Geotechnical Investigation Report

Culver City High School Press Box
4401 Elenda Street
Culver City, CA 90230

Prepared for:

Harris & Associates
One California Plaza
300 S Grand Avenue, Suite 3830
Los Angeles, CA 90071

December 8, 2022 (Revised December 14, 2022)
Project No.: 220695.1

TWINING CONSULTING

18071 Mt. Washington St
Unit A
Fountain Valley, CA 92708

Tel 949.553.0370
Fax 949.553.0371

December 8, 2022 (Revised December 14, 2022)
Project No.: 220695.1

Mr. Sean Dunbar
Vice President
Measure CC Bond Program Manager
Harris & Associates
One California Plaza
300 S Grand Avenue, Suite 3830
Los Angeles, CA 90071

Subject: **Geotechnical Investigation Report**
Culver City High School Press Box
4401 Elenda Street
Culver City, CA 90230

Dear Mr. Dunbar,

In accordance with your request and authorization, we are presenting the results of our geotechnical investigation for the proposed Culver City High School Press Box project located at 4401 Elenda in Culver City, California. The purpose of our investigation is to characterize subsurface conditions of the site, evaluate seismic and geologic hazards at the site, and provide geotechnical engineering recommendations for the proposed improvements, including recommendations for foundations and earthwork.

This report was prepared in accordance with the requirements of the 2022 California Building Code (2022 CBC), ASCE 7-16 (ASCE, 2017), and California Geological Survey (CGS) Note 48 (CGS, 2019). Based on our findings, the proposed project is geotechnically feasible, provided that the recommendations in this report are incorporated into the design and are implemented during construction of the project.

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

Respectfully submitted,
TWINING CONSULTING, INC.

REGISTERED PROFESSIONAL ENGINEER
LIANGCAI HE
GE 3033
EXP. 12/31/2024
GEOTECHNICAL
STATE OF CALIFORNIA

PROFESSIONAL GEOLOGIST
JONATHAN A. BROWNING
No. 2615
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Liangcai He, PhD, PE 73280, GE 3033
Chief Geotechnical Engineer

Jonathan Browning, PG 9012, CEG 2615
Engineering Geologist

TWINING CONSULTING

18071 Mt. Washington St
Unit A
Fountain Valley, CA 92708

Tel 949.553.0370
Fax 949.553.0371

TABLE OF CONTENTS

Page

18071 Mt. Washington St
Unit A
Fountain Valley, CA 92708

Tel 949.553.0370
Fax 949.553.0371

Figures

Appendices

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18071 Mt. Washington St
Unit A
Fountain Valley, CA 92708

Tel 949.553.0370
Fax 949.553.0371

1. INTRODUCTION

This report presents the results of the geotechnical investigation performed by Twining Consulting, Inc. (Twining) for the proposed Culver City High School Press Box project located at 4401 Elenda Street in Culver City, California. A description of the site and the proposed improvements is provided in the following section. The objectives of this investigation have been to characterize subsurface conditions of the site, evaluate seismic and geologic hazards at the site, and provide geotechnical recommendations for design and construction of the proposed development, including recommendations for foundations and earthwork. Our investigation was performed in conformance with the 2022 California Building Code (2022 CBC), ASCE 7-16 (ASCE, 2017), and California Geological Survey (CGS) Note 48 (CGS, 2019).

2. SITE DESCRIPTION AND PROPOSED DEVELOPMENT

The project site is located at 4401 Elenda Street in Culver City, California, as shown on Figure 1 – Site Location Map. The approximate project site coordinates are latitude 34.003560°N and longitude 118.402685°W, on the Beverly Hills, California 7½-Minute Quadrangle, according to the United States Geological Survey (USGS) topographic maps (USGS 2022). The project site is currently occupied by a dirt covered area located behind the southwestern bleachers of the athletic sports field on the south portion of the school campus. The surface elevation of the site is approximately 42 feet above mean sea level (msl).

Based on project plans and information provided to us, the proposed project will consist of the construction of a new Press Box structure with a stairway. The footprint of the proposed structure is anticipated to be approximately 240 square feet. Storm water infiltration BMPs are not anticipated. It is anticipated that minimal grading will be required to achieve desired grades for the project.

The locations and the approximate footprint of the proposed construction are depicted on Figure 2 – Site Plan and Boring Location Map.

3. SCOPE OF WORK

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

3.1. Literature Review

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

3.2. Pre-Field Activities

Before starting our exploration program, we performed a geotechnical site reconnaissance to observe the general surficial conditions at the site and to select field exploration locations. After exploration locations were delineated, Underground Service Alert was notified of the planned locations a minimum of 72 hours prior to excavation.

18071 Mt. Washington St
Unit A
Fountain Valley, CA 92708

Tel 949.553.0370
Fax 949.553.0371

3.3. Field Exploration

The field exploration consisted of drilling, testing, sampling, and logging of 2 exploratory borings (B-1 and B-2). The approximate exploration locations are shown on Figure 2 – Site Plan and Boring Location Map.

The borings were advanced to approximate depths of 31 to 51.5 feet below ground surface (bgs) using a 6-inch-diameter hollow-stem-auger (HSA) on a track-mounted drill rig. The exploration locations were first excavated to approximately 5 feet bgs using a hand-auger to clear potential underground utilities and then switched to HSA drilling.

Drive samples of the soils were obtained from the borings using a Standard Penetration Test (SPT) sampler without room for liner and a modified California split-spoon sampler. Bulk samples were collected from the upper 5-foot soil cuttings. The samples were transported to Twining's geotechnical engineering laboratory in Long Beach, California for examination and testing.

Detailed descriptions of the field exploration are presented in Appendix A – Field Exploration.

3.4. Geotechnical Laboratory Testing

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

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

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

3.5. Engineering Analyses and Report Preparation

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

- Site geology, and subsurface conditions,
- Groundwater conditions,
- Geologic hazards and seismic design parameters,
- Liquefaction potential and seismic settlement,
- Soil corrosion potential,
- Soil collapse and expansion potential,
- Site preparation and earthwork,
- Project feasibility and suitability of on-site soils for foundation support,

- Foundation design parameters including bearing capacity, settlement, and lateral resistance,
- Concrete slab-on-grade support,
- Modulus of subgrade reaction for concrete slab-on-grade design, and
- Temporary excavations.

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

4. GEOLOGY AND SUBSURFACE CONDITIONS

The site geology and subsurface conditions are described in this section, based on our data review and field investigation. A portion of the geologic map is reproduced as Figure 3 – Geologic Map. A cross section illustrating the geologic conditions at the site is presented on Figure 4 – Geologic Cross Section A-A'. Detailed subsurface conditions are presented in Appendix A – Field Exploration.

4.1. Geology

According to Geologic Map of the Beverly Hills and Van Nuys (South ½) Quadrangles prepared by Dibblee and Ehrenspeck (Dibblee, 1991), the site is underlain by alluvial gravel, sand, and clay (Qa). According to Dibblee, the alluvium is "derived mostly from Santa Monica Mountains and includes gravel and sand of stream channels." A portion of the map is depicted in Figure 3 – Geologic Map.

4.2. Surface and Subsurface Conditions

As described earlier, the site is currently occupied by a dirt covered area. In general, the site is underlain by approximately one foot of artificial fill soils consisting of lean clay with sand. Underlying the fill soil, alluvium materials were encountered to the maximum depth of exploration of 51.5 feet bgs. In general, the alluvium consisted of very stiff lean clay with sand to a depth of approximately 10 feet bgs and transitions to dense to very dense poorly graded sand with silt and gravel and poorly graded sand to the maximum depth of exploration at approximately 31 feet bgs in exploratory boring B-1 and to approximately 35 feet bgs in boring B-2. Below 35 feet bgs in boring B-2, the alluvium generally consists of very stiff to hard sandy lean clay to the maximum depth of exploration of 51.5 feet bgs.

Detailed descriptions of the soils encountered during drilling are presented in Appendix A – Field Exploration.

4.3. Groundwater

Groundwater was encountered at a depth of approximately 42 feet bgs in exploratory boring B-2. Based on Seismic Hazard Zone Report prepared for the area by the California Geologic Survey, the historic high groundwater elevation at the site is approximately 10 feet bgs.

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

5. GEOLOGIC HAZARDS AND SEISMIC DESIGN CONSIDERATIONS

Geologic hazards and seismic effects at the project site are discussed in the following sections.

18071 Mt. Washington St
Unit A
Fountain Valley, CA 92708

Tel 949.553.0370
Fax 949.553.0371

5.1. Historical Seismicity

The recorded history of earthquakes prior to the seismograph is sparse and inconsistent. The oldest seismographs (or recordable earthquake devices) originated in Italy in the mid-1800s. The modern seismograph was developed in Japan in 1880. Electromagnetic seismometers (calibrated seismographs) were developed between 1928 and 1930. Townley and Allen (1939) documented earthquakes along the Pacific Coast of the U.S. between 1769 and 1928. The systematic recording of large earthquakes in California began in 1932-1933 by the U.S. Coast and Geodetic Survey (Richter, 1958). As part of our investigation, we reviewed earthquake data recorded between 1700 and 2020 by searching historical accounts and publications cataloging North American earthquake activity, and the current USGS database (USGS, 2020). The epicentral locations of earthquakes with a magnitude 5 and greater in the region are shown on Figure 5 – Historical Earthquake Epicenter Map. A table of earthquakes with a magnitude 5 and greater within 30 miles of the site is presented in Table 1.

5.2. Active Faulting and Surface Fault Rupture

The subject site is not located within a State of California Alquist-Priolo Earthquake Fault Zone (Alquist-Priolo EFZ, formerly known as a Special Studies Zone) (Hart and Bryant, 1997). The boundaries of the closest Alquist-Priolo EFZ are located approximately 1.78 miles northeast of the site associated with the northern section of the Newport-Inglewood fault within the Newport-Inglewood EFZ and 3.65 miles north of the site associated with the Santa Monica fault within the Santa Monica EFZ.

Figure 6 shows the locations of the recognized nearby faults with respect to the site. We also searched and reviewed the active or potentially active faults within 62 miles (100 kilometers) of the site from the 2008 USGS fault database. The faults within 31 miles (50 km) of the site presented in Table 2 are considered to represent the closest and most significant potential hazard to the site with respect to potential ground surface rupture and/or generate strong ground motion in the event of a moderately sized or larger earthquake. Based on our review of geologic and seismologic literature and our site evaluation, it is our opinion that the likelihood of surface fault rupture at the site during the life of the proposed improvements is remote.

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18071 Mt. Washington St
Unit A
Fountain Valley, CA 92708

Tel 949.553.0370
Fax 949.553.0371

Table 1 – Significant Historical Seismicity

Event Time (UTC)	Magnitude	Event Place	Latitude	Longitude	Depth (miles)	Distance to site (miles)
1933-03-11T14:25	5 ml	2 km NE of Carson, California	33.85	-118.266	16	13.2
1988-12-03T11:38	5.02 ml	1km SSE of Pasadena, CA	34.151	-118.13	13.719	18.6
1933-03-11T05:18	5.02 mh	2km ENE of Westminster, CA	33.76666	-117.985	6	29.0
1995-06-26T08:40	5.02 ml	11km SW of Valencia, California	34.394	-118.669	12.76	31.0
1994-01-29T11:20	5.06 ml	6km NNE of Chatsworth, California	34.306	-118.579	0.607	23.2
1994-01-19T21:11	5.07 ml	10km SSW of Valencia, California	34.378	-118.619	10.83	28.7
1997-04-26T10:37	5.07 ml	12km ESE of Piru, California	34.369	-118.67	15.892	29.5
2014-03-29T04:09	5.1 mw	2km NW of Brea, CA	33.9325	-117.9158	5.09	28.3
1941-11-14T08:41	5.12 ml	5km E of Lomita, CA	33.79066	-118.2636	6	16.7
1994-01-17T12:40	5.2 ml	9km N of Chatsworth, California	34.34	-118.614	5.395	26.2
1979-01-01T23:14	5.21 ml	13km S of Malibu Beach, CA	33.9165	-118.6871	13.3	17.4
1994-01-18T00:43	5.24 ml	10km ESE of Piru, California	34.377	-118.698	10.709	30.8
1994-03-20T21:20	5.24 ml	3km WNW of Panorama City, CA	34.231	-118.475	12.442	16.2
1987-10-04T10:59	5.25 ml	2km WSW of Rosemead, CA	34.074	-118.098	7.719	18.1
1930-08-31T00:40	5.25 ms	1 km SW of Las Flores, California	34.03	-118.643	15	13.9
1971-02-09T14:10	5.3 mh	10km SSW of Agua Dulce, CA	34.416	-118.37	6	28.6
1927-08-04T12:24	5.3 uk	2 km SSW of Santa Monica, CA	34	-118.5		5.6
1994-01-17T23:33	5.58 ml	7km NNE of Simi Valley, California	34.326	-118.698	9.083	28.0
1971-02-09T14:02	5.8 mh	10km SSW of Agua Dulce, CA	34.416	-118.37	6	28.6
1971-02-09T14:01	5.8 mh	10km SSW of Agua Dulce, CA	34.416	-118.37	6	28.6
1893-04-04T19:40	5.8 ml	Near Newhall, California	34.3	-118.6		23.4
1991-06-28T14:43	5.8 mw	13km NNE of Sierra Madre, CA	34.27	-117.993	8.049	29.8
1994-01-17T12:31	5.89 ml	1km ENE of Granada Hills, CA	34.275	-118.493	5.317	19.5
1987-10-01T14:42	5.9 mw	2km SSW of Rosemead, CA	34.061	-118.079	14.6	19.0
1855-07-11T04:15	6 ml	Near San Gabriel, California	34.1	-118.1		18.6
1971-02-09T14:00	6.6 mw	10km SSW of Agua Dulce, CA	34.416	-118.37	8.95	28.6
1994-01-17T12:30	6.7 mw	1km NNW of Reseda, CA	34.213	-118.537	18.202	16.4

Notes:

(1) ml = local magnitude, commonly referred to as "Richter magnitude"; mw = moment magnitude; ms = surface wave magnitude; mh = Non-standard magnitude method; uk = uk magnitude.

(2) NA = not available

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18071 Mt. Washington St
Unit A
Fountain Valley, CA 92708

Tel 949.553.0370
Fax 949.553.0371

Table 2 – Nearest Known Active Faults

Distance (miles)	Name	Slip Rate (mm/yr)	Dip (deg)	Dip Direction	Slip Sense	Rupture Top (km)	Rupture Bottom (km)	Length (km)
1.78	Newport-Inglewood, alt 1	1	88		strike slip	0	15	65
1.78	Newport Inglewood Connected alt 1	1.3	89		strike slip	0	11	208
1.95	Newport Inglewood Connected alt 2	1.3	90	V	strike slip	0	11	208
3.65	Santa Monica Connected alt 2	2.4	44		strike slip	0.8	11	93
3.98	Santa Monica Connected alt 1	2.6	51		strike slip	0	16	79
3.98	Santa Monica, alt 1	1	75	N	strike slip	0	18	14
4.33	Puente Hills (LA)	0.7	27	N	thrust	2.1	15	22
5.56	Hollywood	1	70	N	strike slip	0	17	17
7.28	Malibu Coast, alt 2	0.3	74	N	strike slip	0	16	38
7.28	Malibu Coast, alt 1	0.3	75	N	strike slip	0	8	38
8.51	Palos Verdes	3	90	V	strike slip	0	14	99
8.51	Palos Verdes Connected	3	90	V	strike slip	0	10	285
8.70	Anacapa-Dume, alt 2	3	41	N	thrust	1.2	12	65
9.63	Elysian Park (Upper)	1.3	50	NE	reverse	3	15	20
13.09	Raymond	1.5	79	N	strike slip	0	16	22
14.37	Verdugo	0.5	55	NE	reverse	0	15	29
16.53	Puente Hills (Santa Fe Springs)	0.7	29	N	thrust	2.8	15	11
16.89	Anacapa-Dume, alt 1	3	45	N	thrust	0	16	51
18.74	Sierra Madre	2	53	N	reverse	0	14	57
18.74	Sierra Madre Connected	2	51		reverse	0	14	76
19.26	Sierra Madre (San Fernando)	2	45	N	thrust	0	13	18
20.32	Elsinore;W	2.5	75	NE	strike slip	0	14	46
20.32	Elsinore;W+GI	n/a	81	NE	strike slip	0	14	83
20.32	Elsinore;W+GI+T	n/a	84	NE	strike slip	0	14	124
20.32	Elsinore;W+GI+T+J	n/a	84	NE	strike slip	0	16	199
20.32	Elsinore;W+GI+T+J+CM	n/a	84	NE	strike slip	0	16	241
21.14	Northridge	1.5	35	S	thrust	7.4	17	33
21.93	Santa Susana, alt 1	5	55	N	reverse	0	16	27
21.95	Puente Hills (Coyote Hills)	0.7	26	N	thrust	2.8	15	17
22.76	San Gabriel	1	61	N	strike slip	0	15	71
26.03	Clamshell-Sawpit	0.5	50	NW	reverse	0	14	16
26.70	Simi-Santa Rosa	1	60		strike slip	1	12	39
29.73	Holser, alt 1	0.4	58	S	reverse	0	19	20
30.11	San Jose	0.5	74	NW	strike slip	0	15	20

Notes:
(1) Dip directions: N=North; W=west; NE=northeast; NW=northwest; SW=southwest; V=vertical;
(2) n/a = not available; and
(3) km = kilometers.

5.3. Liquefaction and Seismic Settlement

Liquefaction is the phenomenon in which loosely deposited granular soils with silt and clay contents of less than approximately 35 percent, and non-plastic silts located below the water table undergo rapid loss of shear strength when subjected to strong earthquake-induced ground shaking. Ground shaking of sufficient duration results in the loss of grain-to-grain contact due to a rapid rise in pore water pressure and causes the soil to behave as a fluid for a short period of time.

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

The site is located within a state-designated Zone of Required Investigation for Liquefaction, according to California Seismic Hazard Zones Map (Figure 7). However, based on the relatively high penetration resistance of the soil layers and laboratory Atterberg limits and water content testing results, the soil layers are estimated to have negligible liquefaction potential and negligible seismic settlement during risk-targeted maximum considered earthquake (MCE_R) events. As a result, surface manifestation of liquefaction, such as sand boils, ground fissures etc., is also considered negligible.

5.4. Lateral Spreading

Lateral spreading is horizontal/lateral ground movement of gently sloping ground towards downslope or soil deposits towards a free face such as an excavation, channel, or open body of water. Typically, lateral spreading is associated with liquefaction of one or more subsurface layers near the bottom of exposed slope. Based on the relatively flat nature of the site and the negligible liquefaction potential, the potential for lateral spreading at the site is considered negligible.

5.5. Landslides

The site is not located within a Zone of Required Investigation for Earthquake-Induced Landslides designated by the State of California (Figure 7). Furthermore, the site is relatively flat, not susceptible to landslides. There are no known landslides adjacent to the site, and the site is not in the path of any known or potential landslides. It is our opinion that the risk is negligible of earthquake-induced landslides to affect the site.

5.6. Tsunamis and Seiches

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

Seiches are standing wave oscillations of an enclosed or semi-enclosed relatively large 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 enclosed or semi-enclosed large bodies of water in the vicinity of the site.

5.7. Flooding

The Federal Emergency Management Agency (FEMA) has prepared flood insurance rate maps (FIRMs) for use in administering the National Flood Insurance Program, effective September 26, 2008. Based on our review of online FEMA flood mapping, the site is located within a Zone X – Area of Minimal Flood Hazard. FEMA defines Zone X as an “area determined to be outside the 500-year flood and protected by levee from 100-year flood.”

5.8. Deaggregated Seismic Source Parameters

We performed a seismic hazard de-aggregation analysis for the peak ground acceleration with a probability of exceedance of 2% in 50 years. The analysis used the USGS Unified Hazard Tool based on the 2014 USGS seismic source model. The results of the analysis indicate the controlling modal moment magnitude and fault distance are 6.36 Mw and 2.22 miles (3.57 km), respectively.

5.9. Site Class for Seismic Design

Based on subsurface conditions encountered during our field exploration, it is our opinion that Site Class C may be used for the project seismic design according to Chapter 20 of ASCE 7-16.

5.10. Seismic Design Parameters

Seismic design parameters should be based on the 2022 CBC and ASCE 7-16. As the site is classified as seismic Site Class C. Seismic design parameters in Table 3 may be used.

TWINING CONSULTING

18071 Mt. Washington St
Unit A
Fountain Valley, CA 92708

Tel 949.553.0370
Fax 949.553.0371

Table 3 – Seismic Design Parameters Based on 2022 CBC and ASCE 7-16 for Design Based on Exception 2 in Section 11.4.8 of ASCE 7-16

Design Parameters	Value
Site Coordinates	latitude 34.003560°N longitude 118.402685°W
Site Class	C
Mapped Spectral Acceleration Parameter at Period of 0.2-Second, S_s (g)	1.927
Mapped Spectral Acceleration Parameter at Period 1-Second, S_1 (g)	0.68
Site Coefficient, F_a	1.2
Site Coefficient, F_v	1.4
Adjusted MCE_R[1] Spectral Response Acceleration Parameter, S_{MS} (g)	2.312
Adjusted MCE_R[1] Spectral Response Acceleration Parameter, S_{M1} (g)	0.952
Design Spectral Response Acceleration Parameter, S_{DS} (g)	1.542
Design Spectral Response Acceleration Parameter, S_{D1} (g)	0.635
Risk Coefficient, C_{RS}	0.906
Risk Coefficient, C_{R1}	0.902
Peak Ground Acceleration, PGA_M[2] (g)	0.988
Seismic Design Category[3]	D
Long-Period Transition Period, T_L (seconds)	8
$T_S = S_{D1} / S_{DS}$	0.412

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

18071 Mt. Washington St
Unit A
Fountain Valley, CA 92708

Tel 949.553.0370
Fax 949.553.0371

6. GEOTECHNICAL ENGINEERING RECOMMENDATIONS

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

6.1. General Considerations

Geotechnical engineering recommendations presented in this report for the proposed project are based on our understanding of the proposed development, subsurface conditions encountered during our field exploration, the results of laboratory testing on soil samples taken from the site, and our engineering analyses.

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

6.2. Soil Expansion and Collapse Potential

Laboratory expansion index testing indicates the surficial soils at the site have a high expansion potential. Based on our field exploration and laboratory testing results, the soils below the depth of 10 feet have moderate collapse potential (up to about 4%). Due to the presence of the upper 10 feet of very stiff clay, the risk of collapse potential to the project is considered low. Our recommendations for subgrade preparation and foundation excavation are intended to mitigate adverse effects on the project caused by expansive soils.

6.3. 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.

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

6.3.2. Excavation and Subgrade Preparation

Based on our laboratory testing, the near surface soils are considered to have high expansion potential. These results have been incorporated into our recommendations. The proposed structure can be supported on conventional spread footings bearing on at least 3 feet of “very low” expansive engineered fill (i.e., expansion index of 20 or less) and embedded a minimum of

12 inches below the lowest adjacent grade. Engineered fill should be prepared according to the recommendations presented in Section 6.3.3 and 6.3.4.

Minor structures such as slabs-on-grade (SOGs), fencing walls or trash enclosures that are structurally separated from the building may be supported on conventional continuous or spread footings bearing on at least 2 feet of very low expansive engineered fill and embedded a minimum of 12 inches below the lowest adjacent grade. Pavements and hardscape should be over-excavated at least 1 foot as measured from the bottom of pavement or hardscape section.

Laterally, foundation excavation should extend beyond the foundation limits a minimum distance equal to 3 feet or the depth of over-excavation, whichever is greater. Excavation for other improvements should extend laterally at least 2 feet beyond the limits of the improvements.

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

The exposed excavation bottom should be evaluated and approved by the geotechnical engineer. The bottom should then be scarified to a minimum depth of 6 inches and moisture conditioned to achieve generally consistent moisture contents within approximately 3 percent above the optimum moisture content. The scarified bottom should be compacted to at least 90 percent relative compaction in accordance with the latest version of ASTM Test Method D1557 and then evaluated and approved by the geotechnical engineer.

6.3.3. Materials for Fill

Backfill materials used for support of foundations and slabs-on-grade (SOG) should have a very low expansion potential (i.e., expansion index of 20 or less). In general, on-site soils expected to be excavated consist of lean clay with high expansion potential and are not considered suitable for use as foundation and SOG backfill. On-site soil proposed for use as foundation and SOG support should be reviewed by the geotechnical engineer to confirm the low-expansive nature of the soil prior to its use. All fill soils should be free of organics, debris, rocks or lumps over three inches in largest dimension, other deleterious material, and not more than 40 percent larger than ¾ inch. Larger chunks, if generated during excavation, may be broken into acceptably sized pieces or may be disposed of offsite.

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

All fill soils should be evaluated and approved by the geotechnical engineer's representative prior to importing or filling.

6.3.4. Compacted Fill

Unless otherwise recommended, the exposed excavation bottom to receive fill should be prepared in accordance with Section 6.3.2 of this report. Prior to placement of compacted fill, the contractor should request the geotechnical engineer to evaluate the exposed excavation bottoms.

18071 Mt. Washington St
Unit A
Fountain Valley, CA 92708

Tel 949.553.0370
Fax 949.553.0371

Compacted fill should be placed in horizontal lifts of approximately 8 to 10 inches in loose thickness, depending on the equipment used. Prior to compaction, each lift should be moisture conditioned, mixed, and then compacted by mechanical methods. The moisture content should be within approximately 2 percent above the optimum moisture content. Fill materials should be compacted to a minimum relative compaction of 90 percent as determined by ASTM D1557. Successive lifts should be treated in the same manner until the desired finished grades are achieved.

6.3.5. 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 4 feet high will generally be stable. However, sloughing should be expected when materials consisting predominantly of sand are encountered.

Where the space is available, temporary, unsurcharged excavation sides over 4 feet in height should be sloped no steeper than an inclination of 1.5H:1V (horizontal:vertical). Where sloped excavations are created, the tops of the slopes should be barricaded so that vehicles and storage loads do not encroach within 10 feet of the top of the excavated slopes. A greater setback may be necessary when considering heavy vehicles, such as concrete trucks and cranes. The geotechnical engineer should be advised of such heavy vehicle loadings so that specific setback requirements can be established. If the temporary construction slopes are to be maintained during the rainy season, berms are recommended to be graded along the tops of the slopes in order to prevent runoff water from entering the excavation and eroding the slope faces.

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

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

6.3.6. Excavation Bottom Stability

In general, we anticipate that the bottoms of the excavations will be stable and should provide suitable support to the proposed improvements. The condition of the subgrade should be evaluated by the project geotechnical engineer during the scarification and re-compaction efforts.

6.3.7. Backfill for Utility Trench

When parallel to any footings, utility trenches and pipes should be laid above an imaginary 2:1 (H:V) line projected down from a point 9 inches above the bottom edge of the footing, and not closer than 18 inches from the face of such footing. Otherwise, the pipe should be encased to accept the effect from the footing load. Where pipes cross under footings, the footings should be specially designed. Pipe sleeves should be provided where pipes cross through footings or footing walls, and sleeve clearances should provide for possible footing settlement, but not less than 1 inch all around pipe.

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

The bedding materials may consist of clean sand having a minimum sand equivalent (SE) of 20, gravel or crushed rock, or 2-sack sand-cement slurry, and should meet the specifications provided in the latest edition of the “Greenbook” Standard Specifications for Public Works Construction. Samples of materials proposed for use as bedding material should be provided to the project geotechnical engineer for inspection and testing before the material is imported for use on the project. The onsite materials can only be used following the requirement of “Greenbook” bedding specification when the SE is not less than 30. Gravel or crushed rock if used as bedding materials should be wrapped in nonwoven geotextile fabric.

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

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

6.3.8. Rippability

Based on our evaluation of the subsurface conditions of the site, the site earth materials should be generally excavatable with heavy-duty earthwork equipment in good working condition. Some gravels, cobbles and man-made debris should be anticipated.

6.3.9. Construction Dewatering

Groundwater was not encountered in the borings drilled to a maximum depth of approximate 51 feet bgs during our field exploration, dewatering measures are not anticipated during 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.

18071 Mt. Washington St
Unit A
Fountain Valley, CA 92708

Tel 949.553.0370
Fax 949.553.0371

6.4. Corrosive Soil Evaluation

The potential for the near-surface on-site materials to corrode buried steel and concrete improvements was evaluated. Laboratory testing was performed on one selected near-surface soil to evaluate pH and electrical resistivity, as well as chloride and sulfate contents. Laboratory test results are presented in Appendix B – Laboratory Testing.

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

6.4.1. Reinforced Concrete

Laboratory tests indicate that the soil has 181 ppm or 0.0181% of water soluble sulfate (SO_4) by weight. Based on ACI 318, concrete in contact with the site soils will have a sulfate exposure class S0. As a minimum, we recommend that Type II cement and a concrete mix design for foundations and building slabs-on-grade that incorporates a maximum water-cement ratio of 0.50.

Test results indicate that the soil has 128 ppm of water soluble chlorides by weight and the potential for chloride attack of reinforcing steel in concrete structures and pipes in contact with soil is negligible.

6.4.2. Buried Metal

A factor for evaluating corrosivity to buried metal is electrical resistivity. The electrical resistivity of a soil is a measure of resistance to electrical current. Corrosion of buried metal is directly proportional to the flow of electrical current from the metal into the soil. As resistivity of the soil decreases, the corrosivity generally increases. Test results indicate the site soils have minimum electrical resistivity value of 870 ohm-centimeters. Based on the County of Los Angeles (2014) criteria, the soils are considered corrosive to buried metals.

Correlations between resistivity and corrosion potential published by the National Association of Corrosion Engineers (NACE, 1984) indicate that the soils have severely corrosive potential to buried metals. Corrosion protection for metal in contact with site soils may include the use of epoxy or asphalt coatings. If needed, a corrosion specialist should be consulted regarding appropriate protection for buried metals and suitable types of piping.

6.5. Foundation Recommendations

Based upon the excavation/over-excavation and backfill recommendations, proposed structure may be supported on continuous strip footings or isolated footings designed in accordance with the geotechnical recommendations presented below. Structural design of foundations should be performed by the structural engineer and should conform to the 2022 California Building Code.

6.5.1. Bearing Capacity and Settlement

Proposed new footings should be placed on the subgrade prepared in accordance with the requirements described in Section 6.3. Geotechnical parameters presented in Table 4 may be used in the footing design. Twining should be contacted for footing loads, allowable bearing pressures, and settlements that are outside the indicated applicable ranges.

6.5.2. Lateral Resistance

Lateral loads may be resisted by footing base friction and by the passive resistance of the soils based on recommendations provided in Table 4. The total lateral resistance can be taken as the sum of the friction at the base of the footing and passive resistance. The upper one foot of soil should be neglected when calculating the passive resistance.

18071 Mt. Washington St
Unit A
Fountain Valley, CA 92708

Tel 949.553.0370
Fax 949.553.0371

Table 4- Geotechnical Design Parameters for Shallow Foundations

Minimum Footing Dimensions	• <u>Width:</u> 12 inches • <u>Minimum embedment:</u> 12 inches measured from the lowest adjacent grade to the bottom of the footing. • <u>Minimum thickness:</u> 6 inches
Assumed Maximum Vertical Loads on Foundations	• Square Footing: 100 kilo-pounds (kips) • Continuous Footing: 10 kips/foot
Net Allowable Bearing Pressure	• An allowable bearing pressures of 4,000 pounds per square foot (psf) may be used for isolated square and continuous spread footings. • The allowable bearing values correspond to a factor of safety of three. • The allowable bearing values may be increased by one-third for transient loads from wind or earthquake.
Estimated Static Settlement	• Total settlement: less than 1 inch of static settlement. • Differential settlement: less than ½ inches over a horizontal distance of 30 feet between similarly loaded footings. • The majority static settlement is expected to compete upon initial application of loading.
Estimated Seismic Settlement	• Negligible
Allowable Coefficient of Friction Below Footings	0.38, corresponding to a factor of safety of 1.5.
Allowable Lateral Passive Resistance	• 300 pounds per cubic foot (pcf) equivalent fluid pressure (EFP), to a maximum pressure of 3,600 psf, corresponding to a factor of safety of two. • The upper one foot of soil should be neglected when calculating the passive resistance. • The allowable passive resistance value may be increased by one-third for transient loads such as wind or earthquake loads. • Since full passive resistance requires sufficient movement to mobilize, when combined with base shear, the passive resistance should be reduced to half for displacement compatibility.

18071 Mt. Washington St
Unit A
Fountain Valley, CA 92708

Tel 949.553.0370
Fax 949.553.0371

6.6. Modulus of Subgrade Reaction

The modulus of subgrade reaction k for design of combined footing or slabs-on-grade supported on at least 2 feet of imported fill may be obtained from the following equation.

$$K = k_1 \frac{1}{B}\left(\frac{2L + B}{3L}\right)$$

where: k_1 = modulus for a 1-foot by 1-foot plate = 175 pounds per cubic inch (pci);
B = width of combined footing or slab in feet; and
L = length of combined footing or slab in feet.

6.7. Concrete Slabs

Over-excavation for concrete slabs-on-grade (SOG) should extend to at least 2 feet below the bottom of SOG. The over-excavation should be backfilled with engineered fill having a very low expansion potential (i.e., expansion index of 20 or less) compacted to at least 90 percent relative compaction determined according to ASTM D1557. For design of concrete SOG, a subgrade modulus k value discussed in Section 6.6 may be used.

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

The topmost 8 inches below the slab subgrade should be maintained in a moisture condition of approximately within 2 percent above optimum moisture content. The slab subgrade should be tested for moisture and compaction immediately prior to placement of the gravel or sand base, if any. All underslab materials should be adequately compacted prior to the placement of concrete. Care should be taken during placement of the concrete to prevent displacement of the underslab materials. The underslab material should be dry or damp and should not be saturated prior to the placement of concrete. The concrete slab should be allowed to cure properly and should be tested for moisture transmission prior to placing vinyl or other moisture-sensitive floor covering. In moisture sensitive areas, the floor slabs should be dampproofed in accordance with Section 1805.2A of 2022 CBC. Specific recommendations can be provided by a waterproofing consultant.

Table 5 provides general recommendations for various levels of protection against vapor transmission through concrete floor slabs placed over a properly prepared subgrade. Care should be taken not to puncture the plastic membrane during placement of the membrane itself and the overlying silty sand.

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

18071 Mt. Washington St
Unit A
Fountain Valley, CA 92708

Tel 949.553.0370
Fax 949.553.0371

Table 5 - Options for Subgrade Preparation below Concrete Floor Slabs

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

Notes:

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

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

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

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

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

7. DESIGN REVIEW AND CONSTRUCTION MONITORING

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

7.1. Plans and Specifications

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

7.2. Construction Monitoring

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

8. LIMITATIONS

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

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

Site conditions, including groundwater elevation, can change with time as a result of natural processes or the activities of man at the subject site or at nearby sites. Changes to the applicable laws, regulations, codes, and standards of practice may occur as a result of government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Twining has no control.

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

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

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

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

TWINING CONSULTING

18071 Mt. Washington St
Unit A
Fountain Valley, CA 92708

Tel 949.553.0370
Fax 949.553.0371

9. SELECTED REFERENCES

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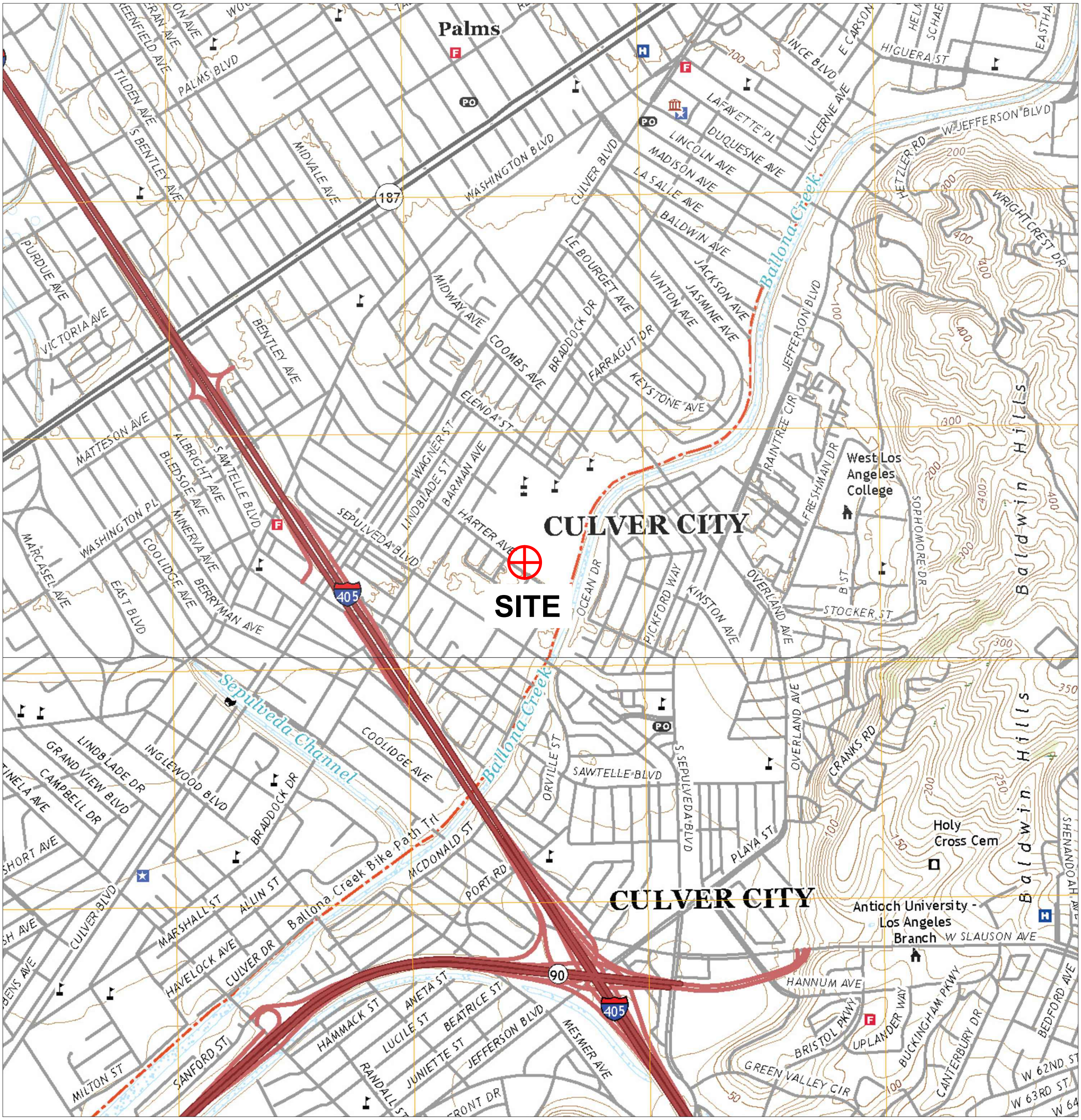
TWINING CONSULTING

18071 Mt. Washington St
Unit A
Fountain Valley, CA 92708

Tel 949.553.0370
Fax 949.553.0371

FIGURES



APPROXIMATE LOCATION OF PROJECT

SCALE IN FEET

0 2000 4000

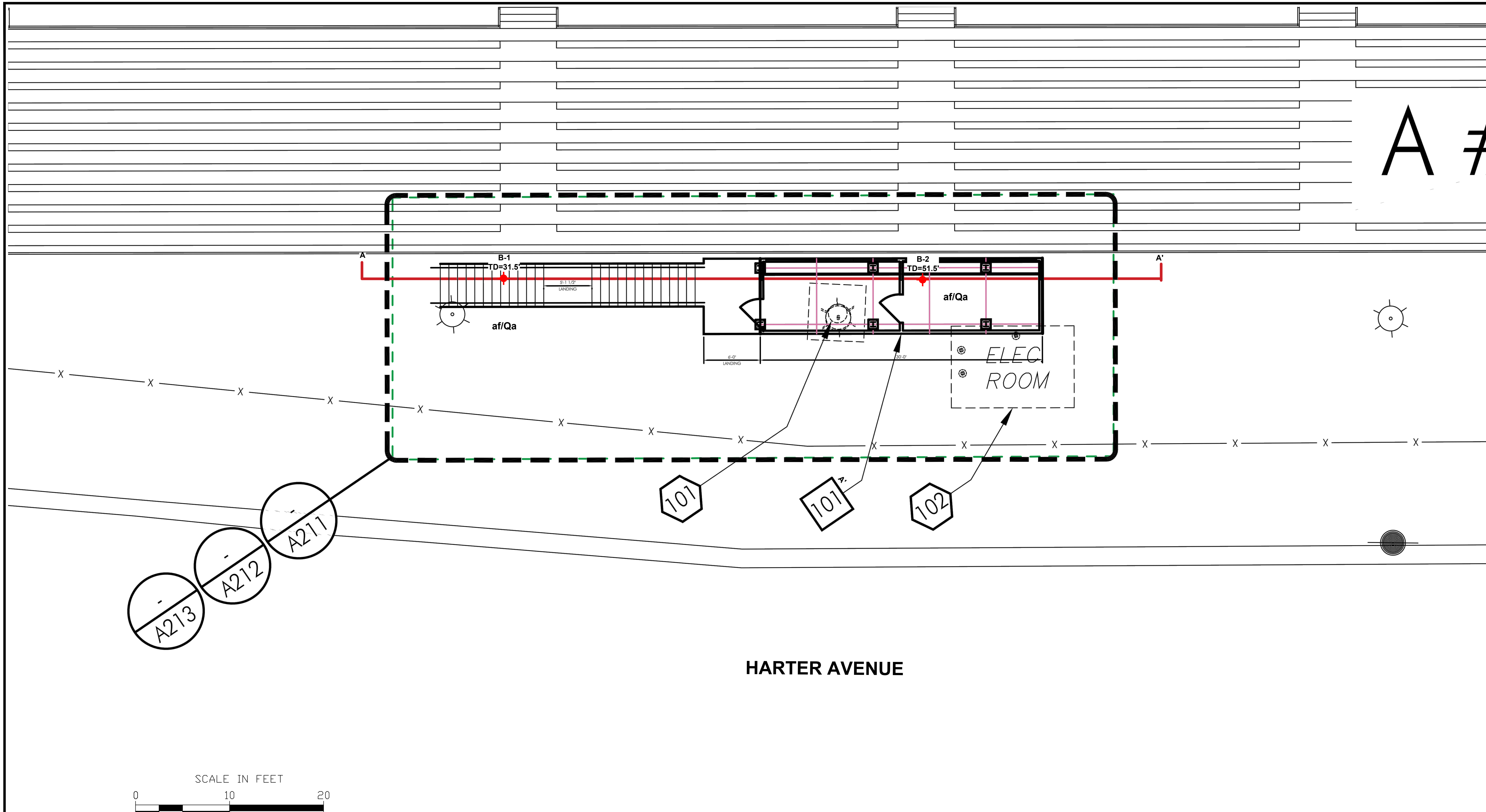
REFERENCE: USGS (2022)

TWINING

SITE LOCATION MAP

CULVER CITY HIGH SCHOOL PRESS BOX
4401 ELENDA STREET
CULVER CITY, CA

PROJECT NO.	REPORT DATE	FIGURE 1
220695.1	December 2022	



NOTE: ALL DIMENSIONS AND LOCATIONS ARE APPROXIMATE

TWINING

REFERENCE: RACHLIN PARTNERS (2022)

LEGEND

B-1 TD=31.5'	APPROXIMATE LOCATION OF BORING BY TWINING TOTAL DEPTH IN FEET
af/Qa	ARTIFICIAL FILL OVER ALLUVIUM
	APPROXIMATE LIMITS OF PROJECT
A A'	GEOLOGIC CROSS SECTION

SITE PLAN AND BORING LOCATION MAP

CULVER CITY HIGH SCHOOL PRESS BOX
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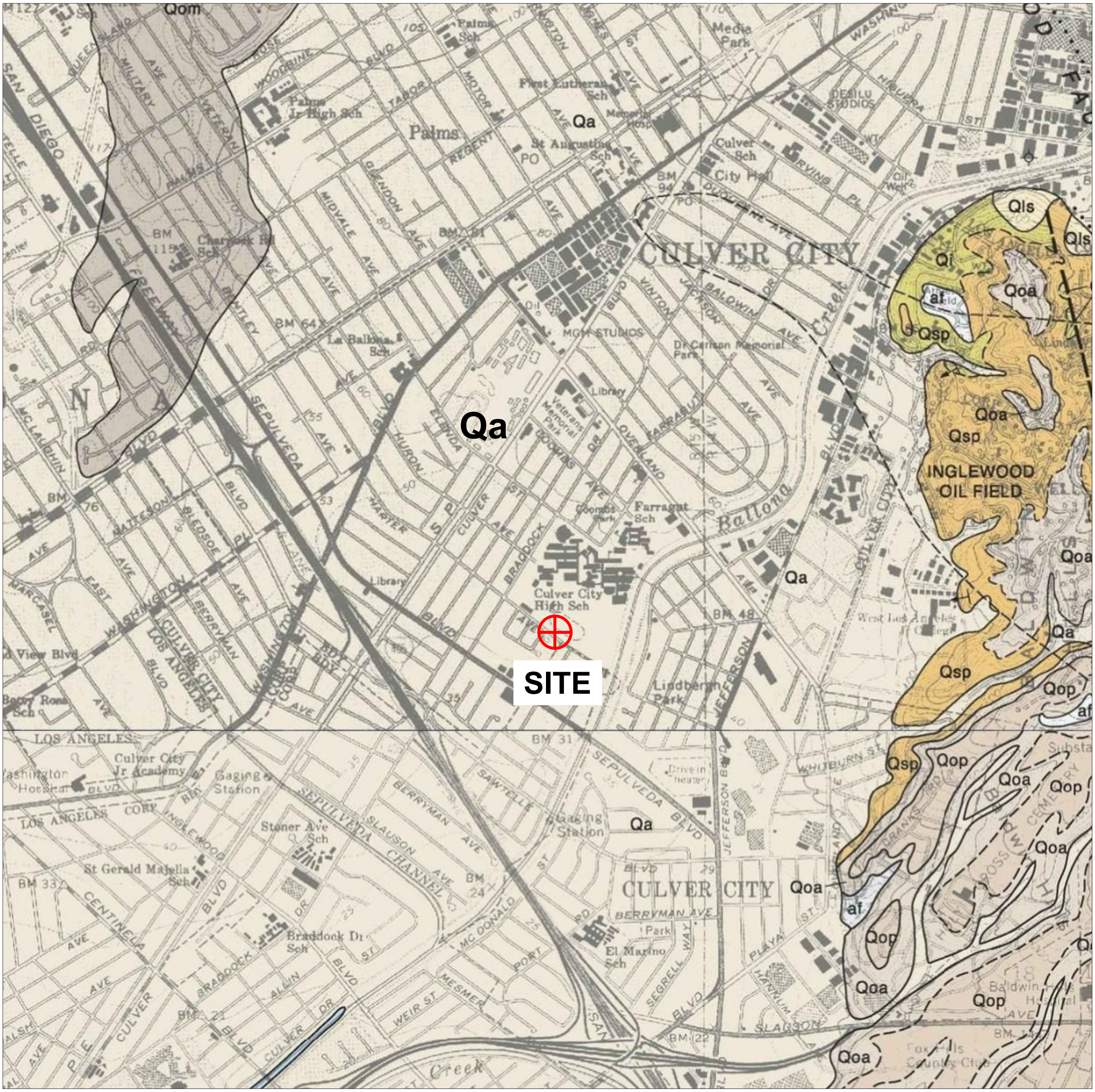
CULVER CITY, CA

PROJECT No.	REPORT DATE	
220695.1	December 2022	FIGURE 2



Qa	Alluvium
Qsp	San Pedro Sand
Qop	Paleosoil
Qoa	Older alluvium

SCALE IN FEET

0 2000 4000

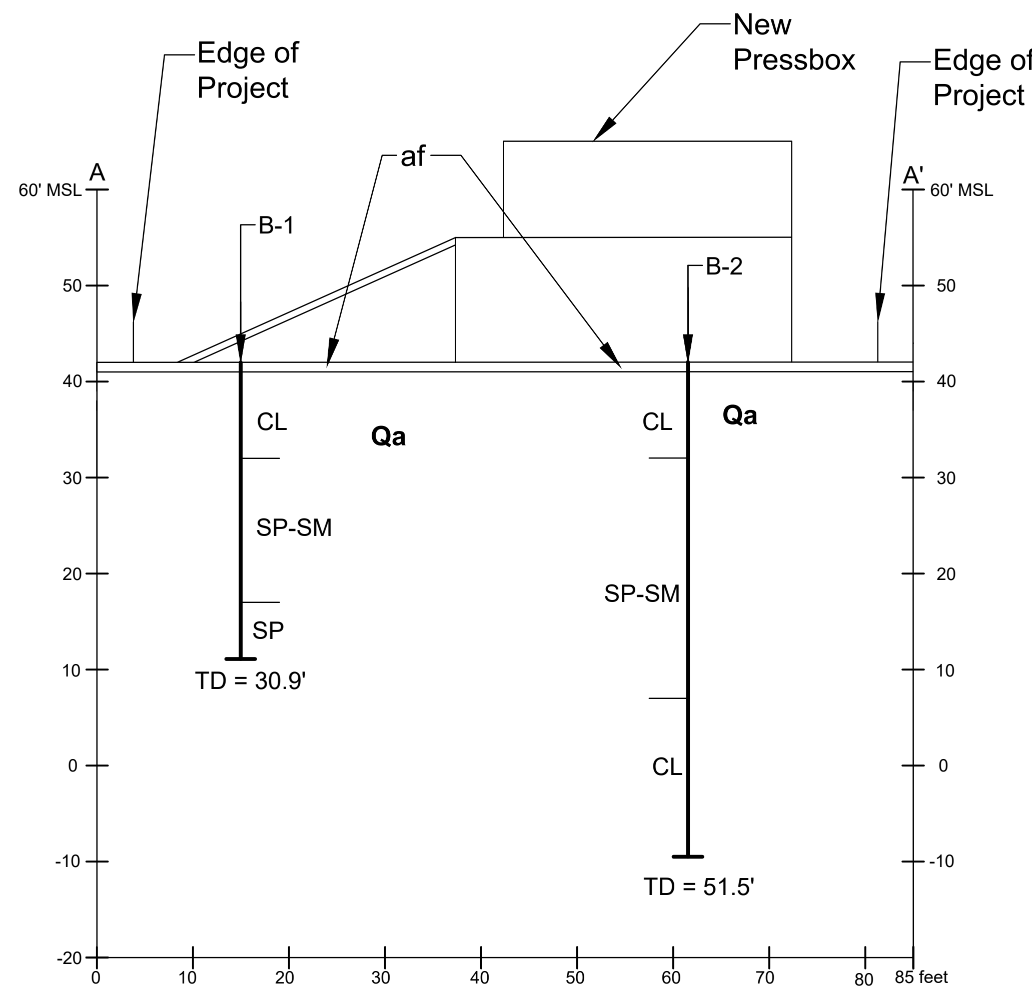
REFERENCE: DIBBLEE AND MINCH (2010)

TWINING

GEOLOGIC MAP

CULVER CITY HIGH SCHOOL PRESS BOX
4401 ELENDA STREET
CULVER CITY, CA

PROJECT NO.	REPORT DATE	
220695.1	December 2022	FIGURE 3



LEGEND

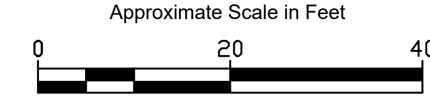
Geologic Contact, Queried Where Uncertain

Af Fill

Qa Young Alluvium

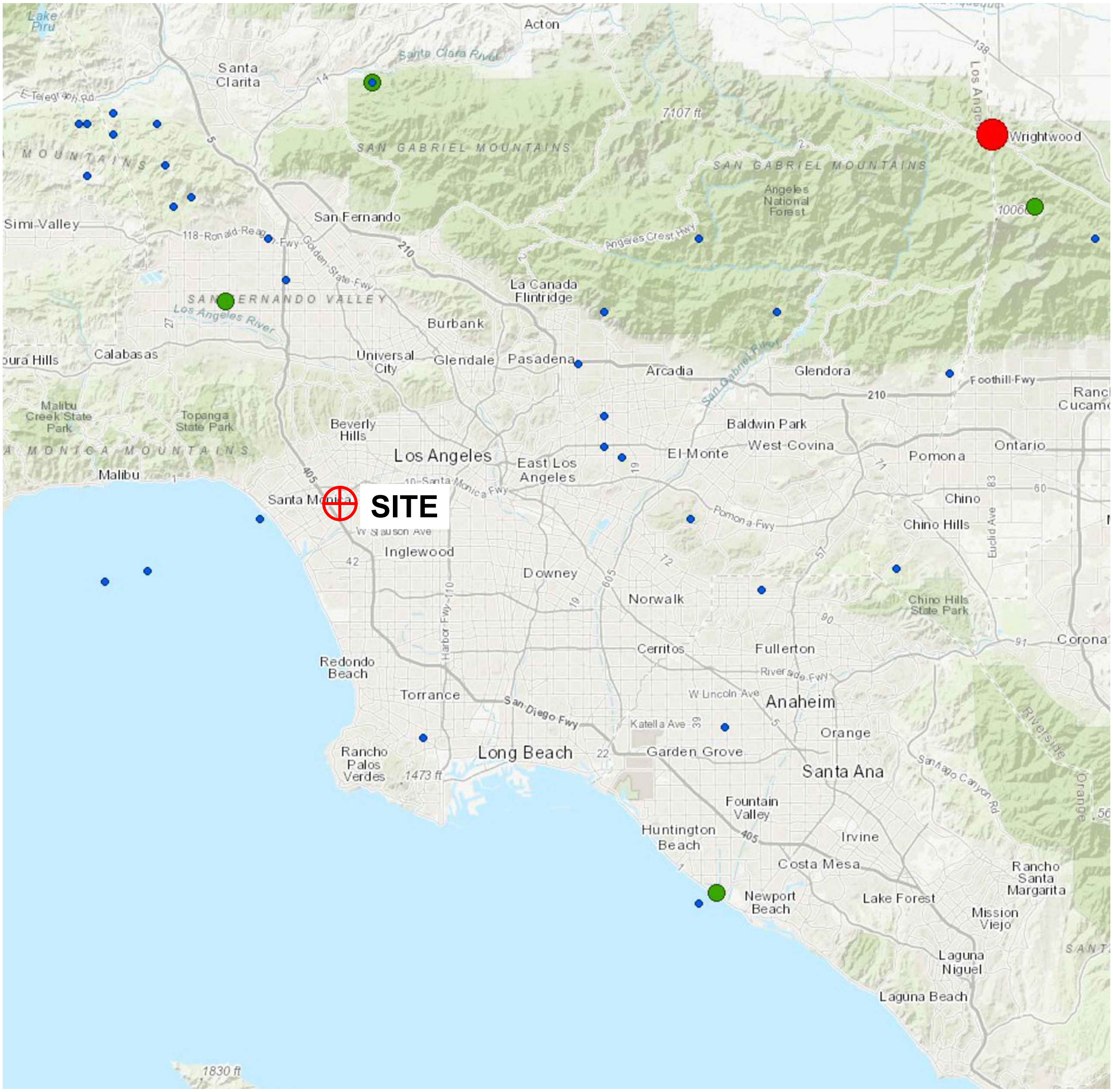
B-1 Location of Boring
TD = Total Depth

Approximate Scale in Feet



TWINING

GEOLOGIC CROSS SECTION A-A'		
CULVER CITY HIGH SCHOOL PRESS BOX 4401 ELENDA STREET CULVER CITY, CA		
PROJECT No. 220695.1	REPORT DATE December 2022	FIGURE 4



MS48: Historic Earthquakes, 1769 to 2015 - California (Magnitude 5.0-plus)

Historic Earthquakes, 1769-2015 - California (Magnitude 5.0 Plus)

- 7+
- 6-7
- 5-6

N

SCALE IN MILES

0 10 20

TWINING

HISTORICAL EARTHQUAKE EPICENTER MAP

CULVER CITY HIGH SCHOOL PRESS BOX
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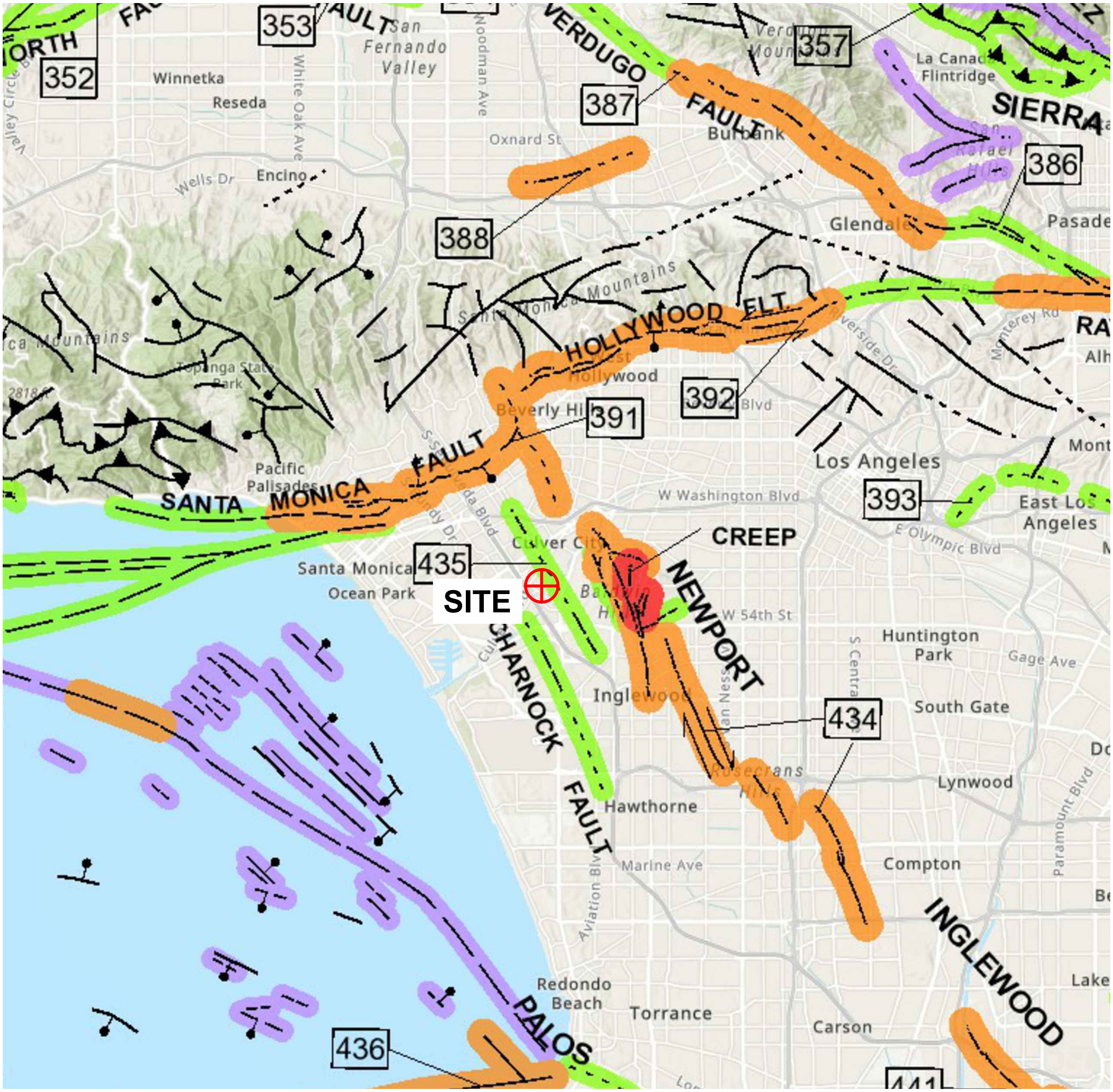
CULVER CITY, CALIFORNIA

PROJECT NO.	REPORT DATE	FIGURE 5
220695.1	December 2022	



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

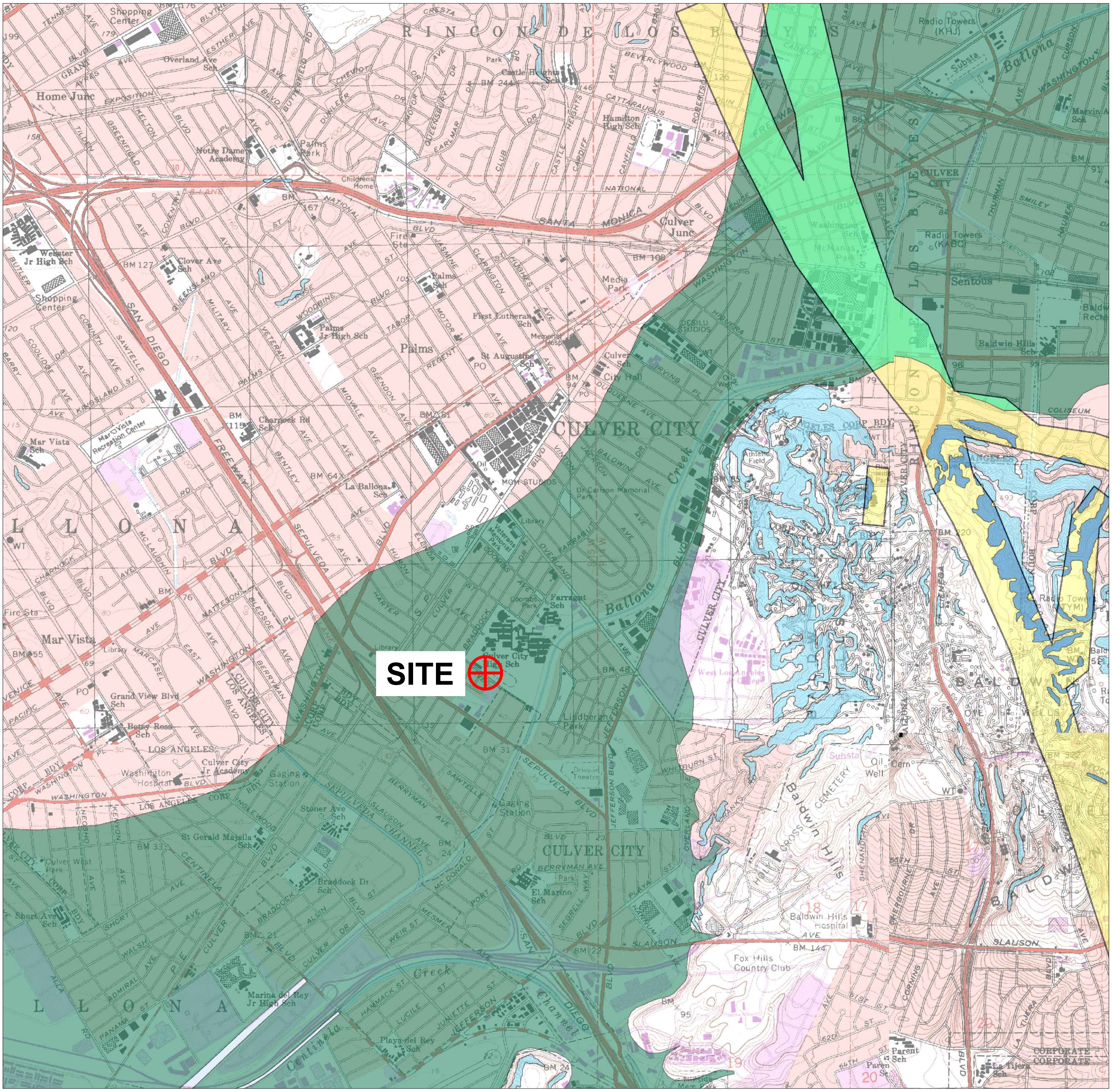
REFERENCE: CGS (2021)

TWINING

REGIONAL FAULT LOCATION MAP

CULVER CITY HIGH SCHOOL PRESS BOX
4401 ELENDA STREET
CULVER CITY, CALIFORNIA

PROJECT NO.	REPORT DATE	FIGURE 6
220695.1	December 2022	



MAP EXPLANATION

EARTHQUAKE FAULT ZONES

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

Active Fault Traces
1906 C
Faults considered to have been active during Holocene time and to have potential for surface rupture: Solid Line in Black or Red where Accurately Located; Long Dash in Black or Solid Line in Purple where Approximately Located; Short Dash in Black or Solid Line in Orange where Inferred; Dotted Line in Black or Solid Line in Rose where Concealed; Query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by fault creep.

SEISMIC HAZARD ZONES

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

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

OVERLAPPING EARTHQUAKE FAULT AND SEISMIC HAZARD ZONES

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

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

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

SCALE IN FEET
0 3000 6000

REFERENCE: CGS (2018)

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SEISMIC HAZARD ZONES MAP

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PROJECT NO.	REPORT DATE	FIGURE 7
220695.1	December 2022	

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CONSULTING

18071 Mt. Washington St
Unit A
Fountain Valley, CA 92708

Tel 949.553.0370
Fax 949.553.0371

APPENDIX A
FIELD EXPLORATION

TWINING CONSULTING

18071 Mt. Washington St
Unit A
Fountain Valley, CA 92708

Tel 949.553.0370
Fax 949.553.0371

Appendix A
Field Exploration

General

The field exploration for the proposed project consisted of drilling, testing, sampling, and logging of 2 exploratory borings (B-1 and B-2). The approximate locations of the exploration are shown on Figure 2 – Site Plan and Exploration Location Map.

The exploration was first excavated to 5 feet below ground surface (bgs) using a hand-auger to clear potential underground utilities. Upon completion of exploration, the borings with neat cement grout. The surface of all locations was repaired to match existing conditions.

Hollow-Stem-Auger Borings

Drilling operation for the borings was performed by Baja Drilling of Escondido, California using a track-mounted drill rig equipped with 6-inch diameter hollow-stem-auger (HSA). The borings were advanced to maximum depths of 31 and 51.5 feet below ground surface (bgs) on October 26, 2022, for B-1 and B-2, respectively.

An explanation of the boring logs is presented as Figure A-1. The boring logs are presented as Figures A-2 through A-3. The boring logs describe the earth materials encountered, samples obtained, and show the field and laboratory tests performed. The logs also show the boring number, drilling date, and the name of the logger and drilling subcontractor. The borings were logged by a Twining engineer using the Unified Soil Classification System under the supervision of a registered California Geotechnical Engineer. The boundaries between soil types shown on the logs are approximate because the transition between different soil layers may be gradual. Drive and bulk samples of representative earth materials were obtained from the borings.

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

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

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

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EXPLANATION FOR LOG OF BORINGS

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Culver City, California

PROJECT NO.	REPORT DATE	
220695.1	December 2022	FIGURE A-1

DATE DRILLED	10/26/2022	LOGGED BY	AB	**BORING NO.**	**B-1**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	N/E
DRILLING METHOD	HSA 6"	DRILLER	Baja Drilling	SURFACE ELEVATION (ft.)	42 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
									CL	ARTIFICAL FILL: Lean CLAY with Sand; fine sand; light yellowish brown; moist
					14.8	178.4	#200, ATT, EI, CORR, MAX, DS		CL	ALLUVIUM: Lean CLAY with Sand; fine sand; reddish brown; moist
37	5			19						- same; very stiff
32	10			50 for 6"	2.4	129.7	C		SP-SM	Poorly graded SAND with Silt and Gravel; fine to medium sand; light reddish brown; moist; very dense; fine to coarse gravel
27	15			77			#200			- same
										- hard drilling; some cobbles
22	20			50 for 5"	4.4	109.7				- same; yellowish brown
17	25			60			#200		SP	Poorly graded SAND; fine sand; light grayish brown; moist; very dense
12	30			50 for 5"	3.6	102.7				- same
										Total Depth = 30.9 feet Backfilled on 10/26/2022 No Groundwater encountered. Boring backfilled with lean cement and surface patched with soil.
7	35									

BORING LOG 220695.1 CULVER CITY HS PRESS BOX.GPJ TWINING LABS.GDT 11/30/22

TWINING

LOG OF BORING

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Culver City, California

PROJECT NO.	REPORT DATE	
220695.1	December 2022	FIGURE A - 2

DATE DRILLED	10/26/2022	LOGGED BY	AB	**BORING NO.**	**B-2**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	42
DRILLING METHOD	HSA 6"	DRILLER	Baja Drilling	SURFACE ELEVATION (ft.)	42 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
									CL	ARTIFICAL FILL: Lean CLAY with sand; fine sand; light yellowish brown; moist; trace roots
		X					ATT, EI		CL	ALLUVIUM: Lean CLAY with Sand; fine sand; reddish brown; moist; trace roots
37	5		X	45	10.2	110.9	DS			- same; very stiff
32	10		X	34			#200		SP-SM	Poorly graded SAND with Silt and Gravel; dark yellowish brown; fine to coarse gravel; moist; dense
										- hard drilling; some cobbles
27	15		X	50 for 5"	1.9	118.0	C			- same; becomes very dense
22	20		X	64			#200			- same
17	25		X	50 for 6"	2.7	106.2	DS			-same; light grayish brown
12	30		X	50 for 4"			#200			- same
7	35									

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LOG OF BORING

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PROJECT NO.	REPORT DATE	
220695.1	December 2022	FIGURE A - 3

DATE DRILLED	10/26/2022	LOGGED BY	AB	**BORING NO.**	**B-2**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	42
DRILLING METHOD	HSA 6"	DRILLER	Baja Drilling	SURFACE ELEVATION (ft.)	42 ±(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	50 for 2"	18.1	108.4			CL	Sandy Lean Clay; fine sand; dark gray; hard; moist
2	40		I	50 for 5"			#200, ATT			- same; micaceous
-3	45		X	50 for 3"	26.0	95.4	C			- becomes wet
-8	50		I	24						- same; becomes very stiff
										Total Depth = 51.5 feet Backfilled on 10/26/2022 Groundwater encountered at 42 feet. Boring backfilled with lean cement and surface patched with soil.
-13	55									
-18	60									
-23	65									
-28	70									

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LOG OF BORING

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PROJECT NO.	REPORT DATE	
220695.1	December 2022	FIGURE A - 3

18071 Mt. Washington St
Unit A
Fountain Valley, CA 92708

Tel 949.553.0370
Fax 949.553.0371

APPENDIX B
LABORATORY TESTING

TWINING CONSULTING
18071 Mt. Washington St
Unit A
Fountain Valley, CA 92708
Tel 949.553.0370
Fax 949.553.0371

Appendix B
Laboratory Testing

Laboratory Moisture Content and Density Tests

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

No. 200 Wash Sieve

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

Atterberg Limits

Tests were performed on selected representative fine-grained soil samples to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D4318. These test results were utilized to evaluate the soil classification in accordance with the Unified Soil Classification System. The test results are summarized in on Figure B-1 and Table B-3.

Expansion Index

The expansion index of a select soil sample was evaluated in general accordance with ASTM D4829. The specimen was molded under a specified compactive energy at approximately 50 percent saturation. The prepared 1-inch thick by 4-inch diameter specimen was loaded with a surcharge of 144 pounds per square foot and was inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The result of expansion index test is presented in Table B-4.

Maximum Density and Optimum Moisture

A Modified Proctor test was also performed on near-surface soils to determine the maximum dry density and optimum water content for compaction. The tests were performed in accordance with ASTM D 1557 Method A. A copy of the curve is attached to this appendix as Figure B-2.

Direct Shear

Direct shear tests were performed on a remolded sample and representative modified-California soil samples in general accordance with the latest version of ASTM D3080 to evaluate the shear strength characteristics of the selected materials. The samples were inundated during shearing to represent adverse field conditions. Test results are presented on Figures B-3 through B-5.

Consolidation

Consolidation tests were performed on a selected modified-California soil sample in general accordance with the latest version of ASTM D2435. The samples were inundated during testing to represent adverse field conditions. The percent consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. Test results are presented on Figures B-6 through B-8.

Corrosivity

Soil pH and resistivity tests were performed by Anaheim Test Lab, Inc. (ATLI) of Anaheim, California on a representative soil sample. The resistivity of the soil assumes saturated soil conditions. The chloride and sulfate contents of the selected samples were evaluated in general accordance with the latest versions of Caltrans test methods CT417, CT422, and CT 643. The test results are presented on Table B-5 and the ATLI report included in this appendix.

Table B-1 - Moisture Content and Dry Density

Boring No.	Depth (feet)	Moisture Content (%)	Dry Density (pcf)
B-1	10	2.4	129.7
B-1	20	4.4	109.7
B-1	30	3.6	102.7
B-2	6	10.2	110.9
B-2	15	1.9	118.0
B-2	25	2.7	106.2
B-2	35	18.1	108.4
B-2	45	26.0	95.4

Table B-2 - Number 200 Wash Results

Boring No.	Depth (feet)	Percent Passing #200
B-1	0-5	72.4
B-1	15	5.5
B-1	25	3.7
B-2	10	10.3
B-2	20	7.0
B-2	30	8.0
B-2	40	58.6

Table B-3 - Atterberg Limits Results

Boring No.	Depth (feet)	Liquid Limit	Plastic Limit	Plasticity Index	U.S.C.S. Classification
B-1	0-5	36	15	21	Lean CLAY with Sand (CL)
B-2	0-5	42	17	25	Lean CLAY with Sand (CL)
B-2	40	36	19	17	Sandy Lean CLAY (CL)

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18071 Mt. Washington St
Unit A
Fountain Valley, CA 92708

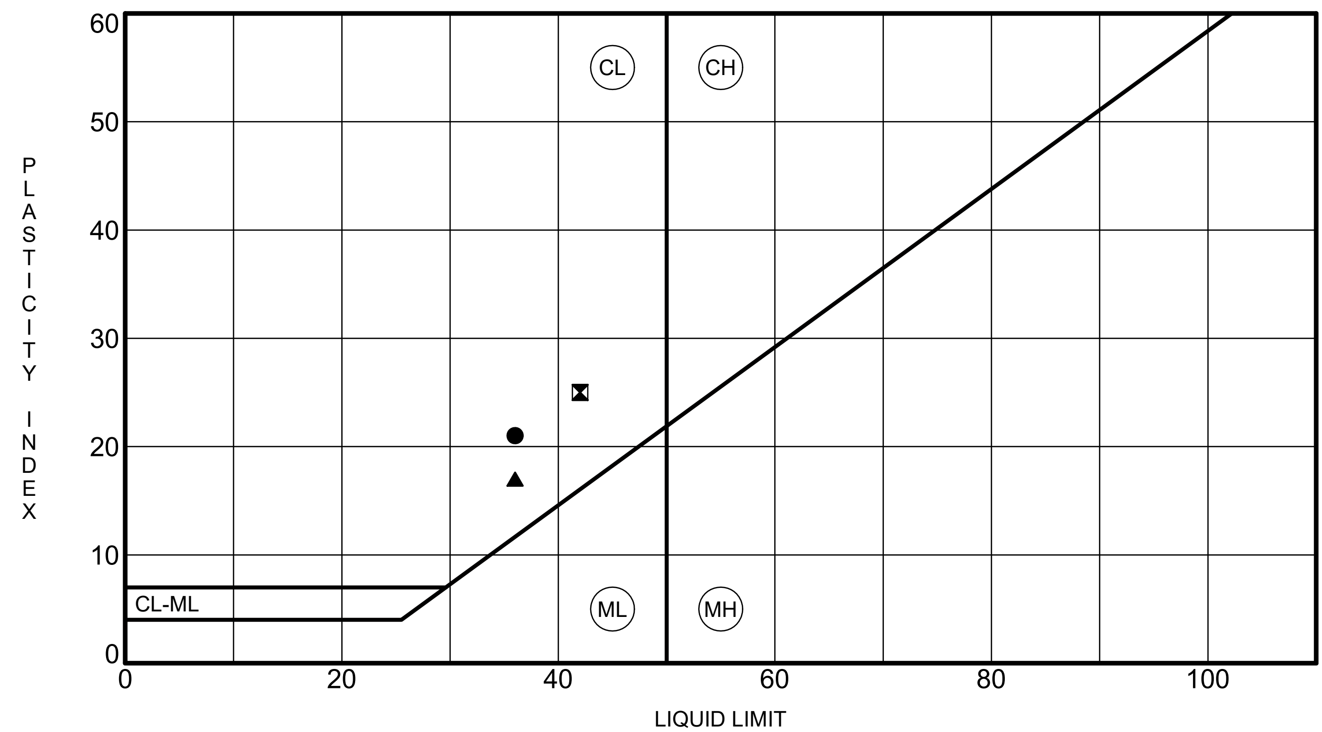
Tel 949.553.0370
Fax 949.553.0371

Table B-4 - Expansion Index

Boring No.	Depth (feet)	Expansion Index	Expansion Potential
B-1	0-5	93	High
B-2	0-5	116	High

Table B-5 – Corrosivity Test Results

Boring No.	Depth (feet)	pH	Minimum Resistivity (ohm-cm)	Water Soluble Sulfate (ppm)	Water Soluble Chloride (ppm)
B-1	0-5	7.2	870	181	128



	Sample Location	LL	PL	PI	U.S.C.S. Classification
●	**B-1 at 1 ft**	**36**	**15**	**21**	**Lean CLAY with Sand**
⧗	**B-2 at 1 ft**	**42**	**17**	**25**	**Lean CLAY with Sand**
▲	**B-2 at 40 ft**	**36**	**19**	**17**	**Lean CLAY with Sand**

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ATTERBERG LIMITS

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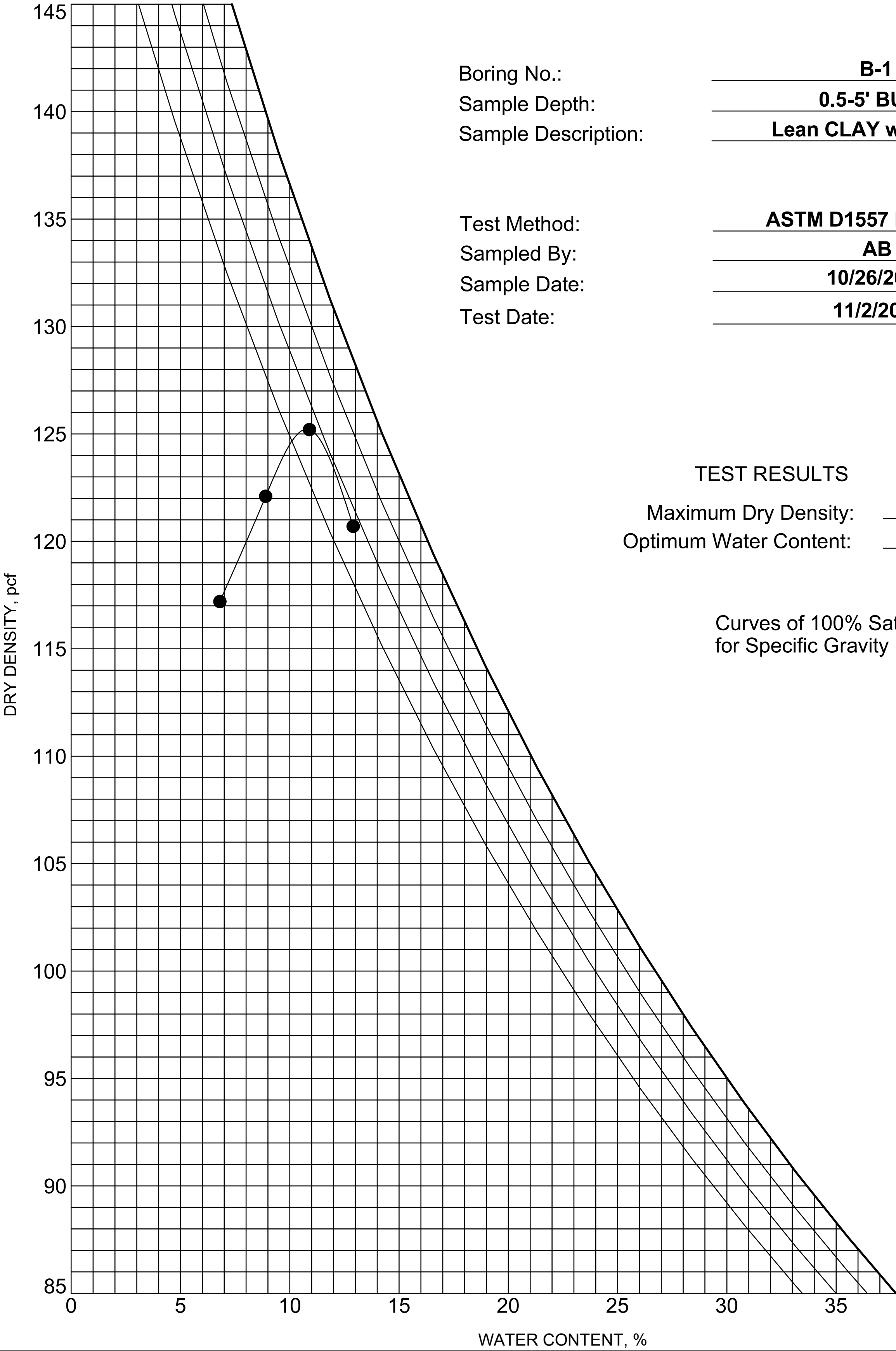
PROJECT NO.	REPORT DATE	
220695.1	December 2022	FIGURE B-1



Boring No.:	**B-1**
Sample Depth:	**0.5-5' BULK**
Sample Description:	**Lean CLAY with Sand**
Test Method:	**ASTM D1557 Method D**
Sampled By:	**AB**
Sample Date:	**10/26/2022**
Test Date:	**11/2/2022**

TEST RESULTS

Maximum Dry Density: **125.0** **pcf**

Optimum Water Content: **11.0** **%**

Curves of 100% Saturation
for Specific Gravity Equal to:

2.80

2.70

2.60

2.50

MOISTURE-DENSITY RELATIONSHIP

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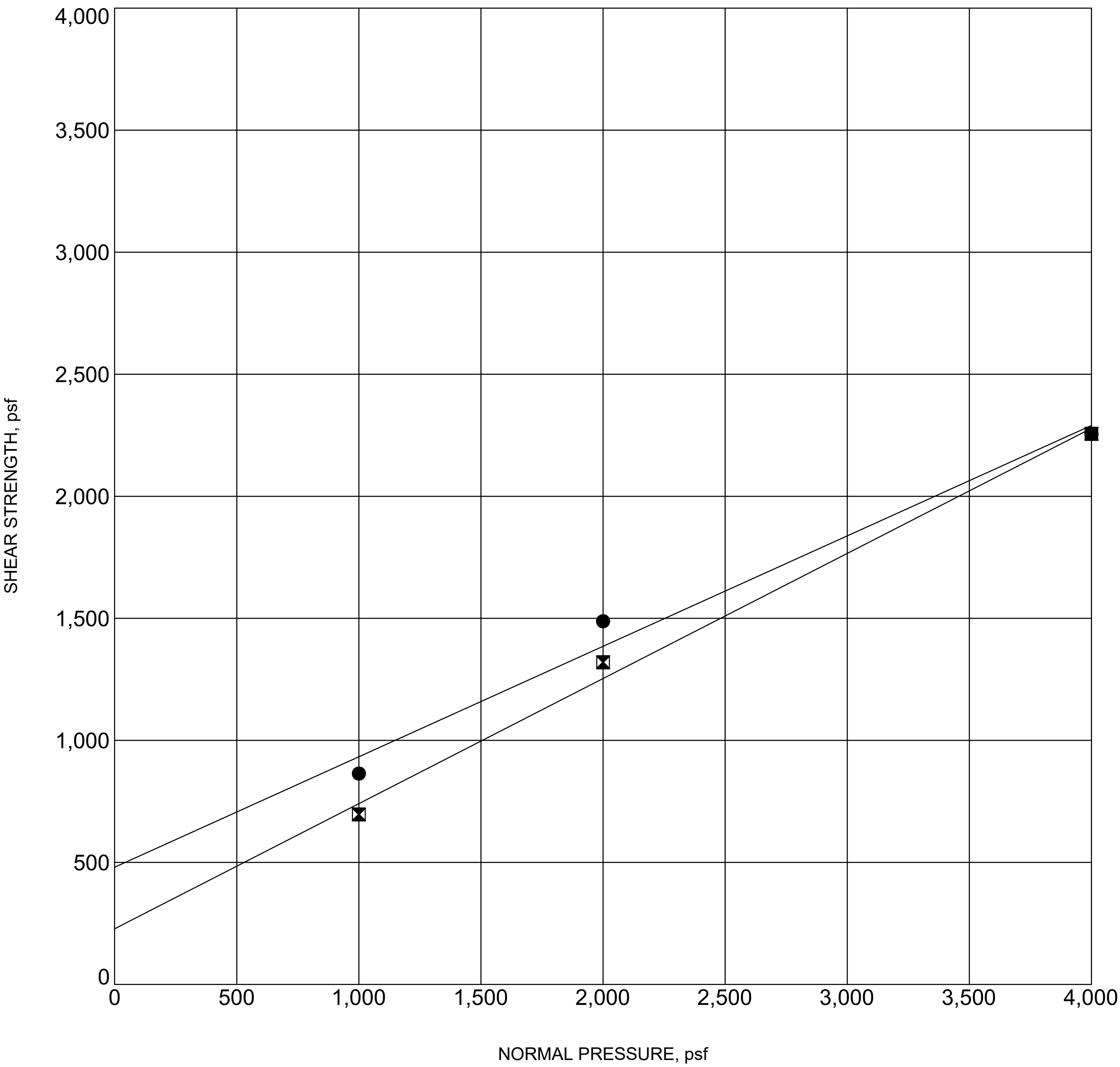
PROJECT NO.	REPORT DATE	
220695.1	December 2022	FIGURE B-2



Shear Strength Parameters

			Peak	Ultimate
Boring No.:	B-1			
Sample Depth (ft):	BULK 0.5-5'	**Cohesion, C (psf):**	480	228
Sample Description:	Lean CLAY with Sand	**Friction Angle, Ø (deg):**	24	27
Strain Rate (in./min):	0.005			
Dry Density (pcf):	125.2	**Initial Moisture (%):**	10.8	
		Final Moisture (%):	14.8	

NOTE: Sample remolded to 90% of maximum density per ASTM D 1557.

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REMOLDED DIRECT SHEAR TEST

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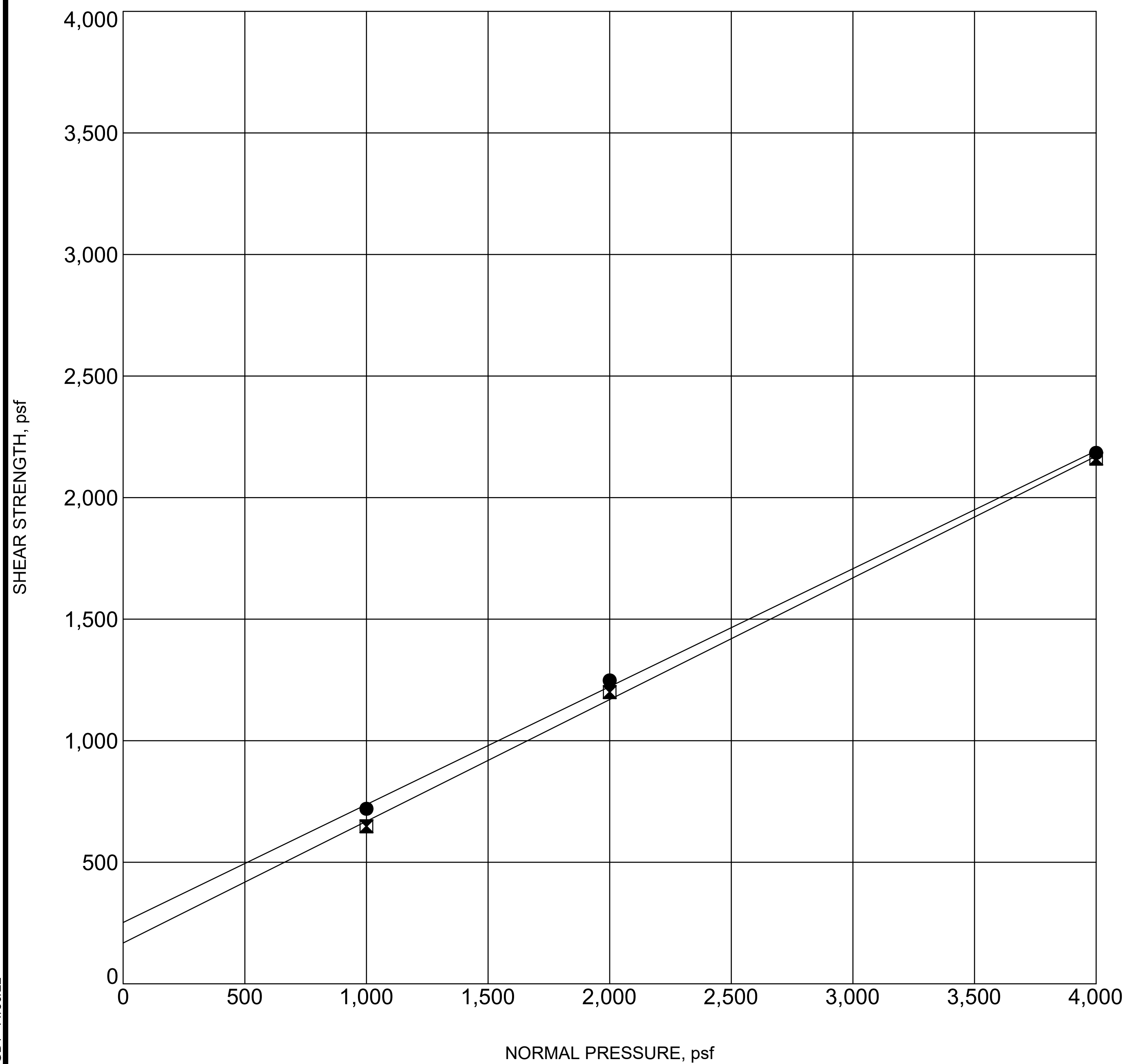
PROJECT NO.	REPORT DATE	
220695.1	December 2022	FIGURE B-3



Boring No.: B-2
Sample Depth (ft): 6
Sample Description: Lean CLAY with Sand
Strain Rate (in./min): 0.005
Dry Density (pcf): 110.9

Shear Strength Parameters

	Peak	Ultimate
Cohesion, C (psf):	252	168
Friction Angle, Ø (deg):	26	27

Initial Moisture (%): 10.2
Final Moisture (%): 16.9

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DIRECT SHEAR TEST

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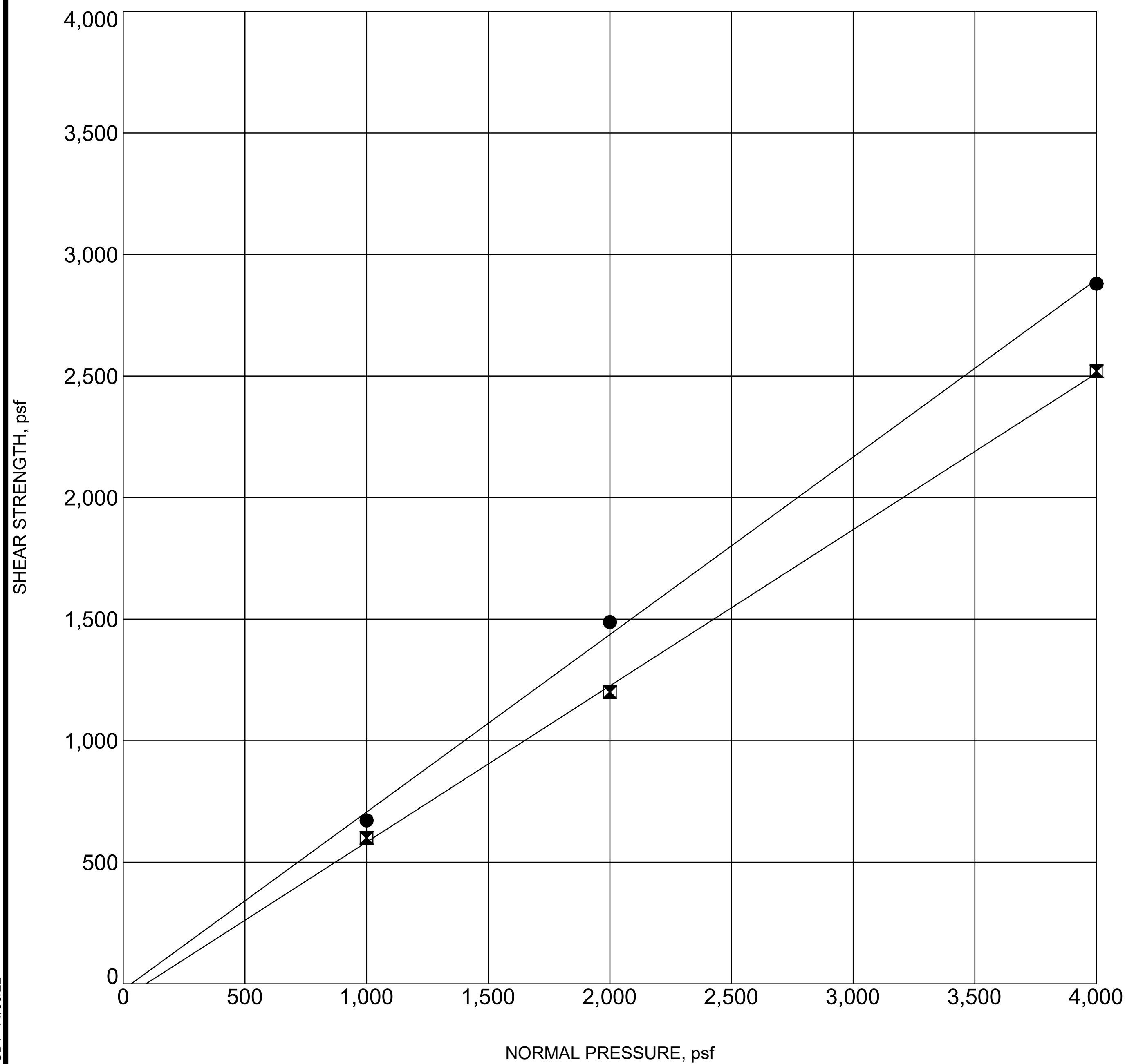
PROJECT NO.	REPORT DATE	FIGURE B-4
220695.1	December 2022	



Boring No.: B-2
Sample Depth (ft): 26
Sample Description: Poorly Graded SAND with Silt and Gravel
Strain Rate (in./min): 0.005
Dry Density (pcf): 106.2

Shear Strength Parameters

	Peak	Ultimate
Cohesion, C (psf):	0	0
Friction Angle, Ø (deg):	36	33

Initial Moisture (%): 2.7
Final Moisture (%): 16.2

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DIRECT SHEAR TEST

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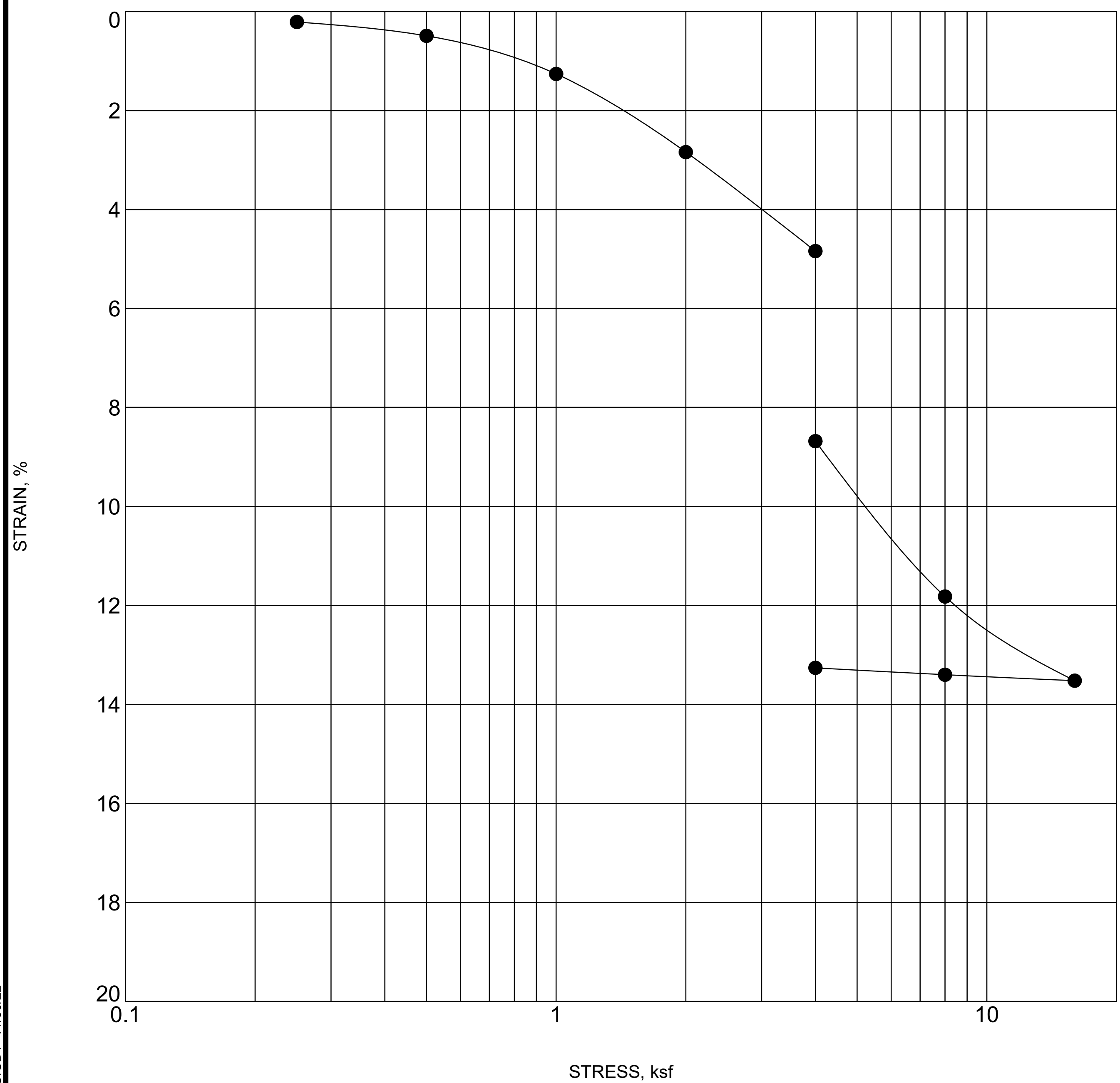
PROJECT NO.	REPORT DATE	
220695.1	December 2022	FIGURE B-5



	Sample Location	Soil Description	Dry Density (pcf)	Moisture Content (%)
●	**B-1 at 11 ft**	**Well graded SAND with Silt and Gravel**	**129.7**	**2.4**

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CONSOLIDATION TEST

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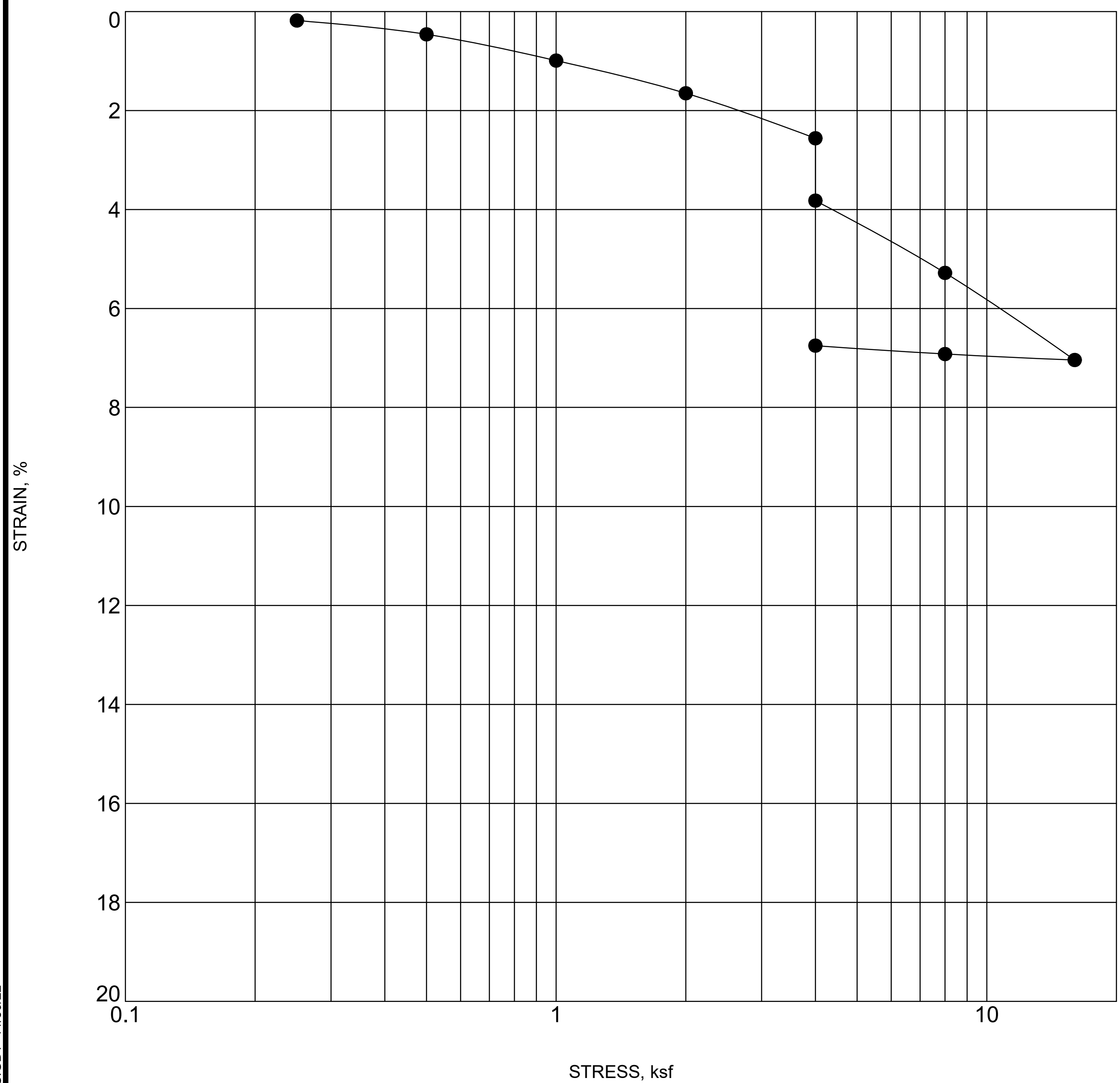
PROJECT NO.	REPORT DATE	
220695.1	December 2022	FIGURE B-6



	Sample Location	Soil Description	Dry Density (pcf)	Moisture Content (%)
●	**B-2 at 15 ft**	**Poorly graded SAND with Silt and Gravel**	**118.0**	**1.9**

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CONSOLIDATION TEST

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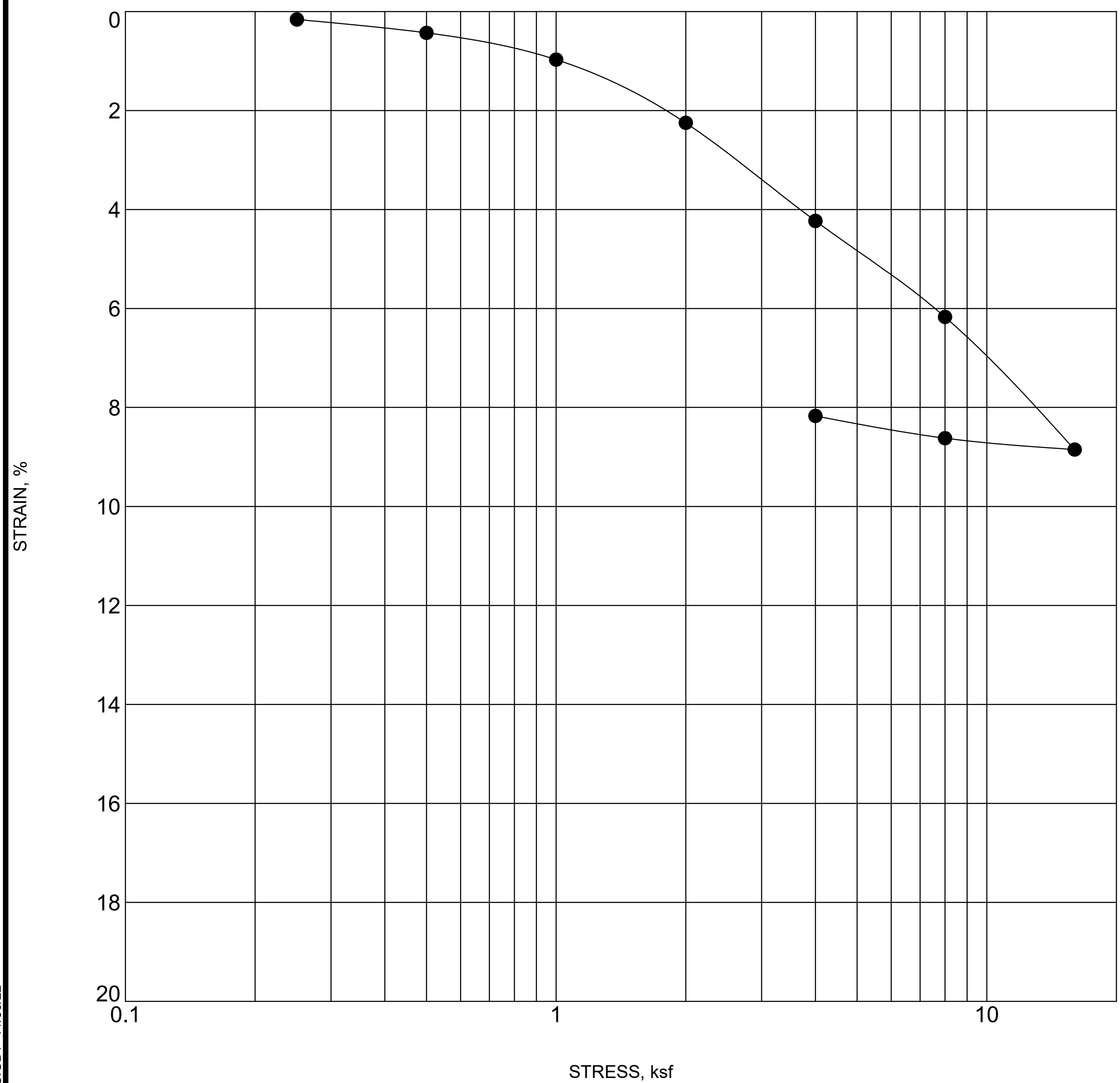
PROJECT NO.	REPORT DATE	
220695.1	December 2022	FIGURE B-7



	Sample Location	Soil Description	Dry Density (pcf)	Moisture Content (%)
●	**B-2 at 46 ft**	**Sandy Lean CLAY**	**95.4**	**26.0**

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CONSOLIDATION TEST

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PROJECT NO.	REPORT DATE	
220695.1	December 2022	FIGURE B-8

ANAHEIM TEST LAB, INC.

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

DATE: 11/8/2022

TWINING LABS
3310 AIRPORT WAY
LONG BEACH, CA 90806

P.O. NO.: Soils11122

LAB NO.: C-6510

SPECIFICATION: CTM-643/417/422

MATERIAL: Soil

Project No.: 220695.1
Project: Culver City HS Press Box
WO No.: W01-22-31576
Sample ID: B-1 (Bulk)
Sample Date: 10/26/2022

ANALYTICAL REPORT

CORROSION SERIES

SUMMARY OF DATA

pH	MIN. RESISTIVITY per CT. 643 ohm-cm	SOLUBLE SULFATES per CT. 417 ppm	SOLUBLE CHLORIDES per CT. 422 ppm
7.2	870	181	128

RESPECTFULLY SUBMITTED

WES BRIDGER LAB MANAGER