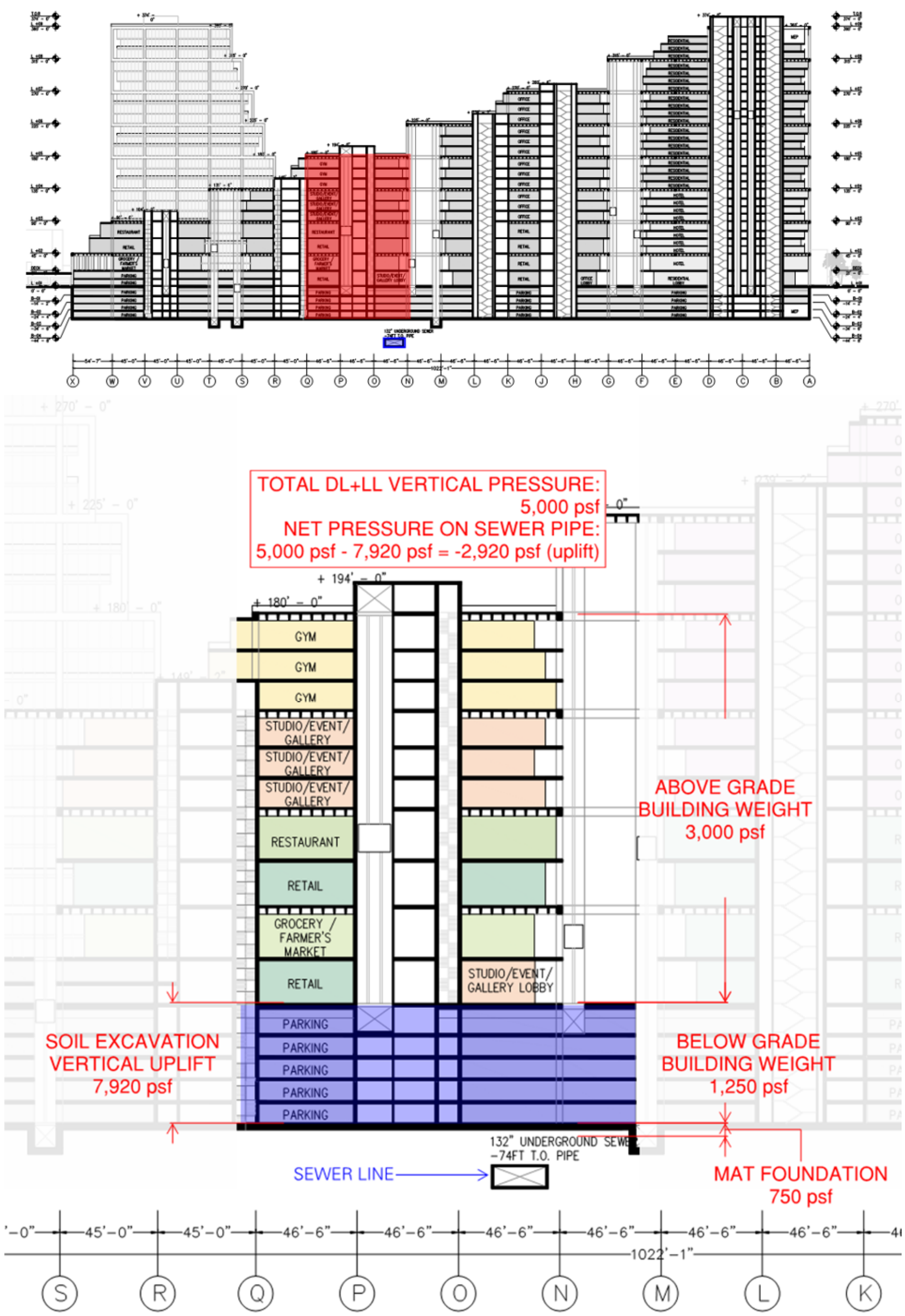
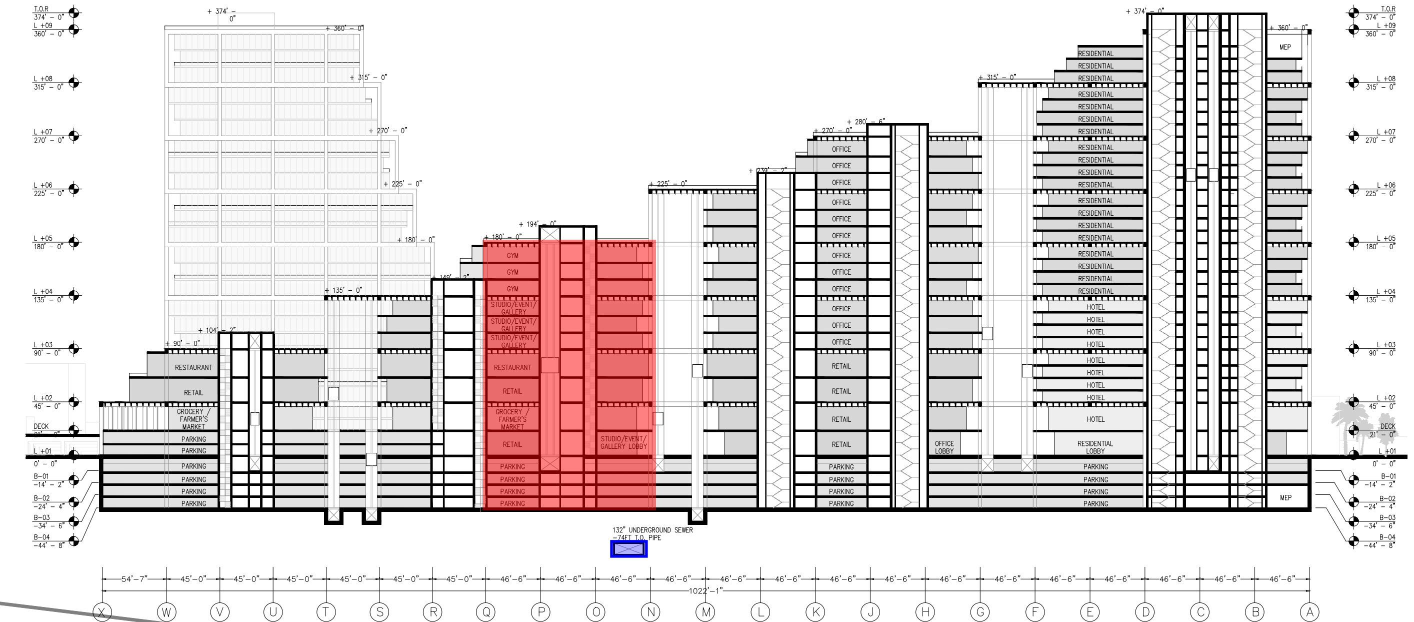


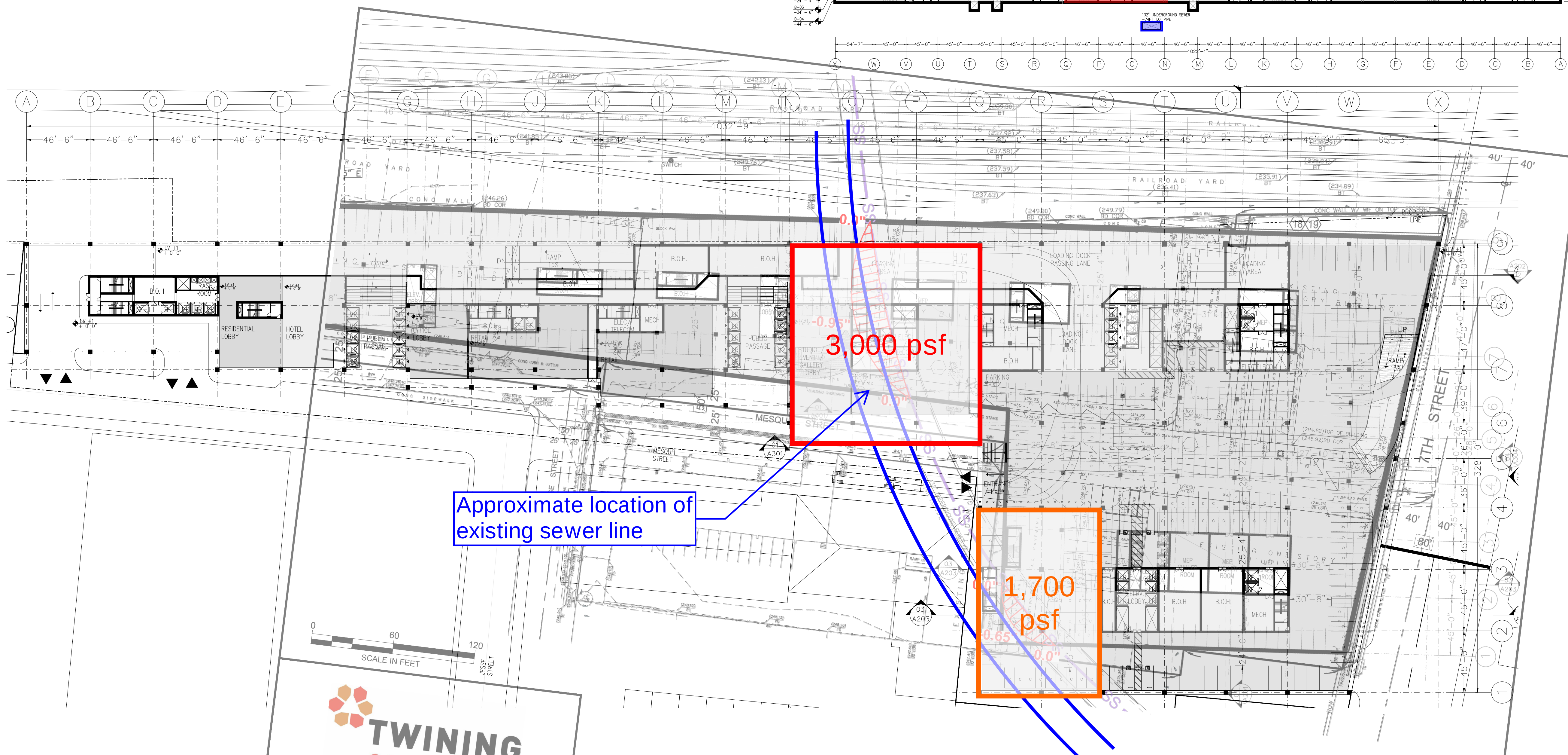
Figure 1: Calculation used to determine pressure on existing sewer line during the lifetime of the new building (Building 3).

Estimated Soil Pressure from Above-Grade Building Weight Above Sewer Line

Dead Load (DL) + Live Load (LL), Unfactored

Units provided in pounds per square foot (psf)





Thornton Tomasetti

PROJECT: 670 Mesquit, Los Angeles, CA

REFERENCE:

BY:

DATE: May 1, 2018

SUBJECT:

APPENDIX A

SSK-02

F-2 Paleontological Resources Assessment

670 MESQUIT PROJECT, CITY OF LOS ANGELES, CALIFORNIA

Paleontological Resources Assessment Report

Prepared for
RCS VE LLC
319 Lafayette Street, Suite 133
New York, NY 10012

August 2020

670 MESQUIT PROJECT, CITY OF LOS ANGELES, CALIFORNIA

Paleontological Resources Assessment Report

Prepared for:
RCS VE LLC
319 Lafayette Street, Suite 133
New York, NY 10012

August 2020

Prepared by:
ESA

Principal Investigator and Report Author:
Alyssa Bell, Ph.D.

Project Manager:
Sara Dietler, B.A.

Project Location:
Los Angeles (CA) USGS 7.5 minute Topographic Quad; Township 1 South, Range 13 West, unsectioned

Acreage: Approx. 5.45 acres

Assessor Parcel Numbers: 5164-016-009; 5164-016-010; a portion of 5164-016-803; 5164-017-002; 5164-017-003; 5164-017-006; 5164-017-008; and 5164-018-009

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EXECUTIVE SUMMARY
670 Mesquit Project - Paleontological Resources Assessment Report

Environmental Science Associates (ESA) has been retained by RCS VE LLC (Applicant) to conduct a Phase I Paleontological Resources Assessment for an Environmental Impact Report (EIR), pursuant to the statutes of the California Environmental Quality Act (CEQA) for the 670 Mesquit Project (Project).

The Applicant proposes to construct a new mixed-use development (Project) totaling approximately 1,792,103 square feet of floor area on an approximately 5.45-acre property at 670 Mesquit Street in the Arts District of Downtown Los Angeles. The Project would include up to six levels of below-grade parking that spans the entire buildings' footprint and would include at-grade and above-grade parking within Building 5 at the southern end of the Project Site.

The Project Site flanks Mesquit Street on the east and west between the former 6th Street Viaduct ROW on the north and the 7th Street Bridge on the south. The majority of the Project Site is on the east side of Mesquit Street; with additional parcels located in the southern portion of the Project Site on the west side of Mesquit Street at 7th Street.

Construction would include approximately 531,319 cubic yards of grading (cut), all of which would be exported from the Project Site. The excavation depth would range from approximatively 61 to 68 feet below ground surface (bgs) for the lowest subterranean parking level. To accommodate elevator pits, maximum excavation depths would range from approximately 71 to 75 feet bgs in isolated areas.

The surficial geology of the Project Site consists of Quaternary Alluvium deposited within Holocene time (Dibblee and Ehrenspeck, 1989).[1] A paleontological records search was conducted for the Project by the Natural History Museum of Los Angeles County (LACM) on March 9, 2018. The results indicate no known fossil localities on the Project Site; however, older Quaternary Alluvium deposited during the Pleistocene epoch[2] can contain significant fossil vertebrate remains, and this alluvium is present in discontinuous areas throughout Downtown Los Angeles and east Los Angeles, including the subsurface of the Project Site. The three closest fossil localities in these sediments known to the LACM have been found between 1.74 and 2.13 miles from the Project Site and have produced fossil specimens of a variety of Ice Age animals

[1] Defined by the International Commission on Stratigraphy (ICS) as 11,700 years ago to the present (ICS, 2017).
[2] Defined by the ICS as 2,588,000 years ago to 11,700 years ago to the present (ICS, 2017).

such as mammoth and ground sloths at depths from 20 to 43 feet bgs. A review of geologic mapping and the scientific literature indicates that the surficial Quaternary Alluvium is too young to preserve fossil resources in the surface or shallow soils of the Project Site; however, the age of the sediments increases with depth and deeper layers may preserve fossil resources. Therefore, the sediments underlying the Project Site are characterized as having variable paleontological sensitivity[3], ranging from low to high, depending on the soil unit. ESA provides recommendations for paleontological impact mitigation in order to ensure that potential impacts remain less than significant. These recommendations are provided in the Conclusions and Recommendations section at the end of this report.

[3] The known potential to produce significant fossils.

INTRODUCTION AND PROJECT DESCRIPTION

Introduction

Environmental Science Associates (ESA) has been retained by RCS VE LLC (Applicant) to conduct a Phase I Paleontological Resources Assessment for an Environmental Impact Report (EIR), pursuant to the statutes of the California Environmental Quality Act (CEQA) for the 670 Mesquit Project (Project). The Applicant proposes to construct a new mixed-use development totaling approximately 1,792,103 square feet on an approximately 5.45-acre property at 670 Mesquit Street in the Arts District of Downtown Los Angeles (Project Site).

Project implementation would require the removal of all existing on-site uses, including warehouses containing freezers, coolers, dry storage, and associated office space, totaling approximately 205,393 gross square feet of floor area. New development would include creative office space (approximately 944,050 square feet); a 236-room hotel; 308 multi-family residential housing units; an Arts District Central Market, a grocery store, and general retail uses totaling approximately 136,152 square feet; restaurants totaling approximately 89,576 square feet; studio/event/gallery space and a potential museum totaling approximately 93,617 square feet; and a gym of approximately 62,148 square feet. Buildings would range between 84 feet to 378 feet tall. The resulting floor area ratio would be approximately 7.5:1, assuming the proposed Mesquit Street vacation.

The Project would provide open space for use by Project residents, hotel guests, employees, and visitors totaling approximately 141,876 square feet. Proposed open space features include at-grade landscaped areas, pedestrian passageways and walkways, viewing platforms, and above-grade landscaped terraces and pool decks.

The Applicant also seeks to construct a Deck over the railway properties if agreements can be obtained with Railway Property owners. The Deck would serve as a connection between the 7th Street Bridge and the Project Site's Northern Landscaped Area, which would provide access to the City's proposed PARC Improvements. The Deck could also provide access directly to the Los Angeles River.

The Project would include up to six levels of below-grade parking that spans the entire buildings' footprint and would include at-grade and above-grade parking in Building 5 at the southern end of the Project Site.

Construction would include approximately 531,319 cubic yards of grading (cut), all of which would be exported from the Project Site. The excavation depth would range from approximatively 61 to 68 feet below ground surface (bgs) for the lowest subterranean parking level. To accommodate elevator pits, maximum excavation depths would range from approximately 71 to 75 feet bgs in isolated areas.

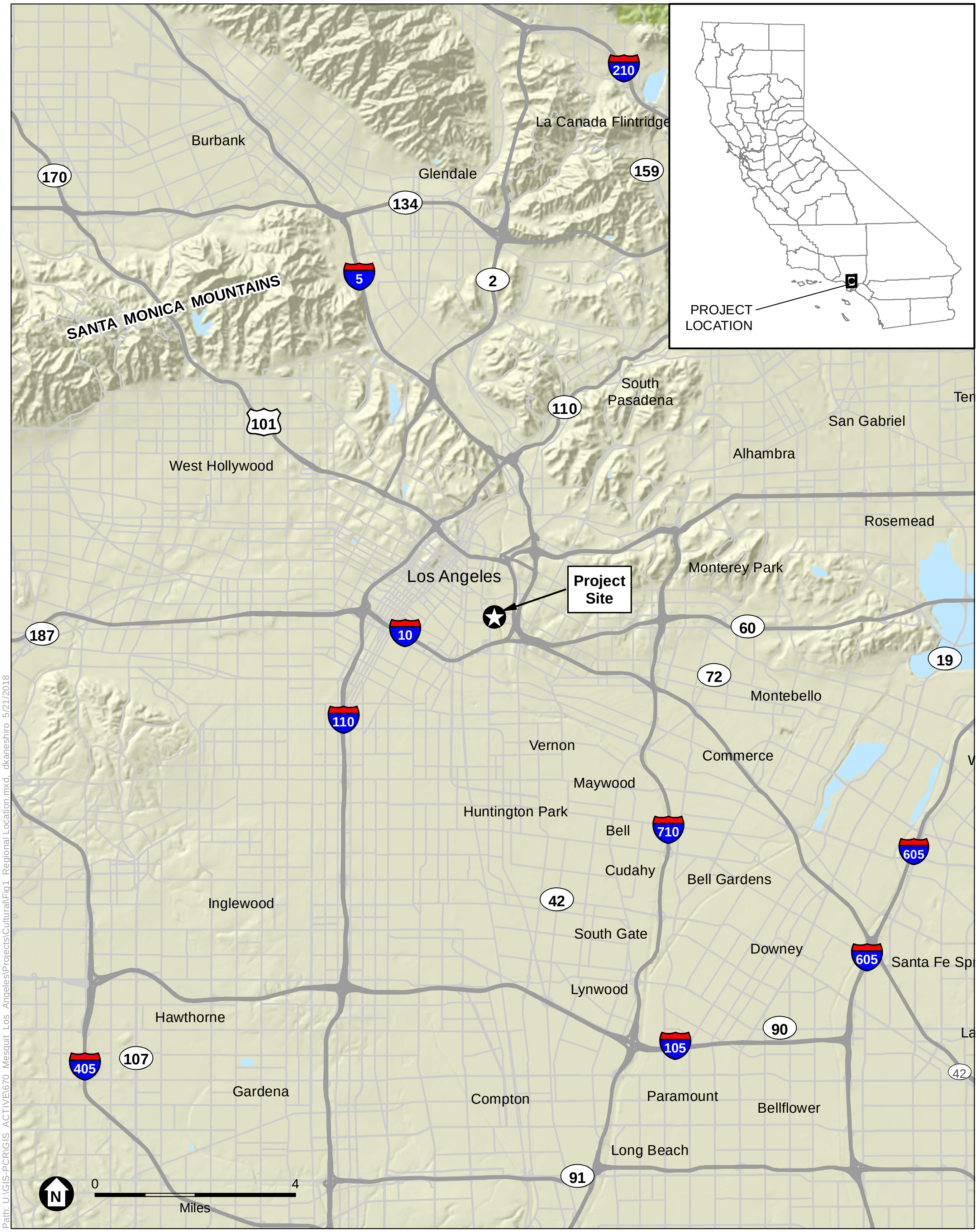
Project construction is anticipated to commence as early as 2021 and be completed as early as 2026, in a single phase, or as late as 2040 if built in separate phases over time. In the event construction is phased, construction of the underground parking may also be phased.

ESA personnel involved in the preparation of this report are as follows: Monica Strauss, M.A., Program Director, Sara Dietler, B.A., Project Manager, Alyssa Bell, Ph.D., report author and Principal Investigator; and Jessie Lee, GIS specialist. Resumes of key personnel are provided in **Appendix A**. The City of Los Angeles (City) is the lead agency for the Project, pursuant to the California Environmental Quality Act (CEQA).

Project Location

The 5.45-acre Project is located within the Central City North Community Plan area within the Arts District of Downtown Los Angeles (Project Site) (**Figure 1, Regional Location Map**). The Project Site flanks Mesquit Street on the east and west between the former 6^{th} Street Viaduct ROW on the north and the 7^{th} Street Bridge on the south. The majority of the Project Site is on the east side of Mesquit Street; with additional parcels located in the southern portion of the Project Site on the west side of Mesquit Street at 7^{th} Street. As part of the Project, Mesquit Street is proposed for vacation between 6^{th} and 7^{th} Streets. More generally, the Project is located in an unsectioned portion of Township 1 South and 2 South, Range 13 West on the Los Angeles USGS 7.5-minute topographic quadrangle (**Figure 2, Project Location Map**).

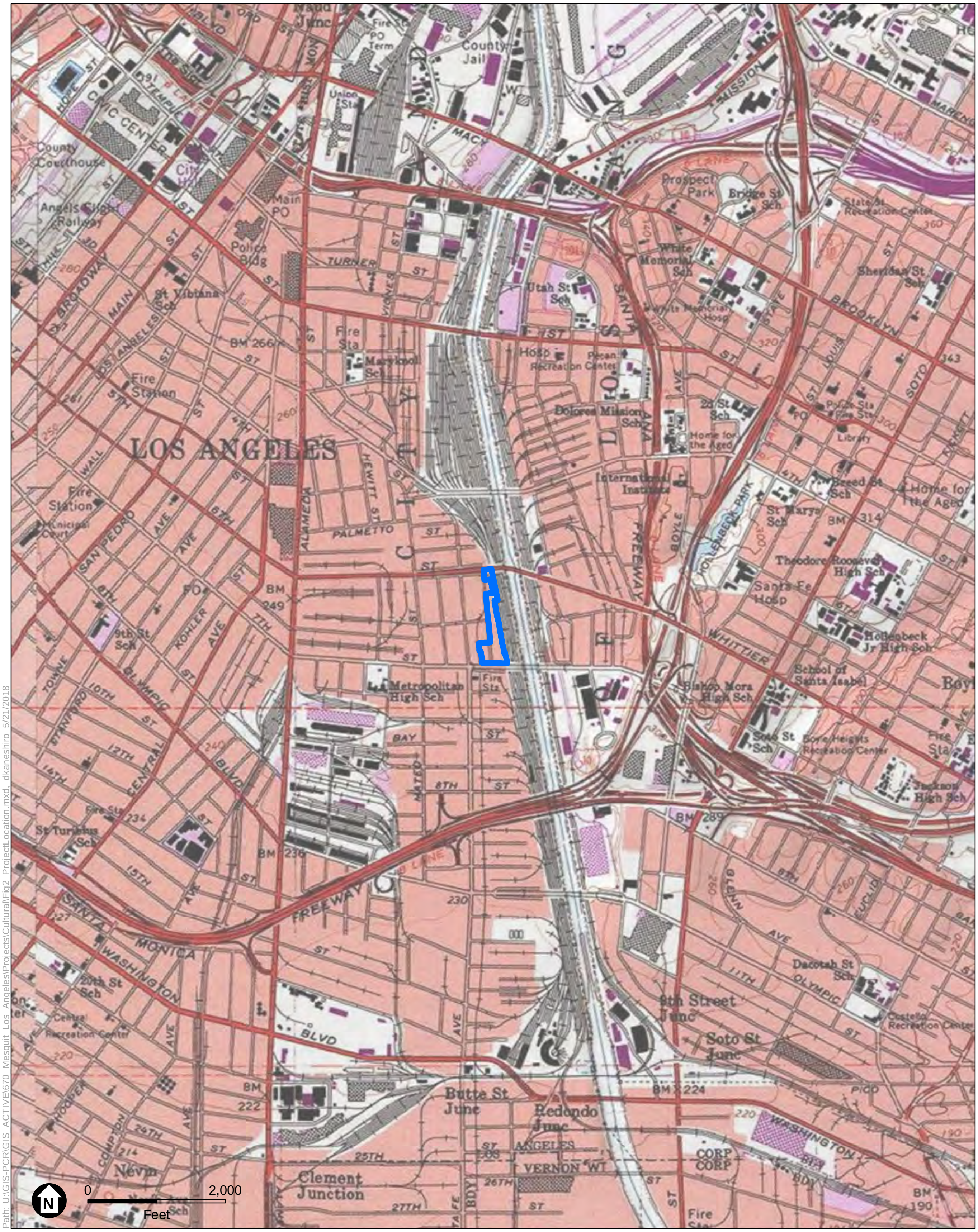
The Project Site is adjacent to property on both sides of Mesquit Street owned by the Los Angeles Department of Water and Power (LADWP) that houses the River Switching Station electricity substation and transmission line ROW (the LADWP Property), just south of 6^{th} Street.



SOURCE: ESRI

670 Mesquit

Figure 1
Regional Location



SOURCE: USGS Topographic Series (Los Angeles Quad).

670 Mesquit

Figure 2
Project Location

ESA

REGULATORY FRAMEWORK

State and Local Regulations

Paleontological resources are limited, nonrenewable resources of scientific, cultural, and educational value that are protected under state laws and regulations. The following section summarizes the applicable federal and state laws and regulations, as well as professional standards provided by the Society of Vertebrate Paleontology (SVP).

State Regulations

California Environmental Quality Act

The CEQA Guidelines (Title 14, Chapter 3 of the California Code of Regulations, Section 15000 *et seq*.), define the procedures, types of activities, individuals, and public agencies required to comply with CEQA. As part of CEQA's Initial Study process, one of the questions for lead agencies relates to paleontological resources: "Will the proposed project directly or indirectly destroy a unique paleontological resource or site or unique geologic feature?" (CEQA Guidelines Section 15023, Appendix G, Section XIV, Part a).

The loss of any identifiable fossil that could yield information important to prehistory, or that embodies the distinctive characteristics of a type of organism, environment, period of time, or geographic region, would be a significant environmental impact. Direct impacts to paleontological resources primarily concern the potential destruction of nonrenewable paleontological resources and the loss of information associated with these resources. This includes the unauthorized collection of fossil remains. If fossiliferous bedrock or surficial sediments are disturbed, the disturbance could result in the destruction of paleontological resources and subsequent loss of information (significant impact). At the project-specific level, the implementation of paleontological mitigation can mitigate direct impacts to a less than significant level.

The CEQA threshold of significance for a significant impact to paleontological resources is when a project is determined to "directly or indirectly destroy a significant paleontological resource or unique geologic feature." In general, where paleontologically sensitive geologic units underlie Project Sites, the greater the amount of ground disturbance, the higher the potential for significant impacts to paleontological resources. Where geologic units with no paleontological sensitivity directly underlie Project Sites, there is no potential for impacts on paleontological resources, unless sensitive geologic units which underlie the non-sensitive units are also affected.

Public Resources Code Section 5097.5 and Section 30244

Other state requirements for paleontological resource management are included in PRC Section 5097.5 and Section 30244. These statutes prohibit the removal of any paleontological site or feature from public lands without permission of the jurisdictional agency, define the removal of paleontological sites or features as a misdemeanor, and require reasonable mitigation of adverse impacts to paleontological resources from developments on public (state, county, city, or district) lands.

Local Regulations

City of Los Angeles – General Plan

The Conservation Element of the City of Los Angeles General Plan recognizes paleontological resources in Section 3: "Archeological and Paleontological" (II-3), specifically the La Brea Tar Pits, and identifies protection of paleontological resources as an objective (II-5). The General Plan identifies site protection as important, stating, "Pursuant to CEQA, if a land development project is within a potentially significant paleontological area, the developer is required to contact a bona fide paleontologist to arrange for assessment of the potential impact and mitigation of potential disruption of or damage to the site. If significant paleontological resources are uncovered during project execution, authorities are to be notified and the designated paleontologist may order excavations stopped, within reasonable time limits, to enable assessment, removal or protection of the resources" (City of Los Angeles, 2001).

City of Los Angeles CEQA Thresholds of Significance

The City of Los Angeles' CEQA Thresholds of Significance Guide (City of Los Angeles, 2006) Section D:1 specifies that the determination of significance for paleontological resources shall be made on a case-by-case basis, taking into consideration the following factors:

- Whether, or the degree to which, the project might result in the permanent loss of, or loss of access to, a paleontological resource; and
- Whether the paleontological resource is of regional or statewide significance. [City of Los Angeles, 2006]

Society for Vertebrate Paleontology Definitions and Guidelines

The SVP has established standard guidelines (SVP, 1995, 2010) that outline professional protocols and practices for conducting paleontological resource assessments and surveys, monitoring and mitigation, data and fossil recovery, sampling procedures, and specimen preparation, identification, analysis, and curation. Most practicing professional vertebrate paleontologists adhere closely to the SVP's assessment, mitigation, and monitoring requirements pursuant to the standard guidelines. Most state regulatory agencies with paleontological resource-specific laws, ordinances, regulations, and standards (LORS) likewise accept and use the professional standards set forth by the SVP.

The SVP also establishes definitions of paleontological resources and the fossiliferous deposits that may contain them, and provides guidance for determining the geographic extent of a given deposit that may be sensitive and the circumstances under which such resources might be disturbed and an impact occur. As defined by the SVP (1995:26), significant paleontological resources are:

> Fossils and fossiliferous deposits[,] here restricted to vertebrate fossils and their taphonomic and associated environmental indicators. This definition excludes invertebrate or paleobotanical fossils except when present within a given vertebrate assemblage. [However,] [c]ertain invertebrate and plant fossils may be defined as significant by a project paleontologist, local paleontologist, specialists, or special interest groups, or by lead agencies or local governments.

As defined by the SVP (1995:26), significant fossiliferous deposits are:

> A rock unit or formation which contains significant nonrenewable paleontologic resources, here defined as comprising one or more identifiable vertebrate fossils, large or small, and any associated invertebrate and plant fossils, traces, and other data that provide taphonomic, taxonomic, phylogenetic, ecologic, and stratigraphic information (ichnites and trace fossils generated by vertebrate animals, e.g., trackways, or nests and middens which provide datable material and climatic information). Paleontologic resources are considered to be older than recorded history and/or older than 5,000 years BP [before present].

Based on the significance definitions of the SVP (1995), all identifiable vertebrate fossils have scientific value and are therefore considered scientifically significant. This position is maintained because vertebrate fossils are relatively uncommon, and only rarely will a fossil locality yield a large number of specimens of the same genus; thus, abundance of fossils is not a requirement for designating a given rock unit as a significant fossiliferous deposit. Therefore, every vertebrate fossil found has the potential to provide important new scientific information regarding the taxon it represents, its paleoenvironment, and/or its distribution. Furthermore, all geologic units that have previously yielded vertebrate fossils are considered to have high sensitivity for the presence of fossils in the future. Identifiable plant and invertebrate fossils are considered significant if found in association with vertebrate fossils or if defined as scientifically significant by project paleontologists, specialists, or local government agencies.

A geologic unit known to contain scientifically significant fossils is considered “sensitive” to adverse impacts if there is a high probability that earth-moving or ground-disturbing activities in that rock unit will either directly or indirectly disturb or destroy fossil remains. Paleontological sites indicate that the associated sedimentary rock unit or formation is fossiliferous. Therefore, the known limits of the entirety rock unit or formation, both areal and stratigraphic, are considered to define the scope or extent of paleontological sensitivity in each case (SVP, 1995).

Fossils are contained within surficial sediments or bedrock, and are therefore not observable or detectable unless exposed by erosion or human activity. In summary, paleontologists cannot know either the quality or quantity of fossils prior to natural erosion or human-caused exposure. As a result, even in the absence of surface fossils, it is necessary to assess the sensitivity of rock units based on their known potential to produce significant fossils elsewhere within the same or similar geologic unit (both within and outside of the study area), or whether the unit in question was deposited in a favorable environment for fossil preservation. Monitoring by experienced paleontologists greatly increases the probability of fossil discovery during ground-disturbing activities and, if these remains are significant, successful mitigation and salvage efforts to prevent adverse impacts to these resources.

Paleontological Sensitivity

Paleontological sensitivity is the potential for a geologic unit to produce scientifically significant fossils. This is determined by rock type, past history of the geologic unit in producing significant fossils, and fossil localities recorded from that unit; for this reason, paleontological sensitivity depends on the known fossil data collected from the entire geologic unit, not just a specific survey. In its "Standard Guidelines for the Assessment and Mitigation of Adverse Impacts to Non-renewable Paleontologic Resources," the SVP (2010:1-2) defines four categories of paleontological sensitivity or, per the SVP guidelines, potential, for the presence of paleontological resources – high, low, undetermined, and no potential – as follows:

- **High Potential.** Rock units that have yielded vertebrate or significant invertebrate, plant, or trace fossils are considered to have a high potential for containing additional significant paleontological resources. Rocks units classified as having high potential for producing paleontological resources include, but are not limited to, (1) sedimentary formations and some volcaniclastic formations (e. g., ashes or tephras [rock fragments and particles from volcanic eruptions]), (2) some low-grade metamorphic rocks which contain significant paleontological resources anywhere within their geographical extent, (3) and sedimentary rock units temporally or lithologically suitable for the preservation of fossils. The latter includes middle Holocene and older, fine-grained fluvial sandstones, argillaceous (i.e., clay-bearing) and carbonate-rich paleosols (rock units representing former, now lithified, soils), cross-bedded point bar sandstones, fine-grained marine sandstones, etc.

- **Low Potential.** Some rock units have been concluded to contain low potential for yielding scientifically significant fossils, based on field survey findings reported reports in the paleontological literature by qualified professional paleontologists. These conclusions may be based on the fact that certain rock units are poorly represented by fossil specimens in institutional collections, leading to the determination that they are not generally fossil-bearing, or on general scientific consensus that a given rock unit only preserves fossils in rare circumstances and their presence of fossils is an exception in such units, not the rule, as in basalt flows or colluvium deposited during Holocene time. Rock units with low potential typically do not require impact mitigation measures to protect fossils.

- **Undetermined Potential.** Rock units for which little information is available concerning their paleontological content, geologic age, and depositional environment are considered to have undetermined potential. Further study is necessary to determine if these rock units have high or low potential to contain significant paleontological resources. A field survey by a qualified professional paleontologist to specifically determine the paleontological resource potential of these rock units is required before development of a paleontological resource impact mitigation program. In cases where no subsurface data are available, strategically located excavations into subsurface stratigraphy can determine paleontological potential.

- **No Potential.** Some rock units have no potential to contain significant paleontological resources. An example is high-grade metamorphic rocks, which have typically been distorted or recrystallized through intense processes of heat or other stresses (e.g., gneisses and schists). Likewise, plutonic igneous rocks such as granite are considered to have no potential to yield fossils, as they are formed from (liquid) magma that has dissolved the original rock matrix including any fossils it may once have contained. Rock units with no potential to yield fossils require no protections; no impacts are anticipated on such units and no mitigation is not required.

For geologic units with high potential, full-time monitoring is appropriate during any project-related ground disturbance because of the risk to paleontological resources. For geologic units with low potential, protection or salvage efforts is not generally required because of the low risk of encountering paleontological resources. For geologic units with undetermined potential, accepted professional practice recommends field surveys conducted by a qualified vertebrate paleontologist to specifically determine the paleontologic potential of the rock units present in the study area which in turn prescribes how mitigation measures should be assigned.

Paleontological Resources Significance Criteria

Numerous paleontological studies have developed criteria for the assessment of significance for fossil discoveries (Eisentraut and Cooper, 2002; Murphey and Daitch, 2007). In general, these studies assess fossils as significant if one or more of the following criteria apply:

1. The fossils provide information on the evolutionary relationships and developmental trends among organisms, living or extinct;

2. The fossils provide data useful in determining the age(s) of the rock unit or sedimentary stratum, including data important in determining the depositional history of the region and the timing of geologic events therein;

3. The fossils provide data regarding the development of biological communities or interaction between paleobotanical and paleozoological biotas;

4. The fossils demonstrate unusual or spectacular circumstances in the history of life; or

5. The fossils are in short supply and/or in danger of being depleted or destroyed by the elements, vandalism, or commercial exploitation, and are not found in other geographic locations.

As stated earlier, all indentifiable vertebrate fossils (and fossil assemblages) are considered scientifically significant. Fossils can include remains of large to very small aquatic and terrestrial vertebrates or remains of plants and animals previously not represented in certain portions of the stratigraphy. Fossil assemblages might aid stratigraphic correlation, particularly those offering data for the interpretation of tectonic events, geomorphologic evolution, and paleoclimatology, and are therefore also critically important (Scott and Springer 2003; Scott et al. 2004).

METHODS AND RESULTS

Archival Research

The Project Site was the subject of thorough background research and analysis. Research included a paleontological records search from the Natural History Museum of Los Angeles County (LACM) as well as geologic map and literature reviews. The Project Site is also the subject of a geotechnical study (Twining Consulting, 2017).

Geologic Setting

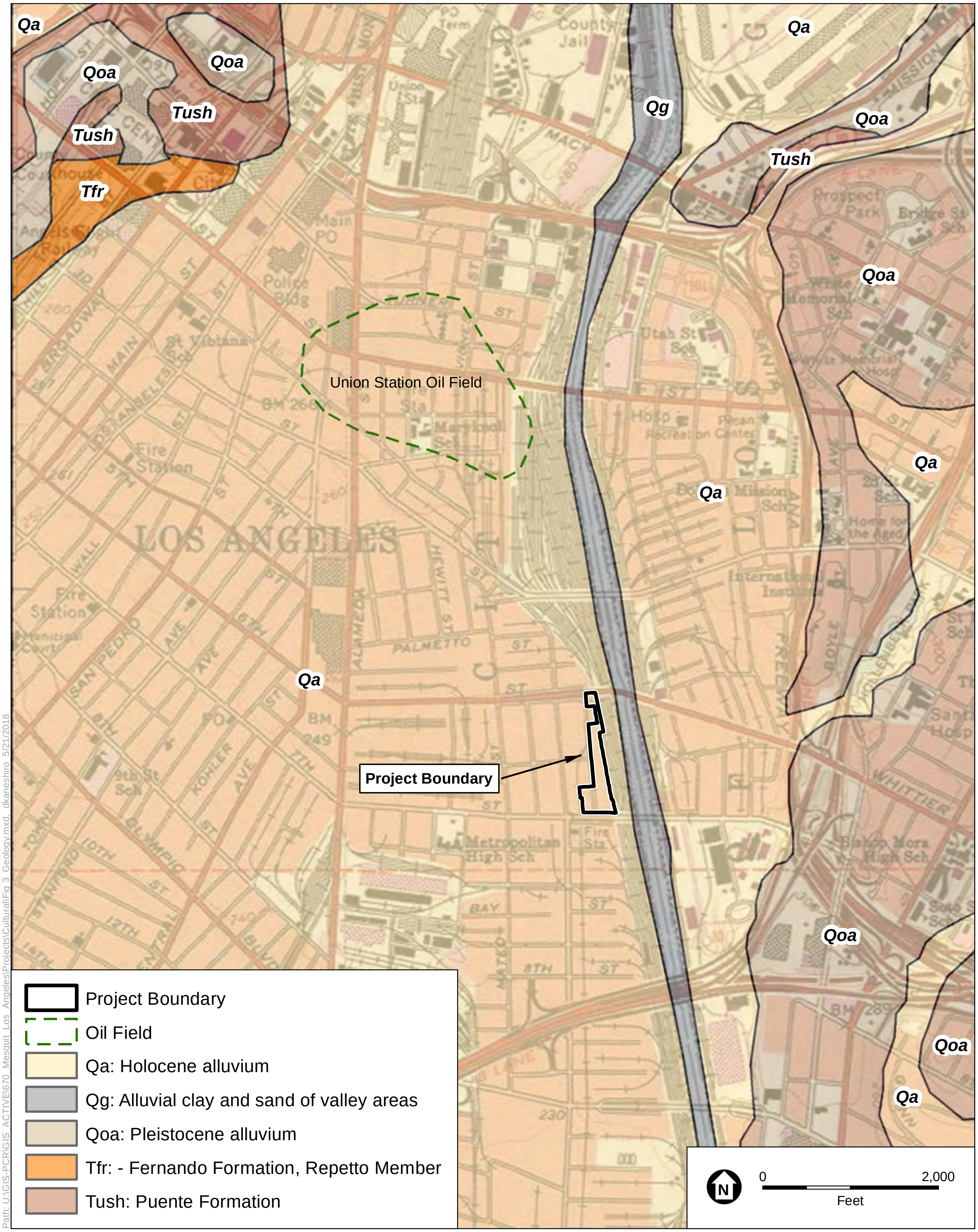
The Project Site is located in the Los Angeles Basin, a structural depression approximately 50 miles long and 20 miles wide in the northernmost Peninsular Ranges Geomorphic Province (Ingersoll and Rumelhart, 1999). The Los Angeles basin developed as a result of tectonic forces and the San Andreas fault zone, with subsidence occurring between 18 and three million years ago (Mya) (Critelli et al., 1995). While sediments dating back to the Cretaceous (66 million years ago) are preserved in the basin, continuous sedimentation began in the middle Miocene (around 13 million years ago) (Yerkes et al., 1965). Since that time, sediments have eroded into the basin from the surrounding highlands, resulting in thousands of feet of accumulation (Yerkes et al., 1965). Most of these sediments are marine, until sea level dropped in the Pleistocene and deposition of the alluvial sediments that compose the uppermost units in the Los Angeles Basin began.

The Los Angeles Basin is subdivided into four structural blocks, with the Project Site occurring in the Central Block, where sediments range from 32,000 to 35,000 feet thick (Yerkes et al., 1965). The Central Block is wedge-shaped, extending from the Santa Monica Mountains in the northwest, where it is about 10 miles wide, to the San Joaquin Hills to the southeast, where it widens to approximately 20 miles across (Yerkes et al., 1965).

Geologic Map & Literature Review

Geologic mapping by Dibblee and Ehrenspeck (1989) indicates that[4] Quaternary Alluvium deposited during Holocene time covers the surface of the Project Site (mapped as Qa in **Figure 3. Geology**). During mid- to late Holocene time, before the river was channelized, the floodplain of the Los Angeles River received deposition of the alluvial sediments, which consist of well-sorted silts and sands interbedded with stream channel deposits of sands and gravels (Dibblee and Ehrenspeck, 1989). Geotechnical analysis of the Project Site indicates that artificial fill appears to

[4] Defined by the International Commission on Stratigraphy (ICS) as 11,700 years to the present (ICS, 2017).



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SOURCE: Dibblee Geological Foundation

670 Mesquit

Figure 3
Geology

ESA

be shallow, to depths of 5-6 feet below ground surface (bgs) recorded in borings at the site (Twining Consulting, 2017).

Below the artificial fill is younger Quaternary Alluvium, deposited in Holocene times, as mapped by Dibblee and Ehrenspeck (1989). At greater depths, Pleistocene-aged older Quaternary Alluvium (mapped as Qoa in **Figure 3**) and the Pliocene-aged Fernando Formation (mapped as Tfr **Figure 3**) underlie the surficial Holocene Quaternary Alluvium. The nearest outcrops of older alluvium to the Project Site are just east of the US-101 (Hollywood) Freeway, roughly 0.7 miles away (Dibblee and Ehrenspeck, 1989). The Fernando Formation is a marine and nonmarine semi-friable, massively bedded sandstone that crops out near the intersection of Broadway Avenue and 1st Street, roughly one mile from the Project Site (Dibblee and Ehrenspeck, 1989). The Fernando Formation occurs between 100-150 feet bgs in the Project Site (Yerkes et al., 1965).

Geotechnical analysis to determine the depth at which younger alluvium transitions to older alluvium has not been determined within the Project Site (Twining Consulting, 2017). Geotechnical analysis is not always able to identify a clear division between newer and older alluvium because of limited sample size (i.e., number of borings), potential historic disturbance (i.e. chaotic flood deposits) of the alluvial layers that may prevent clear stratification, interfingering of layers (i.e., old and new layers intermixed), and the fact that core samples were not dated, as that was beyond the scope of work for this geotechnical analysis. To the northwest and north of the Project Site, along the US-110 (Harbor) Freeway and US-101, a study correlating well and boring logs found that the depths of the older alluvium are highly variable, ranging from 10 and 200 feet bgs (Yerkes et al., 1965).

The Holocene-aged Quaternary Alluvium is relatively recent in age in the upper layers and therefore is not old enough to contain fossil remains, which the SVP defines as over 5,000 years old (SVP, 2010). However, these sediments increase in age with depth, such that while the surficial sediments are too young to preserve fossils, the underlying older Quaternary Alluvium dates to the late Holocene or Pleistocene and therefore may preserve fossil resources. These sediments have a rich fossil history in Los Angeles (Brattstrom and Sturn, 1959; Steadman, 1980) and throughout southern California (Jefferson 1991a and b; Miller 1971; Scott and Cox 2008). The most common fossils include the bones of mammoth, bison, horse, lion, cheetah, wolf, camel, antelope, peccary, mastodon, capybara, and giant ground sloth, as well as small animals such as rodents and lizards (Graham and Lundelius, 1994). In addition to illuminating the differences between Southern California in the Pleistocene and today, this abundant fossil record has been vital in studies of extinction (e.g. Sandom, et al., 2014; Scott, 2010), ecology (e.g. Connin et al., 1998), and climate change (e.g., Roy et al., 1996).

Natural History Museum of Los Angeles County Records Search

On February 23, 2018 ESA requested a database search of the LACM collection[5] for records of fossil localities on and around the Project Site.[6] The purpose of the LACM records search was to: (1) determine whether any previously recorded fossil localities occur in the Project Site, (2) assess the potential for disturbance of these localities during construction, and (3) evaluate the paleontological sensitivity in the Project Site. The records search returned no known localities on the Project Site; however, similar sedimentary deposits in Downtown Los Angeles have yielded a number of vertebrate fossils (McLeod, 2018).

The closest fossil locality on record at the LACM is approximately 1.74 miles west of the Project Site, where a fossil horse (*Equus*) was recovered from 43 feet bgs (McLeod, 2018). Approximately 1.89 miles northeast of the Project Site, fossil specimens of pond turtle, (*Clemmys mamorata*), ground sloth (*Paramylodon harlani*), mastodon (*Mammut americanum*), mammoth (*Mammuthus imperator*), horse (*Equus*), and camel (*Camelops*) were recovered from a depth of 20-35 feet bgs (McLeod, 2018). Just north of that locality, 2.13 miles northeast of the Project Site, excavations for a storm drain recovered fossil specimens of turkey (*Meleagris californicus*), sabre-toothed cat (*Smilodon fatalis*), horse (*Equus*), and deer (*Odocoileus*) at an unstated depth (McLeod, 2018).

The results of the database search are provided in **Appendix B**.

Paleontological Sensitivity Analysis

Based on the review of the scientific literature and geologic mapping and the records search from the LACM, ESA assigned paleontological sensitivities to the geologic units present at the surface or in the subsurface of the Project Site. The assignment of sensitivity follows the guidelines of the SVP (1995, 2010).

- **Quaternary younger Alluvium (Qa) –** Surficial sediments; **low to high sensitivity**, increasing with depth. While the shallow layers of this unit are too young to preserve fossil resources (i.e., <5,000 years old), these sediments increase in age with depth and may preserve fossils in deeper layers. These potential fossils include a wide variety of Ice Age animals, as reviewed above.

- **Quaternary older Alluvium (Qoa) –** Subsurface, **high sensitivity.** These sediments have yielded a wide variety of Ice Age fossils across the Los Angeles Basin, as discussed

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5 LACM is the official repository for paleontological resources in Los Angeles County and the research standard for universities, colleges, and professionals in the Southern California region.

6 The precise locations of paleontological sites are not provided by the LACM; record searches only return the general localities of the closest recorded sites. This is the established practice because cultural resources are nonrenewable, and their scientific, cultural, and aesthetic values can be significantly impaired by disturbance. To deter vandalism, artifact hunting, and other activities that can damage cultural resources, the locations of paleontological and cultural resources are confidential.

above, including multiple specimens belonging to ten taxa within 1.74 miles of the Project Site (McLeod, 2017).

As previously stated, the exact depth at which the alluvium becomes old enough to preserve fossils (i.e., >5,000 years old) is unknown at the Project Site. The closest study to identify the depth of this transition correlated well and boring logs from northwest and north of the Project Site, along US-110 and US-101 in downtown Los Angeles (Yerkes et al., 1965). This study found that the depth to older alluvial sediments was highly variable, ranging from 10 to 200 feet bgs (Yerkes et al., 1965). The LACM records search indicated fossil recovery at depths of as little as 20 feet bgs in the area (McLeod, 2018). Given the lack of definitive information on the depth of the transition to high sensitivity sediments at the Project Site, an estimated depth of 10 feet bgs is assumed, using the depths from Yerkes et al. (1965) and the LACM fossil localities (McLeod, 2018).

It should be noted that while the older Fernando Formation is present in the subsurface of the Project Site, it occurs between 100-150 feet bgs in the area (Yerkes et al., 1965), and therefore would not be impacted by construction activities associated with the Project, which are expected to only extend down to a maximum of 63 feet bgs in isolated areas.

CONCLUSIONS AND RECOMMENDATIONS

This study concluded that the surficial sediments underlying the Project Site, identified as younger Quaternary Alluvium, have low paleontological sensitivity as they are too young to preserve fossils, and occur to an undetermined depth in the Project Site. However, the Late Holocene-Pleistocene older Alluvium, present at an undetermined depth in the subsurface of the Project Site, has high paleontological sensitivity. Based upon the known depth to the older Alluvium north and northwest of the Project Site (as little as 10 feet bgs; Yerkes et al., 1965) and the depth at which fossils have been found within 1.74 to 2.13 miles of the Project Site (as little as 20 feet bgs; McLeod, 2018), it is estimated that the transition from low to high sensitivity sediments could occur at or around 10 feet bgs in the vicinity of the Project Site and on the Project Site itself. The Project proposes deep excavation and excavation shoring during the construction of subterranean parking structures, building foundations, and infrastructure and utility improvements (e.g., sewer, electrical, water, and drainage systems) at depths that could impact older Alluvium with a high sensitivity for fossils.

The recommendations would reduce impacts to unique paleontological resources or unique geological feature to a less than significant level:

1. A qualified paleontologist meeting the Society of Vertebrate Paleontology (SVP) Standards (SVP, 2010) (Qualified Paleontologist) shall be retained prior to the approval of demolition or grading permits. The Qualified Paleontologist shall provide technical and compliance oversight of all ground-disturbing activities (e.g., clearing, grading and excavation) that relate to paleontological resources, shall attend the Project kick-off meeting and any construction progress meetings, and shall report to the Project Site in the event potential paleontological resources are encountered in order to assess the significance of the discovery and determine appropriate documentation and/or salvage.

2. The Qualified Paleontologist shall conduct construction worker paleontological resources sensitivity training prior to the start of ground-disturbing activities (including vegetation removal, pavement removal, etc.), in accordance with SVP Standards (SVP, 2010). In the event construction crews are phased, additional trainings shall be conducted for new construction personnel. The training session shall focus on recognition of the types of paleontological resources that could be encountered within the Project Site and the procedures to be followed if they are found. Documentation shall be retained demonstrating that all construction personnel attended the training.

3. Full-time paleontological resources monitoring shall be conducted for all ground-disturbing activities in previously undisturbed sediments that exceed 10 feet in depth, and are, therefore, likely to impact high-sensitivity older Alluvial sediments. The

surficial Alluvium has low paleontological sensitivity, and, therefore, work in the upper 10 feet of the Project Site does not need to be monitored. The Qualified Paleontologist shall spot-check the excavation on an intermittent basis and recommend revision of the depth of required monitoring based on his/her observations. The frequency of spot-checks shall be determined based on the pace of excavations, both vertically and laterally. Paleontological resources monitoring shall be performed by a qualified paleontological monitor (meeting the standards of the SVP, 2010) under the direction of the Qualified Paleontologist. Full-time monitoring can be reduced to part-time inspections or ceased entirely if determined adequate by the qualified paleontologist. Monitors shall have the authority to temporarily halt or divert work away from exposed fossils in order to recover the fossil specimens. Any significant fossils that could yield information important to prehistory, or that embody the distinctive characteristics of a type of organism, environment, period of time, or geographic region, collected during Project-related excavations shall be prepared to the point of identification and curated into an accredited repository with retrievable storage. Monitors shall prepare daily logs detailing the types of activities and soils observed, and any discoveries. The Qualified Paleontologist shall prepare a final monitoring and mitigation report to document the results of the monitoring effort, and shall provide the final report to the Department of City Planning.

4. If construction or other Project personnel discover any potential fossils during construction, regardless of the depth of work or location, work at the discovery location shall cease within a 50-foot radius of the discovery until the Qualified Paleontologist has assessed the discovery and made recommendations as to the appropriate treatment. If the find is deemed significant, it shall be salvaged following the standards of the SVP and curated with a certified repository. If there are significant discoveries, fossil locality information and final disposition will be included within the final report which will be submitted to the appropriate repository and the Department of City Planning.

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APPENDIX A

Personnel Qualifications

Alyssa Bell, PhD

Paleontologist

EDUCATION

Ph.D., Vertebrate Paleontology; University of Southern California

M.S., Environmental Microbiology; University of Tennessee

B.A. with honors, Ecology and Systematics; William Jewell College & Homerton College, Cambridge University

10 YEARS EXPERIENCE

Dr. Alyssa Bell has supervised and peformed field work, authored project reports, and provided scientific and compliance direction and quality control for paleontological projects throughout Southern California. Dr. Bell has accumulated a wealth of field experience, working with crews from a variety of institutions on field sites in California, Arizona, New Mexico, South Dakota, and Utah, and has led her own expeditions in Montana. She has performed all manner of investigations from surveys and assessments to monitoring and fossil idenfitication over the last 15 years as a part of her academic pursuits and professional consultation, with the last three years being exclusively professional endeavors.

In addition to consulting, Dr. Bell serves as a postdoctoral fellow at the Dinosaur Institute of the Natural History Museum of Los Angeles County (LACM). There she is involved in pursuing her own research into fossil birds as well as working with the Institute's field projects and museum-wide education and outreach initiatives. She has also published peer-reviewed articles and book chapters and given numerous presentations at scientific conferences on both her paleontological and microbiological research.

Relevant Experience

ICHA Area 10 (PA 10-2 & 10-4) Archaeological and Paleontological Monitoring, Irvine, CA. *Principal Investigator & Project Paleontologist.* Dr. Bell managed the curatorial process for fossils collected during monitoring of pre-construction activities at the University of California, Irvine, and authored the final report.

Suncrest Reactive Power Support Project, San Diego County, CA. *Principal Investigator.* Dr. Bell authored the paleontological assessment for the Proponent's Environmental Assessment (PEA) in support for a dynamic reactive power support facility and associated 230-kilovolt (kV) transmission line near Alpine, California. The application for Certificate of Public Convenience and Necessary was filed in summer 2015 and the PEA was deemed complete in December 2015.

Washington National Archaeological and Paleontological Monitoring (Access Culver City), Culver City, CA. *Principal Investigator & Project Paleontologist.* Dr. Bell managed the curatorial process for fossils collected during monitoring of pre-construction activities at the Washington national site in Culver City, CA and authored the final report.

OTO Hotels Santa Monica Archaeological and Paleontological Service, Santa Monica, CA. *Principal Investigator.* Dr. Bell supervised paleontological monitoring and mitigation services during construction excavations and grading. Services included implementation of a paleontological mitigation monitoring program and reporting.

Sacred Heart Specific Plan Environmental Impact Report (EIR), La Canada Flintridge, CA. *Principal Investigator.* Dr. Bell prepared paleontological studies and developed monitoring & mitigation recommendations for the Sacred Heart development project.

Sixth & Bixel Paleontological Monitoring Services Project, Los Angeles, CA. *Principal Investigator & Project Paleontologist.* Dr. Bell supervised paleontological monitoring of preconstruction activities in support of a development project encompassing two parcels in downtown Los Angeles. During these activities, monitors identified and recovered numerous significant vertebrate fossils. Dr. Bell supervised the excavation of fossilized whale remains discovered on-site, and oversaw the collection and curation of all fossil specimens.

Natural and Cultural Support for the Gordon Mull Subdivision EIR, Glendora, CA. *Principal Investigator.* Dr. Bell collected the necessary data to prepare the technical sections and mitigation recommendations to support an EIR prepared by another firm to address the Gordon Mull Subdivision in the city of Glendora. The project is proposes to redevelop a 71-acre, 19-lot located in the San Gabriel Foothills.

Lake Elsinore Lakeshore Town Center Permitting, Riverside County, CA. *Principal Investigator.* Dr. Bell provided paleontological studies and developed monitoring and mitigation recommendations for the Lake Elsinore Town Center project in Riverside County.

San Pedro Plaza Park - Phase III Archaeological Monitor, Los Angeles, CA. *Principal Investigator.* Dr. Bell identified fossils during the mitigation measurement-required archaeological monitoring of earthmoving activities in San Pedro Park Plaza. She is also responsible for curation of the fossil material and authorship of the paleontological section of the final report.

City of Hope Specific Plan and EIR, Duarte, CA. *Principal Investigator.* Dr. Bell provided paleontological resource studies for the City of Hope Specific Plan Project.

Blythe Solar Power Project, Units 1 & 2, Riverside County, CA. *Project Paleontologist.* Dr. Bell supervised paleontological monitoring of preconstruction activities for a solar photo-voltaic cell power-generating facility outside the city of Blythe. As a part of her role, she provided oversight and management of paleontological monitors and development of the final monitoring report.

Industrial Project Environmental Impact Report, Colton, CA. *Principal Investigator.* Dr. Bell provided a paleontological resources study for a six-acre industrial project site at the southwest corner of Agua Mansa Road and Rancho Avenue in the city of Colton.

Mojave Solar Project Paleontological Reporting, San Bernardino County, CA. *Principal Investigator.* Dr. Bell managed curation of fossil materials and authored the final report of paleontological monitoring services provided for construction activities in support of a solar field development project in San Bernardino County.

El Camino Real Bridge Replacement Environmental Services, Atascadero, CA. *Principal Investigator.* Dr. Bell provided environmental services, including preparation of all California Environmental Quality Act (CEQA)/National Environmental Policy Act (NEPA) documentation, technical studies, and permitting, for the replacement of the El Camino Real Bridge over Santa Margarita Creek in Atascadero.

Recycled Water Transmission Water Main Paleo Monitoring, Fresno, CA. *Principal Investigator.* Dr. Bell developed a monitoring and mitigation plan for the city of Fresno recycled water main construction project.

Shafter Wasco Irrigation District Natural and Cultural Resource Evaluations and Air Quality, Kern County CA. *Principal Investigator.* Dr. Bell provided paleontological studies and developed recommendations for the monitoring and mitigation of paleontological resources for the project.

Valentine EIR, Kern County, CA. *Principal Investigator.* Dr. Bell provided paleontological resources support for a 2,000-acre solar PV project in the Mojave Desert. Deliverables included comprehensive technical reports, GIS impact analysis, strategic and permitting support, and a paleontological field survey in the preparation of an EIR and other permitting requirements.

Valentine Solar EIR 115MW Supplemental Reports, Kern County, CA. *Principal Investigator.* Dr. Bell provided paleontological studies in support of changes to the previously established Valentine Solar project.

Valentine Solar Biological and Paleontological Study Updates, Rosamond, Kern County, CA. *Principal Investigator & Project Paleontologist.* Dr. Bell provided paleontological studies, carried out a paleontological survey, and developed monitoring and mitigation guidelines for the Valentine Solar project.

Field Research

2006-Present. The Dinosaur Institute, LACM. Coordinator and Team Leader on expeditions in Montana (Niobrara and Pierre Shale Formations) and Arizona (Chinle Formation). Field assistant on expeditions to Montana (Hell Creek Formation), Utah (Morrison Formation), Arizona (Chinle Formation), New Mexico (Kirtland Formation), and California (Aztec Sandstone). During this period approximately four-six weeks are spent in the field in various locations every year.

2015. Principal Investigator, Field Manager. SWCA Environmental Consultants. Supervision of all paleontological field work, including excavation of a partial whale fossil from a downtown Los Angeles construction site and numerous monitoring projects.

2014. University of Southern California. Field Assistant on an expedition to South Africa (Pre-Cambrian).

2005. Cambridge University. Field Assistant on an expedition in Badlands National Park, South Dakota (White River Group).

2002-2004. Montana State University Northern. Field Assistant on excavations in Montana (Judith River Formation).

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APPENDIX B

Record Search Results (Confidential)