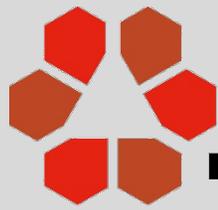


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**APPENDIX D7**  
**Geotechnical Report**

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**TWINING**

Engineering a Better Tomorrow

# **Geotechnical Investigation Report**

**Kaiser Redlands – MOB 2  
1301 California Street  
Redlands, California**

Prepared for:

Kaiser Foundation Health Plan, Inc.  
SBC NFS Office  
14520 Hawthorne Avenue  
Fontana, CA 92335

March 31, 2023  
Project No.: 220729.3



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Mr. James Fung  
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Kaiser Foundation Health Plan, Inc.  
SBC NFS Office  
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Fontana, CA 92335

**Subject:** Geotechnical Investigation Report  
Kaiser Redlands – MOB 2  
1301 California Street  
Redlands, California

Dear Mr. Fung,

In accordance with your request and authorization, we are presenting the results of our geotechnical investigation for the Medical Office Building 2 (MOB 2) project at the Kaiser Redlands site located at 1301 California Street in Redlands, California. The purpose of our investigation is to characterize subsurface conditions at the site, to identify seismic and geologic hazards present at the site, and to provide geotechnical engineering recommendations for the proposed improvements. This report was prepared in accordance with the requirements of the 2022 California Building Code (2022 CBC) and ASCE 7-16 (ASCE 2017). It is our understanding that the project will be submitted to the California Geological Survey (CGS) as it will fall under the jurisdiction of the Department of Health Care Access and Information (HCAI). Additionally, the report will be reviewed by the City of Redlands as part of the City permit approval process. Our report has been prepared to satisfy CGS, HCAI and the City of Redlands requirements.

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.

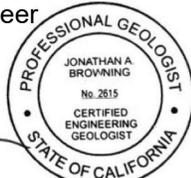
Respectfully submitted,

**TWINING, INC.**

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Table of Contents	Page
<b>1. INTRODUCTION .....</b>	<b>1</b>
<b>2. SITE DESCRIPTION AND PROPOSED DEVELOPMENT .....</b>	<b>1</b>
<b>3. SCOPE OF WORK .....</b>	<b>2</b>
<b>3.1. LITERATURE REVIEW .....</b>	<b>2</b>
<b>3.2. PRE-FIELD ACTIVITIES AND FIELD EXPLORATION .....</b>	<b>2</b>
<b>3.3. GEOTECHNICAL LABORATORY TESTING.....</b>	<b>3</b>
<b>3.4. ENGINEERING ANALYSES AND REPORT PREPARATION .....</b>	<b>3</b>
<b>4. GEOLOGY AND SUBSURFACE CONDITIONS .....</b>	<b>4</b>
<b>4.1. SITE GEOLOGY AND SUBSURFACE CONDITIONS .....</b>	<b>4</b>
4.1.1. Artificial Fill .....	4
4.1.2. Alluvial Fan Deposits.....	4
<b>4.2. GROUNDWATER CONDITIONS .....</b>	<b>4</b>
<b>5. GEOLOGIC HAZARDS AND SEISMIC DESIGN CONSIDERATIONS .....</b>	<b>5</b>
<b>5.1. HISTORICAL SEISMICITY .....</b>	<b>5</b>
<b>5.2. ACTIVE FAULTING AND SURFACE FAULT RUPTURE .....</b>	<b>7</b>
<b>5.3. LIQUEFACTION POTENTIAL AND SEISMIC SETTLEMENT .....</b>	<b>10</b>
<b>5.4. OTHER LIQUEFACTION EFFECTS .....</b>	<b>10</b>
<b>5.5. LANDSLIDES .....</b>	<b>10</b>
<b>5.6. FLOODING, INUNDATION, TSUNAMI AND SEICHE .....</b>	<b>10</b>
<b>5.7. DEAGGREGATED SEISMIC SOURCE PARAMETERS .....</b>	<b>11</b>
<b>5.8. SITE CLASS FOR SEISMIC DESIGN .....</b>	<b>11</b>
<b>5.9. MAPPED CBC SEISMIC DESIGN PARAMETERS .....</b>	<b>11</b>
<b>6. GEOTECHNICAL ENGINEERING RECOMMENDATIONS .....</b>	<b>12</b>
<b>6.1. GENERAL CONSIDERATIONS.....</b>	<b>13</b>
<b>6.2. SOIL COLLAPSE AND EXPANSION POTENTIAL .....</b>	<b>13</b>
<b>6.3. CORROSIVE SOIL EVALUATION .....</b>	<b>13</b>
6.3.1. Reinforced Concrete .....	13
6.3.2. Buried Metal .....	14
<b>6.4. SITE PREPARATION AND EARTH WORK.....</b>	<b>14</b>
6.4.1. Site Preparation .....	14
6.4.2. Temporary Excavations .....	14
6.4.3. Subgrade Preparation .....	15
6.4.4. Materials for Fill .....	16
6.4.5. Compacted Fill .....	16
6.4.6. Fill Slopes.....	16
6.4.7. Excavation Bottom Stability.....	17
6.4.8. Backfill for Utility Trench.....	17
6.4.9. Rippability.....	18
<b>6.5. FOUNDATION RECOMMENDATIONS.....</b>	<b>18</b>
6.5.1. Bearing Capacity and Settlement .....	18
6.5.2. Lateral Resistance .....	18
<b>6.6. SLAB-ON-GRADE .....</b>	<b>20</b>
<b>6.7. BELOW-GRADE WALLS.....</b>	<b>22</b>
6.7.1. Lateral Earth Pressure .....	22
6.7.2. Seismic Lateral Earth Pressure.....	22
6.7.3. Backfill and Drainage of Walls .....	23
<b>6.8. PAVEMENT RECOMMENDATIONS.....</b>	<b>24</b>



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6.8.1.	Flexible Pavement Design .....	24
6.8.2.	Rigid Pavement Design.....	24
<b>6.9.</b>	<b>STORMWATER INFILTRATION FACILITY .....</b>	<b>25</b>
<b>6.10.</b>	<b>DRAINAGE CONTROL .....</b>	<b>25</b>
<b>6.11.</b>	<b>SHORING RECOMMENDATIONS .....</b>	<b>26</b>
<b>7.</b>	<b>DESIGN REVIEW AND CONSTRUCTION MONITORING .....</b>	<b>26</b>
7.1.	PLANS AND SPECIFICATIONS .....	26
7.2.	CONSTRUCTION MONITORING.....	26
<b>8.</b>	<b>LIMITATIONS .....</b>	<b>27</b>
<b>9.</b>	<b>SELECTED REFERENCES .....</b>	<b>28</b>

### Figures

- Figure 1 – Site Location Map
- Figure 2 – Site Plan, Geologic Map, and Boring Location Map
- Figure 3 – Geologic Map
- Figure 4A – Cross Section A-A'
- Figure 4B – Cross Section B-B'
- Figure 5 – Historical Earthquake Epicenter Map
- Figure 6 – Regional Fault Map
- Figure 7 – Liquefaction and Landslide Hazards Zones Map

### Appendices

- Appendix A – Field Exploration
- Appendix B – Laboratory Testing
- Appendix C – Percolation Testing
- Appendix D – Geovision Results
- Appendix E – Seismic Settlement Analysis



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## 1. INTRODUCTION

This report presents the results of our geotechnical investigation for the Medical Office Building 2 (MOB 2) project at the Kaiser Redlands site located at 1301 California Street in Redlands, California. A description of the site and the proposed improvements are provided in the following section. The purpose of our investigation is to characterize subsurface conditions at the site, to identify seismic and geologic hazards present at the site, and to provide geotechnical engineering recommendations for the proposed improvements.

This report was prepared in accordance with the requirements of the 2022 California Building Code (2022 CBC) and ASCE 7-16 (ASCE 2017). It is our understanding that the project will be submitted to the California Geological Survey (CGS) as it will fall under the jurisdiction of the Department of Health Care Access and Information (HCAI). Additionally, the report will be reviewed by the City of Redlands as part of the City permit approval process. Our report has been prepared to satisfy CGS, HCAI and the City of Redlands requirements.

## 2. SITE DESCRIPTION AND PROPOSED DEVELOPMENT

The Kaiser Redlands site is situated at the northwest corner of California Street and Lugonia Avenue in the City of Redlands. The site is bound by California Street to the east, Lugonia Avenue to the south, Almond Avenue to the north and a commercial property to the west. The site is currently partly developed with the existing 3-story MOB 1 building situated near the southeast corner of the property and an asphalt-paved parking lot that surrounds MOB 1. The remainder of the site (approximately  $\frac{3}{4}$  of the site area) is undeveloped.

The proposed project is Phase 1 of the further development of the Kaiser Redlands site. The Phase 1 portion of the project consists of the development of MOB 2, which will be 4 stories tall with a footprint of approximately 41,250 square feet. Future phases of development at the site will include a 7-level parking structure, up to 7-story hospital building, 2-story central utility plant, and a 4-story medical office building (MOB 3). We note that our report presents recommendations for the MOB 2 and additional field exploration, laboratory testing, engineering analyses and reporting will be necessary for the other proposed structures upon determination of exact locations and details of the proposed structures other than MOB 2.

At the northwest corner of the project site, a stormwater retention basin is planned and our scope includes the investigation and feasibility of infiltrating stormwater within this basin. We have provided a separate report that presents the results of investigation of percolation and corresponding recommendations for design and implementation of stormwater infiltration devices.

Additionally, new asphalt-paved surface parking lots to the north, south and west of the proposed MOB 2 location are planned as part of Phase 1. A new entry drive from California Street and Almond Avenue is proposed as part of Phase 1.

The location of the project site is shown on Figure 1 – Site Location Map. The site plan and borings performed during this investigation are depicted on Figure 2 – Site Plan and Boring Location Map.

The approximate site coordinates where the proposed building is located are latitude 34.0717°N and longitude 117.2287°W, and the site is located on the Redlands, California 7½-Minute Quadrangle, based

on the United States Geological Survey (USGS) topographic maps (USGS 2021). The elevation of the existing grade at the site varies from 1,146 feet to 1,155 feet above mean sea level (MSL).

### **3. SCOPE OF WORK**

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

#### **3.1. Literature Review**

We reviewed readily available background data including proposed site improvement plans, published geologic maps, topographic maps, aerial photographs, seismic hazard maps and literature, and flood hazard maps relevant to the subject site. Relevant information has been incorporated into this report.

We reviewed a previous report prepared by LeRoy Crandall and Associates (Crandall, 1990), that was performed during the purchase of the property. Crandall performed 9 borings on the property that ranged in depth from 20 feet below ground surface (bgs) to 60 feet bgs. During the preparation of this report, we reviewed the report and logs prepared by Crandall. However, the plan prepared by Crandall showing the location of the borings was not available for review by us.

#### **3.2. Pre-Field Activities and Field Exploration**

Before starting our exploration program, we performed a site reconnaissance to observe the general surficial conditions at the site, to select field exploration locations, and to plan field logistics including health and safety. After exploration locations were delineated, Underground Service Alert was notified of the planned locations a minimum of 72 hours prior to excavation. Prior to the exploration, Geovision was used to perform utility scanning using Ground-Penetrating Radar at the proposed exploration locations to minimize the chance of damaging existing underground utilities. The results of the scanning are attached below in Appendix D.

The field exploration was conducted on February 2 and 3, 2023 and consisted of drilling, testing, sampling, and logging of 13 exploratory hollow-stem-auger (HSA) borings (B-1 through B-13). The borings deeper than 5 feet bgs were drilled using a CME-75 truck-mounted drill rig equipped with 8-inch-diameter HSA. Borings B-5 through B-13 were advanced to approximately 31.5 to 51.5 feet bgs and were within the footprint of the proposed building. Borings B-1 through B-4 were advanced to approximate 5 feet bgs and were within the proposed parking lot areas. All borings were first advanced to 5 feet bgs using a 5-inch-diameter hand auger to minimize the chance of damaging existing utilities. The approximate locations of the borings are shown on Figure 2 – Site Plan and Boring Location Map.

Drive samples of the subsurface materials were obtained from the borings using a Standard Penetration Test (SPT) sampler without liners and a modified California split spoon sampler. The samplers were driven using a 140-pound automatic hammer falling approximately 30 inches. The blow-counts to drive the samplers were recorded, and subsurface conditions encountered in the borings were logged by a Twining Engineer. Samples obtained from the borings were transported to Twining's geotechnical engineering laboratory for examination and testing.

Percolation testing was also performed in accordance with the 2013 Technical Guidance Document for Water Quality Management Plans of the San Bernardino County. The testing was performed to

provide an estimate of infiltration rate of the site soils for use in preliminary design of a storm water management system. The locations of the percolation tests and their results are attached in Appendix C – Percolation Testing.

Upon completion of drilling, sampling and testing, the borings were backfilled by the drilling subcontractor using neat cement grout.

Detailed descriptions of the borings and the soils encountered during drilling are presented in Appendix A.

### **3.3. Geotechnical Laboratory Testing**

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

- In-situ moisture and density;
- #200 Wash;
- Grain size analysis;
- Atterberg Limits;
- Expansion Index;
- Consolidation;
- Direct shear;
- R-Value; and
- Corrosivity.

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

### **3.4. 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;
- Temporary excavations;
- Project feasibility and suitability of on-site soils for foundation support;
- Foundation design parameters including bearing capacity, settlement, and lateral resistance;

- Modulus of subgrade reaction for mat foundation and concrete slab-on-grade design;
- Lateral earth pressures for below-grade walls;
- Concrete slab-on-grade support; and
- Pavement section recommendations.

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

#### **4. GEOLOGY AND SUBSURFACE CONDITIONS**

The geology and subsurface conditions at the site are based on the results of our field investigation (Appendix A) and our review of published geologic maps (Figure 3).

##### **4.1. Site Geology and Subsurface Conditions**

According to the geologic map prepared by Dibblee, the project site is underlain by Holocene-aged alluvial deposits (Geologic Symbol Qa) consisting of “alluvial sand and clay of valley areas” (Dibblee, 2004). A portion of the geologic map is reproduced as Figure 3 – Geologic Map. A generalized description of the subsurface conditions encountered is provided below. Detailed descriptions of the earth materials encountered in the exploratory borings are presented in Appendix A – Field Exploration.

###### **4.1.1. Artificial Fill**

Subsurface conditions encountered in our and previous other’s borings consisted of approximately 5 feet of artificial fill in borings B-5 and B-11 on the eastern edge of the building to approximately 3 feet of artificial fill on borings B-8, B-9, and B-12 on the western edge of the building. The fill in the proposed building area and other areas consisted of silty sand with some sandy silt. The fill is generally dark brown to reddish brown, slightly moist, and consists mostly of fine sand.

It appears that the fill was associated with previous grading operations performed at the site, however, no documentation for the placement and compaction of the fill was available for our review. It is recommended that any compaction reports related to the fill placement be provided to us for an evaluation; if these documents are not available, the fill materials will require removal and replacement as part of the grading for the new MOB building pad.

###### **4.1.2. Alluvial Fan Deposits**

Holocene-aged old alluvial deposits (Qa) underly the site to the maximum depth of exploration. The deposits in the proposed building area and most other areas consist predominantly of medium dense to very dense silty sand with some sandy silt that varied primarily from stiff to very stiff.

##### **4.2. Groundwater Conditions**

Based on the previous report prepared by Leroy Crandall and Associates (Crandall, 1990), two wells (1S/3W-19J2 and 1S/3W-19L1) were located within 1300 feet of the site. Groundwater has been as shallow as 33 feet below ground surface (bgs) in the 1930’s and 1940’s and fell to a maximum of 210 feet bgs in the 1960s. During 1986, water was at approximately 74 feet bgs in one of the wells (1,054 feet MSL). Woodward-Clyde prepared a geotechnical report for the larger parcel which included the site in 1989 and encountered water at a depth of 50 feet bgs in one of their borings.



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Groundwater was not encountered in any of the borings performed by Crandall to a maximum depth of 60 feet bgs.

According to the California Department of Water Resources Water Data Library, there are two wells in the vicinity of the site: State Well Number 01S03W19L001S (Site Code 340686N1172353W001, ground surface elevation 1,130.53 feet MSL) and State Well Number 01S03W19G002S (Site Code 340739N1172334W001, ground surface elevation 1,137.52 feet MSL). The first is approximately 2,200 feet southwest of the site and the second is approximately 1,500 feet to the northwest. At the first well, the groundwater elevation varied from 944 feet MSL to 981 feet MSL from 2005 to 2008, which corresponds to a groundwater depth of approximately 166 feet bgs to 203 feet bgs at the current site. The groundwater elevation at the second site varied from a high of 1,122 feet MSL in 1945 to a low of 912 feet MSL in 1966. While the groundwater levels fell from the late 1940s to 1966, they steadily increased until 1987 when they reached a local high of 1,061 feet MSL. However, from 1991 to 2000, when measurement stopped, the water level remained relatively constant at approximately 1,018 feet above MSL which corresponds to approximately 129 feet bgs at the current site.

During the current drilling for the office building, three borings were advanced by Twining to a depth of 51.5 feet bgs and groundwater was not encountered in any of the borings.

Based on the findings by Crandall, the data from the Water Data Library, and our current borings, the depth to groundwater can be assumed to be greater than 50 feet bgs for design purposes.

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

## **5. GEOLOGIC HAZARDS AND SEISMIC DESIGN CONSIDERATIONS**

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

### **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.

**Table 1 – Significant Historical Seismicity**

Event Time (UTC)	Magnitude	Event Place	Latitude	Longitude	Depth (miles)	Distance to site (miles)
1910-05-13T06:20	5 ml	7 km WNW of Lake Elsinore, California	33.70	-117.40	-	27.5
1910-04-11T07:57	5 ml	7 km WNW of Lake Elsinore, California	33.70	-117.40	-	27.5
1918-06-06T22:32	5 ml	4 km WNW of San Jacinto, California	33.80	-117.00	-	22.9
1944-06-12T10:45	5.06 ml	10km NNE of Cabazon, CA	33.99	-116.73	6	29.1
1930-01-16T00:34	5.1 uk	4 km SSE of Big Bear Lake, California	34.20	-116.90	-	20.8
1970-09-12T14:30	5.22 ml	3km W of Lytle Creek, CA	34.25	-117.53	10.75	21.6
1938-05-31T08:34	5.23 ml	8km ENE of Trabuco Canyon, CA	33.70	-117.51	10.19	30.4
1992-08-17T20:41	5.23 ml	7km SE of Big Bear Lake, California	34.20	-116.86	9.44	22.6
1944-06-12T11:16	5.24 ml	12km WSW of Morongo Valley, CA	34.00	-116.70	6	30.7
1930-01-16T00:24	5.25 ms	4 km SSE of Big Bear Lake, California	34.20	-116.90	-	20.8
1992-06-28T17:05	5.26 ml	1km N of Big Bear Lake, California	34.26	-116.91	6.553	22.1
1992-12-04T02:08	5.26 ml	10km SE of Lucerne Valley, California	34.37	-116.90	1.325	27.9
1943-08-29T03:45	5.28 ml	6km WNW of Big Bear Lake, CA	34.27	-116.97	6	20.2
1963-09-23T14:41	5.29 ml	6km SSE of Hemet, CA	33.70	-116.94	10.66	30.4
1992-11-27T16:00	5.29 ml	10km NNW of Big Bear City, California	34.34	-116.90	-0.402	26.4
1907-09-20T01:54	5.3 mw	1 km SE of Running Springs, California	34.20	-117.10	-	11.5
1910-05-15T15:47	5.3 mw	7 km WNW of Lake Elsinore, California	33.70	-117.40	-	27.5
1892-06-14T13:25	5.5 ml	Northeast of Rancho Cucamonga, California	34.20	-117.50	-	17.9
1990-02-28T23:43	5.51 ml	6km NNE of Claremont, CA	34.14	-117.70	3.292	27.3
1992-06-28T14:43	5.53 ml	11km SSE of Big Bear Lake, California	34.16	-116.85	9.615	22.4
1999-10-16T09:59	5.6 mb	7km ENE of Running Springs, CA	34.24	-117.04	6	15.9
1889-02-07T05:20	5.6 ml	San Bernardino Mountains northeast of Beaumont, California	34.10	-116.70	-	30.3
1880-12-19T23:35	5.9 ml	Near San Bernardino, California	34.00	-117.00	-	14.0
1899-07-22T00:46	5.9 ml	Near San Bernardino, California	34.20	-117.40	-	13.2
1858-12-16T10:00	6 ml	Near San Bernardino, California	34.20	-117.40	-	13.2
1894-07-30T05:12	6.2 mw	Northwest of San Bernardino, California	34.30	-117.60	-	26.4
1923-07-23T07:30	6.21 mw	3 km SE of San Bernardino, California	34.09	-117.26	5	2.1
1992-06-28T15:05	6.3 mw	The 1992 Big Bear Earthquake, California	34.20	-116.83	3.634	24.7
1899-07-22T20:32	6.36 mw	Cajon Pass area, northwest of San Bernardino, California	34.30	-117.50	-	22.1
1918-04-21T22:32	6.7 mw	1 km N of Hemet, California	33.76	-116.97	10	26.0
1899-12-25T12:25	6.7 mw	Near San Jacinto, California	33.80	-117.00	-	22.9
1910-05-13T06:20	5 ml	7 km WNW of Lake Elsinore, California	33.70	-117.40	-	27.5
1910-04-11T07:57	5 ml	7 km WNW of Lake Elsinore, California	33.70	-117.40	-	27.5
1918-06-06T22:32	5 ml	4 km WNW of San Jacinto, California	33.80	-117.00	-	22.9

**Notes:**

- (1) ml = local magnitude, commonly referred to as "Richter magnitude"; mw = moment magnitude; ms = surface wave magnitude; mb = body wave magnitude; mh = Non-standard magnitude method; uk = uk magnitude.
- (2) NA = not available

## 5.2. Active Faulting and Surface Fault Rupture

The site is not located within an Alquist-Priolo Earthquake Fault Zone (EFZ) (CGS 2016) or a fault zone mapped by the City of Redlands. The boundary of the closest Alquist-Priolo EFZs are located approximately 2.75 miles southwest of the site and 4.75 miles northeast of the site and are associated with the San Bernardino South and Harrison Mountain fault zones, respectively. Based on our search of the 2008 national fault database (Petersen et al., 2008), the known active fault closest to the site is the San Jacinto fault system which encompasses the San Bernardino South fault system.

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.

**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)
2.87	San Jacinto;SBV+SJV+A	n/a	90	V	strike slip	0	16	134
2.87	San Jacinto;SBV+SJV+A+CC+B+SM	n/a	90	V	strike slip	0.1	15	241
2.87	San Jacinto;SBV+SJV+A+CC+B	n/a	90	V	strike slip	0.1	15	215
2.87	San Jacinto;SBV+SJV+A+CC	n/a	90	V	strike slip	0	16	181
2.87	San Jacinto;SBV	6	90	V	strike slip	0	16	45
2.87	San Jacinto;SBV+SJV+A+C	n/a	90	V	strike slip	0	17	181
2.87	San Jacinto;SBV+SJV	n/a	90	V	strike slip	0	16	88
3.80	San Jacinto;SJV+A+CC+B+SM	n/a	90	V	strike slip	0.1	15	196
3.80	San Jacinto;SJV+A+CC+B	n/a	90	V	strike slip	0.1	15	170
3.80	San Jacinto;SJV+A+CC	n/a	90	V	strike slip	0	16	136
3.80	San Jacinto;SJV+A+C	n/a	90	V	strike slip	0	17	136
3.80	San Jacinto;SJV+A	n/a	90	V	strike slip	0	17	89
3.80	San Jacinto;SJV	18	90	V	strike slip	0	16	43
5.07	S. San Andreas;SSB+BG	n/a	71		strike slip	0	13	101
5.07	S. San Andreas; CH+CC+BB+NM+SM+NSB+SSB +BG+CO	n/a	86		strike slip	0.1	13	512
5.07	S. San Andreas;NSB+SSB+BG+CO	n/a	79		strike slip	0.2	12	206
5.07	S. San Andreas; CC+BB+NM+SM+NSB+SSB	n/a	90	V	strike slip	0	14	322
5.07	S. San Andreas; CC+BB+NM+SM+NSB+SSB+BG	n/a	85		strike slip	0	14	380



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5.07	S. San Andreas; CC+BB+NM+SM+NSB+SSB+BG +CO	n/a	86		strike slip	0.1	13	449
5.07	S. San Andreas; CH+CC+BB+NM+SM+NSB+SSB	n/a	90	V	strike slip	0	14	384
5.07	S. San Andreas; CH+CC+BB+NM+SM+NSB+SSB +BG	n/a	86		strike slip	0	14	442
5.07	S. San Andreas;NM+SM+NSB+SSB	n/a	90	V	strike slip	0	13	213
5.07	S. San Andreas; NM+SM+NSB+SSB+BG	n/a	83		strike slip	0	14	271
5.07	S. San Andreas; NM+SM+NSB+SSB+BG+CO	n/a	84		strike slip	0.1	13	340
5.07	S. San Andreas;NSB+SSB	n/a	90	V	strike slip	0	13	79
5.07	S. San Andreas;NSB+SSB+BG	n/a	75		strike slip	0	14	136
5.07	S. San Andreas; PK+CH+CC+BB+NM+SM+NSB+ SSB	n/a	90	V	strike slip	0.1	13	421
5.07	S. San Andreas;PK+CH+CC+BB+NM+S M+NSB+SSB+BG	n/a	86		strike slip	0.1	13	479
5.07	S. San Andreas; PK+CH+CC+BB+NM+SM+NSB+ SSB+BG+CO	n/a	86		strike slip	0.1	13	548
5.07	S. San Andreas;SM+NSB+SSB	n/a	90	V	strike slip	0	13	176
5.07	S. San Andreas;SM+NSB+SSB+BG	n/a	81		strike slip	0	13	234
5.07	S. San Andreas; SM+NSB+SSB+BG+CO	n/a	83		strike slip	0.1	13	303
5.07	S. San Andreas;SSB	16	90	V	strike slip	0	13	43
5.07	S. San Andreas; SSB+BG+CO	n/a	77		strike slip	0.2	12	170
5.07	S. San Andreas; BB+NM+SM+NSB+SSB	n/a	90	V	strike slip	0	14	263
5.07	S. San Andreas; BB+NM+SM+NSB+SSB+BG	n/a	84		strike slip	0	14	321
5.07	S. San Andreas; BB+NM+SM+NSB+SSB+BG+CO	n/a	85		strike slip	0.1	13	390
5.41	S. San Andreas; BB+NM+SM+NSB	n/a	90	V	strike slip	0	14	220
5.41	S. San Andreas; NM+SM+NSB	n/a	90	V	strike slip	0	13	170
5.41	S. San Andreas; CH+CC+BB+NM+SM+NSB	n/a	90	V	strike slip	0	14	341
5.41	S. San Andreas; PK+CH+CC+BB+NM+SM+NSB	n/a	90	V	strike slip	0.1	13	377
5.41	S. San Andreas;SM+NSB	n/a	90	V	strike slip	0	13	133
5.41	S. San Andreas; CC+BB+NM+SM+NSB	n/a	90	V	strike slip	0	14	279
5.41	S. San Andreas;NSB	22	90	V	strike slip	0	13	35
14.08	Cleghorn	3	90	V	strike slip	0	16	25
14.24	San Jacinto;A+CC+B	n/a	90	V	strike slip	0.1	15	152
14.24	San Jacinto;A	9	90	V	strike slip	0	17	71
14.24	San Jacinto;A+C	n/a	90	V	strike slip	0	17	118

14.24	San Jacinto;A+CC+B+SM	n/a	90	V	strike slip	0.1	15	178
14.24	San Jacinto;A+CC	n/a	90	V	strike slip	0	16	118
14.36	Cucamonga	5	45	N	thrust	0	8	28
17.07	North Frontal (West)	1	49	S	reverse	0	16	50
24.95	S. San Andreas;SM	29	90	V	strike slip	0	13	98
24.95	S. San Andreas; CC+BB+NM+SM	n/a	90	V	strike slip	0	14	243
24.95	S. San Andreas;NM+SM	n/a	90	V	strike slip	0	14	134
24.95	S. San Andreas; PK+CH+CC+BB+NM+SM	n/a	90	V	strike slip	0.1	13	342
24.95	S. San Andreas; CH+CC+BB+NM+SM	n/a	90	V	strike slip	0	14	306
24.95	S. San Andreas;BB+NM+SM	n/a	90	V	strike slip	0	14	184
25.60	Elsinore;W+GI+T+J+CM	n/a	84	NE	strike slip	0	16	241
25.60	Elsinore;W+GI	n/a	81	NE	strike slip	0	14	83
25.60	Elsinore;W+GI+T	n/a	84	NE	strike slip	0	14	124
25.60	Elsinore;W+GI+T+J	n/a	84	NE	strike slip	0	16	199
25.60	Elsinore;GI+T	5	90	V	strike slip	0	14	78
25.60	Elsinore;GI+T+J	n/a	86	NE	strike slip	0	17	153
25.60	Elsinore;GI+T+J+CM	n/a	86	NE	strike slip	0	16	195
25.60	Elsinore;GI	5	90	V	strike slip	0	13	37
25.80	S. San Andreas;BG+CO	n/a	72		strike slip	0.3	12	125
25.80	S. San Andreas;BG	n/a	58		strike slip	0	13	56
25.83	Chino, alt 2	1	65	SW	strike slip	0	14	29
26.06	Chino, alt 1	1	50	SW	strike slip	0	9	24
26.66	San Jose	0.5	74	NW	strike slip	0	15	20
26.70	Elsinore;W	2.5	75	NE	strike slip	0	14	46
29.13	Pinto Mtn	2.5	90	V	strike slip	0	16	74
29.35	Elsinore;T	5	90	V	strike slip	0	14	52
29.35	Elsinore;T+J	n/a	86	NE	strike slip	0	17	127
29.35	Elsinore;T+J+CM	n/a	85	NE	strike slip	0	16	169
29.39	Helendale-So Lockhart	0.6	90	V	strike slip	0	13	114
29.58	Sierra Madre Connected	2	51		reverse	0	14	76
29.58	Sierra Madre	2	53	N	reverse	0	14	57
30.58	North Frontal (East)	0.5	41	S	thrust	0	16	27

**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 Potential and Seismic Settlement**

Liquefaction is the phenomenon in which loosely deposited granular soils with silt and clay contents generally 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 not within areas susceptible to liquefaction mapped by the City of Redlands (Figure 7). Additionally, groundwater is likely deeper than 50 feet. It is our opinion that liquefaction potential at the site is negligible.

Seismic settlement can occur when medium dense granular materials densify during seismic shaking and/or liquefaction. Seismically-induced settlement may occur in dry, unsaturated, as well as saturated soils. In light of deep groundwater at the site, we performed seismic settlement analysis for the unsaturated soils using data from the 50-foot-deep borings B-5, B-10 and B-12. Details of the analysis are provided in Appendix E. Results of the analysis indicate a total seismic settlement of approximately one to two inches. Based on the distances between the borings and the calculated total settlement, the differential settlement is expected to be less than ½ inches over a horizontal distance of 50 feet.

### **5.4. Other Liquefaction Effects**

The potential for other liquefaction effects (i.e., liquefaction-induced bearing capacity failure, surface manifestation, and lateral spread) at the site is considered negligible because the site has negligible liquefaction potential.

### **5.5. Landslides**

According to the Flood Hazards map in the General Plan of the City of Redlands, the site is not located within an area susceptible to landslides. There are no known landslides at the site, nor is the site in the path of any known or potential landslides. In addition, the site is not adjacent to significant slopes. The potential for earthquake-induced landslides to occur at the site is considered low.

### **5.6. Flooding, Inundation, Tsunami and Seiche**

Tsunamis are waves generated by massive landslides near or under sea water. Seiches are standing wave oscillations of an enclosed or semi-enclosed water body (e.g., a lake, reservoir, or bay) after the original driving force has dissipated. Resulting oscillation could cause waves up to tens of feet high, which in turn could cause extensive damage along the shoreline. The most serious consequences of a seiche would be the overtopping and failure of a dam.

The site is not located within a coastal area or near an enclosed or semi-enclosed water body. According to the Flood Hazards map in the General Plan of the City of Redlands, the site is not

located within a 100- or 500-year floodplain, or a dam inundation area. Therefore, flood, inundation, tsunami and seiche hazards at the site are considered low.

#### **5.7. 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  $M_w$  and fault distance  $R$  are 8.1 and 3.85 miles (6.2 km), respectively.

#### **5.8. Site Class for Seismic Design**

During the field exploration, Geovision performed multi-channel analysis of surface waves (MASW) to determine the shear wave velocity profile to a depth of 100 feet ( $V_{s,30}$ ). They determined that the average shear wave velocity in the area of the proposed structure is 944 ft/s in the upper 100 feet. Based on the  $V_{s,30}$ , Site Class D may be used for the project seismic design according to Chapter 20 of ASCE 7-16. Based on Table 20.3-1 of ASCE 7-16, the ranges of shear wave velocities in the upper 100 feet for Site Class D are greater than 600 ft/s and less than or equal to 1,200 ft/s.

#### **5.9. Mapped CBC Seismic Design Parameters**

Our recommendations for seismic design parameters have been developed in accordance with the 2022 CBC and ASCE 7-16 (ASCE 2017) standards. As the site is classified as seismic Site Class D and the mapped spectral acceleration parameter at period 1-second,  $S_1$ , is greater than 0.2 g, a site-specific ground motion hazard analysis is required according to Section 11.4.7 of ASCE 7-16.

As an alternative, Exception 2 in Section 11.4.8 of ASCE 7-16 may be used for the project. For structural design based on this exception, Table 3 presents the seismic design parameters for the site based on coordinates of latitude 34.0717°N and longitude 117.2287°W.

**Table 3 – 2022 California Building Code Seismic Design Parameters  
for Design Based on Exception 2 in Section 11.4.8 of ASCE 7-16**

Design Parameters	Value
Site Class	D
Mapped Spectral Acceleration Parameter at Period of 0.2-Second, $S_s$ (g)	1.956
Mapped Spectral Acceleration Parameter at Period 1-Second, $S_1$ (g)	0.772
Site Coefficient, $F_a$	1
Site Coefficient, $F_v$	1.7
Adjusted $MCE_R^1$ Spectral Response Acceleration Parameter, $S_{MS}$ (g)	1.956
Adjusted $MCE_R^1$ Spectral Response Acceleration Parameter, $S_{M1}$ (g)	1.312
Design Spectral Response Acceleration Parameter, $S_{DS}$ (g)	1.304
Design Spectral Response Acceleration Parameter, $S_{D1}$ (g)	0.874
Risk Coefficient $C_{RS}$	0.916
Risk Coefficient $C_{R1}$	0.891
Peak Ground Acceleration, $PGA_M^2$ (g)	0.908
Seismic Design Category <sup>3</sup>	E
Long-Period Transition Period, $T_L$ (seconds)	8
$T_S = S_{D1} / S_{DS}$	0.670
When using the above parameters for seismic design, the seismic design coefficient $C_s$ should be calculated as follows: For $T \leq 1.5T_S$ , $C_s = S_{DS}/(R/I_e)$ For $T_L \geq T > 1.5T_S$ , $C_s = 1.5 S_{D1}/(T R/I_e)$ For $T > T_L$ , $C_s = 1.5 (S_{D1} T_L)/(T^2 R/I_e)$ where T = the fundamental period of the structure(s) determined in Section 12.8.2 of ASCE 7-16; R = the response modification factor determined in Table 12.2-1 of ASCE 7-16; and $I_e$ = the importance factor determined in accordance with Section 11.5.1 of ASCE 7-16.	
Notes: <sup>1</sup> Risk-Targeted Maximum Considered Earthquake. <sup>2</sup> Peak Ground Acceleration adjusted for site effects. <sup>3</sup> For $S_1$ greater than or equal to 0.75 g, the Seismic Design Category is E for risk category I, II, and III structures and F for risk category IV structures.	

## 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.

Fill encountered in the borings drilled in the building footprint was up to 5 feet and could be more or less between borings. At the time of preparing this report, no information is available for our review regarding the proposed site grades of the project.

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 Collapse and Expansion Potential**

Based on our consolidation test results, site subsurface materials have negligible collapse. We note that one sample (boring B-12 at 5 feet bgs) exhibited approximately 8.3% collapse potential when wetted but this result is not considered reliable because of obvious relatively large sample disturbance observed.

Based on the results of our laboratory potential expansion index tests, surficial soils have very low expansion potential.

## **6.3. 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 a selected near-surface soil sample 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.

Corrosive soil may be defined as the soil has minimum electrical resistivity less than 1,000 ohm-centimeters, or chloride concentration equal to or greater than 500 parts per million (ppm), or sulfate concentration in soils equal to or greater than 2,000 ppm, or a pH equal to or less than 5.5 (e.g., based on the County of Los Angeles criteria).

### **6.3.1. Reinforced Concrete**

Based on ACI 318 and laboratory test results, concrete in contact with the site soils will have a sulfate exposure class S0. As a minimum, we recommend that Type II cement and a water-cement ratio of no greater than 0.50 be used on the project.

Test results indicate that the potential is low for chloride attack of reinforcing steel in concrete structures and pipes in contact with subsurface materials. However, if needed, a corrosion specialist may be consulted for protection from chloride attack.

### **6.3.2. Buried Metal**

A factor for evaluating corrosivity to buried metal is electrical resistivity. The electrical resistivity of a soil is a measure of resistance to electrical current. Corrosion of buried metal is directly proportional to the flow of electrical current from the metal into the soil. As resistivity of the soil decreases, the corrosivity generally increases. Test results indicate the site soils have a minimum electrical resistivity value of 2,200 ohm-centimeters. Based on the criteria of the County of Los Angeles, the soils are not considered to be corrosive to buried metals.

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

## **6.4. Site Preparation and Earth Work**

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

### **6.4.1. Site Preparation**

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

### **6.4.2. Temporary Excavations**

Unsurcharged temporary excavations less than with vertical sides less than 4 feet high are generally expected stable. Where space is available, temporary, un-surcharged excavation sides over 4 feet in height should be sloped back at 1.5H:1V (horizontal:vertical) or flatter.

The tops of the excavation sides should be barricaded so that vehicles and storage loads are away from the top edge of the excavated slopes with a distance at least equal to the height of the slopes. A greater setback may be necessary when considering heavy vehicles, such as concrete trucks and cranes. Twining should be advised of such heavy vehicle loadings so that specific setback requirements can be established. If the temporary construction slopes are to be maintained during the rainy season, berms are recommended to be graded along the tops of the slopes to prevent runoff water from entering the excavation and eroding the slope faces.

Excavations should not undermine existing adjacent footings. We recommend that excavations for the proposed improvements do not encroach within an imaginary 1:1 plane projected down and outward from the closest bottom edge of any existing foundations of at-grade or below-grade existing facilities including foundations of existing structures, trenches, underground

pipelines. Otherwise, temporary shoring or slot-cut should be implemented to maintain support of adjacent facilities.

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

#### **6.4.3.Subgrade Preparation**

Footings should bear on at least 3 feet of properly compacted engineered fill. Undocumented fill in the building footprint should be removed to its full depth. Undocumented fill below foundations of retaining walls should also be removed to its full depth and the footings should bear on at least 3 feet of properly compacted fill. Laterally, excavations should extend 5 feet beyond the limits of the foundations or the depth of over-excavation, whichever is greater.

Footings for other minor structures and slabs-on-grade that are structurally separated from the building should be supported on at least 2 feet of properly compacted engineered fill. Excavation for pavements and hardscape should be over-excavated at least 1 foot as measured from the bottom of the pavement or hardscape section. Due to potential settlement of underlying undocumented fill, the risk of future maintenance for these improvements should be considered. If such risks cannot be tolerated, undocumented fill should be removed to its full depth below the minor structures, slabs-on-grade, pavement, and hardscape.

Laterally, excavation for foundation of minor structures should extend beyond the limits of the foundation a minimum distance equal to two feet or the depth of over-excavation, whichever is greater. Excavation for other improvements (e.g., concrete walkways, flatwork, pavement, and slab-on-grade that are structurally separated from the building) should extend laterally at least two 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 as unsuitable for use as fill by a Twining representative, additional removals may be recommended. For example, deeper removal may be required in areas where soft, saturated, or organic materials are encountered.

The exposed excavation bottom should be evaluated and approved by Twining. The excavation bottom to receive fill should be scarified to a minimum depth of 6 inches and moisture conditioned to achieve generally consistent moisture contents approximately 2 percent above the optimum moisture content. The scarified bottom should be compacted to at least 90 percent relative compaction in accordance with the latest version of ASTM Test Method D1557 and then evaluated and approved by Twining.

Fill and backfill materials should be compacted fill in accordance with Sections 6.4.4 and 6.4.5 of this report. Prior to placement of any fill, the geotechnical engineer or their representative should review the bottom of the excavation for conformance with the recommendations of this report.

#### **6.4.4. Materials for Fill**

In general, on-site soils with low expansion potential are considered suitable for use as fill materials. All fill soils should be free of organics, debris, rocks or lumps over 3 inches in largest dimension, other deleterious material, and not more than 40 percent larger than  $\frac{3}{4}$  inch. Larger chunks, if generated during excavation, may be broken into acceptably-sized pieces or may be disposed offsite.

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

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

#### **6.4.5. Compacted Fill**

Unless otherwise recommended, the exposed excavation bottom to receive fill should be prepared in accordance with Section 6.4.3 of this report. Prior to placement of compacted fill, the contractor should request Twining to evaluate the exposed excavation bottoms.

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

#### **6.4.6. Fill Slopes**

Any areas where new fill slopes will be placed over existing fills should be evaluated by the geotechnical engineer. This may require test pits and/or other exploration methods. Depending on the assessment, it may be necessary to partially or completely excavate the existing fills and/or recompact them.

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units) and the depth of new fill measured from the top to the toe of the new fill exceeds 5 feet, the ground should be stepped or benched. The lowest bench or key should be a minimum of 10 feet wide and at least 2 feet deep into competent material as evaluated by Twining. Other benches should be excavated a minimum height of 4 feet into competent material. Fill placed on ground sloping flatter than 5:1 should also be benched or otherwise over-excavated to provide a flat subgrade for the fill. Fill slopes no steeper than 2:1 are expected to have adequate factors of safety for stability. If fill slopes are steeper than 2:1, Twining should be contacted for evaluation.

Fill and backfill materials should be compacted fill in accordance with Sections 6.4.4 and 6.4.5 of this report. Prior to placement of any fill, the geotechnical engineer or their representative

should review the bottom of the excavation for conformance with the recommendations of this report.

#### **6.4.7. 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. Unstable bottom conditions may be mitigated by over-excavation of the bottom to suitable depths and replacement with a one-foot-thick crushed aggregate base underlain by geogrid (Tensor TX7 or equivalent). Any loose, soft, or deleterious material should be removed prior to placement of aggregate base. Recommendations for stabilizing excavation bottoms should be based on evaluation in the field by the geotechnical consultant at the time of construction.

#### **6.4.8. Backfill for Utility Trench**

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 12 inches of crushed aggregate base (CAB) or crushed miscellaneous base (CMB) as defined in the "Greenbook" Standard Specifications for Public Works Construction (SSPWC).

The trench should be bedded with clean sand extending to at least 6 inches below the bottom of the pipe and one foot over the top of pipe. Pipe bedding as specified in SSPWC can be used. Bedding material should consist of clean sand having a sand equivalent (SE) of 20 or greater, 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 should be provided to the 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 20. Gravel or crushed rock if used as bedding materials should be wrapped in nonwoven geotextile fabric. The pipe 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 and mechanically compacted to reduce the potential for unbalanced loads. No void or uncompacted areas should be left beneath the pipe haunches.

Above pipe bedding, trench backfill may be onsite soils with low expansion potential 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 approximately 2 percent above the optimum moisture content.

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

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

#### **6.4.9. Rippability**

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

### **6.5. Foundation Recommendations**

Based upon the excavation/over-excavation and backfill recommendations provided in Section 6.4, the proposed building may be supported by conventional shallow foundations, 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. We note that the allowable bearing capacity provided for shallow foundations is governed by the potential for settlement of the underlying site soil.

#### **6.5.1. Bearing Capacity and Settlement**

Continuous strip footings or isolated footings for the proposed building should be placed on the subgrade prepared in accordance the requirements for the building pad as described in Section 6.4. Geotechnical design parameters for these footings presented in Table 5 may be used. Twining should be contacted for footing dimensions, allowable bearing pressures, and settlements that are outside the indicated applicable ranges.

#### **6.5.2. Lateral Resistance**

Lateral loads may be resisted by footing base friction and by the passive resistance of the soils based on recommendations provided in Table 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 resistance should be neglected when calculating the passive resistance. The passive resistance value may be increased by one-third for transient loads from wind or earthquake.

**Table 4 - Geotechnical Design Parameters for Footing Foundations**

<b>Minimum Footing Dimensions</b>	<ul style="list-style-type: none"> <li>• <u>Minimum width</u>: 18 inches.</li> <li>• <u>Minimum embedment</u>: 18 inches measured from the lowest adjacent firm grade to the bottom of the footing.</li> <li>• <u>Minimum thickness</u>: 6 inches</li> </ul>
<b>Allowable Bearing Pressure</b>	<ul style="list-style-type: none"> <li>• Assuming less than 680 kips (dead + live loads) on shallow spread footings and less than 18 kips per lineal foot on continuous strip footings</li> <li>• An allowable bearing pressure of 2,000 pounds per square foot (psf) may be used. The allowable may be increased by 300 psf for each additional foot of embedment, up to a maximum of 3,500 psf.</li> <li>• The allowable bearing values correspond to a factor of safety of 3.</li> <li>• The allowable bearing values may be increased by one-third for transient loads from wind or earthquake.</li> </ul>
<b>Estimated Static Settlement</b>	<ul style="list-style-type: none"> <li>• Approximately one inch of total settlement with differential settlement estimated to be approximately ½ inch over 30 feet.</li> <li>• The majority of static settlement of the foundation system is expected to occur upon initial application of loading.</li> </ul>
<b>Allowable Coefficient of Friction</b>	<ul style="list-style-type: none"> <li>• 0.4</li> <li>• The allowable bottom friction values correspond to a factor of safety of 1.5.</li> </ul>
<b>Allowable Lateral Passive Resistance</b>	<ul style="list-style-type: none"> <li>• 360 psf per foot of depth (i.e., 360 pcf equivalent fluid pressure).</li> <li>• The allowable passive resistance corresponds to a factor of safety of 2.</li> <li>• The upper one foot of resistance should be neglected when calculating the passive resistance.</li> <li>• The allowable passive resistance value may be increased by one-third for transient loads such as wind or earthquake loads.</li> <li>• 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.</li> </ul>

## 6.6. Slab-On-Grade

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

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

where:  $k_1$  = modulus for a 1-foot by 1-foot plate = 150 pci;  
 $B$  = width of slab in feet; and  
 $L$  = length of slab in feet.

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

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

**Table 5 - Options for Subgrade Preparation below Concrete Floor Slabs**

Primary Objective	Recommendation
<b>Enhanced protection against vapor transmission</b>	<ul style="list-style-type: none"> <li>• 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)</li> <li>• 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<sup>1</sup> under the membrane</li> </ul>
<b>Above-standard protection against vapor transmission</b>	<p>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:</p> <ul style="list-style-type: none"> <li>• 2 inches of dry silty sand<sup>1</sup>; over</li> <li>• Waterproofing plastic membrane 10 mils in thickness; over</li> <li>• At least 4 inches of ¾-inch crushed rock<sup>2</sup> or clean gravel<sup>3</sup> to act as a capillary break</li> </ul>
<b>Standard protection against vapor transmission</b>	<ul style="list-style-type: none"> <li>• 2 inches of dry silty sand<sup>1</sup>; over</li> <li>• Waterproofing plastic membrane 10 mils in thickness</li> <li>• 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<sup>1</sup> under the membrane.</li> </ul>
<p>Notes:</p> <p><sup>1</sup> 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.</p> <p><sup>2</sup> 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).</p> <p><sup>3</sup> 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.</p>	

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.

## **6.7. Below-Grade Walls**

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

### **6.7.1.Lateral Earth Pressure**

The values presented below assume that the supported grade is level, and Twining should be contacted for sloping backfill conditions.

Walls that are free to move and rotate at the top (such as cantilevered walls) and have adequate drainage may be designed for the active earth pressure equivalent to a fluid weighting 40 pcf. Where adequate drainage is not provided behind walls, an undrained or submerged EFP of 82 pcf should be used in the design.

Walls that are restricted to move horizontally at the top (such as by a floor deck) and have adequate drainage may be designed for the “at-rest” earth pressure equivalent to a fluid weighting 60 pcf. Where adequate drainage is not provided behind walls, an undrained or submerged EFP of 92 pcf should be used in the design.

Vertical surcharge loads within a 1:1 plane projected up and outward from the bottom of the wall distributed over retained soils should be considered as additional uniform horizontal pressures acting on the wall. These additional pressures can be estimated as approximately 1/3 and 1/2 of the magnitude of the vertical surcharge pressures for the “active” and “at-rest” conditions, respectively.

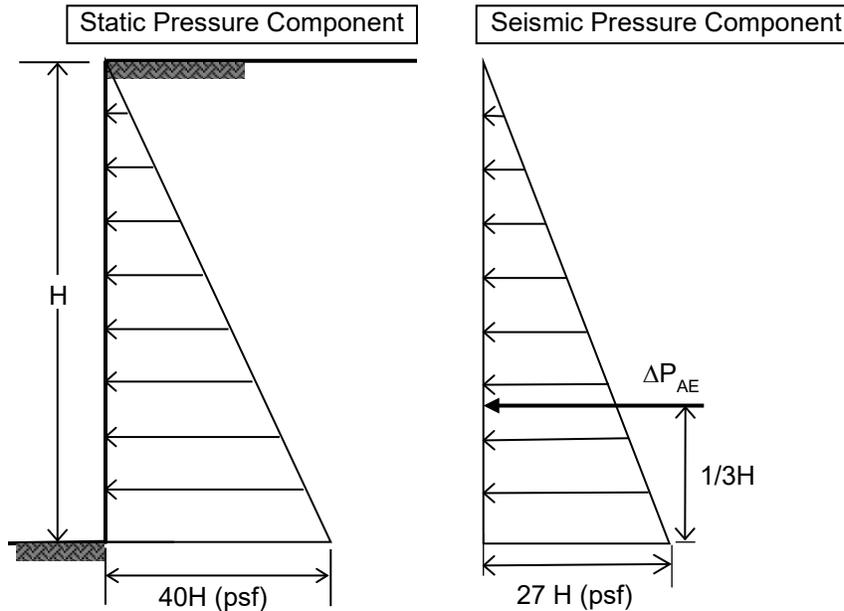
Surcharge loads may be simplified as a line load along the ground surface behind the walls. For walls under “at-rest” conditions, lateral pressure on the wall due to the line load surcharge can be approximated by a uniform pressure on the wall and the resultant of the pressure should be equal to 1/2 of the line load. For walls under “active” conditions (i.e., cantilevered walls), lateral pressures due to the line load surcharge can be approximated by a triangular pressure on the wall with the maximum value at the bottom of the wall and the resultant of the pressure equal to 1/3 of the line load. For a wall under “at-rest” conditions subject to surcharge from an adjacent footing with dimensions L and B in the directions parallel and perpendicular to the wall respectively, the corresponding lateral load on the wall may be approximated as a uniform pressure  $p = 0.5 \cdot q \cdot L / (L+x)$ , where q = footing pressure, and x = distance from the wall to the closest edge of the footing. The pressure p should be applied on the upper portion of the wall for a height equal to B over a wall length equal to (L+x).

Loads behind the wall at a distance greater than the height of the wall need not be considered. All permanent surcharge loading conditions should be evaluated on a case-by-case basis by the geotechnical engineer.

### **6.7.2.Seismic Lateral Earth Pressure**

Walls retaining more than 6 feet high earth should be designed for seismic lateral earth pressure. The seismic pressure distribution may be considered a triangle with the maximum pressure at the bottom. We estimated the seismic earth pressure increment for walls retaining level ground based on Seed and Whitman (1970) and a horizontal seismic coefficient ( $k_h$ ) equivalent to one-

half of two-thirds of  $PGA_M$  provide in Table 4. The following combination of static and incremental seismic pressures shown in the following diagram may be used for seismic design for both cantilever and restrained walls.



where H is in feet

**Diagram 1 - Seismic Earth Pressure Distribution on Walls**

### 6.7.3. Backfill and Drainage of Walls

The backfill material behind walls should consist of granular non-expansive material with an expansion index no greater than 40 and be approved by the project geotechnical engineer. Based on the soil materials encountered during our exploration, most on-site soils will meet this requirement.

Wall backfill should be adequately drained. Adequate backfill drainage is essential to provide a free-drained backfill condition and to limit hydrostatic buildup behind walls. Drainage behind walls may be provided by a geosynthetic drainage composite such as TerraDrain, MiraDrain, or equivalent, attached to the outside perimeter of the wall and installed in accordance with the manufacturer's recommendations. The drainage system should meet the minimum requirements of Sections 1805.4.2 and 1805.4.3 of 2022 CBC.

The walls should be appropriately waterproofed in accordance with the recommendations of the project architect.

## 6.8. Pavement Recommendations

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

We performed laboratory R-value testing for preliminary pavement section design. The tests indicate R values of 72 and 70. A value of 50 was used in our pavement structural calculations. Sections 6.8.1 and 6.8.2 present our recommendations for preliminary design of flexible and rigid pavement sections, respectively. Final pavement design should be based on field observations, additional R-value tests during construction should the materials exposed differ than what is expected based on our field exploration, and the anticipated traffic index as determined by the project civil engineer.

### 6.8.1. Flexible Pavement Design

Our flexible pavement structural design is in accordance with Chapter 630 of the Caltrans Highway Design Manual, which is based on a relationship between the gravel equivalent (GE) of the pavement structural materials, the traffic index (TI), and the R-value of the underlying subgrade soil. For preliminary design of flexible pavement section, Table 6 provides recommended minimum thicknesses for hot mix asphalt (HMA) and aggregate base sections for different traffic indices.

**Table 6 – Recommended Minimum HMA and Base Section Thicknesses**

Traffic Index	5.0	6.0	7.0
HMA Thickness (in)	4	4	5.5
Aggregate Base Thickness (in)	5	5	5

### 6.8.2. Rigid Pavement Design

For preliminary design of rigid pavement section, Table 7 provides recommended minimum thicknesses for Portland Cement Concrete (PCC) pavement section and Class 2 Aggregate Base (AB) section for different traffic indices. The recommended values are based on a minimum 28-day concrete compressive strength of 3,500 psi. Positive drainage should be provided away from all pavement areas to prevent seepage of surface and/or subsurface water into the pavement base and/or subgrade.

**Table 7 – Recommended Minimum Rigid Pavement Thicknesses**

Traffic Index	5.0	6.0	7.0
PCC Thickness (in)	4	4	5.5
Aggregate Base Thickness (in)	4	4	4

### **6.9. Stormwater Infiltration Facility**

Infiltration testing was performed at the site by Twining. The testing, location, and recommended design rates are detailed below in Appendix C and in the separate report that was prepared.

### **6.10. 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.

### 6.11. Shoring Recommendations

Temporary shoring may consist of a soldier beam and lagging appropriately designed by a qualified shoring engineer. The shoring should be designed to resist lateral earth pressure from retained soils and any surcharge loads from traffic, adjacent buildings, or construction equipment and materials. Lateral earth pressures are provided in Table 8 for design of temporary shoring assuming a maximum excavation depth of 20 feet, drained level backfill conditions and no surcharge loading.

**Table 8 - Recommended Lateral Earth Pressure and Resistance for Temporary Shoring**

Shoring Conditions	Lateral Earth Pressure <sup>(1) (2)(3)</sup>
Cantilevered Shoring	40 pcf
Braced Shoring	25H psf
Ultimate Passive Resistance	800 psf uniform passive pressure

Notes: H = depth of excavation in feet.

The lateral pressure due to a uniform surcharge load located immediately behind the temporary shoring may be calculated by multiplying the vertical surcharge pressure by 1/3 for cantilevered shoring and 1/2 for braced-shoring, corresponding to the “active” and “at-rest” conditions, respectively. Lagging may be designed for a maximum lateral pressure of 400 psf in the middle between adjacent soldier piles and zero psf at the soldier piles.

Support of adjacent structures and utilities without distress is the contractor’s responsibility.

## 7. DESIGN REVIEW AND CONSTRUCTION MONITORING

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

### 7.1. Plans and Specifications

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

### 7.2. Construction Monitoring

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



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## 8. LIMITATIONS

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

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

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

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

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

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

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

## 9. SELECTED REFERENCES

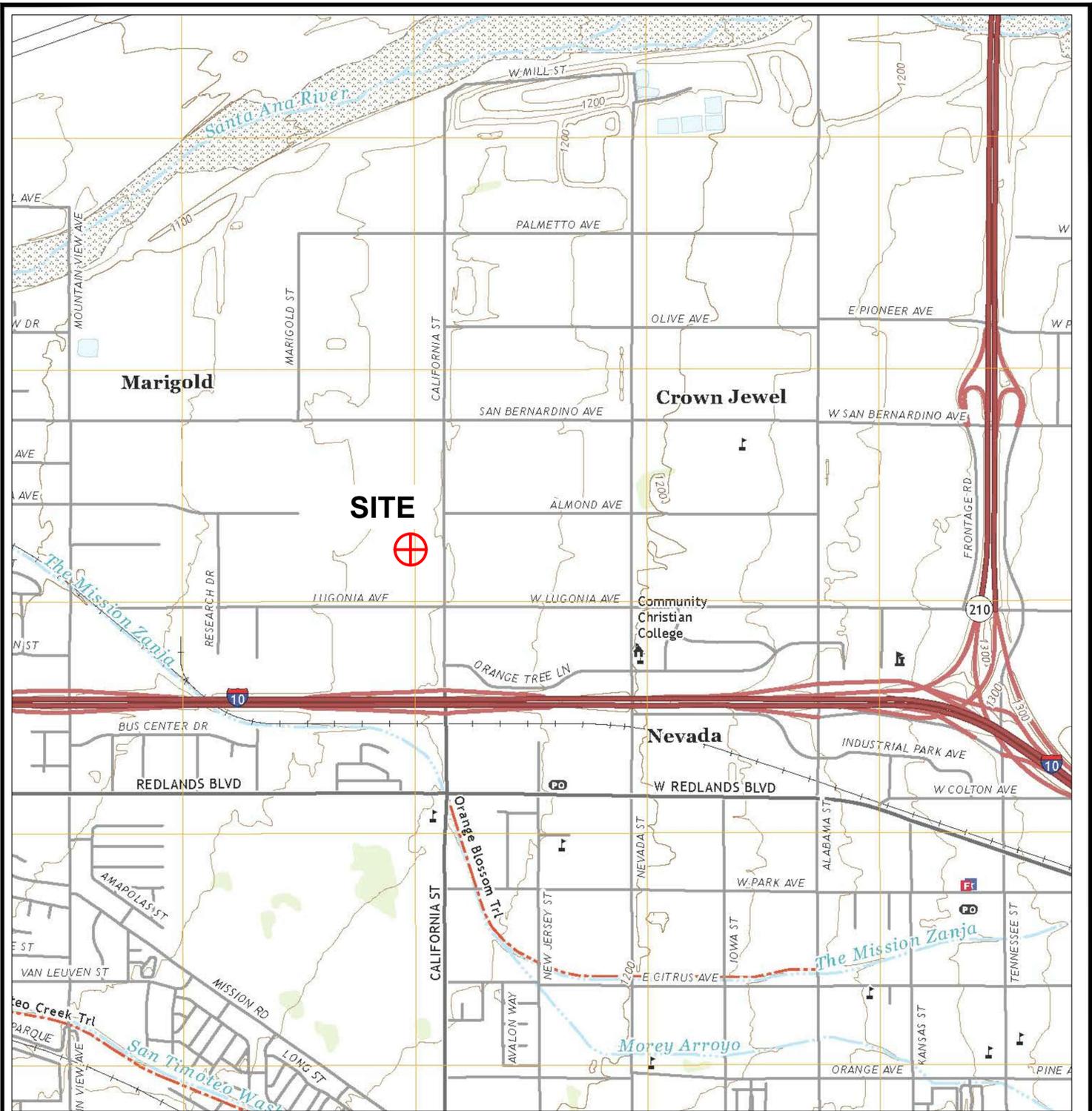
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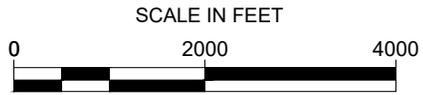
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## FIGURES



 APPROXIMATE LOCATION OF PROJECT



REFERENCE: USGS (2021)



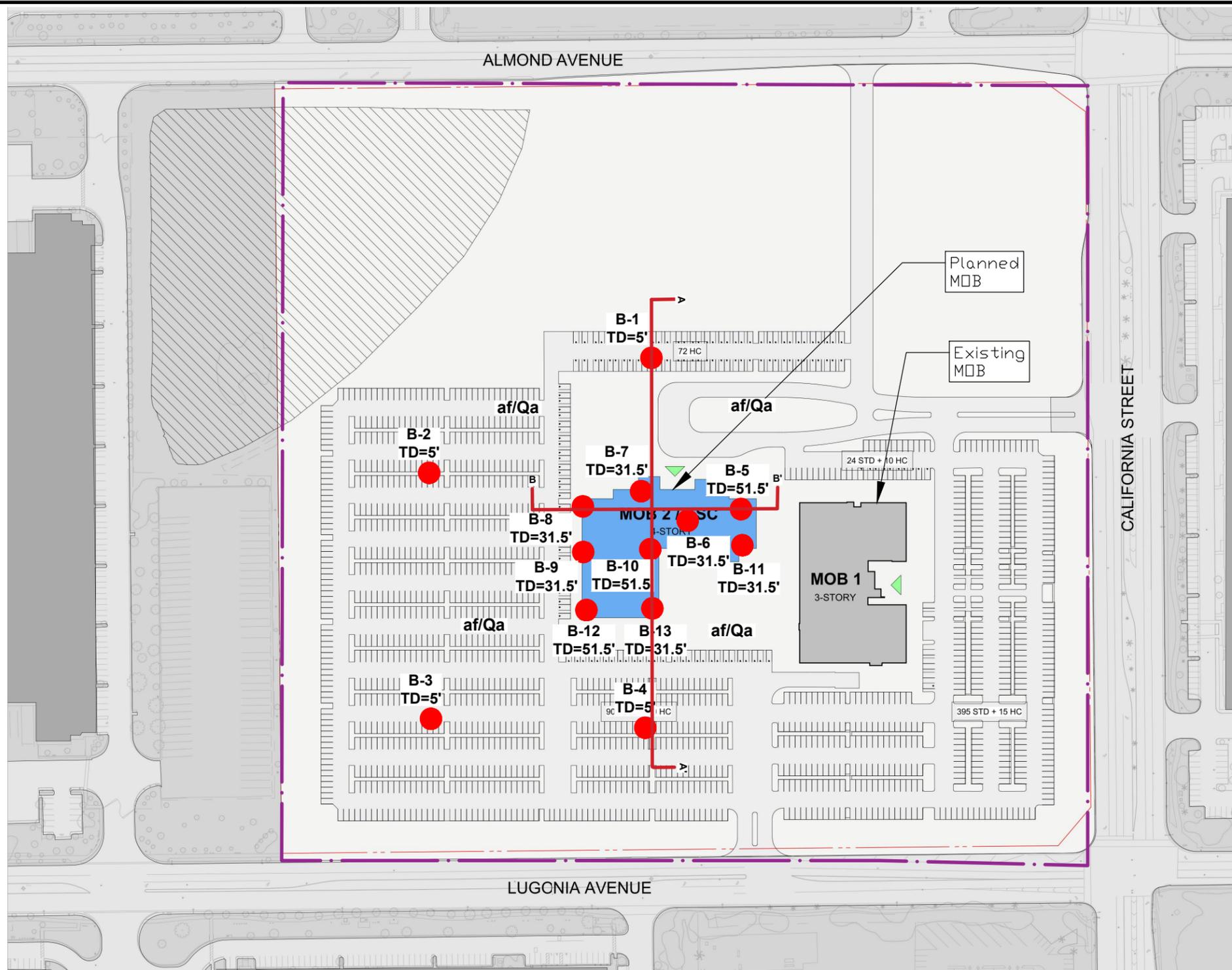
**SITE LOCATION MAP**

KAISER REDLANDS MOB 2  
1301 CALIFORNIA STREET  
REDLANDS, CALIFORNIA

PROJECT NO.  
220729.3

REPORT DATE  
March 2023

FIGURE 1



SCALE IN FEET



NOTE: ALL DIMENSIONS AND LOCATIONS ARE APPROXIMATE

**LEGEND**

- B-1**  
TD=5'  
 APPROXIMATE LOCATION OF BORING BY TWINING TOTAL DEPTH IN FEET
- GEOLOGIC CROSS SECTION
- af/Qa** ARTIFICIAL FILL OVER ALLUVIUM



REFERENCE: CO ARCHITECTS 2023

**SITE PLAN, GEOLOGIC MAP, AND BORING LOCATION MAP**

KAISER REDLANDS MOB 2  
1301 CALIFORNIA STREET  
REDLANDS, CA

PROJECT No.  
220729.3

REPORT DATE  
March 2023

FIGURE 2

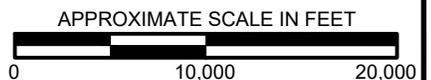




Qa - Alluvial sand and clay of valley areas  
 Qg - Alluvial gravel and sand of stream channels



REFERENCE: DIBBLEE (2004)



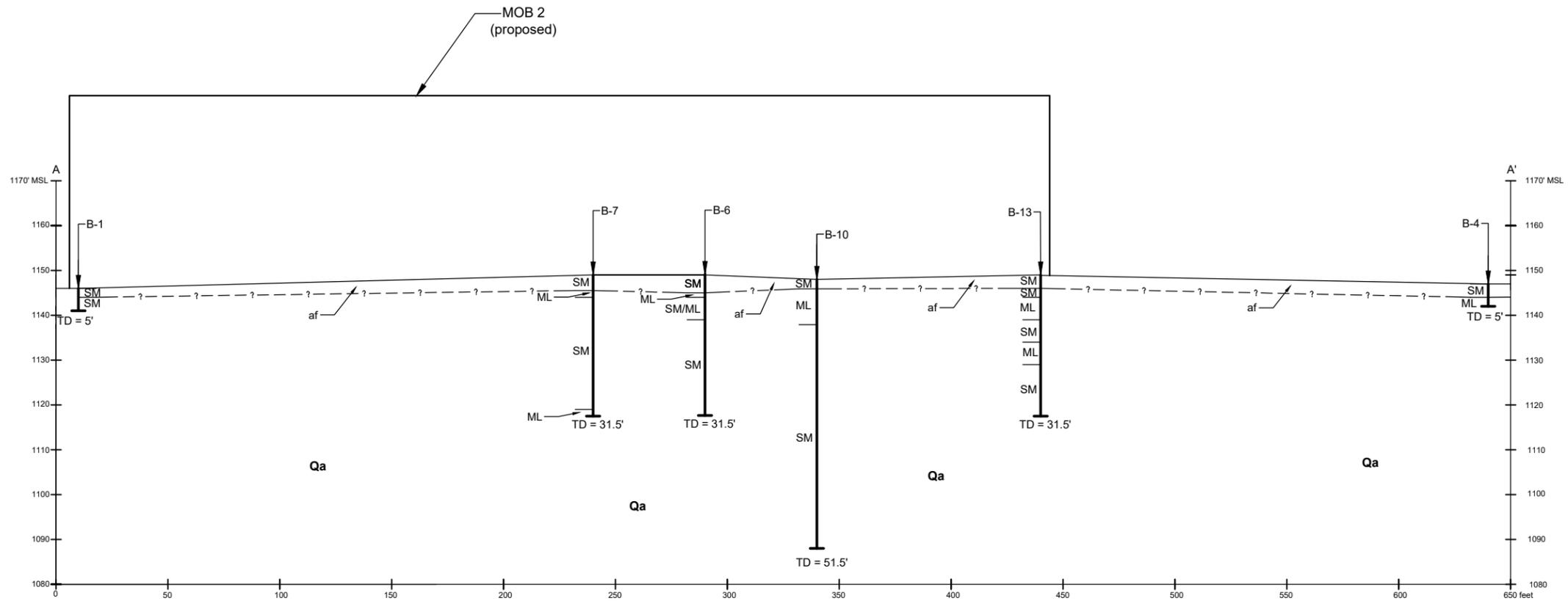
**GEOLOGIC MAP**

KAISER REDLANDS MOB 2  
 1301 CALIFORNIA STREET  
 REDLANDS, CALIFORNIA

PROJECT NO.  
 220729.3

REPORT DATE  
 March 2023

FIGURE 3



**LEGEND**

— Geologic Contact,  
Queried Where Uncertain

**Af** Artificial Fill

**Qa** Alluvial Sand and Clay of Valley Areas

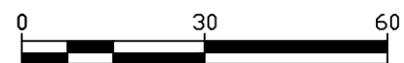
**B-1**  
┆ Location of Boring  
TD = Total Depth

SCALE IN FEET  
VERTICAL AND HORIZONTAL SCALE DIFFERENT

Approximate Horizontal Scale in Feet



Approximate Vertical Scale in Feet



**GEOLOGIC CROSS SECTION A-A'**

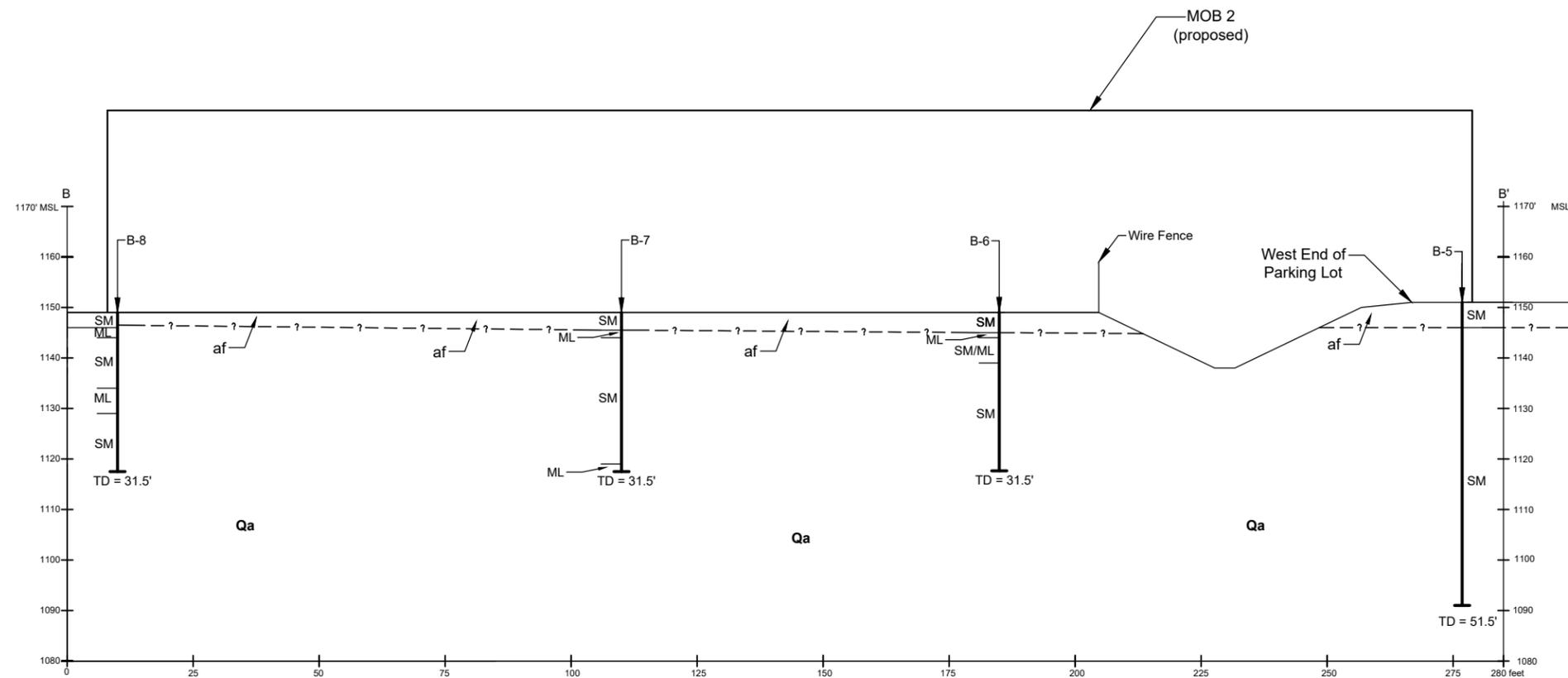
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REDLANDS, CA

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220729.3

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FIGURE 4A

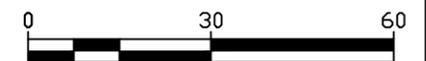




**LEGEND**

- Geologic Contact, Queried Where Uncertain
- Af** Artificial Fill
- Qa** Alluvial Sand and Clay of Valley Areas
- B-5  
┆ Location of Boring  
TD = Total Depth

Approximate Scale in Feet



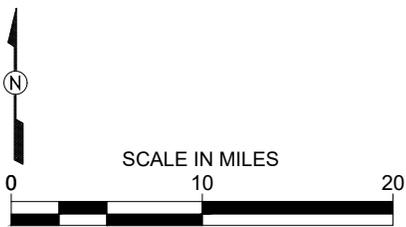
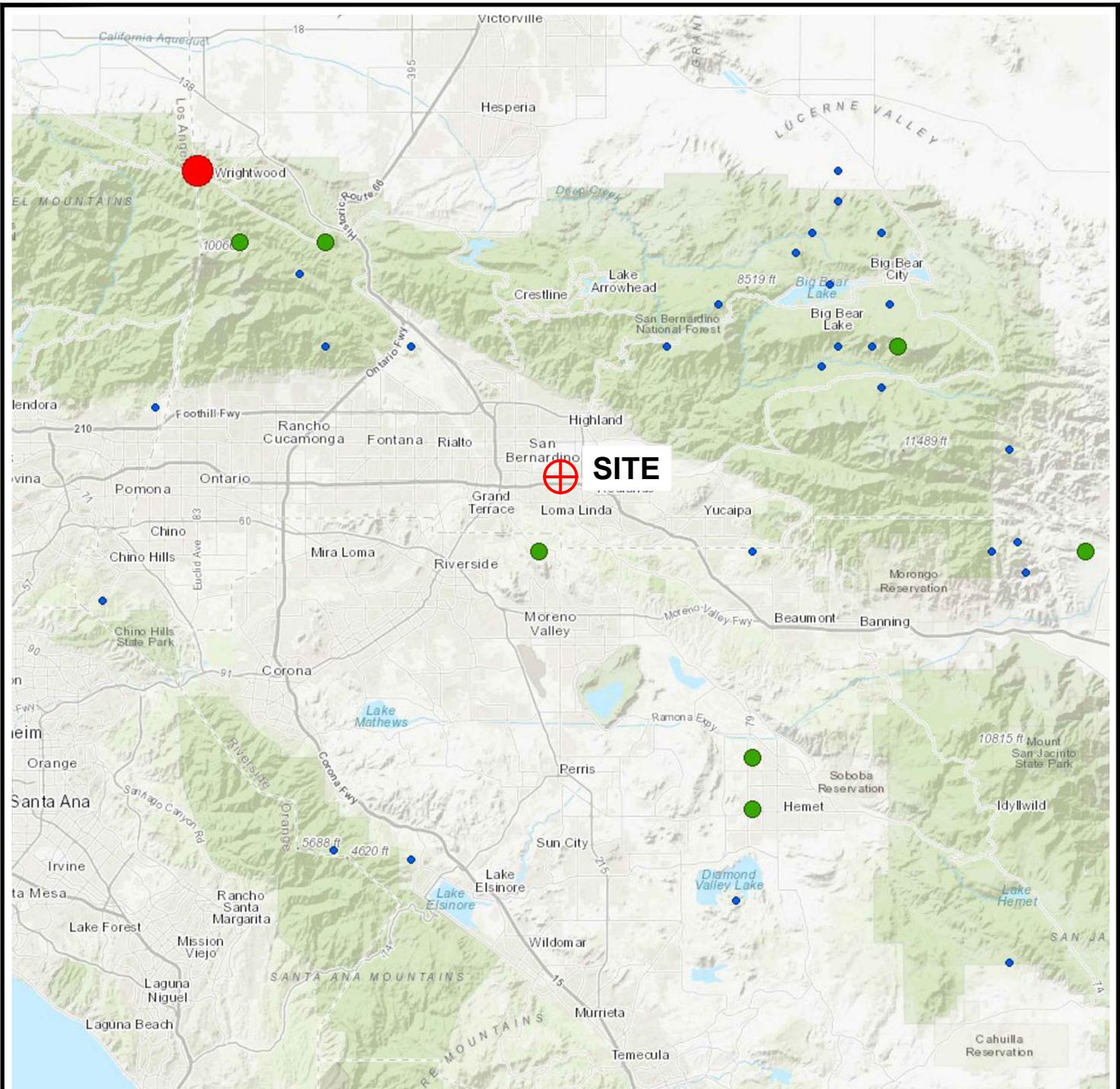
**GEOLOGIC CROSS SECTION B-B'**

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REDLANDS, CA

PROJECT No.  
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FIGURE 4B



**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



**TWINING**

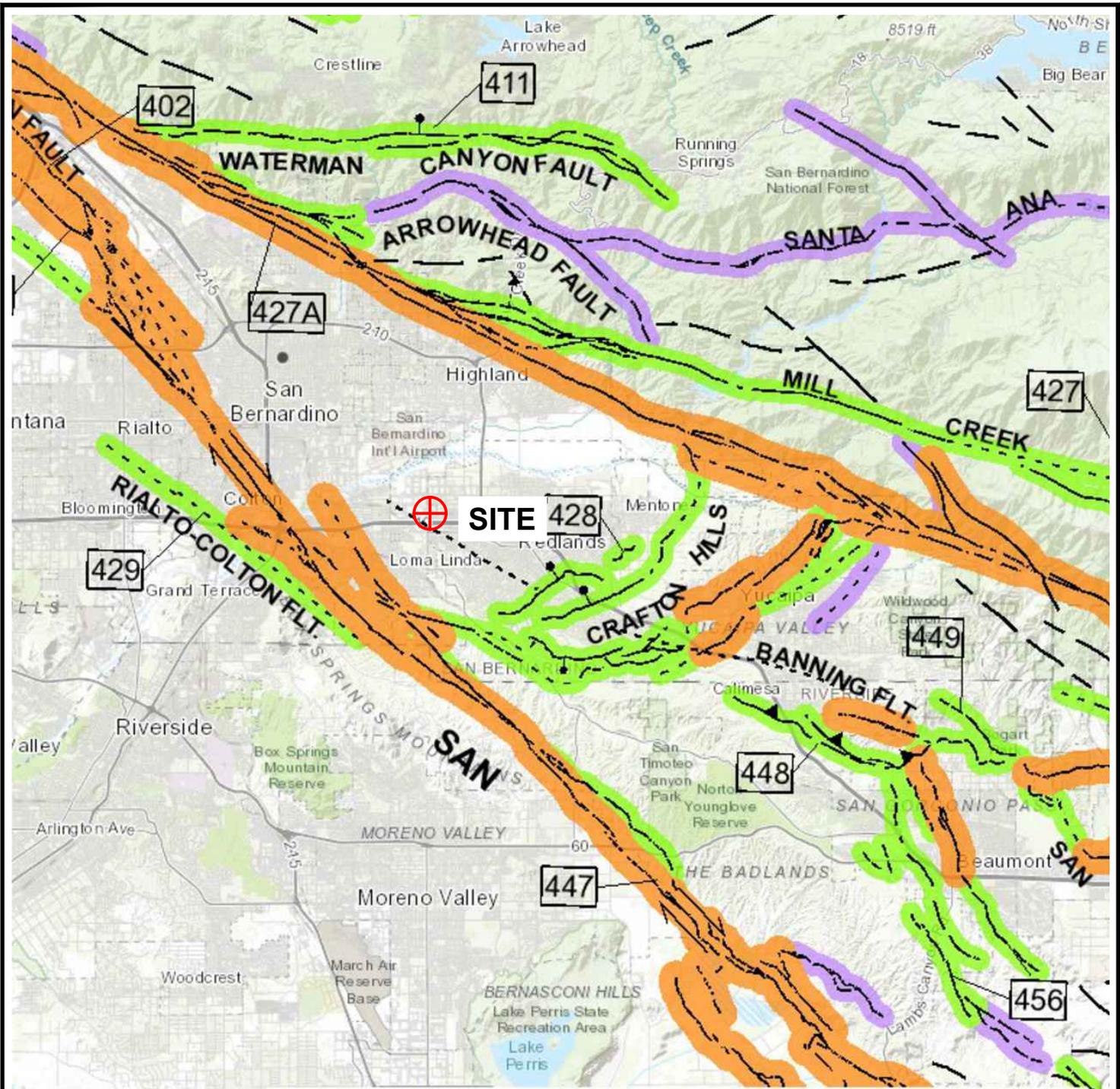
**HISTORICAL EARTHQUAKE EPICENTER MAP**

KAISER REDLANDS MOB 2  
1301 CALIFORNIA STREET  
REDLANDS, CALIFORNIA

PROJECT NO.  
220729.3

REPORT DATE  
March 2023

FIGURE 5



-  FAULT ALONG WHICH HISTORIC DISPLACEMENT HAS OCCURRED
-  HOLOCENE FAULT DISPLACEMENT
-  LATE QUATERNARY FAULT DISPLACEMENT
-  QUATERNARY FAULT DISPLACEMENT
-  PRE-QUATERNARY FAULT DISPLACEMENT



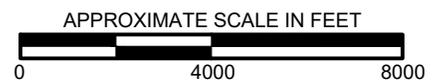
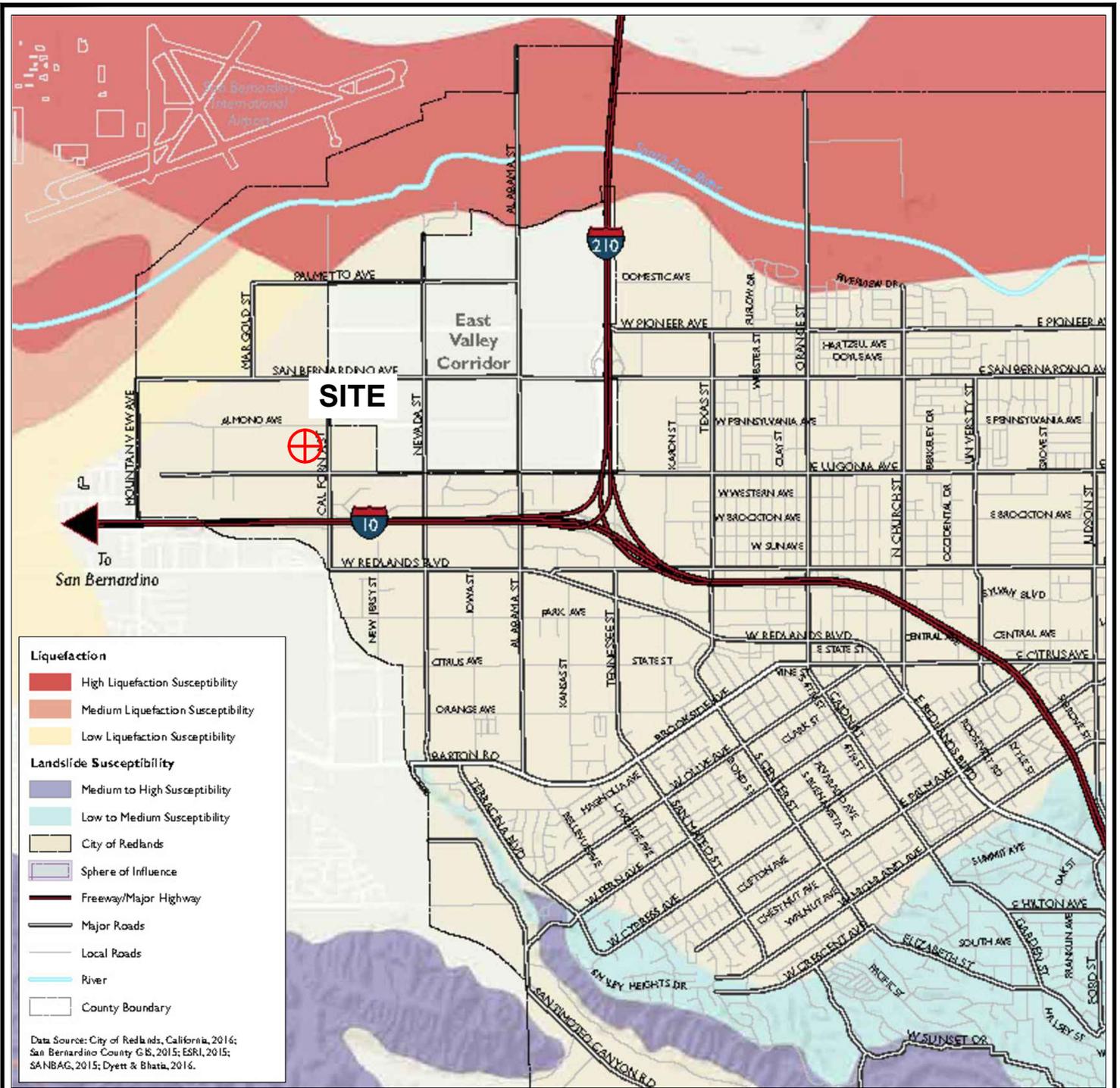
REFERENCE: CGS (2021)



**REGIONAL FAULT LOCATION MAP**

KAISER REDLANDS MOB 2  
1301 CALIFORNIA STREET  
REDLANDS, CALIFORNIA

PROJECT NO. 220729.3	REPORT DATE March 2023	FIGURE 6
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REFERENCE: CITY OF REDLANDS (2017)



**LIQUEFACTION AND LANDSLIDE HAZARDS ZONES MAP**

KAISER REDLANDS MOB 2  
1301 CALIFORNIA STREET  
REDLANDS, CALIFORNIA

PROJECT NO.  
220729.3

REPORT DATE  
March 2023

FIGURE 7



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# APPENDIX A FIELD EXPLORATION



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## Appendix A Field Exploration

### General

The field exploration was conducted on February 2 and 3, 2023 and consisted of drilling, testing, sampling, and logging of 13 exploratory hollow-stem-auger (HSA) borings (B-1 through B-13). Borings B-5 through B-13 were drilled using a CME-75 truck-mounted drill rig equipped with 8-inch-diameter HSA. Borings B-5 through B-13 were advanced to approximately 31.5 to 51.5 feet bgs and were within the footprint of the proposed building. Borings B-1 through B-4 were advanced to approximate 5 feet bgs and were within the proposed parking lot areas. All borings were first advanced to 5 feet bgs using a 5-inch-diameter hand auger to minimize the chance of disturbing any existing utilities.

Percolation testing was also performed in accordance with the 2013 Technical Guidance Document for Water Quality Management Plans of the San Bernardino County. The testing was performed to provide an estimate of infiltration rate of the site soils for use in preliminary design of a storm water management system. The locations of the percolation tests and their results are attached in Appendix C – Percolation Testing.

Before starting our exploration program, we performed a site reconnaissance to observe the general surficial conditions at the site, to select field exploration locations, and to plan field logistics including health and safety. After exploration locations were delineated, Underground Service Alert was notified of the planned locations a minimum of 72 hours prior to excavation. Prior to the exploration, Geovision was used to perform utility scanning using Ground-Penetrating Radar at the proposed exploration locations in order to minimize the chance of disturbing any existing utilities. The results of the scanning are attached below in Appendix D.

The approximate locations of the borings are shown on Figure 2 – Site Plan and Boring Location Map.

### Drilling and Sampling

An explanation of the boring logs is presented as Figure A-1. The boring logs are presented as Figures A-2 through A-10. 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 an 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 with room for liner but liner was not used. 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



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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.

During drilling, groundwater was not encountered in any of the borings.

Upon completion of the borings or percolation testing, the boreholes were backfilled with neat cement grout and the surface was repaired to match existing conditions.

DATE DRILLED 2/2/2023 LOGGED BY AB **BORING NO.** B-1  
 DRIVE WEIGHT N/A DROP N/A DEPTH TO GROUNDWATER (ft.) N/E  
 DRILLING METHOD 5" Hand Auger DRILLER Baja Exploration SURFACE ELEVATION (ft.) 1146 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES		BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
		Bulk Driven								
1141	5				6.3		R-Value		SM	ARTIFICIAL FILL: Silty SAND; dark grayish brown; slightly moist to moist
1136	10								SM	ALLUVIUM: Silty SAND; dark grayish brown; slightly moist to moist
1131	15									Total Depth = 5.0 feet Backfilled on 2/2/2023 Backfilled with cuttings. Groundwater not encountered.
1126	20									
1121	25									
1116	30									
1111	35									



**TWINING**

**LOG OF BORING**

Kaiser Redlands MOB 2  
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PROJECT NO. 220729.3 REPORT DATE March 2023 FIGURE A - 2

DATE DRILLED 2/2/2023 LOGGED BY AB **BORING NO.** B-2  
 DRIVE WEIGHT N/A DROP N/A DEPTH TO GROUNDWATER (ft.) N/E  
 DRILLING METHOD 5" Hand Auger DRILLER Baja Exploration SURFACE ELEVATION (ft.) 1146 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES		BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
		Bulk Driven								
1141	5				7.7		EI		SM	ARTIFICIAL FILL: Silty SAND; reddish brown; slightly moist to moist; fine sand
1136	10								SM	ALLUVIUM: Silty SAND; reddish brown; moist; fine sand
1141	5									Total Depth = 5.0 feet Backfilled on 2/2/2023 Backfilled with cuttings. Groundwater not encountered.
1126	20									
1121	25									
1116	30									
1111	35									



**TWINING**

**LOG OF BORING**

Kaiser Redlands MOB 2  
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PROJECT NO.  
220729.3

REPORT DATE  
March 2023

FIGURE A - 3

DATE DRILLED 2/2/2023 LOGGED BY AB **BORING NO.** B-3  
 DRIVE WEIGHT N/A DROP N/A DEPTH TO GROUNDWATER (ft.) N/E  
 DRILLING METHOD 5" Hand Auger DRILLER Baja Exploration SURFACE ELEVATION (ft.) 1147 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES		BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
		Bulk Driven								
1142	5				8.3		R-Value	SM	ML	ARTIFICIAL FILL: Silty SAND; reddish brown; slightly moist to moist; fine sand
					5.4			ML		ALLUVIUM: Sandy SILT; light brown; slightly moist  Total Depth = 5.0 feet Backfilled on 2/2/2023 Backfilled with cuttings. Groundwater not encountered.
1137	10									
1132	15									
1127	20									
1122	25									
1117	30									
1112	35									



**TWINING**

**LOG OF BORING**

Kaiser Redlands MOB 2  
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 Redlands, California

PROJECT NO. 220729.3 REPORT DATE March 2023 FIGURE A - 4

DATE DRILLED 2/2/2023 LOGGED BY AB **BORING NO.** B-4  
 DRIVE WEIGHT N/A DROP N/A DEPTH TO GROUNDWATER (ft.) N/E  
 DRILLING METHOD 5" Hand Auger DRILLER Baja Exploration SURFACE ELEVATION (ft.) 1147 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES		BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
		Bulk Driven								
1142	5				7.0		EI		SM	ARTIFICIAL FILL: Silty SAND; reddish brown; moist; fine sand
1137	10								ML	ALLUVIUM: Sandy SILT; light grayish brown; slightly moist; fine sand Total Depth = 5.0 feet Backfilled on 2/2/2023 Backfilled with cuttings. Groundwater not encountered.
1132	15									
1127	20									
1122	25									
1117	30									
1112	35									



**TWINING**

**LOG OF BORING**

Kaiser Redlands MOB 2  
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PROJECT NO.  
 220729.3

REPORT DATE  
 March 2023

FIGURE A - 5

DATE DRILLED 2/2/2023 LOGGED BY CDD **BORING NO.** B-5  
 DRIVE WEIGHT 140 lbs. DROP 30 inches DEPTH TO GROUNDWATER (ft.) N/E  
 DRILLING METHOD 8" HSA DRILLER Baja Exploration SURFACE ELEVATION (ft.) 1155 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES		BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
		Bulk	Driven							
1150	5			64	10.8 7.4	111.5 117.9	DS C		SM	4 inches of asphalt concrete over 4 inches of base ARTIFICIAL FILL: Silty SAND; dark brown; slightly moist; fine sand with some fine gravel -- some debris: pieces of concrete ALLUVIUM: Silty SAND; dense; dark brown; moist; fine sand
1145	10			9			#200, ATT		SM	-- same; loose; light olive brown; slightly moist; fine sand
1140	15			24	8.2	91.2	DS		SM	-- same; medium dense; moist; fine to medium sand
1135	20			13			#200		SM	-- same; medium dense; dry; fine sand
1130	25			77	2.5	112.3			SM	-- same; dense; dry
1125	30			29			#200, ATT		SM	-- same; medium dense; slightly moist
1120	35									



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 1301 California Street  
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PROJECT NO.  
 220729.3

REPORT DATE  
 March 2023

FIGURE A - 6

DATE DRILLED 2/2/2023 LOGGED BY CDD **BORING NO.** B-5  
 DRIVE WEIGHT 140 lbs. DROP 30 inches DEPTH TO GROUNDWATER (ft.) N/E  
 DRILLING METHOD 8" HSA DRILLER Baja Exploration SURFACE ELEVATION (ft.) 1155 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES		BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
		Bulk	Driven							
1115	40			57	12.7	97.6	#200		SM	-- same; very dense; slightly moist; fine sand
1110	45			50 for 4"	11.9	99.3			SM	-- same; very dense; dark grayish brown; moist
1105	50			74					SM	-- same; very dense; light olive brown; slightly moist; fine sand; laminations of clay
1100	55									Total Depth = 51.5 feet Backfilled on 2/2/2023 Backfilled with neat cement. Groundwater not encountered. Surface patched with dyed black PCC.
1095	60									
1090	65									
1085	70									



**TWINING**

**LOG OF BORING**

Kaiser Redlands MOB 2  
 1301 California Street  
 Redlands, California

PROJECT NO. 220729.3 REPORT DATE March 2023 FIGURE A - 6

DATE DRILLED 2/3/2023 LOGGED BY CDD **BORING NO.** B-6  
 DRIVE WEIGHT 140 lbs. DROP 30 inches DEPTH TO GROUNDWATER (ft.) N/E  
 DRILLING METHOD 8" HSA DRILLER Baja Exploration SURFACE ELEVATION (ft.) 1149.4(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES		BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
		Bulk	Driven							
1144	5			23	6.9	101.7	DS		SM	ARTIFICIAL FILL: Silty SAND; reddish brown; slightly moist; fine sand
							#200, ATT		ML	ALLUVIUM: Sandy SILT; light grayish brown; slightly moist; fine sand
									SM/ML	Silty SAND/Sandy SILT; medium dense/stiff; light olive brown; dry; fine sand
1139	10			9					SM	Silty SAND; loose; light olive brown; slightly moist; fine sand
1134	15			26	5.3	97.2			SM	-- same; medium dense; trace iron oxide staining; some silt laminations; fine sand
1129	20			12					SM	-- same; medium dense; slightly moist
1124	25			49	8.3	96.9			SM	-- same; dense; moist; fine to medium sand; trace fine gravel
1119	30			29					SM	-- same; medium dense; slightly moist; fine to medium sand; some silty laminations
1114	35									Total Depth = 31.5 feet Backfilled on 2/3/2023 Backfilled with neat cement. Groundwater not encountered.



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**LOG OF BORING**

Kaiser Redlands MOB 2  
 1301 California Street  
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PROJECT NO.  
220729.3

REPORT DATE  
March 2023

FIGURE A - 7

DATE DRILLED 2/3/2023 LOGGED BY CDD **BORING NO.** B-7  
 DRIVE WEIGHT 140 lbs. DROP 30 inches DEPTH TO GROUNDWATER (ft.) N/E  
 DRILLING METHOD 8" HSA DRILLER Baja Exploration SURFACE ELEVATION (ft.) 1149.4(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES		BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
		Bulk	Driven							
1144	5			9					SM	ALLUVIUM: Silty SAND; reddish brown; slightly moist to moist; fine sand; trace gravel
					6.0				ML	Sandy SILT; light grayish brown; slightly moist; fine sand
1139	10			16	10.7	97.3			SM	Silty SAND; loose; light olive brown; dry to slightly moist; fine sand
										-- same; medium dense; moist; trace iron oxide staining; some laminations of silt
1134	15			11					SM	-- same; medium dense; some silt laminations
										-- same; medium dense; dark grayish brown; wet; some silt laminations
1129	20			26	23.7	88.7			SM	-- same; medium dense; moist
										-- same; medium dense; moist
1124	25			24					SM	
1119	30			22	31.5	88.3	ATT		ML	Sandy SILT; stiff; light olive brown; wet
1114	35									Total Depth = 31.5 feet Backfilled on 2/3/2023 Backfilled with neat cement. Groundwater not encountered.



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Kaiser Redlands MOB 2  
 1301 California Street  
 Redlands, California

PROJECT NO.  
220729.3

REPORT DATE  
March 2023

FIGURE A - 8

DATE DRILLED 2/2/2023 LOGGED BY CDD **BORING NO.** B-8  
 DRIVE WEIGHT 140 lbs. DROP 30 inches DEPTH TO GROUNDWATER (ft.) N/E  
 DRILLING METHOD 8" HSA DRILLER Baja Exploration SURFACE ELEVATION (ft.) 1149.4(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES		BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
		Bulk	Driven							
1144	5			8	1.4	100.1			SM	ARTIFICIAL FILL: Silty SAND; reddish brown; moist; fine sand
									ML	ALLUVIUM: Sandy SILT; light brown; moist; very fine sand
1139	10			24			#200, ATT		SM	-- same; medium dense; dry; some mica
1134	15			29	6.7	104.4	DS		ML	Sandy SILT; very stiff; light olive brown; slightly moist; some iron oxide staining
1129	20			17					SM	Silty SAND; medium dense; light yellowish brown; dry
1124	25			42	2.5	105.1			SM	-- same; medium dense; slightly moist; some iron oxide staining
1119	30			22					SM	-- same; medium dense; light olive brown; slightly moist
1114	35									Total Depth = 31.5 feet Backfilled on 2/2/2023 Backfilled with neat cement. Groundwater not encountered.



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Kaiser Redlands MOB 2  
 1301 California Street  
 Redlands, California

PROJECT NO.  
220729.3

REPORT DATE  
March 2023

FIGURE A - 9

DATE DRILLED 2/2/2023 LOGGED BY CDD **BORING NO.** B-9  
 DRIVE WEIGHT 140 lbs. DROP 30 inches DEPTH TO GROUNDWATER (ft.) N/E  
 DRILLING METHOD 8" HSA DRILLER Baja Exploration SURFACE ELEVATION (ft.) 1149.4(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES		BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
		Bulk Driven								
1144	5			11			#200		SM	ARTIFICIAL FILL: Silty SAND; reddish brown; moist; fine sand; fine to medium gravel -- finer sand
					7.8				ML	ALLUVIUM: Sandy SILT; light grayish brown; fine sand
1139	10			40	2.2	101.9	DS		SM	Silty SAND; medium dense; light yellowish brown; dry; disturbed sample
1134	15			13					SM	-- same; medium dense; dry to slightly moist
1129	20			34	2.5	105.4			SM	-- same; medium dense; dry; fine to medium sand
1124	25			14					SM	-- same; medium dense; slightly moist
1119	30			36	4.6	103.7			SM	--same; medium dense; slightly moist
1114	35									Total Depth = 31.5 feet Backfilled on 2/2/2023 Backfilled with neat cement. Groundwater not encountered.



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**LOG OF BORING**

Kaiser Redlands MOB 2  
 1301 California Street  
 Redlands, California

PROJECT NO.  
220729.3

REPORT DATE  
March 2023

FIGURE A - 10

DATE DRILLED 2/3/2022 LOGGED BY CDD **BORING NO.** B-10  
 DRIVE WEIGHT 140 lbs. DROP 30 inches DEPTH TO GROUNDWATER (ft.) N/E  
 DRILLING METHOD 8" HSA DRILLER Baja Exploration SURFACE ELEVATION (ft.) 1148 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES		BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
		Bulk Driven								
1143	5			17	1.8	100.3	#200, ATT		ML	ARTIFICIAL FILL: Silty SAND; reddish brown; slightly moist to moist; fine sand  ALLUVIUM: Sandy SILT; light grayish brown; slightly moist; fine sand  -- same; stiff; light olive brown; dry
1138	10			10					SM	Silty SAND; medium dense; dark grayish brown; slightly moist
1133	15			19	7.9 9.9	101.4 90.0	C DS		SM	-- same; medium dense; increased silt; some clay; some iron oxide staining
1128	20			11			#200, ATT		SM	-- same; medium dense; light olive brown; slightly moist
1123	25			29	5.3	96.8			SM	-- same; medium dense; light olive brown; slightly moist
1118	30			30					SM	-- same; medium dense; moist; laminations of clay and silt
1113	35									



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**LOG OF BORING**

Kaiser Redlands MOB 2  
 1301 California Street  
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PROJECT NO.  
 220729.3

REPORT DATE  
 March 2023

FIGURE A - 11

DATE DRILLED 2/3/2022 LOGGED BY CDD **BORING NO.** B-10  
 DRIVE WEIGHT 140 lbs. DROP 30 inches DEPTH TO GROUNDWATER (ft.) N/E  
 DRILLING METHOD 8" HSA DRILLER Baja Exploration SURFACE ELEVATION (ft.) 1148 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES		BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
		Bulk Driven								
1108	40			26					SM	-- same; medium dense; laminations of silt; fine sand
1103	45			77	11.3	104.2			SM	-- same; very dense; some iron oxidation staining; fine to medium sand; moist
1098	50			53					SM	-- same; very dense
1093	55									Total Depth = 51.5 feet Backfilled on 2/3/2023 Backfilled with neat cement. Groundwater not encountered.
1088	60									
1083	65									
1078	70									



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**LOG OF BORING**

Kaiser Redlands MOB 2  
 1301 California Street  
 Redlands, California

PROJECT NO. 220729.3 REPORT DATE March 2023 FIGURE A - 11

DATE DRILLED 2/2/2023 LOGGED BY CDD **BORING NO.** B-11  
 DRIVE WEIGHT 140 lbs. DROP 30 inches DEPTH TO GROUNDWATER (ft.) N/E  
 DRILLING METHOD 8" HSA DRILLER Baja Exploration SURFACE ELEVATION (ft.) 1155.4(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES		BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
		Bulk	Driven							
1150	5			31	11.1		EI, CORR		SM	3 inches of asphalt concrete over 5 inches of base ARTIFICIAL FILL: Silty SAND; dark brown; moist; some fine gravel
1145	10			31	4.9	102.9	#200, C		SM	ALLUVIUM: Silty SAND; dense; dark brown; moist; fine sand; some mica -- same; medium dense; light olive brown; slightly moist; fine to medium sand
1140	15			17					SM	-- same; medium dense
1135	20			33	12.0	104.8			SM	-- same; medium dense; fine sand; moist
1130	25			12			#200		ML	Sandy SILT; stiff; fine sand; moist
1125	30			63	8.9	111.5			ML	-- same; hard; moist
1120	35									Total Depth = 31.5 feet Backfilled on 2/2/2023 Backfilled with neat cement. Groundwater not encountered. Surface patched with dyed black PCC.



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**LOG OF BORING**

Kaiser Redlands MOB 2  
 1301 California Street  
 Redlands, California

PROJECT NO.  
220729.3

REPORT DATE  
March 2023

FIGURE A - 12

DATE DRILLED 2/2/2023 LOGGED BY CDD **BORING NO.** B-12  
 DRIVE WEIGHT 140 lbs. DROP 30 inches DEPTH TO GROUNDWATER (ft.) N/E  
 DRILLING METHOD 8" HSA DRILLER Baja Exploration SURFACE ELEVATION (ft.) 1148 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES		BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
		Bulk Driven								
1143	5			29	2.6 2.4	85.7 99.4	DS C		SM	ALLUVIUM: Silty SAND; light brown; slightly moist; fine sand -- same; medium dense; light gray; dry
1138	10			9			#200		SM	-- same; loose; light yellowish brown; slightly moist
1133	15			22	4.4 3.7	92.9 92.7			ML	SILT with sand; stiff; light gray; dry; some mica
1128	20			11			#200			-- same; stiff; dry
1123	25			51	11.0 5.8	87.8 102.4			SM	Silty SAND; dense; light yellowish brown; dry to slightly moist
1118	30			32					SM	-- same; dense; slightly moist
1113	35									



**TWINING**

**LOG OF BORING**

Kaiser Redlands MOB 2  
 1301 California Street  
 Redlands, California

PROJECT NO.  
220729.3

REPORT DATE  
March 2023

FIGURE A - 13

DATE DRILLED 2/2/2023 LOGGED BY CDD **BORING NO.** B-12  
 DRIVE WEIGHT 140 lbs. DROP 30 inches DEPTH TO GROUNDWATER (ft.) N/E  
 DRILLING METHOD 8" HSA DRILLER Baja Exploration SURFACE ELEVATION (ft.) 1148 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES		BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
		Bulk Driven								
1078	70									
1083	65									
1088	60									
1093	55									
1098	50			61			#200		SM	Silty SAND; very dense; light yellowish brown; slightly moist; fine sand Total Depth = 51.5 feet Backfilled with neat cement. Groundwater not encountered.
1103	45			75	3.8	103.4	#200, ATT		ML	Sandy SILT; hard; light yellowish brown; moist; fine sand
1108	40			24			#200		SM	Silty SAND; medium dense; light yellowish brown; dry; fine sand
				43	16.6	96.4			SM	Silty SAND; dense; light yellowish brown; moist
					9.8	108.0			ML	Sandy SILT; very stiff; light gray; moist; some mica



**TWINING**

**LOG OF BORING**

Kaiser Redlands MOB 2  
 1301 California Street  
 Redlands, California

PROJECT NO.  
 220729.3

REPORT DATE  
 March 2023

FIGURE A - 13

DATE DRILLED 2/3/2023 LOGGED BY CDD **BORING NO.** B-13  
 DRIVE WEIGHT 140 lbs. DROP 30 inches DEPTH TO GROUNDWATER (ft.) N/E  
 DRILLING METHOD 8" HSA DRILLER Baja Exploration SURFACE ELEVATION (ft.) 1149.4(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES		BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
		Bulk Driven								
1114	35								SM	ARTIFICIAL FILL: Silty SAND; reddish brown; slightly moist to moist; fine sand
					6.8		#200		SM	ALLUVIUM: Silty SAND; light grayish brown; fine sand
1144	5			11					ML	SILT; stiff; light olive brown; dry
1139	10			14	3.1	98.0			SM	Silty SAND; loose; light olive brown; dry; fine sand
1134	15			7			#200, ATT		ML	SILT; medium stiff; light olive brown; slightly moist; fine sand
1129	20			25	5.2	90.7			SM	Silty SAND; medium dense; light olive brown; slightly moist; fine sand
1124	25			21					SM	-- same; medium dense; fine to medium sand; some laminations of silt and clay
1119	30			41	3.6	103.7			SM	-- same; medium dense; fine to medium sand; trace iron oxide staining Total Depth = 31.5 feet Backfilled on 2/3/2023 Backfilled with neat cement. Groundwater not encountered.



**TWINING**

**LOG OF BORING**

Kaiser Redlands MOB 2  
 1301 California Street  
 Redlands, California  
 PROJECT NO. 220729.3 REPORT DATE March 2023 FIGURE A - 14



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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 results are shown on the boring logs in Appendix A, and also summarized in Table B-1.

### **No. 200 Wash Sieve**

The amount of fines passing the No. 200 sieve was evaluated in accordance with ASTM D 1140. 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 D 4318. These test results were utilized to evaluate the soil classification in accordance with the Unified Soil Classification System. The test results are summarized in Figure B-1 and Table B-3.

### **Resistance Value (R-value)**

R-value testing was performed on a select bulk sample of the near-surface soils encountered at the site. The test was performed in general accordance with ASTM D 2844. The result is summarized in Table B-4.

### **Expansion Index**

The expansion index of a select soil sample was evaluated in general accordance with ASTM D 4829. The specimen was molded under a specified compactive energy at approximately 50 percent saturation. The prepared 1-inch thick by 4-inch diameter specimen was loaded with a surcharge of 144 pounds per square foot (psf) and was inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The result of expansion index test is presented in Table B-5.

### **Consolidation**

Consolidation tests were performed on select modified-California soil samples in general accordance with the latest version of ASTM D2435. The samples were inundated during testing to represent adverse field conditions. The percent consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The results of the tests are presented in Figures B-2 through B-5.

### **Direct Shear**

Direct shear tests were performed on representative soil samples in general accordance with the latest version of ASTM D 3080 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-6 through B-12.

### **Corrosivity**

Soil pH and resistivity tests were performed by Anaheim Test Lab, Inc. (ATLI) of Anaheim, California on representative soil samples. The resistivity of the soil assumes saturated soil



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conditions. The chloride and sulfate contents of the selected samples were evaluated in general accordance with the latest versions of Caltrans test methods CT417, CT422, and CT 643. The test results are presented on Table B-6 and the ATLI report included in this appendix.

**Table B-1  
 Moisture Content and Dry Density**

Boring No.	Depth (feet)	Moisture Content (%)	Dry Density (pcf)
B-1	0.5-3	6.3	N/A*
B-2	2-3	7.7	N/A
B-3	0.5-2	8.3	N/A
B-3	2-3	5.4	N/A
B-4	2-3	7.0	N/A
B-6	2-3	6.9	N/A
B-7	2-3	6.0	N/A
B-9	2-3	7.8	N/A
B-10	2-3	5.3	N/A
B-11	2	11.1	N/A
B-12	2-3	7.0	N/A
B-13	2-3	6.8	N/A
B-5	5.5	10.8	111.5
B-5	15.5	8.2	91.2
B-5	6	7.4	117.9
B-5	25.5	2.5	112.3
B-5	35.5	12.7	97.6
B-5	45	11.9	99.3
B-6	5	1.8	101.7
B-6	15	5.3	97.2
B-6	25	8.3	96.9
B-7	10	10.7	97.3
B-7	25	23.7	88.7
B-7	30	31.5	88.3
B-8	5	1.4	100.1
B-8	15	6.7	104.4
B-8	25	2.5	105.1
B-9	10	2.2	101.9
B-9	20	2.5	105.4
B-9	30	4.6	103.7
B-10	5.5	1.8	100.3
B-10	15.5	7.9	101.4
B-10	16	9.9	90.0
B-10	25.5	5.3	96.8
B-10	35.5	2.4	113.0
B-10	45.5	11.3	104.2



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B-11	10	4.9	102.9
B-11	20	12.0	104.8
B-11	30	8.9	111.5
B-12	5.5	2.6	85.7
B-12	6	2.4	99.4
B-12	15.5	4.4	92.9
B-12	16	3.7	92.7
B-12	25.5	11.0	87.8
B-12	26	5.8	102.4
B-12	35.5	16.6	96.4
B-12	36	9.8	108.0
B-12	45.5	3.8	103.4
B-12	46	11.1	102.2
B-13	10	3.1	98.0
B-13	20	5.2	90.7
B-13	30	3.6	103.7

\* Only moisture readings taken for bulk samples



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**Table B-2  
 Number 200 Wash Results**

Boring No.	Depth (feet)	Percent Passing #200
B-5	10	39.2
B-5	20	33.6
B-5	30	46.3
B-5	40	38.6
B-6	3-5	33.1
B-8	10	36.2
B-9	5	46.1
B-10	6	54.3
B-10	20	49.2
B-11	10	25.5
B-11	25	64.1
B-12	3-5	48.8
B-12	10	13.7
B-12	20	75.6
B-12	40	47.6
B-12	46	55.9
B-12	50	15.6
B-13	3-5	46.9
B-13	15	84.8

**Table B-3  
 Atterberg Limits Results**

Boring No.	Depth (feet)	Liquid Limit	Plastic Limit	Plasticity Index	U.S.C.S. Classification
B-5	10	NP	NP	NP	Silty SAND (SM)
B-5	30	NP	NP	NP	Silty SAND (SM)
B-6	3	NP	NP	NP	Silty SAND (SM)
B-7	30	NP	NP	NP	Sandy SILT (ML)
B-8	10	NP	NP	NP	Silty SAND (SM)
B-10	6	NP	NP	NP	Sandy SILT (ML)
B-10	20	NP	NP	NP	Silty SAND (SM)
B-12	46	NP	NP	NP	Sandy SILT (ML)
B-13	15	NP	NP	NP	SILT (ML)

**Table B-4  
Resistance Value (R-value)**

Boring No.	Depth (feet)	R Value
B-1	0.5-3	72
B-3	0.5-2	70

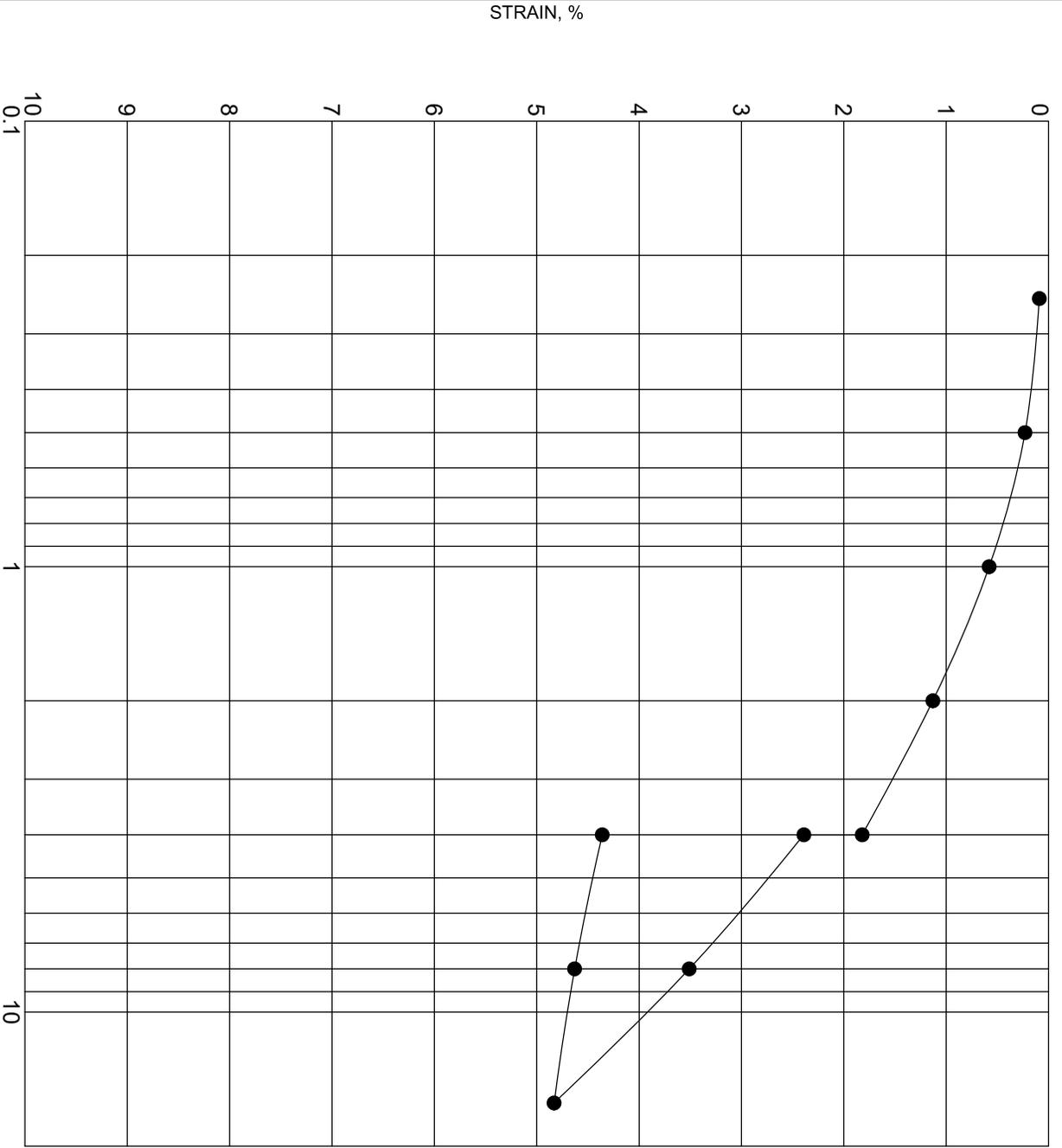
**Table B-5  
Expansion Index**

Boring No.	Depth (feet)	Expansion Index	Expansion Potential
B-2	0.5-2	0	Very Low
B-4	0.5-2	2	Very Low
B-10	0.5-2	0	Very Low
B-11	1-5	1	Very Low

**Table B-6  
Corrosivity Test Results**

Boring No.	Depth (feet)	pH	Water Soluble Sulfate (ppm)	Water Soluble Chloride (ppm)	Minimum Resistivity (ohm-cm)
B-10	0.5-2	7.5	172	51	2,200
B-11	1-5	7.4	189	39	6,400

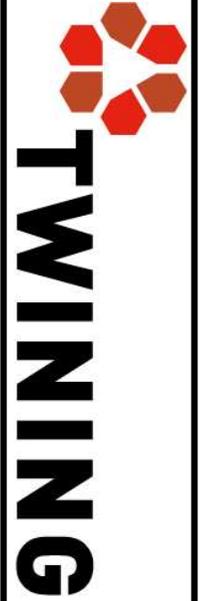




Sample Location	Soil Description	Dry Density (pcf)	Moisture Content (%)
● B-5 at 6 ft	Silty SAND	117.9	7.4

**CONSOLIDATION TEST**

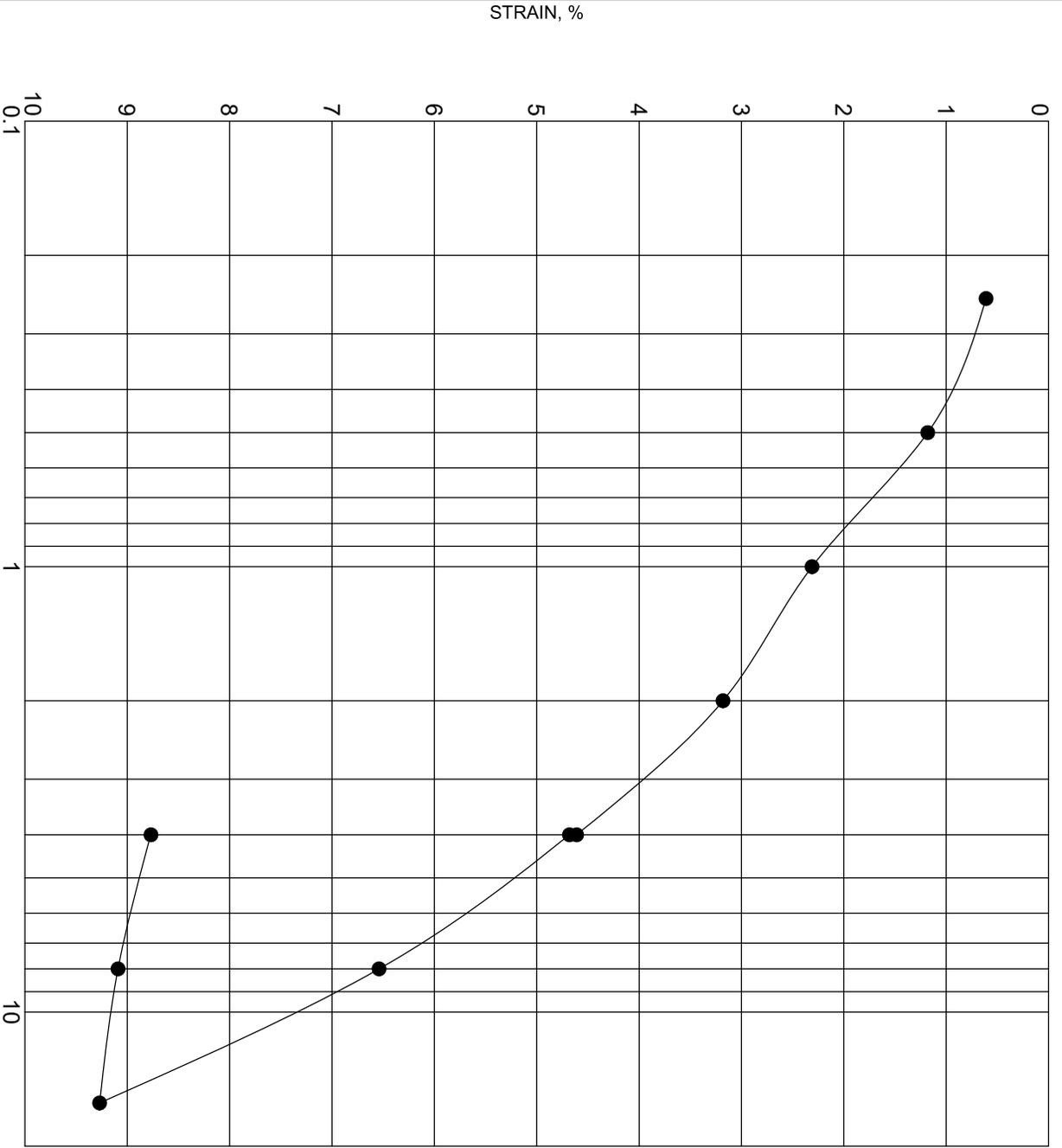
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FIGURE B-2



Sample Location	Soil Description	Dry Density (pcf)	Moisture Content (%)
● B-10 at 16 ft	SILTY SAND	101.4	7.9



**TWINING**

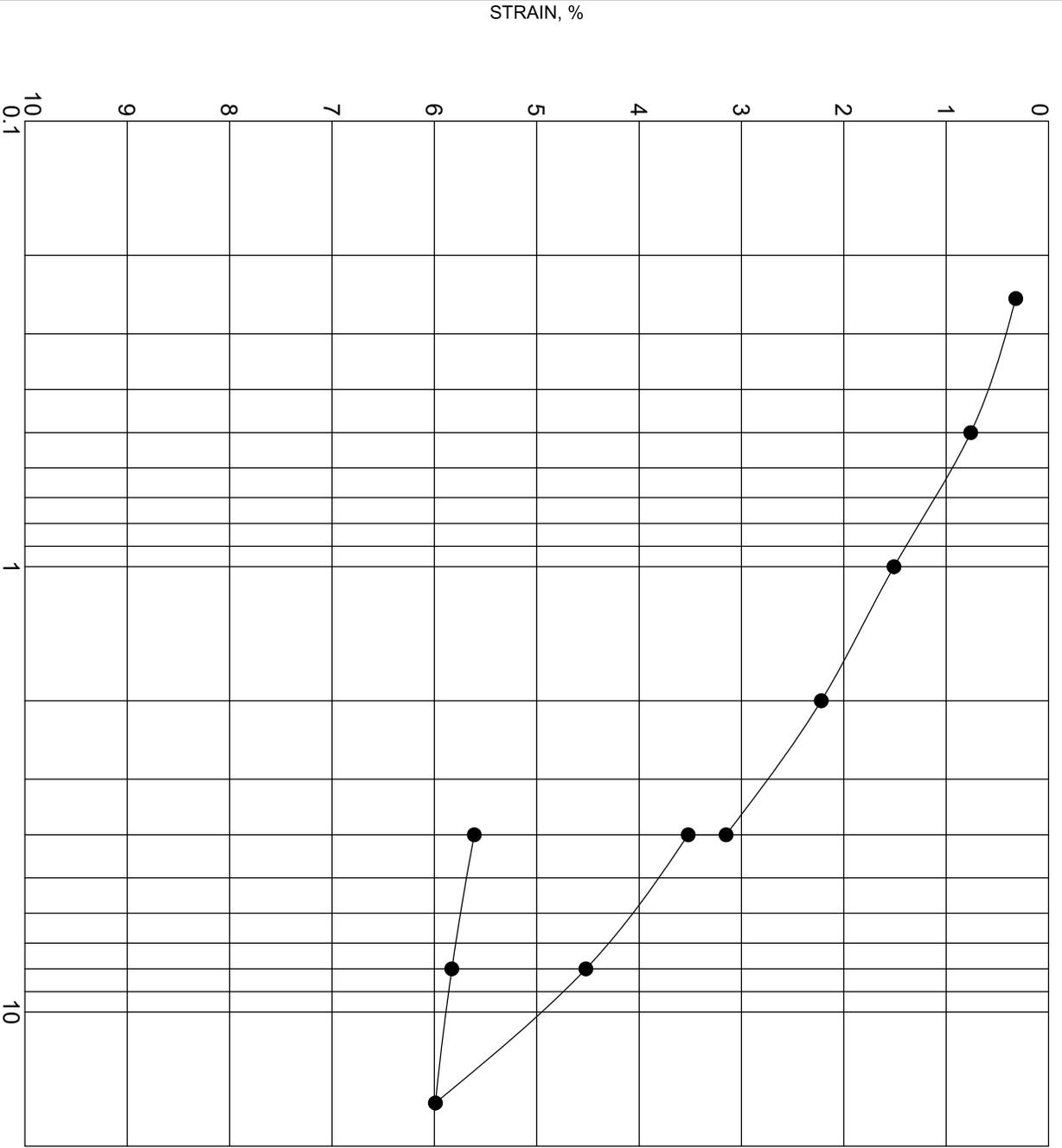
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FIGURE B-3



Sample Location	Soil Description	Dry Density (pcf)	Moisture Content (%)
● B-11 at 10 ft	Silty SAND	102.9	4.9

**CONSOLIDATION TEST**

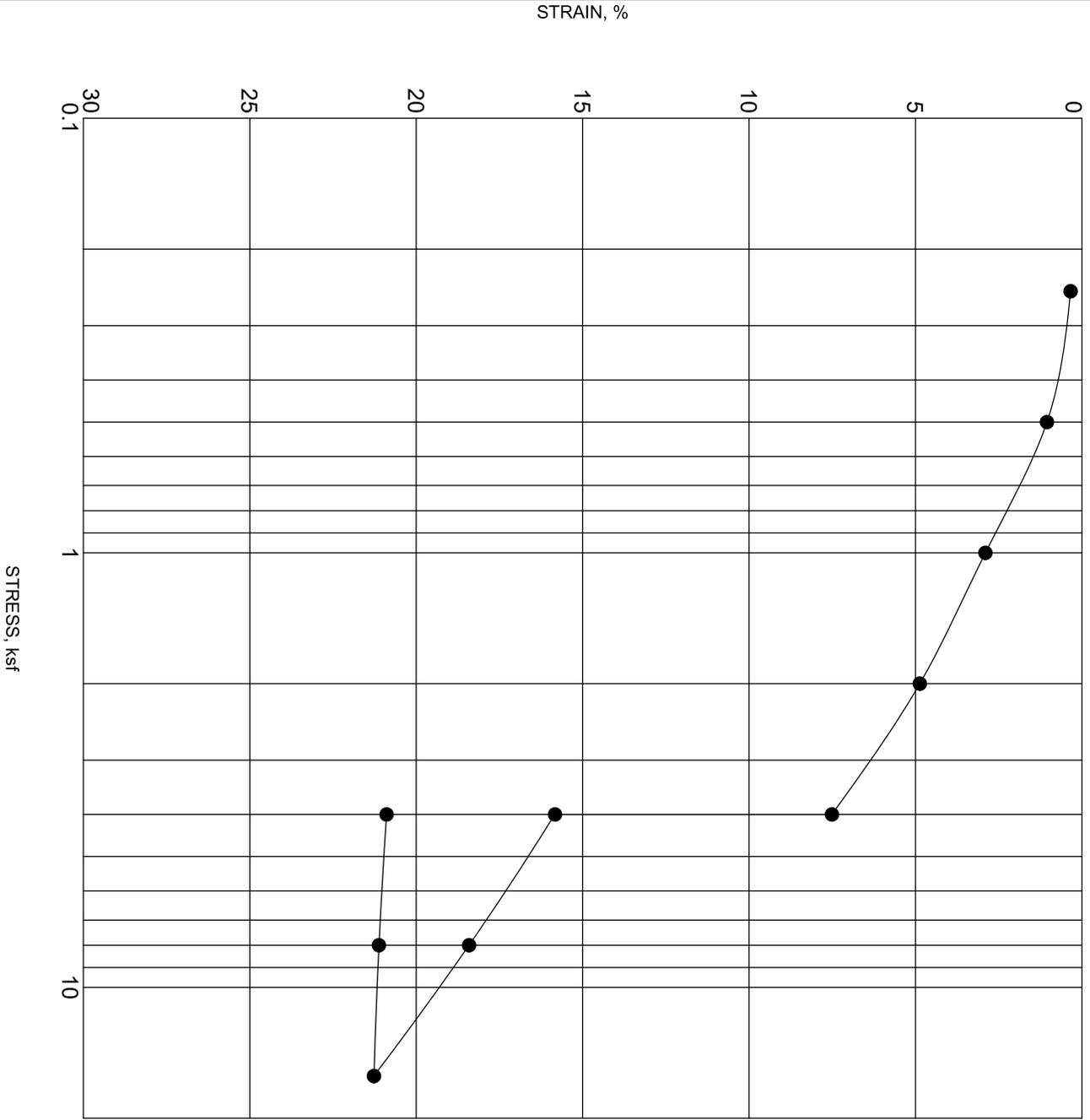
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FIGURE B-4



Sample Location	Soil Description	Dry Density (pcf)	Moisture Content (%)
● B-12 at 5.5 ft	Silly SAND	92.9	4.4



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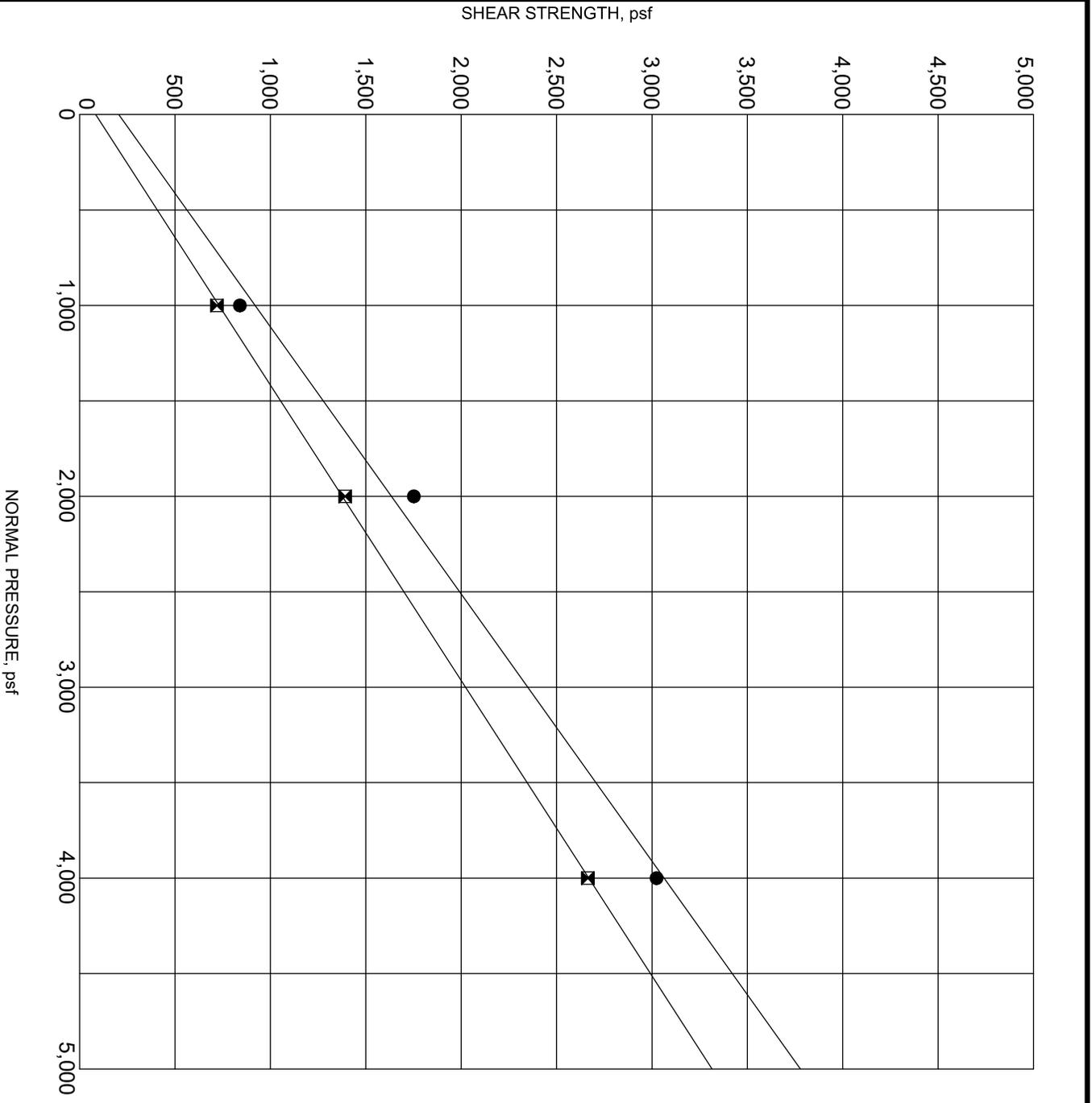
**CONSOLIDATION TEST**

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FIGURE B-5



Boring No.: B-5  
 Sample Depth (ft): 6  
 Sample Description: Silty SAND  
 Strain Rate (in./min): 0.005  
 Dry Density (pcf): 111.5

Cohesion, C (psf): 204  
 Friction Angle,  $\phi$  (deg): 36  
 Initial Moisture (%): 10.8  
 Final Moisture (%): 11.8

Shear Strength Parameters  
 Peak —●— Ultimate —x—

84  
33

**DIRECT SHEAR TEST**

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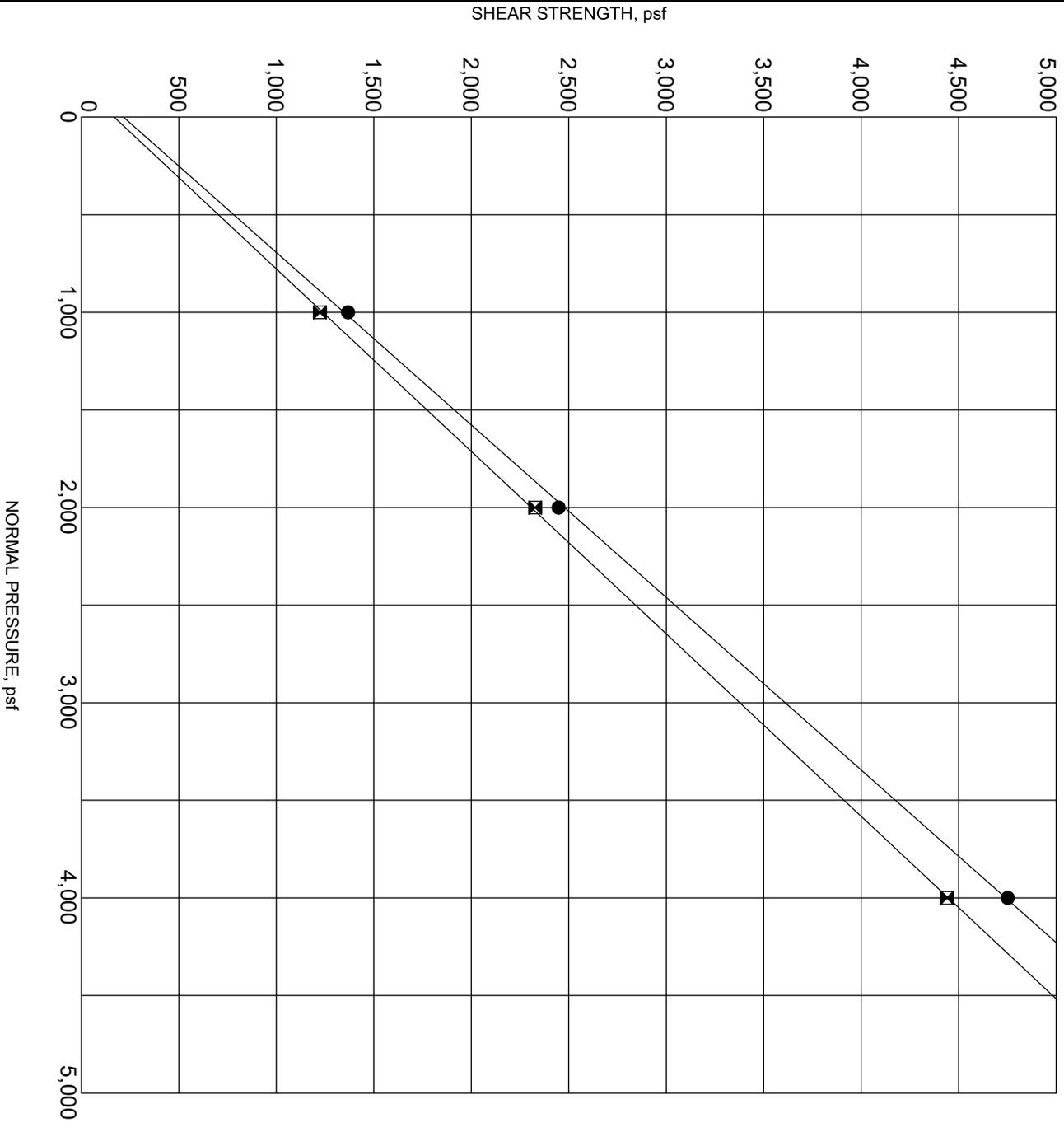
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FIGURE B-6



**TWINING**



**Boring No.:** B-5  
**Sample Depth (ft):** 16  
**Sample Description:** Silty SAND  
**Strain Rate (in./min):** 0.005  
**Dry Density (pcf):** 91.2

**Shear Strength Parameters**  
**Peak** ● **Ultimate** ✕  
**Cohesion, C (psf):** 216      168  
**Friction Angle,  $\phi$  (deg):** 49      47  
**Initial Moisture (%):** 8.2  
**Final Moisture (%):** 19.8

**DIRECT SHEAR TEST**

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 Redlands, California

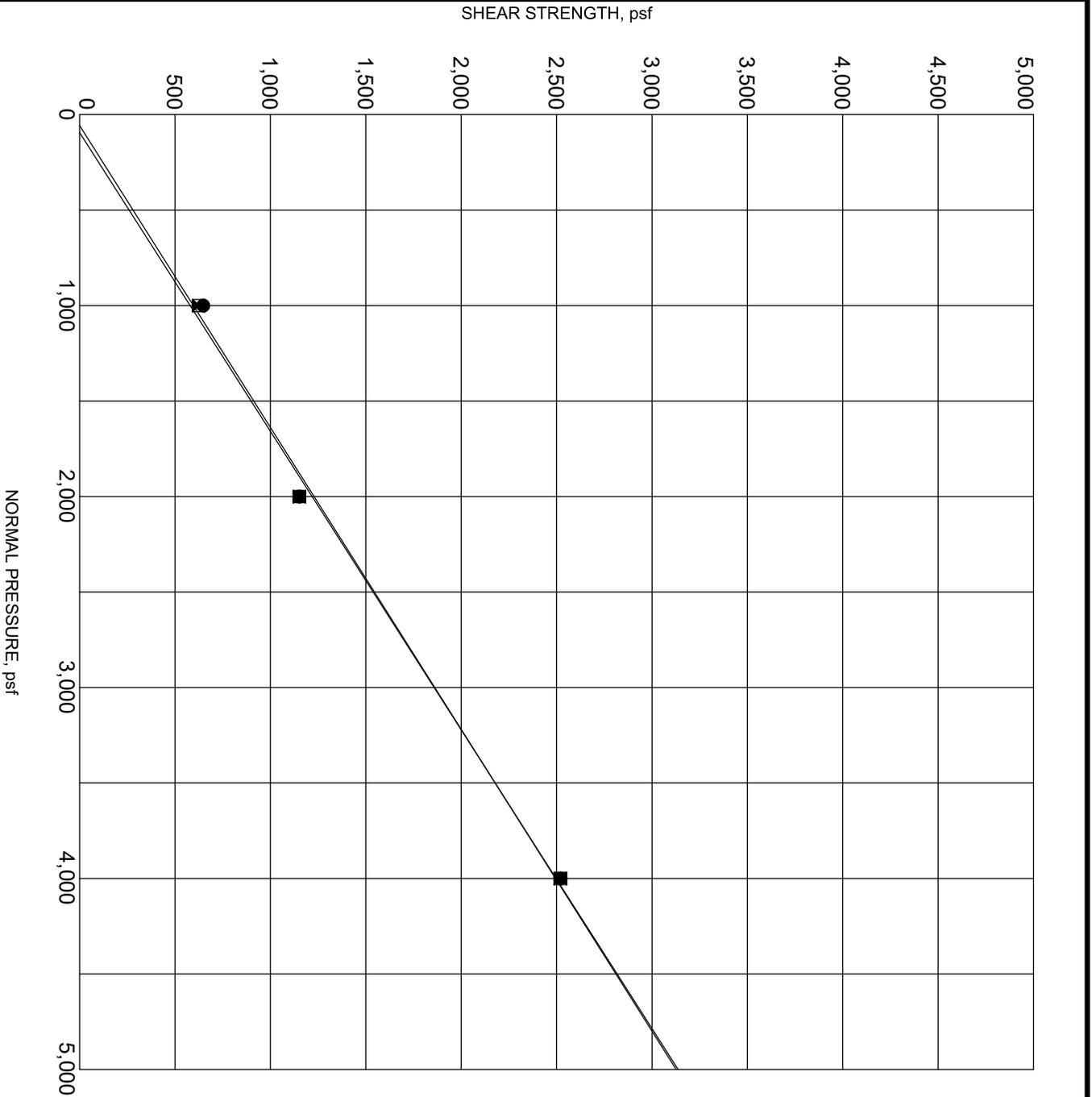
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**FIGURE B-7**



**TWINING**



Boring No.: B-6  
 Sample Depth (ft): 5  
 Sample Description: Silty SAND/Sandy SILT  
 Strain Rate (in./min): 0.005  
 Dry Density (pcf): 101.7

**Shear Strength Parameters**  
 Peak —●— Ultimate —■—  
 Cohesion, C (psf): 0  
 Friction Angle,  $\phi$  (deg): 32  
 Initial Moisture (%): 1.8  
 Final Moisture (%): 17.4

**DIRECT SHEAR TEST**

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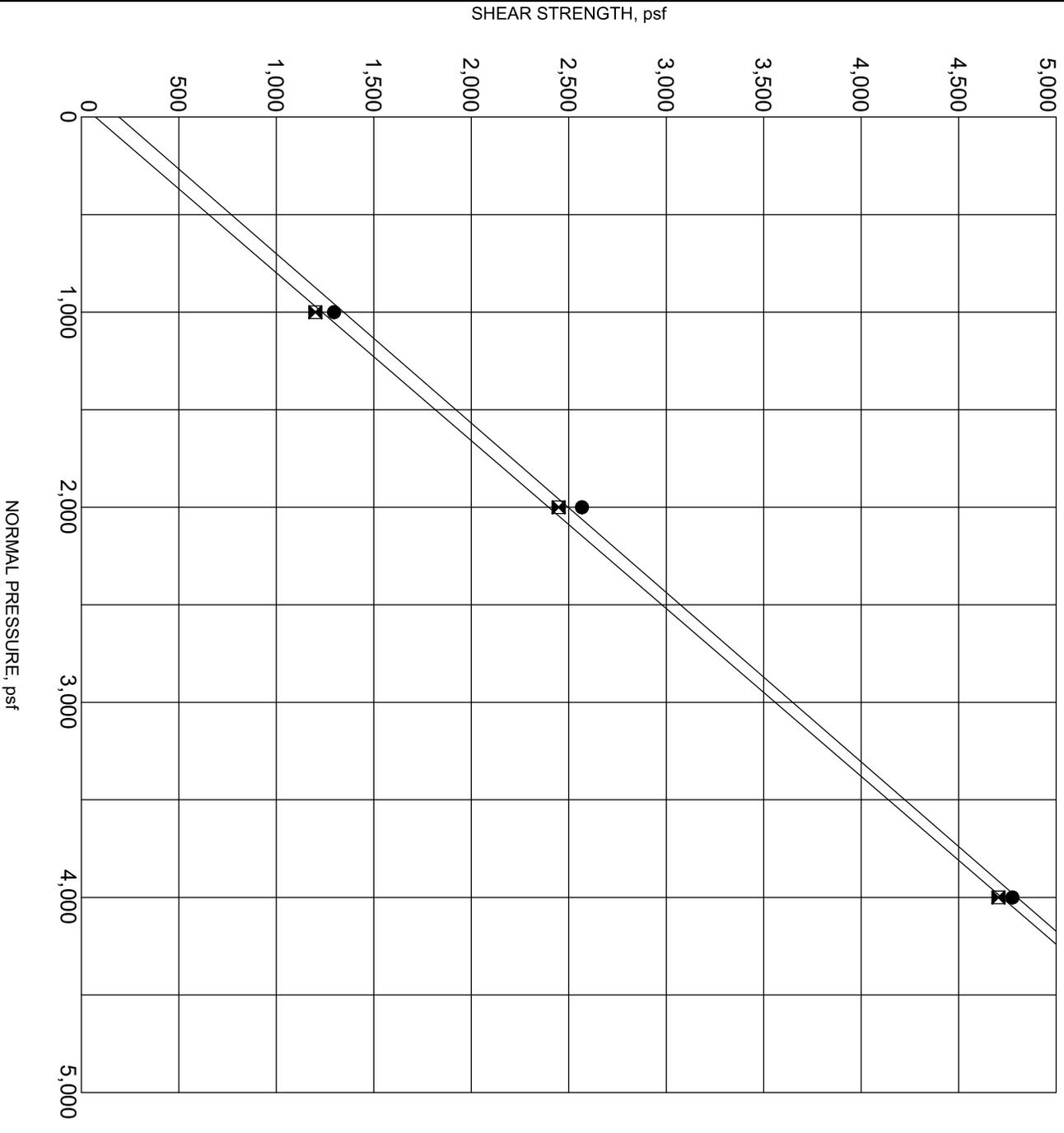
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FIGURE B-8



**TWINING**



**Shear Strength Parameters**

Peak —●— Ultimate —X—  
 Cohesion, C (psf): 192  
 Friction Angle,  $\phi$  (deg): 49

72  
49

Initial Moisture (%): 6.7  
 Final Moisture (%): 15.1

Boring No.: B-8  
 Sample Depth (ft): 15  
 Sample Description: Sandy SILT  
 Strain Rate (in./min): 0.005  
 Dry Density (pcf): 104.4

**DIRECT SHEAR TEST**

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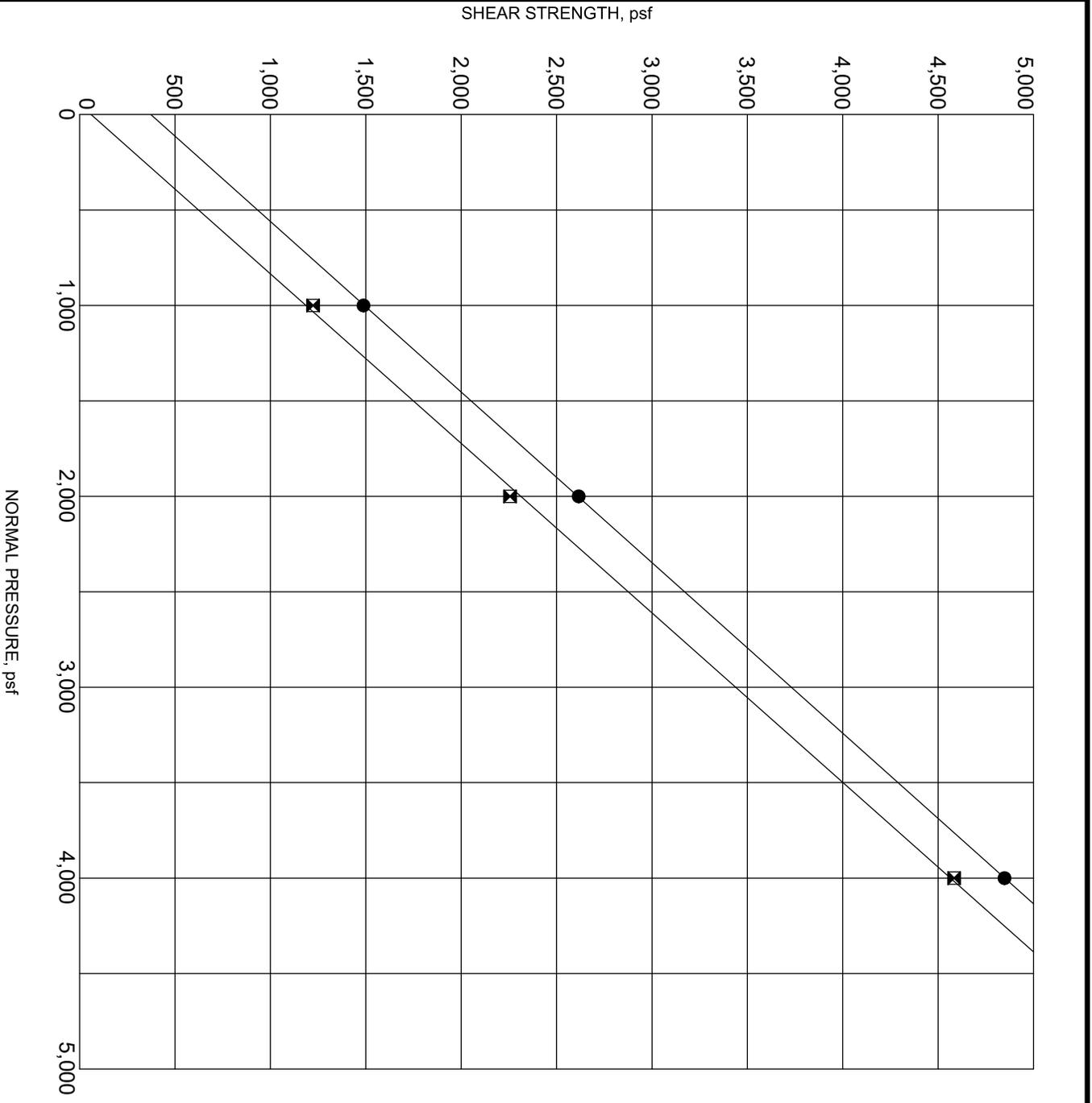
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FIGURE B-9



**TWINING**



Boring No.: B-9  
 Sample Depth (ft): 10  
 Sample Description: Silty SAND  
 Strain Rate (in./min): 0.005  
 Dry Density (pcf): 101.9

**Shear Strength Parameters**  
 Peak —●— Ultimate —x—  
 Cohesion, C (psf): 372  
 Friction Angle,  $\phi$  (deg): 48  
 Initial Moisture (%): 2.2  
 Final Moisture (%): 18.3

**DIRECT SHEAR TEST**

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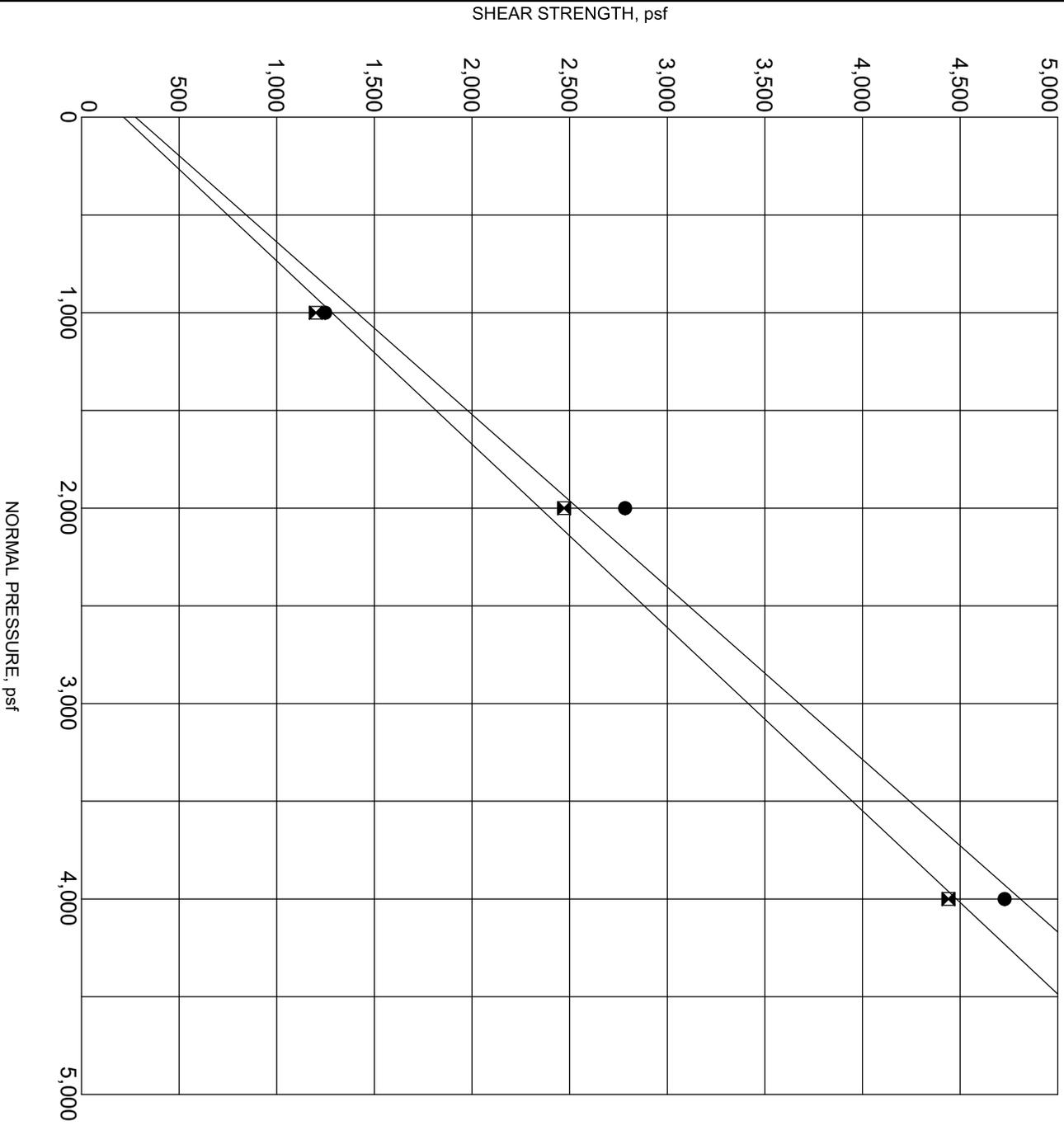
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FIGURE B-10



**TWINING**



**Boring No.:** B-10  
**Sample Depth (ft):** 16  
**Sample Description:** Silty SAND  
**Strain Rate (in./min):** 0.005  
**Dry Density (pcf):** 90.0

**Shear Strength Parameters**  
**Peak** ● **Ultimate** ✕  
**Cohesion, C (psf):** 276      216  
**Friction Angle,  $\phi$  (deg):** 49      47  
**Initial Moisture (%):** 9.9  
**Final Moisture (%):** 20.5

**DIRECT SHEAR TEST**

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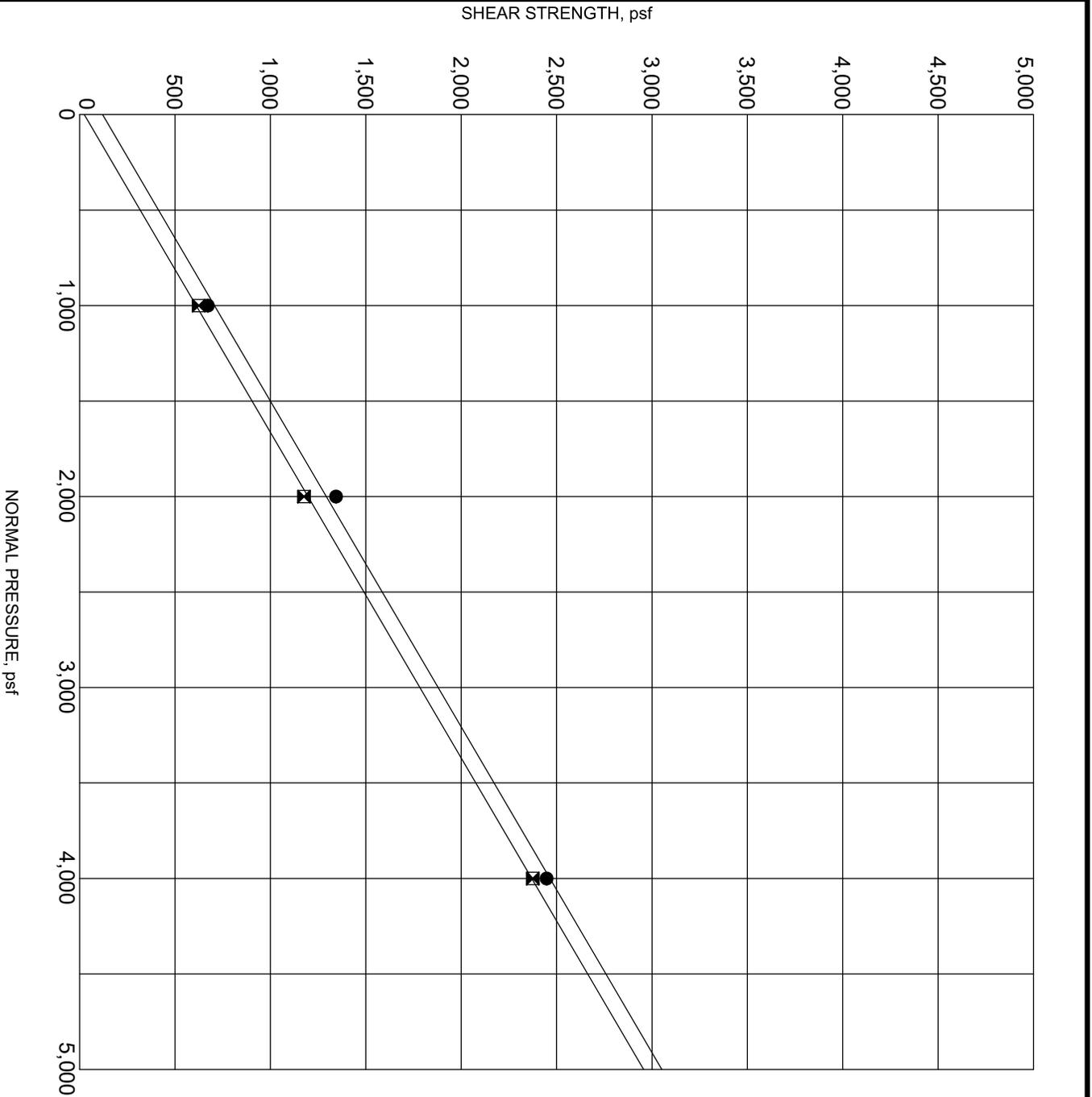
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FIGURE B-11



**TWINING**



**Boring No.:** B-12  
**Sample Depth (ft):** 6  
**Sample Description:** Silty SAND  
**Strain Rate (in./min):** 0.005  
**Dry Density (pcf):** 99.4

**Shear Strength Parameters**  
**Peak** ● **Ultimate** ✕  
**Cohesion, C (psf):** 120      24  
**Friction Angle,  $\phi$  (deg):** 30      30  
**Initial Moisture (%):** 2.4  
**Final Moisture (%):** 16.9

**DIRECT SHEAR TEST**

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**FIGURE B-12**



**TWINING**



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# APPENDIX C PERCOLATION TESTING



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March 16, 2023  
Project No. 220729.3

James Fung  
Team Manager, National Facilities Services  
Southern California Service Delivery Team  
Kaiser Foundation Health Plan  
Los Angeles Medical Center  
Service Building  
1550 N. Edgemont Street  
Los Angeles, CA 90027

**Subject: Percolation Testing**  
Kaiser Redlands  
1301 California Street  
Redlands, CA

Dear Mr. Fung:

In accordance with your request and authorization, we are presenting this percolation testing report prepared for the proposed Kaiser Redlands project to be located at 1301 California Street in Redlands, California – see Figure 1 – Site Location Map. The purpose of this letter report is to provide design infiltration rates for the proposed infiltration systems.

### **Field Exploration**

On January 27, 2023, we were provided by email with a map showing locations and depths to be tested by Michael Baker, International. The locations were first scanned by Geovision on February 20, 2023, using ground-penetrating radar to minimize the potential to disrupt any underground utilities. The percolation testing was performed on March 7 and 8, 2023 by Twining, Inc.

The field exploration consisted of excavating six borings to depths of approximately 5 to 8.75 feet below existing grade using a 5-inch-diameter hand auger. After logging the borings, the borings were converted to perform borehole percolation testing. Additionally, three test pits were excavated to perform percolation testing using the Open Pit Falling Head Procedure. P-7 was originally scheduled to be a test pit; however, the bottom of the existing basin exposed approximately 4 feet of compacted fill so it was converted to a borehole percolation. Finally, a double ring infiltrometer was used to test a location adjacent to one of the test pits. The locations of the borings and test pits are shown on Figure 2 – Site Plan and Percolation Location Map. Groundwater was not encountered in any of the borings or test pits. The existing surface elevation of the site is approximately 1148 feet above mean sea level (MSL). The bottom of the large existing basin in the northwest corner of the site is at approximately 1132 feet above MSL. The bottom of the existing basin at the northwest corner of the existing building is at approximately 1140 feet above MSL. The existing basin that runs north-south to the west of the existing medical office building has a bottom elevation of approximately 1143 feet above MSL.

### Borehole Percolation, Open Pit Falling Head and Double Ring Infiltrometer Testing

Borehole percolation, open pit falling head and double ring infiltrometer testing was conducted to provide design infiltration rates for the proposed infiltration systems. The testing was performed in accordance with the requirements of Appendix D of the San Bernardino County “Technical Guidance Document for Water Quality Management Plans,” dated June 7, 2013.

For the borehole percolation testing, the borings were advanced to the depths and locations requested by the project civil engineer. The materials encountered in the excavations generally consisted of olive to medium brown silty sand to the maximum depth of exploration.

After the borings were advanced to the planned depth, perforated PVC pipe was installed, the annular area surrounding the pipe was filled with gravel, and the percolation test was performed according to the requirements of San Bernardino County. Immediately upon completion of the percolation testing, the holes were backfilled with soil cuttings.

The percolation test rates have been converted to infiltration rates using the Porchet Method and a minimum factor of safety of 2 was applied. The factor of safety was determined using Worksheet H of the Technical Guidance Document. An additional factor of safety should be added by the project civil engineer based on the Factor Category B – see attached Worksheet H.

The following formula was used to determine the tested infiltration rate:

$$\text{Tested Infiltration Rate (inch/hour)} = \left( \frac{\Delta H(60r)}{\Delta t(r+2H_{avg})} \right)$$

where  $\Delta H$  is water drop in pipe in inches,  $\Delta t$  time interval in minutes,  $r$  borehole diameter in inches,  $H_{avg}$  average water height in inches in the pipe at the beginning and the end during the time interval.

$$\text{Design Infiltration Rate} = (\text{Tested Infiltration Rate}) / (\text{Factor of Safety} = 2)$$

The lowest recorded reading of the last three consecutive readings was used to determine the infiltration rate. A summary of test results with an applied factor of safety of 2 is presented in Table 1 below.

**Table 1 - Summary of Boring Percolation Test Results**

Location	Depth of Test Hole (ft)	Time Interval, $\Delta t$ (min.)	Design Infiltration Rate (in/hr)
P-3	8.75	10	0
P-4	7	10	3.45
P-5	5	10	2.43
P-6	7	10	3.91
P-7	5	10	0.42
P-8	6.5	10	2.11

The test pits were used to perform the Open Pit Falling Head Procedure. At the chosen locations, a 2x4 foot pit was excavated to a depth of 2 feet below the existing surface. The pit was then filled with water to one foot above the bottom of the pit. Readings were taken of the height of the water at 10-minute intervals. The infiltration rate was determined by calculating the volumetric flow rate and dividing by the average wetted area of infiltration. A factor of safety as determined by Worksheet H was then applied. As in the boring percolation tests, an additional factor of safety should be added by the project civil engineer based on the Factor Category B – see attached Worksheet H.

The lowest recorded reading of the last three consecutive readings was used to determine the infiltration rate. A summary of test results with an applied factor of safety of 2 is presented in Table 2 below.

**Table 2 – Open Pit Falling Head Test Results**

Location	Depth of Test Hole (ft)	Time Interval, $\Delta t$ (min.)	Design Infiltration Rate (in/hr)
P-1	2	10	0.73
P-2	2	10	1.43
P-9	2.2	10	0.87

At one location in the large existing basin located in the northwest corner of the property, a double-ring infiltrometer was used to perform infiltration testing adjacent to test pit P-2 that was used to perform an Open Pit Falling Head test. Two cylinders of sheet metal (one 12-inch diameter and one 24-inch diameter) were driven approximately 6 inches into the ground. Water was then added to the inside of the smaller cylinder and in the annular space between the larger and smaller at approximately equal heights. Mariotte tubes were then used to maintain a constant head and the volume of water infiltrated was measured at 10-minute intervals. The volumetric flow rate was then divided by the infiltration area to obtain an infiltration rate. A factor of safety was then applied as determined by Worksheet H. As in the boring percolation tests, an additional factor of safety should be added by the project civil engineer based on the Factor Category B – see attached Worksheet H.

The lowest recorded reading of the last three consecutive readings was used to determine the infiltration rate. A summary of test results with an applied factor of safety of 2 is presented in Table 3 below.

**Table 3 - Summary of Double Ring Infiltrimeter Test Results**

Location	Time Interval, $\Delta t$ (min.)	Design Infiltration Rate (in/hr)
DR-1	5	0.68

The results of the infiltration testing for all three methods are attached in Appendix B – Percolation Testing Results.

### Groundwater

Based on the previous report prepared by Leroy Crandall and Associates (Crandall, 1990), two wells were located within 1300 feet of the site. Groundwater has been as shallow as 33 feet below ground

surface (bgs) in the 1930's and 1940's and fell to a maximum of 210 feet bgs in the 1960s. During 1986, water was at approximately 74 feet bgs in one of the wells (1,054 feet MSL). Woodward-Clyde prepared a geotechnical report for the larger parcel which included the site in 1989 and encountered water at a depth of 50 feet bgs in one of their borings. Groundwater was not encountered in any of the borings performed by Crandall to a maximum depth of 60 feet bgs.

According to the California Department of Water Resources Water Data Library, there are two wells in the vicinity of the site: 340686N1172353W001 and 340739N1172334W001. The first is approximately 2,200 feet southwest of the site and the second is approximately 1,500 feet to the northwest. At the first well, the groundwater elevation varied from 944 feet MSL to 981 feet MSL from 2005 to 2008, which corresponds to a groundwater depth of approximately 166 feet bgs to 203 feet bgs at the current site. The groundwater elevation at the second site varied from a high of 1122 feet MSL in 1945 to a low of 912 feet MSL in 1966. While the groundwater levels fell from the late 1940s to 1966 they steadily increased until 1987 when they reached a local high of 1061 feet MSL. However, from 1991 to 2000, when measurement stopped, the water level remained relatively constant at approximately 1018 feet above MSL which corresponds to approximately 129 feet bgs at the current site.

During the current drilling for the office building, three borings were advanced by Twining to a depth of 51.5 feet bgs and groundwater was not encountered in any of the borings.

Based on the findings by Crandall, the data from the Water Data Library, and our current borings, the depth to groundwater can be assumed to be greater than 50 feet bgs for infiltration purposes.

### Recommendations

Based on the results of our testing and analyses, the installation of infiltration BMPs at the locations selected by the project architect is considered feasible. Table 4 shows recommended infiltration rates for each basin as shown on Figure 2 – Site Plan and Percolation Location Map.

**Table 4 - Recommend Infiltration Rate by Basin**

Basin	Recommended Design Infiltration Rate (in/hr)
A	1.77
B	0.95
C	3.91
D	1.38
E	2.11
F	0.87

Infiltration devices for managing stormwater should have a minimum setback from property lines and foundations recommended in Table 5. In addition, the bottom of the infiltration facility should be at least 10 feet above the seasonal high groundwater.

**Table 5 - Recommended Infiltration Facility Setback Requirements**

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

**Closure**

We appreciate the opportunity to be of service on this project. Should you have any questions or comments, please contact the undersigned at your convenience.

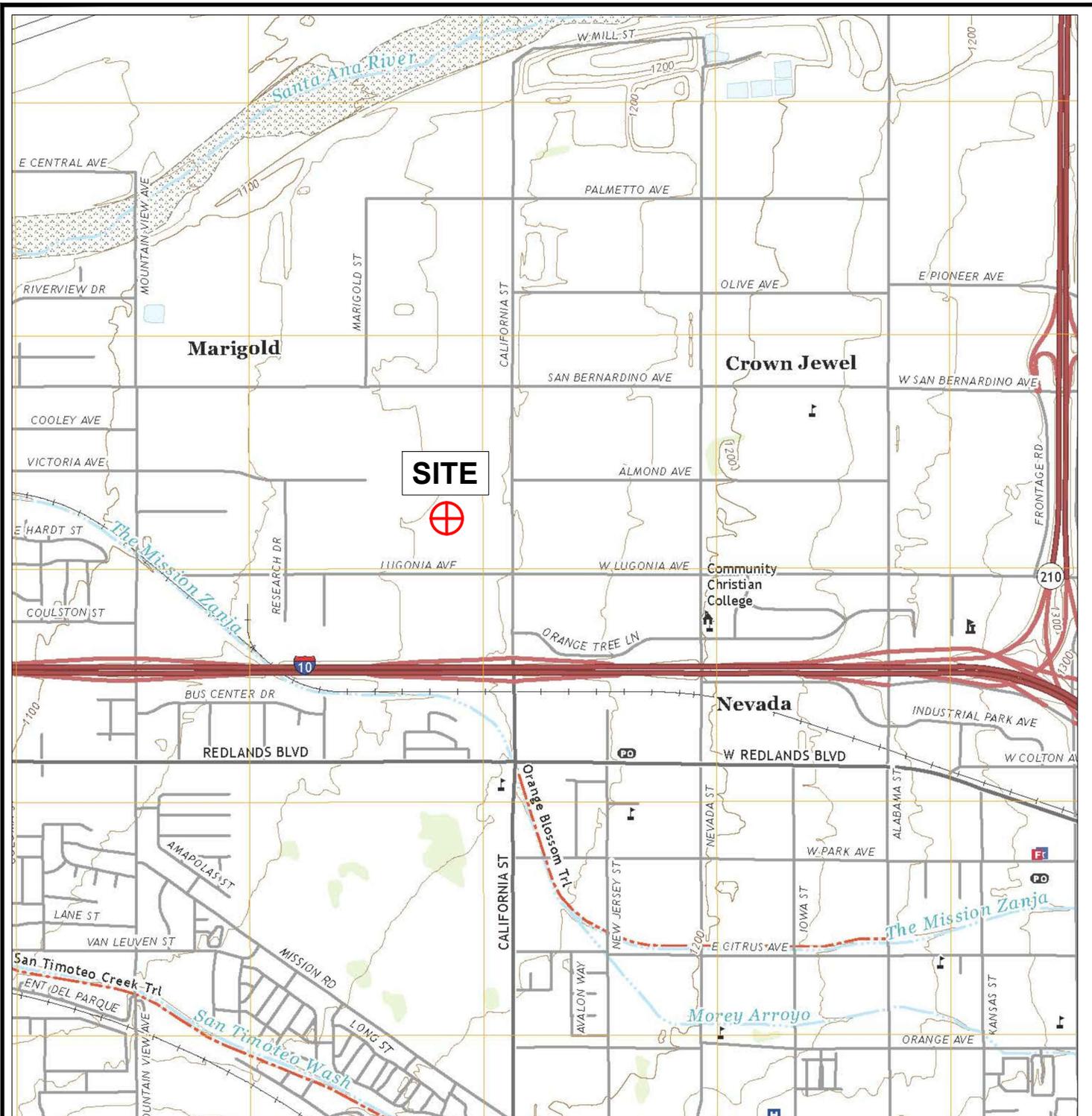
Respectfully submitted,  
**TWINING, INC.**



Paul Soltis, RCE 56140, GE 2606  
 Vice President, Geotechnical Engineering

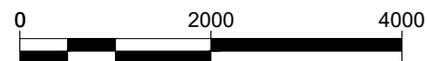
Doug Crayton  
 Staff Engineer

- Attachment(s):
- Figure 1 – Site Location Map
  - Figure 2 – Site Plan and Percolation Test Location Map
  - Worksheet H – Factor of Safety and Design Infiltration Rate and Worksheet
  - Appendix A – Field Exploration
  - Appendix B – Percolation Testing Results



APPROXIMATE LOCATION OF PROJECT

SCALE IN FEET



REFERENCE: USGS (2021)



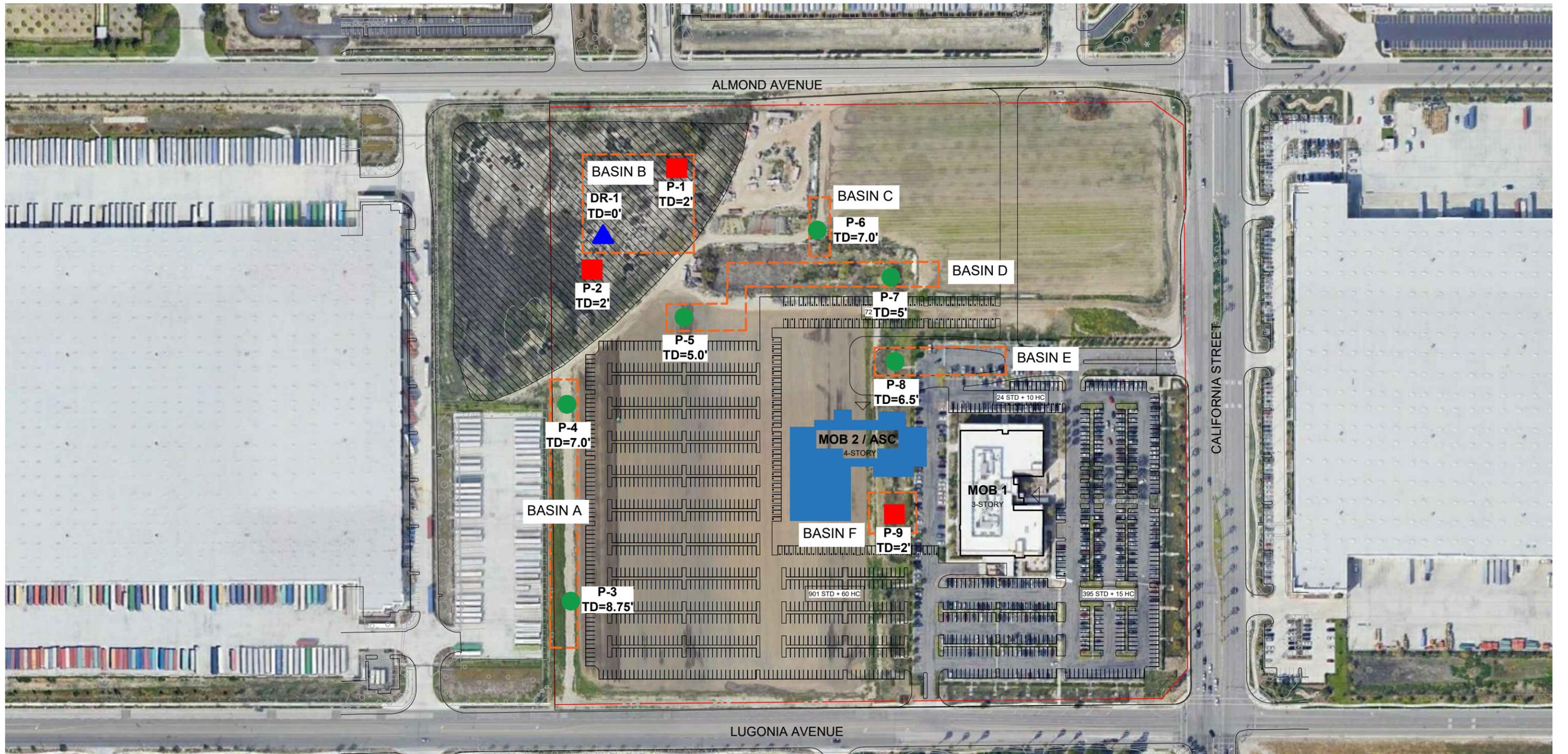
**SITE LOCATION MAP**

KAISER REDLANDS MOB 2  
1301 CALIFORNIA STREET  
REDLANDS, CALIFORNIA

PROJECT NO.  
220729.3

REPORT DATE  
March 2023

FIGURE 1



SCALE IN FEET



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

**LEGEND**

● P-3  
TD=8.75'

▲ DR-1  
TD=0'

BOREHOLE PERCOLATION TEST AND TOTAL DEPTH

DOUBLE-RING INFILTRIMETER TEST AND TOTAL DEPTH

■ P-1  
TD=2'

□ APPROXIMATE LOCATION OF PLANNED INFILTRATION BASIN

OPEN PIT TEST AND TOTAL DEPTH



**SITE PLAN AND PERCOLATION LOCATION MAP**

KAISER REDLANDS MOB 2  
1301 CALIFORNIA STREET  
REDLANDS, CALIFORNIA

PROJECT NO.  
220729.3

REPORT DATE  
March 2023

FIGURE 2

**Worksheet H: Factor of Safety and Design Infiltration Rate and Worksheet**

Factor Category		Factor Description	Assigned Weight (w)	Factor Value (v)	Product (p) $p=w \times v$
A	Suitability Assessment	Soil Assessment Methods	0.25	1	0.25
		Predominant soil texture	0.25	2	0.5
		Site soil variability	0.25	1	0.25
		Depth to groundwater / impervious layer	0.25	1	0.25
		Suitability Assessment Safety Factor, $S_A = \sum p$			
B	Design	Tributary area size	0.25		
		Level of pretreatment/ expected sediment loads	0.25		
		Redundancy	0.25		
		Compaction during construction	0.25		
		Design Safety Factor, $S_B = \sum p$			
Combined Safety Factor, $S_{TOT} = S_A \times S_B$					
Measured Infiltration Rate, inch/hr, $K_M$ (corrected for test-specific bias)					
Design Infiltration Rate, inch/hr, $K_{DESIGN} = S_{TOT} \times K_M$					
<b>Supporting Data</b>					
Briefly describe infiltration test and provide reference to test forms:					

**Note:** The minimum combined adjustment factor shall not be less than 2.0 and the maximum combined adjustment factor shall not exceed 9.0.



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## **APPENDIX A FIELD EXPLORATION**



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## **Appendix A Field Exploration**

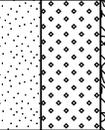
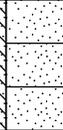
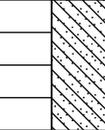
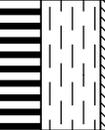
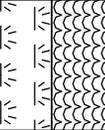
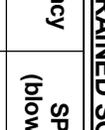
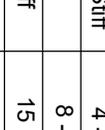
### **General**

The field exploration consisted of excavating six borings drilled to depths of 5 to 8.75 feet below existing grade using a 5-inch-diameter hand auger to perform borehole percolation testing. Additionally, three test pits were excavated to perform percolation testing using the Open Pit Falling Head Procedure. The pits measured 2x4' and were 2 feet deep and were excavated using a shovel. Finally, a double-ring infiltrometer was used to test a location adjacent to one of the test pits. Groundwater was not encountered in any of the borings or test pits. The existing surface elevation of the site is approximately 1148 feet above MSL. The bottom of the large existing basin in the northwest corner of the site is approximately 1132 feet above MSL. The bottom of the existing basin at the northwest corner of the existing building is approximately 1140 feet above MSL. The basin that runs north-south on the west of the building has an existing bottom elevation of approximately 1143 feet above MSL.

The approximate location of the borings and test pits is shown on Figure 2 – Site Plan and Percolation Location Map.

The attached logs show the soils encountered during our exploration.

## UNIFIED SOIL CLASSIFICATION CHART

MAJOR DIVISIONS		SYMBOLS		TYPICAL DESCRIPTIONS
		GRAPH	LETTER	
<b>GRAVEL AND GRAVELLY SOILS</b>	CLEAN GRAVELS <small>(LITTLE OR NO FINES)</small>		<b>GW</b>	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
			<b>GP</b>	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
	GRAVELS WITH FINES <small>(APPRECIABLE AMOUNT OF FINES)</small>		<b>GM</b>	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
			<b>GC</b>	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
<b>SAND AND SANDY SOILS</b>	CLEAN SANDS <small>(LITTLE OR NO FINES)</small>		<b>SW</b>	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
			<b>SP</b>	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
	SANDS WITH FINES <small>(APPRECIABLE AMOUNT OF FINES)</small>		<b>SM</b>	SILTY SANDS, SAND - SILT MIXTURES
			<b>SC</b>	CLAYEY SANDS, SAND - CLAY MIXTURES
<b>SILTS AND CLAYS</b>	LIQUID LIMIT LESS THAN 50		<b>ML</b>	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			<b>CL</b>	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
	LIQUID LIMIT GREATER THAN 50		<b>MH</b>	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
			<b>CH</b>	INORGANIC CLAYS OF HIGH PLASTICITY
<b>HIGHLY ORGANIC SOILS</b>	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY		<b>OH</b>	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
			<b>PT</b>	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

### COARSE-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

### FINE-GRAINED SOILS

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

## EXPLANATION FOR LOG OF BORINGS

Kaiser Redlands MOB 2  
1301 California Street  
Redlands, California

PROJECT NO.  
220729.3

REPORT DATE  
March 2023

FIGURE A-1



# TWINNING

DATE DRILLED 3/7/2023 LOGGED BY CDD **BORING NO.** P-1  
 DRIVE WEIGHT N/A DROP N/A DEPTH TO GROUNDWATER (ft.) N/E  
 DRILLING METHOD Shovel DRILLER Twining SURFACE ELEVATION (ft.) 1136 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES		BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
		Bulk Driven							
1126	10							SM	approximately 2 inches of topsoil <u>ARTIFICIAL FILL:</u> Silty SAND; interbedded layers; yellowish brown to olive brown; moist; fine to medium sand; very loose; micaceous; rootlets ALLUVIUM: Silty SAND; loose to medium dense; olive brown; moist; fine sand; some mica  Total Depth = 2.0 feet Backfilled on 3/7/2023 Converted to open test pit. Backfilled with cuttings. Groundwater not encountered.
1131	5							SM	



**TWINING**

**LOG OF BORING**

Kaiser Redlands MOB 2  
 1301 California Street  
 Redlands, California

PROJECT NO. 220729.3 REPORT DATE March 2023 FIGURE A - 2

DATE DRILLED 3/7/2023 LOGGED BY CDD **BORING NO.** P-2  
 DRIVE WEIGHT N/A DROP N/A DEPTH TO GROUNDWATER (ft.) N/E  
 DRILLING METHOD Shovel DRILLER Twining SURFACE ELEVATION (ft.) 1134 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES		BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
		Bulk Driven							
1124	10							SM	approximately 2 inches of topsoil
1129	5							SM	ARTIFICIAL FILL: Silty SAND; loose; olive brown; moist; fine to medium sand; micaceous; some rootlets
								SM	ALLUVIUM: Silty SAND; loose to medium dense; olive brown; moist; fine sand; micaceous
									Total Depth = 2.0 feet Backfilled on 3/7/2023 Converted to open test pit. Backfilled with cuttings. Groundwater not encountered.



**TWINING**

**LOG OF BORING**

Kaiser Redlands MOB 2  
 1301 California Street  
 Redlands, California

PROJECT NO. 220729.3      REPORT DATE March 2023      FIGURE A - 3

DATE DRILLED 3/7/2023 LOGGED BY DHC **BORING NO.** P-3  
 DRIVE WEIGHT N/A DROP N/A DEPTH TO GROUNDWATER (ft.) N/E  
 DRILLING METHOD 5" Hand Auger DRILLER Twining SURFACE ELEVATION (ft.) 1145 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES		BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
		Bulk Driven							
1135	10							SM	ARTIFICIAL FILL: Silty SAND; medium brown; slightly moist; mostly fine sand
1140	5							SM	ALLUVIUM: Silty SAND; medium brown; slightly moist; mostly fine sand
									- same; slightly lighter brown
									- same; light brown

Total Depth = 8.75 feet  
 Backfilled on 3/7/2023  
 Converted into percolation test  
 Backfilled with cuttings.  
 Groundwater not encountered.



**TWINING**

**LOG OF BORING**

Kaiser Redlands MOB 2  
 1301 California Street  
 Redlands, California

PROJECT NO. 220729.3 REPORT DATE March 2023 FIGURE A - 4

DATE DRILLED 3/7/2023 LOGGED BY DHC **BORING NO.** P-4  
 DRIVE WEIGHT N/A DROP N/A DEPTH TO GROUNDWATER (ft.) N/E  
 DRILLING METHOD 5" Hand Auger DRILLER Twining SURFACE ELEVATION (ft.) 1143.4(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES		BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
		Bulk Driven							
1133	10							SM	ARTIFICIAL FILL: Silty SAND; medium brown; slightly moist; mostly fine sand
1138	5							SM	ALLUVIUM: Silty SAND; medium brown; slightly moist; mostly fine sand  - same; slightly coarser sand
									Total Depth = 7.0 feet Backfilled on 3/7/2023 Converted into percolation test Backfilled with cuttings. Groundwater not encountered.



**TWINING**

**LOG OF BORING**

Kaiser Redlands MOB 2  
 1301 California Street  
 Redlands, California

PROJECT NO. 220729.3 REPORT DATE March 2023 FIGURE A - 5

DATE DRILLED 3/7/2023 LOGGED BY DHC **BORING NO.** P-5  
 DRIVE WEIGHT N/A DROP N/A DEPTH TO GROUNDWATER (ft.) N/E  
 DRILLING METHOD 5" Hand Auger DRILLER Twining SURFACE ELEVATION (ft.) 1146 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES		BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
		Bulk	Driven						
1136	10							SM	<u>ARTIFICIAL FILL:</u> Silty SAND; medium brown; slightly moist; mostly fine sand
1141	5							SM	<u>ALLUVIUM:</u> Silty SAND; medium brown; slightly moist; mostly fine sand - same; light brown
									Total Depth = 5.0 feet Backfilled on 3/7/2023 Converted into percolation test Backfilled with cuttings. Groundwater not encountered.



**TWINING**

**LOG OF BORING**

Kaiser Redlands MOB 2  
 1301 California Street  
 Redlands, California

PROJECT NO.  
220729.3

REPORT DATE  
March 2023

FIGURE A - 6

DATE DRILLED 3/7/2023 LOGGED BY DHC **BORING NO.** P-6  
 DRIVE WEIGHT N/A DROP N/A DEPTH TO GROUNDWATER (ft.) N/E  
 DRILLING METHOD 5" Hand Auger DRILLER Twining SURFACE ELEVATION (ft.) 1152.4(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES		BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
		Bulk Driven							
1142	10							SM	ARTIFICIAL FILL: Silty SAND; medium brown; slightly moist; fine sand
1147	5							SM	ALLUVIUM: Silty SAND; light brown; slightly moist; fine sand
									Total Depth = 7.0 feet Backfilled on 3/7/2023 Converted into percolation test Backfilled with cuttings. Groundwater not encountered.



**TWINING**

**LOG OF BORING**

Kaiser Redlands MOB 2  
 1301 California Street  
 Redlands, California

PROJECT NO. 220729.3 REPORT DATE March 2023 FIGURE A - 7

DATE DRILLED 3/8/2023 LOGGED BY            CDD            **BORING NO.** P-7  
 DRIVE WEIGHT            N/A DROP            N/A DEPTH TO GROUNDWATER (ft.)            N/E  
 DRILLING METHOD 5" Hand Auger DRILLER Twining SURFACE ELEVATION (ft.) 1147 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES		BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
		Bulk	Driven						
1137	10								
								ML	approximately 2 inches of topsoil
								ML	ARTIFICIAL FILL: Sandy SILT; olive brown; moist; some mica
									- same; trace semi-angular gravel
									- same; increased silt
								ML	ALLUVIUM: Sandy SILT; loose; light olive brown; moist
1142	5								Total Depth = 5.0 feet Backfilled on 3/8/2023 Converted into percolation test Backfilled with cuttings. Groundwater not encountered.



**TWINING**

**LOG OF BORING**

PROJECT NO. 220729.3 REPORT DATE March 2023 FIGURE A - 8  
 Kaiser Redlands MOB 2  
 1301 California Street  
 Redlands, California

DATE DRILLED 3/8/2023 LOGGED BY AB **BORING NO.** P-8  
 DRIVE WEIGHT N/A DROP N/A DEPTH TO GROUNDWATER (ft.) N/E  
 DRILLING METHOD 5" Hand Auger DRILLER Twining SURFACE ELEVATION (ft.) 1148 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES		BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
		Bulk	Driven						
1138	10							SM	ALLUVIUM: Silty SAND; medium brown; slightly moist; mostly fine sand  - same; becomes moist  - same; increased silt; some oxidation staining  - same; increased course sand; slightly moist; light brown  Total Depth = 6.5 feet Backfilled on 3/8/2023 Converted into percolation test Backfilled with cuttings. Groundwater not encountered.
1143	5								



**TWINING**

**LOG OF BORING**

Kaiser Redlands MOB 2  
 1301 California Street  
 Redlands, California

PROJECT NO. 220729.3 REPORT DATE March 2023 FIGURE A - 9

DATE DRILLED 3/8/2023 LOGGED BY AB **BORING NO.** P-9  
 DRIVE WEIGHT N/A DROP N/A DEPTH TO GROUNDWATER (ft.) N/E  
 DRILLING METHOD Shovel DRILLER Twining SURFACE ELEVATION (ft.) 1148 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES		BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
		Bulk Driven							
1138	10							SM/ML	Approximately 2 inches of topsoil Silty SAND/ Sandy SILT; Olive brown; moist; micaceous; some rootlets  Total Depth = 2.0 feet Backfilled on 3/8/2023 Converted to open test pit. Backfilled with cuttings. Groundwater not encountered.
1143	5								



**TWINING**

**LOG OF BORING**

Kaiser Redlands MOB 2  
 1301 California Street  
 Redlands, California

PROJECT NO. 220729.3 REPORT DATE March 2023 FIGURE A - 10



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## **APPENDIX B PERCOLATION TESTING RESULTS**

## Infiltration Rate Calculation Sheet

Project :	Kaiser Redlands	Project No. :	220729.3	Date :	3/7/2023
Test Hole No.:	P-1	Tested by :	CD		
Depth of Test Hole, $D_T$ (in.):	24	USCS Soil Classification :	SM		
Test Hole Dimension (inches)			Length	Width	
Diameter (if round) (inches) =		Sides (if rectangular) =	48	24	

### Sandy Soil Criteria Test\*

Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Height of Water (in.)	Final Height of Water (in.)	Change in Water Level (in.)	
1	10:33 AM	10:43 AM	10	12.0	10.25	1.75	
2	10:45 AM	10:55 AM	10	12.0	11.13	0.88	

Trial No.	Start Time	Stop Time	$\Delta t$	$H_o$	$H_f$	$\Delta H$	
Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Water Height (inches)	Final Water Height (inches)	Change in Water Level (inches)	Tested Infiltration Rate (in/hr)
1	11:00 AM	11:20 AM	20	12.00	10.25	1.75	2.20
2	11:20 AM	11:40 AM	20	10.25	8.88	1.38	1.88
3	11:40 AM	12:00 PM	20	8.88	7.63	1.25	1.85
4	12:00 PM	12:20 PM	20	7.63	6.50	1.13	1.79
5	12:20 PM	12:40 PM	20	6.50	5.63	0.88	1.49
6	12:40 PM	12:50 PM	10	5.63	4.13	1.50	5.59
7	12:56 PM	1:06 PM	10	11.50	10.50	1.00	2.53
8	1:06 PM	1:16 PM	10	10.50	10.13	0.38	0.98
9	1:16 PM	1:26 PM	10	10.13	9.63	0.50	1.34
10	1:26 PM	1:36 PM	10	9.63	9.13	0.50	1.38
11	1:36 PM	1:46 PM	10	9.13	8.63	0.50	1.42
12	1:46 PM	1:56 PM	10	8.63	8.13	0.50	1.47
13	1:56 PM	2:06 PM	10	8.125	7.63	0.50	1.51
14	2:06 PM	2:16 PM	10	7.625	7.13	0.50	1.56

**Raw Infiltration Rate (no factor of safety) = 1.47 inch /hr**  
**Factor of Safety = 2.00**  
**Design Infiltration Rate = 0.73 inch/hr**

## Infiltration Rate Calculation Sheet

Project :	Kaiser Redlands	Project No. :	220729.3	Date :	3/7/2023
Test Hole No.:	P-2	Tested by :	CD		
Depth of Test Hole, $D_T$ (in):	24	USCS Soil Classification :	SM		
Test Hole Dimension (inches)			Length	Width	
Diameter (if round) (inches) =		Sides (if rectangular) =	48	24	

### Sandy Soil Criteria Test\*

Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Height of Water (in.)	Final Height of Water (in.)	Change in Water Level (in.)	
1	1:07 PM	1:17 PM	10	12.00	10.50	1.50	
2	1:20 PM	1:30 PM	10	10.50	9.50	1.00	

Trial No.	Start Time	Stop Time	$\Delta t$	$H_o$	$H_f$	$\Delta H$	
Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Water Height (inches)	Final Water Height (inches)	Change in Water Level (inches)	Tested Infiltration Rate
1	1:30 PM	1:40 PM	10	12.00	10.50	1.50	3.74
2	1:40 PM	1:50 PM	10	10.50	9.50	1.00	2.67
3	1:50 PM	2:00 PM	10	9.50	8.13	1.38	3.93
4	2:00 PM	2:10 PM	10	8.13	7.25	0.88	2.68
5	2:10 PM	2:30 PM	20	7.25	6.13	1.13	1.84
6	2:30 PM	2:40 PM	10	6.13	5.25	0.88	3.07
7	2:40 PM	2:50 PM	10	5.25	4.25	1.00	3.76
8	2:50 PM	3:00 PM	10	4.25	3.75	0.50	2.00
9	3:00 PM	3:10 PM	10	9.25	8.63	0.63	1.77
10	3:10 PM	3:20 PM	10	8.63	7.88	0.75	2.22
11	3:20 PM	3:30 PM	10	7.88	6.50	1.38	4.35
12	3:30 PM	3:40 PM	10	6.50	6.00	0.50	1.68
13	3:40 PM	3:50 PM	10	6.00	5.00	1.00	3.56
14	3:50 PM	4:00 PM	10	5.00	4.25	0.75	2.85
15	4:00 PM	4:10 PM	10	4.25	3.50	0.75	3.03

**Raw Infiltration Rate (no factor of safety) = 2.85 inch /hr**  
**Factor of Safety = 2.00**  
**Design Infiltration Rate = 1.43 inch/hr**

## Infiltration Rate Calculation Sheet

Project :	Kaiser Redlands	Project No. :	220729.3	Date :	3/7/2023
Test Hole No.:	P-3	Tested by :	AB		
Depth of Test Hole, $D_T$ (in):	105	USCS Soil Classification :	SM		
Test Hole Dimension (inches)			Length	Width	
Diameter (if round) (inches) =	5.00	Sides (if rectangular) =			

### Sandy Soil Criteria Test\*

Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6" ? (Y/N)
1	11:23 AM	11:38 AM	15	24.0	32.1	8.1	Y
2	11:39 AM	11:54 AM	15	27.6	35.1	7.5	Y

\*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	Stop Time	$\Delta t$	$H_o$	$H_f$	$\Delta H$	Tested Infiltration Rate
1	1:17 PM	1:27 PM	10	81.25	75.50	5.75	0.54
2	1:30 PM	1:40 PM	10	75.50	71.00	4.50	0.45
3	1:41 PM	1:51 PM	10	82.75	79.13	3.63	0.33
4	1:53 PM	2:03 PM	10	79.13	76.63	2.50	0.24
5	2:03 PM	2:13 PM	10	80.50	77.88	2.63	0.24
6	2:15 PM	2:25 PM	10	77.50	75.50	2.00	0.19
7	2:26 PM	2:36 PM	10	81.00	78.88	2.13	0.20
8	2:37 PM	2:47 PM	10	78.88	77.00	1.88	0.18
9	2:51 PM	3:01 PM	10	77.00	75.13	1.88	0.18
10	3:03 PM	3:13 PM	10	81.50	79.75	1.75	0.16

**Raw Infiltration Rate (no factor of safety) = 0.16 inch /hr**  
**Factor of Safety = 2.00**  
**Design Infiltration Rate = 0.08 inch/hr**

## Infiltration Rate Calculation Sheet

Project :	Kaiser Redlands	Project No. :	220729.3	Date :	3/7/2023
Test Hole No.:	P-4	Tested by :	AB		
Depth of Test Hole, $D_T$ (in):	84	USCS Soil Classification :	SM		
Test Hole Dimension (inches)			Length	Width	
Diameter (if round) (inches) =	5.00	Sides (if rectangular) =			

### Sandy Soil Criteria Test\*

Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6" ? (Y/N)
1	9:43 AM	10:08 AM	25	36.0	75.6	39.6	Y
2	10:09 AM	10:34 AM	25	30.0	73.2	43.2	Y

\*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	Stop Time	$\Delta t$ Time Interval (min.)	$H_o$ Initial Water Height (inches)	$H_f$ Final Water Height (inches)	$\Delta H$ Change in Water Level (inches)	Tested Infiltration Rate
1	10:40 AM	10:50 AM	10	50.04	13.20	36.84	8.41
2	10:52 AM	11:02 AM	10	60.00	14.40	45.60	8.89
3	11:04 AM	11:14 AM	10	50.40	13.56	36.84	8.31
4	11:16 AM	11:26 AM	10	48.00	14.64	33.36	7.68
5	11:27 AM	11:37 AM	10	48.00	16.20	31.80	7.15
6	11:38 AM	11:48 AM	10	49.80	15.60	34.20	7.56
7	11:50 AM	12:00 PM	10	49.20	15.60	33.60	7.49
8	12:01 PM	12:11 PM	10	48.00	16.80	31.20	6.95
9	12:14 PM	12:24 PM	10	47.76	16.20	31.56	7.12
10	12:29 PM	12:39 PM	10	49.20	17.40	31.80	6.90

**Raw Infiltration Rate (no factor of safety) = 6.90 inch /hr**  
**Factor of Safety = 2.00**  
**Design Infiltration Rate = 3.45 inch/hr**

## Infiltration Rate Calculation Sheet

Project :	Kaiser Redlands	Project No. :	220729.3	Date :	3/7/2023
Test Hole No.:	P-5	Tested by :	TT		
Depth of Test Hole, $D_T$ (in):	60	USCS Soil Classification :	SM		
Test Hole Dimension (inches)			Length	Width	
Diameter (if round) (inches) =	5.00	Sides (if rectangular) =			

### Sandy Soil Criteria Test\*

Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6" ? (Y/N)
1	12:33 PM	12:48 PM	15	10.8	48.4	37.6	Y
2	12:55 PM	1:10 PM	15	11.5	51.3	39.8	Y

\*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	Stop Time	$\Delta t$	$H_o$	$H_f$	$\Delta H$	Tested Infiltration Rate
1	1:20 PM	1:30 PM	10	37.20	11.40	25.80	7.57
2	1:30 PM	1:40 PM	10	36.00	13.80	22.20	6.37
3	1:40 PM	1:50 PM	10	13.80	3.60	10.20	7.69
4	1:50 PM	2:00 PM	10	36.00	22.80	13.20	3.23
5	2:00 PM	2:10 PM	10	36.00	16.80	19.20	5.21
6	2:10 PM	2:20 PM	10	49.20	25.20	24.00	4.68
7	2:20 PM	2:30 PM	10	25.20	10.80	14.40	5.61

**Raw Infiltration Rate (no factor of safety) = 4.68 inch /hr**

**Factor of Safety = 2.00**

**Design Infiltration Rate = 2.34 inch/hr**

## Infiltration Rate Calculation Sheet

Project :	Kaiser Redlands	Project No. :	220729.3	Date :	3/7/2023
Test Hole No.:	P-6	Tested by :	TT		
Depth of Test Hole, $D_T$ (in):	84	USCS Soil Classification :	SM		
Test Hole Dimension (inches)			Length	Width	
Diameter (if round) (inches) =	5.00	Sides (if rectangular) =			

Sandy Soil Criteria Test*							
Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6" ? (Y/N)
1	12:29 PM	12:39 PM	10	24.0	58.2	34.2	Y
2	12:55 PM	1:05 PM	10	25.7	59.4	33.7	Y

\*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

			$\Delta t$	$H_o$	$H_f$	$\Delta H$	
Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Water Height (inches)	Final Water Height (inches)	Change in Water Level (inches)	Tested Infiltration Rate
1	1:25 PM	1:35 PM	10	54.00	18.00	36.00	7.25
2	1:35 PM	1:45 PM	10	60.00	24.00	36.00	6.24
3	1:45 PM	1:55 PM	10	60.00	27.60	32.40	5.39
4	1:55 PM	2:05 PM	10	60.00	28.80	31.20	5.13
5	2:05 PM	2:15 PM	10	74.40	34.80	39.60	5.32
6	2:15 PM	2:25 PM	10	69.60	28.80	40.80	6.07
7	2:25 PM	2:35 PM	10	60.00	18.00	42.00	7.83
8	2:35 PM	2:45 PM	10	36.00	9.60	26.40	8.23
9	2:45 PM	2:55 PM	10	36.00	4.20	31.80	11.17

<b>Raw Infiltration Rate (no factor of safety) =</b>	<b>7.83</b>	<b>inch /hr</b>
<b>Factor of Safety =</b>	<b>2.00</b>	
<b>Design Infiltration Rate =</b>	<b>3.91</b>	<b>inch/hr</b>

Reference: Appendix VII. Infiltration Rate Evaluation Protocol and Factor of Safety Recommendations dated 05/19/2011

## Infiltration Rate Calculation Sheet

Project :	Kaiser Redlands	Project No. :	220729.3	Date :	3/8/2023
Test Hole No.:	P-7	Tested by :	AB/CD		
Depth of Test Hole, $D_T$ (in):	60	USCS Soil Classification :	SM		
Test Hole Dimension (inches)			Length	Width	
Diameter (if round) (inches) =	5.00	Sides (if rectangular) =			

### Sandy Soil Criteria Test\*

Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6" ? (Y/N)
1	1:28 PM	1:48 PM	20	27.6	39.4	11.8	Y
2	1:50 PM	2:10 PM	20	28.8	39.4	10.6	Y

\*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	Stop Time	$\Delta t$ Time Interval (min.)	$H_o$ Initial Water Height (inches)	$H_f$ Final Water Height (inches)	$\Delta H$ Change in Water Level (inches)	Tested Infiltration Rate
1	2:40 PM	2:50 PM	10	33.60	28.80	4.80	1.11
2	2:52 PM	3:02 PM	10	33.00	30.60	2.40	0.54
3	3:02 PM	3:12 PM	10	34.56	30.60	3.96	0.88
4	3:13 PM	3:23 PM	10	34.80	30.60	4.20	0.93
5	3:23 PM	3:33 PM	10	38.40	34.20	4.20	0.84
6	3:33 PM	3:43 PM	10	38.40	34.20	4.20	0.84

**Raw Infiltration Rate (no factor of safety) = 0.84 inch /hr**  
**Factor of Safety = 2.00**  
**Design Infiltration Rate = 0.42 inch/hr**

Reference: Appendix VII. Infiltration Rate Evaluation Protocol and Factor of Safety Recommendations dated 05/19/2011

## Infiltration Rate Calculation Sheet

Project :	Kaiser Redlands	Project No. :	220729.3	Date :	3/8/2023
Test Hole No.:	P-8	Tested by :	AB/CD		
Depth of Test Hole, $D_T$ (in):	78	USCS Soil Classification :	SM		
Test Hole Dimension (inches)			Length	Width	
Diameter (if round) (inches) =	5.00	Sides (if rectangular) =			

**Sandy Soil Criteria Test\***

Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6" ? (Y/N)
1	8:48 AM	9:03 AM	15	24.0	61.5	37.5	Y
2	9:05 AM	9:20 AM	15	22.5	59.1	36.6	Y

\*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	Stop Time	$\Delta t$	$H_o$	$H_f$	$\Delta H$	
Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Water Height (inches)	Final Water Height (inches)	Change in Water Level (inches)	Tested Infiltration Rate
1	9:35 AM	9:45 AM	10	52.20	24.00	28.20	5.37
2	9:45 AM	9:55 AM	10	51.60	27.60	24.00	4.41
3	9:56 AM	10:06 AM	10	54.60	26.64	27.96	5.01
4	10:07 AM	10:17 AM	10	52.80	26.40	26.40	4.85
5	10:18 AM	10:28 AM	10	54.00	27.60	26.40	4.71
6	10:29 AM	10:39 AM	10	54.00	27.60	26.40	4.71
7	10:39 AM	10:49 AM	10	54.24	27.84	26.40	4.68
8	10:56 AM	11:06 AM	10	58.80	32.40	26.40	4.23
9	11:06 AM	11:16 AM	10	54.00	27.84	26.16	4.65

<b>Raw Infiltration Rate (no factor of safety) =</b>	<b>4.23</b>	<b>inch /hr</b>
<b>Factor of Safety =</b>	<b>2.00</b>	
<b>Design Infiltration Rate =</b>	<b>2.11</b>	<b>inch/hr</b>

Reference: Appendix VII. Infiltration Rate Evaluation Protocol and Factor of Safety Recommendations dated 05/19/2011

Infiltration Rate Calculation Sheet							
Project :	Kaiser Redlands		Project No. :	220729.3		Date :	3/8/2023
Test Hole No.:	P-9		Tested by :	AB			
Depth of Test Hole, $D_T$ (in):	26		USCS Soil Classification :	SM			
Test Hole Dimension (inches)				Length	Width		
Diameter (if round) (inches) =			Sides (if rectangular) =	48	24		
Sandy Soil Criteria Test*							
Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Height of Water (in.)	Final Height of Water (in.)	Change in Water Level (in.)	
1	10:10 AM	10:20 AM	10	12.00	10.50	1.50	
2	10:20 AM	10:30 AM	10	10.50	9.50	1.00	
Trial No.	Start Time	Stop Time	$\Delta t$ Time Interval (min.)	$H_o$ Initial Water Height (inches)	$H_f$ Final Water Height (inches)	$\Delta H$ Change in Water Level (inches)	Tested Infiltration Rate
1	10:31 AM	10:41 AM	10	12.00	11.00	1.00	2.46
2	10:41 AM	10:51 AM	10	11.00	10.00	1.00	2.59
3	10:51 AM	11:01 AM	10	10.00	9.25	0.75	2.04
4	11:01 AM	11:11 AM	10	9.25	8.50	0.75	2.13
5	11:11 AM	11:21 AM	10	8.50	8.00	0.50	1.48
6	11:21 AM	11:31 AM	10	8.00	6.88	1.13	3.50
7	11:49 AM	11:59 AM	10	12.00	11.25	0.75	1.83
8	11:59 AM	12:09 PM	10	11.25	10.25	1.00	2.56
9	12:09 PM	12:19 PM	10	10.25	9.75	0.50	1.33
10	12:19 PM	12:29 PM	10	9.75	9.00	0.75	2.07
11	12:29 PM	12:39 PM	10	9.00	8.25	0.75	2.17
12	12:39 PM	12:49 PM	10	8.25	7.50	0.75	2.27
13	12:56 PM	1:06 PM	10	12.00	11.13	0.88	2.15
14	1:06 PM	1:16 PM	10	11.13	10.13	1.00	2.58
15	1:16 PM	1:26 PM	10	10.13	9.63	0.50	1.34
16	1:26 PM	1:36 PM	10	9.63	9.00	0.63	1.73
17	1:36 PM	1:46 PM	10	9.00	8.38	0.63	1.80
18	1:46 PM	1:56 PM	10	8.38	7.75	0.63	1.87
<b>Raw Infiltration Rate (no factor of safety) =</b>						<b>1.73</b>	<b>inch /hr</b>
<b>Factor of Safety =</b>						<b>2.00</b>	
<b>Design Infiltration Rate =</b>						<b>0.87</b>	<b>inch/hr</b>

Reference: Appendix VII. Infiltration Rate Evaluation Protocol and Factor of Safety Recommendations dated 05/19/2011

## DOUBLE-RING INFILTROMETER RESULTS

**Project No.:** 220729.3  
**Project Name:** Kaiser Redlands  
**Date:** Tuesday, March 7, 2023  
**Engineers:** PS  
**Boring No.:** DR-1

	<b>Inner</b>	<b>Outer</b>	
<b>Ring Diameter:</b>	<b>29.21</b>	<b>58.42</b>	<b>cm</b>
<b>Ring Area:</b>	<b>670.12</b>	<b>2,680.48</b>	<b>cm<sup>2</sup></b>
<b>Ring Area:</b>	<b>103.86</b>	<b>415.45</b>	<b>in<sup>2</sup></b>

Water source: Small Mariotte Tube - 53.53 cc per cm division

Water source: Large Mariotte Tube - 167.53 cc per cm division

Trial No.		Time (hr:min)	Elapsed (min)	Inner Reading (cm)	Outer Reading (cm)	Inner Flow (cm <sup>3</sup> )	Outer Flow (cm <sup>3</sup> )	Inner Flow (in <sup>3</sup> )	Outer Flow (in <sup>3</sup> )	Inner Infiltration Rate (in/hr)	Outer Infiltration Rate (in/hr)
1	Start	1:05	10	21.7	48.7	401.4	1223.0	24.5	74.6	1.415	1.437
	End	1:15		14.2	41.4						
2	Start	1:15	5	14.2	41.4	208.7	787.4	12.7	48.0	1.472	1.851
	End	1:20		10.3	36.7						
3	Start	1:20	5	10.3	36.7	235.5	603.1	14.4	36.8	1.660	1.417
	End	1:25		5.9	33.1						
4	Start	1:25	5	5.9	33.1	198.0	686.9	12.1	41.9	1.396	1.614
	End	1:30		2.2	29						
5	Start	1:35	5	29.5	25	214.1	569.6	13.1	34.8	1.509	1.339
	End	1:40		25.5	21.6						
6	Start	1:40	5	25.5	21.6	208.7	636.6	12.7	38.8	1.472	1.496
	End	1:45		21.6	17.8						
7	Start	1:45	5	21.6	17.8	406.8	1239.7	24.8	75.7	2.868	2.914
	End	1:55		14	10.4						
8	Start	1:55	5	14	10.4	214.1	686.9	13.1	41.9	1.509	1.614
	End	2:00		10	6.3						
9	Start	2:12	5	44	52.7	187.3	753.9	11.4	46.0	1.321	1.772
	End	2:17		40.5	48.2						
10	Start	2:17	5	40.5	48.2	326.5	938.2	19.9	57.3	2.302	2.205
	End	2:22		34.4	42.6						
11	Start	2:22	5	34.4	42.6	192.7	619.9	11.8	37.8	1.358	1.457
	End	2:27		30.8	38.9						
12	Start	2:27	5	30.8	38.9	214.1	670.1	13.1	40.9	1.509	1.575
	End	2:32		26.8	34.9						
13	Start	2:32	5	26.8	34.9	214.1	703.6	13.1	42.9	1.509	1.654
	End	2:37		22.8	30.7						
14	Start	2:37	5	22.8	30.7	171.3	603.1	10.5	36.8	1.208	1.417
	End	2:42		19.6	27.1						
15	Start	2:42	5	19.6	27.1	262.2	753.9	16.0	46.0	1.849	1.772
	End	2:47		14.7	22.6						
16	Start	2:47	5	14.7	22.6	203.4	720.4	12.4	44.0	1.434	1.693
	End	2:52		10.9	18.3						
17	Start	2:52	5	10.9	18.3	192.7	653.4	11.8	39.9	1.358	1.536
	End	2:57		7.3	14.4						
18	Start	2:52	5	7.3	14.4	214.1	737.1	13.1	45.0	1.509	1.732
	End	2:57		3.3	10						

**Raw Infiltration Rate (no factor of safety) = 1.358**  
**Factor of Safety = 2**  
**Design Infiltration Rate = 0.68**



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# APPENDIX D GEOVISION RESULTS



## **REPORT**

### **SURFACE WAVE MEASUREMENTS**

#### **KAISER REDLANDS MEDICAL CENTER REDLANDS, CALIFORNIA**

**GEOVision** Project No. 22462

*Prepared for*

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Report 22462-01 Rev 0

December 20, 2022

# TABLE OF CONTENTS

<b>1</b>	<b>INTRODUCTION</b> .....	<b>1</b>
<b>2</b>	<b>OVERVIEW OF SURFACE WAVE TECHNIQUES</b> .....	<b>3</b>
2.1	INTRODUCTION .....	3
2.2	SURFACE WAVE TECHNIQUES .....	3
2.2.1	MASW Technique .....	3
2.2.2	Array Microtremor Technique .....	4
2.3	SURFACE WAVE DISPERSION CURVE MODELING .....	5
<b>3</b>	<b>FIELD PROCEDURES</b> .....	<b>7</b>
<b>4</b>	<b>DATA REDUCTION</b> .....	<b>8</b>
<b>5</b>	<b>DATA MODELING</b> .....	<b>10</b>
<b>6</b>	<b>INTERPRETATION AND RESULTS</b> .....	<b>11</b>
6.1	ARRAYS 1 AND 2 .....	11
6.2	ARRAYS 3 AND 4 .....	13
<b>7</b>	<b>REFERENCES</b> .....	<b>15</b>
<b>8</b>	<b>CERTIFICATION</b> .....	<b>18</b>

## LIST OF TABLES

<b>TABLE 1</b>	<b>LOCATION OF SURFACE WAVE ARRAYS</b> .....	<b>7</b>
<b>TABLE 2</b>	<b>ARRAYS 1 AND 2 <math>V_s</math> MODEL</b> .....	<b>11</b>
<b>TABLE 3</b>	<b>ARRAYS 3 AND 4 <math>V_s</math> MODEL</b> .....	<b>13</b>

## LIST OF FIGURES

<b>FIGURE 1</b>	<b>SITE MAP</b> .....	<b>2</b>
<b>FIGURE 2</b>	<b>SURFACE WAVE MODEL – ARRAYS 1 AND 2</b> .....	<b>12</b>
<b>FIGURE 3</b>	<b>SURFACE WAVE MODEL – ARRAYS 3 AND 4</b> .....	<b>14</b>

# 1 INTRODUCTION

In-situ seismic measurements using an active-source surface wave technique were performed at the Kaiser Redlands Medical Center in Redlands, California on December 5, 2022. The purpose of this investigation was to provide a shear (S) wave velocity profile to a depth of 100 ft, or greater, and estimate the average S-wave velocity of the upper 100 ft ( $V_{S100ft}$ ) at two locations at the site. The active-source surface wave technique utilized during this investigation consisted of the multi-channel analysis of surface waves (MASW) method. The passive-source surface wave technique consisted of the array microtremor method. The locations of the active- and passive-source surface wave arrays are shown on Figure 1. MASW measurements were made along two linear arrays (Arrays 1 and 3) and array microtremor measurements were made using a T-shaped array (Array 2) and L-shaped array (Array 4).

For seismic design, the 2019 California Building Code (CBC) and 2018 International Building Code (IBC) reference the provisions in ASCE/SEI 7-16 (Minimum Design Loads and Associated Criteria for Buildings and Other Structures). The Site Classes and associated S-wave velocity ranges outlined in Table 20.3-1 of ASCE/SEI 7-16 are as follows:

Site Class A – Hard rock –  $V_{S100ft} > 5,000$  ft/s

Site Class B – Rock –  $2,500 < V_{S100ft} \leq 5,000$  ft/s

Site Class C – Very dense soil and soft rock –  $1,200 < V_{S100ft} \leq 2,500$  ft/s

Site Class D – Stiff soil –  $600 < V_{S100ft} \leq 1,200$  ft/s

Site Class E – Soft clay soil –  $V_{S100ft} < 600$  ft/s

Site Class F – Soils requiring site response analysis

At many sites, active-source surface wave techniques with the utilization of portable energy sources, such as hammers and weight drops, are sufficient to obtain S-wave velocity sounding to 100 ft depth. However, at sites with high ambient noise levels and/or very soft soils, these energy sources may not be sufficient to image to 100 ft depth and a larger energy source, such as a bulldozer, is necessary. Alternatively, passive-source surface wave techniques, such as the array microtremor method, can be used to extend the depth of investigation providing the site has sufficient ambient noise. Two-dimensional passive-source surface wave arrays (e.g., triangular, circular, or L-shaped arrays) are expected to perform significantly better than linear arrays.

This report provides the results of the active and passive surface wave measurements conducted at the site. An overview of the surface wave methods is given in Section 2. Field and data reduction procedures are discussed in Sections 3 and 4, respectively. Data modeling is presented in Section 5 and interpretation and results are presented in Section 6. References and our professional certification are presented in Sections 7 and 8, respectively.



- Active Surface Wave Array (MASW)
- Passive Surface Wave Array (L-shaped)
- Passive Surface Wave Array (T-shaped)

Notes:  
 Coordinate System: NAD 1983 StatePlane California V FIPS 0405 Feet  
 Base map source: San Bernardino County, Maxar, Microsoft

<b>GE</b> <i>Vision</i> geophysical services	
Date:	12/19/2022
GV Project:	22462
Developed by:	A Martin
Drawn by:	T Rodriguez
Approved by:	A Martin
File Name:	GV-22462

<b>FIGURE 1</b> <b>SITE MAP</b>
<b>KAISER REDLANDS MEDICAL CENTER</b> <b>REDLANDS, CALIFORNIA</b>
<b>PREPARED FOR</b> <b>TWINING CONSULTING, INC.</b>

## 2 OVERVIEW OF SURFACE WAVE TECHNIQUES

### 2.1 Introduction

Active- and passive-source (ambient vibration) surface wave techniques are routinely utilized for site characterization. Active-source surface wave techniques include the spectral analysis of surface waves (SASW) and multi-channel array surface wave (MASW) methods. Passive-source surface wave techniques include the horizontal over vertical spectral ratio (HVSr) method, and the array and refraction microtremor methods.

The basis of surface wave methods is the dispersive characteristic of Rayleigh and Love waves when propagating in a layered medium. Surface waves of different wavelengths ( $\lambda$ ) or frequencies ( $f$ ) sample different depths. As a result of the variance in the shear stiffness of the distinct layers, waves with different wavelengths propagate at different phase velocities; hence, dispersion. A surface wave dispersion curve is the variation of  $V_R$  or  $V_L$  with  $\lambda$  or  $f$ . The Rayleigh wave phase velocity ( $V_R$ ) depends primarily on the material properties ( $V_S$ , mass density, and Poisson's ratio or compression wave velocity) over a depth of approximately one wavelength. The Love wave phase velocity ( $V_L$ ) depends primarily on  $V_S$  and mass density. Rayleigh and Love wave propagation are also affected by damping or seismic quality factor ( $Q$ ). Rayleigh wave techniques are utilized to measure vertically polarized S-waves ( $S_V$ -wave); whereas Love wave techniques are utilized to measure horizontally polarized S-waves ( $S_H$ -wave).

### 2.2 Surface Wave Techniques

The MASW and array microtremor techniques were utilized during this investigation and are discussed below.

#### 2.2.1 MASW Technique

A description of the MASW method is given by Park, 1999a and 1999b and Foti, 2000. Ground motions are typically recorded by 24, or more, geophones typically spaced 1 to 3 m apart along a linear array and connected to a seismograph. Energy sources for shallow investigations include various sized hammers and vehicle mounted weight drops. When applying the MASW technique to develop a one-dimensional (1-D)  $V_S$  model, it is preferable to use multiple-source offsets from both ends of the array. The most routinely applied MASW technique is the Rayleigh-wave based MASW method, which we refer to as  $MAS_RW$  to distinguish from Love-wave based MASW ( $MAS_LW$ ).  $MAS_RW$  and  $MAS_LW$  acquisition can easily be combined with P- and S-wave seismic refraction acquisition, respectively.  $MAS_RW$  data are generally recorded using a vertical source and vertical geophone but may also be recorded using a horizontal geophone with radial (in-line) orientation.  $MAS_LW$  data are recorded using transversely orientated horizontal source and transverse horizontal geophone.

A wavefield transform is applied to the time-history data to convert the seismic record from time-offset space to frequency-wavenumber ( $f$ - $k$ ) space in which the fundamental or higher surface-wave modes can be easily identified as energy maxima and picked. Frequency and/or wavenumber can easily be mapped to phase velocity, slowness, or wavelength using the following properties:  $k = 2\pi/\lambda$ ,  $\lambda = v/f$ . Common wave-field transforms include: the  $f$ - $k$  transform (a 2D fast Fourier transform), slant-stack transform (also referred to as intercept-

slowness or  $\tau$ -p transform and equivalent to linear Radon transform), frequency domain beamformer, and phase-shift transform. The minimum wavelength that can be recovered from MASW data set without spatial aliasing is equal to the minimum receiver spacing. Occasionally, SASW analysis procedures are used to extract surface wave dispersion data, from fixed receiver pairs, at smaller wavelengths than can be recovered by wavefield transformation. Construction of a dispersion curve over the wide frequency/wavelength range necessary to develop a robust  $V_S$  model while also limiting the maximum wavelength based on an established near-field criterion (e.g., Yoon and Rix, 2009; Li and Rosenblad, 2011), generally requires multiple source offsets.

Although the clear majority of MASW surveys record Rayleigh waves, it has been shown that Love wave techniques can be more effective in some environments, particularly shallow rock sites and sites with a highly attenuative, low velocity surface layer (Xia, et al., 2012; *GEOVision*, 2012; Yong, et al., 2013; Martin, et al., 2014). Rayleigh wave techniques, however, are generally more effective at sites where velocity gradually increases with depth because larger energy sources are readily available for the generation of Rayleigh waves. Rayleigh wave techniques are also more applicable to sites with high velocity layers and/or velocity inversions because the presence of such structures is more apparent in the Rayleigh wave dispersion curves than in Love wave dispersion curves. Rayleigh wave techniques are preferable at sites with a high velocity surface layer because Love waves do not theoretically exist in such environments. Occasionally, the horizontal radial component of a Rayleigh wave may yield higher quality dispersion data than the vertical component because different modes of propagation may have more energy in one component than the other. Recording both the vertical and horizontal components of the Rayleigh wave is particularly useful at sites with complex modes of propagation or when attempting to recover multiple Rayleigh wave modes for multi-mode modeling as demonstrated in Dal Moro, et al, 2015. Joint inversion of Rayleigh and Love wave data may yield more accurate  $V_S$  models and also offers a means to investigate anisotropy, where  $S_V$ - and  $S_H$ -wave velocity are not equal, as shown in Dal Moro and Ferigo, 2011.

### **2.2.2 Array Microtremor Technique**

A detailed discussion of the array microtremor method can be found in Okada, 2003. Unlike active source techniques which use an active energy source (e.g., hammer), the array microtremor technique (also referred to as passive surface wave or array ambient vibration method) records background noise (ambient vibrations) emanating from ocean wave activity, wind noise, traffic, industrial activity, construction, etc. The technique uses 4, or more, receivers aligned in a 2-dimensional array. Triangle, circle, semi-circle, and “L” shaped arrays are commonly used, although any 2-dimensional arrangement of receivers can be used. For investigations of the upper 100 m, receivers typically consist of 1 to 4.5 Hz geophones. For deeper investigations, 5 to 120 s seismometers are generally utilized. The nested triangle array, which consists of several embedded equilateral triangles, is popular as it provides accurate dispersion curves with a relatively small number of geophones. The “L” array is useful at sites located at the corner of intersecting streets. The maximum receiver separation in an array should be at a minimum equal to the desired depth of investigation. Typically, 15 to 60 minutes of ambient vibration data is recorded depending on the size of the array, desired depth of investigation, and noise conditions. Investigations to depths on the order of 1 km may require that ambient vibrations are recorded for a much longer duration. The surface wave dispersion curve is typically estimated from array microtremor data using various f-k methods such as

beamforming (Lacoss, et al., 1969), and maximum-likelihood (Capon, 1969), and the spatial-autocorrelation (SPAC) method. The beam-forming and maximum-likelihood methods are generally referred to as the frequency wavenumber (FK) and high-resolution frequency wavenumber (HRFK or HFK) methods. The SPAC method was originally based on work by Aki, 1957 and has since been extended and modified (Ling and Okada, 1993 and Ohori *et al.*, 2002) to permit the use of noncircular arrays, and is now collectively referred to as extended spatial autocorrelation (ESPAC or ESAC). Further modifications to the SPAC method permit the use of irregular or random arrays (Bettig *et al.*, 2001). Although it is common to apply SPAC methods to obtain a surface wave dispersion curve for modeling, other approaches involve direct modeling of the coherency data, also referred to as SPAC coefficients (Asten, 2006 and Asten, *et al.*, 2015). The beam-forming and maximum-likelihood methods are generally referred to as the frequency wavenumber (FK) and high-resolution frequency wavenumber (HRFK or HFK) methods, respectively. More recently, a Rayleigh wave three-component beamforming method (RTBF) has been developed (Wathelet, et al., 2018) and appears to offer significant resolution enhancements over other methods.

FK, HRFK and RTBF methods are generally expected to perform better when ambient vibration sources are not azimuthally well-distributed (e.g., rural area where the primary noise source is a large industrial facility). SPAC methods are expected to perform better when noise sources are azimuthally well-distributed (e.g., in a large, urbanized area).

The minimum wavelength surface wave that can be extracted from an array microtremor dataset acquired utilizing a symmetric array is typically set equal to the minimum receiver spacing. The maximum wavelength is often set equal to twice the maximum receiver separation for SPAC analysis and the maximum receiver spacing for FK analysis.

### **2.3 Surface Wave Dispersion Curve Modeling**

The dispersion curves generated from the active and passive surface wave soundings are generally combined and modeled using iterative forward and inverse modeling routines. The final model profile is assumed to represent actual site conditions. The theoretical model used to interpret the dispersion curve assumes horizontally layered, laterally invariant, homogeneous-isotropic material. Although these conditions are seldom strictly met at a site, the results of active and/or passive surface wave testing provide a good “global” estimate of the material properties along the array. The results may be more representative of the site than a borehole “point” estimate.

The surface wave forward problem is typically solved using the Thomson-Haskell transfer-matrix (Thomson, 1950; Haskell, 1953) later modified by Dunkin (1965) and Knopoff (1964), dynamic stiffness matrix (Kausel and Roësset, 1981), or reflection and transmission coefficient (Kennett, 1974) methods. All of these methods can determine fundamental- and higher-mode phase velocities, which correspond to plane waves in 2-D space. The transfer-matrix method is often used in MASW and passive surface-wave software packages, whereas the dynamic stiffness matrix is utilized in many SASW software packages. MAS<sub>R</sub>W and/or passive surface-wave modeling may involve modeling of the fundamental mode, some form of effective mode, or multiple individual modes (multi-mode). As outlined in Roësset et al. (1991), several options exist for forward modeling of Rayleigh wave SASW data. One formulation considers only fundamental mode plane Rayleigh-wave motion (called the 2-D solution), whereas another

includes all stress waves (e.g., body, fundamental, and higher mode surface waves) and incorporates a generalized receiver geometry (3-D global solution) or actual receiver geometry (3-D array solution).

The fundamental mode assumption is generally applicable to modeling Rayleigh-wave dispersion data collected at normally dispersive sites, providing there are not abrupt increases in velocity or steep velocity gradients. Effective-mode or multi-mode approaches are often required for irregularly dispersive sites and sites with steep velocity gradients at shallow depth. If active and passive surface wave data are combined or MAS<sub>R</sub>W data are combined from multiple seismic records with different source offsets and receiver gathers, then effective-mode computations are limited to algorithms that assume far-field plane Rayleigh wave propagation. Local search (e.g., linearized matrix inversion methods) or global search methods (e.g., Monte Carlo approaches such as simulated annealing, generic algorithm, and neighborhood algorithm) are typically used to solve the inverse problem.

The maximum wavelength ( $\lambda_{\max}$ ) recovered from a surface wave data set is typically used to estimate depth of investigation although a sensitivity analysis of the  $V_S$  models would be a more robust means to estimate depth of investigation. For normally dispersive velocity profiles with a gradual increase in  $V_S$  with depth, the maximum depth of investigation is on the order of  $\lambda_{\max}/2$  for both Rayleigh and Love wave dispersion data. For velocity profiles with an abrupt increase in  $V_S$  at depth, the maximum depth of investigation is on the order of  $\lambda_{\max}/3$  for Rayleigh wave dispersion data but less than  $\lambda_{\max}/3$  for Love wave dispersion data. The depth of investigation can be highly variable for sites with complex velocity structure (e.g., high velocity layers).

As with all other surface geophysical methods, the inversion of surface wave dispersion data does not yield a unique  $V_S$  model and multiple possible solutions may equally fit the experimental data. Based on experience at other sites, the shear wave velocity models ( $V_S$  and layer thicknesses) determined by surface wave testing are within 20% of the velocities and layer thicknesses that would be determined by other seismic methods (Brown, 1998). The average velocity of the upper 30 m, however, is much more accurate, often to better than 5%, because it is not sensitive to the layering in the model.  $V_{S30}$  does not appear to suffer from the non-uniqueness inherent in  $V_S$  models derived from surface wave dispersion curves (Martin et al., 2006, Comina et al., 2011). Therefore,  $V_{S30}$  is more accurately estimated from the inversion of surface wave dispersion data than the resulting  $V_S$  models.

It may not always be possible to develop a coherent, fundamental mode dispersion curve over sufficient frequency range for modeling due to dominant higher modes with the higher modes not clearly identifiable for multi-mode modeling. It may, however, be possible to identify the Rayleigh wave phase velocity of the fundamental mode at 40 m wavelength ( $V_{R40}$ ) in which case  $V_{S30}$  can at least be estimated using the Brown et al., 2000 relationship:

$$V_{S30} = 1.045V_{R40}$$

This relationship was established based on a statistical analysis of many of surface wave data sets from sites with borehole velocity control and has been further evaluated by Martin and Diehl, 2004, and Albarello and Gargani, 2010. Further investigation of this approach has revealed that  $V_{S30}$  is generally between  $V_{R40}$  and  $V_{R45}$  with  $V_{R40}$  often being most appropriate for shallow groundwater sites and  $V_{R45}$  for deep ground water sites.

### 3 FIELD PROCEDURES

The active- and passive-source surface wave sounding locations at the site are shown in Figure 1 with array locations presented in Table 1. Two types of surface wave data were acquired at the site: an active-source surface wave array to characterize near-surface velocity structure and a passive-source surface wave array to characterize deeper velocity structure. Active-source surface wave data were acquired along Arrays 1 and 3 using the MASW technique. Passive-source surface wave data were acquired along Arrays 2 and 4 using the array microtremor method. The surface wave arrays were surveyed using a Trimble R10 GPS system with the RTX differential correction service.

**Table 1 Location of Surface Wave Arrays**

Location	Latitude	Longitude
Array 1 MASW, South End of Array	34.07112	-117.22939
Array 1 MASW, North End of Array	34.07209	-117.22938
Array 2 Passive, South End of Array	34.07110	-117.22939
Array 2 Passive, Northwest End of Array	34.07210	-117.22993
Array 2 Passive, Intersection of Legs of Array	34.07209	-117.22938
Array 2 Passive, Northeast End of Array	34.07209	-117.22879
Array 3 MASW, West End of Array	34.07299	-117.22820
Array 3 MASW, West End of Array	34.07299	-117.22704
Array 4 Passive, Northwest End of Array	34.07381	-117.22820
Array 4 Passive, Corner of Array	34.07299	-117.22820
Array 4 Passive, Southeast End of Array	34.07299	-117.22686

Notes: 1) WGS84 Coordinate System (decimal degrees)

MASW equipment used during this investigation consisted of Geometrics Geode signal enhancement seismographs, 4.5 Hz vertical geophones, seismic cables, a 4 lb hammer, 10 lb sledgehammer, and 240 lb accelerated weight drop (AWD). MASW data were acquired along a linear array of 48 geophones spaced 7.5 ft apart for a length of 352.5 ft (Arrays 1 and 3). Source locations were located between 7.5 and 97.5 ft from the end geophone locations and at 60 ft intervals in the interior of the array. The AWD was used as the energy source for all off-end source locations. The 4 and 10 lb hammers were utilized at the near offset source locations and all interior source locations. Data from the transient impacts (hammers) were generally averaged 5 to 10 times to improve the signal-to-noise ratio. All field data were saved to hard disk and documented on field data acquisition forms.

Array microtremor equipment consisted of Geometrics Geode signal enhancement seismographs, 4.5 Hz vertical geophones, and seismic cables. Array 2 is a T-shaped array with a 15 ft geophone spacing and leg lengths of 345 and 360 ft. Array 4 is a L-shaped array with a 15 ft geophone spacing and leg lengths of 300 and 405 ft. Ambient noise measurements were made on each passive array for about 30 to 45 minutes with a 2-millisecond sample rate. Array microtremor data were downloaded to a laptop computer for later processing. The field geometry and associated files names were documented in field notes.

## 4 DATA REDUCTION

The MASW data were reduced using the software Seismic Pro Surface V9 developed by Geogiga and multiple in-house scripts for various data extraction and formatting tasks, with all data reduction documented in a Microsoft Excel spreadsheet.

The following steps were used for data reduction:

- Input seismic records to be used for analysis into software package.
- Check and correct source and receiver geometry as necessary.
- Select offset range used for analysis (multiple offset ranges utilized for each seismic record as discussed below) and document in spreadsheet.
- Apply phase shift transform to seismic record to convert the data from time – offset to frequency – phase velocity space.
- Identify, pick, save, and document dispersion curve.
- Change the receiver offset range and repeat process.
- Repeat process for all seismic records.
- Use in-house script to apply near-field criteria with maximum wavelength set equal to 1.0 times the source to midpoint of receiver array distance.
- Use in-house script to merge multiple dispersion curves extracted from the MASW data collected along each seismic line for a specific source type (different source locations, different receiver offset ranges, etc.).
- Edit dispersion data, as necessary (e.g., delete poor quality curves and outliers).
- Calculate a representative dispersion curve at equal log-frequency or log-wavelength spacing for the MASW dispersion data using a moving average, polynomial curve fitting routine.

This unique data reduction strategy, which can involve combination of over 50 dispersion curves for a 1-D sounding, is designed for characterizing sites with complex velocity structure that do not yield surface wave dispersion data over a wide frequency range from a single source type or source location. The data reduction strategy ensures that the dispersion curve selected for modeling is representative of average conditions beneath the array and spans as broad a frequency/wavelength range as possible while considering near field effects.

The array microtremor data were reduced using the SeisImager software package developed by Oyo Corporation/Geometrics, Inc., and the following steps:

The processing sequence for implementation of the ESAC method in the SeisImager software package is as follows:

- Input seismic record(s) for a dataset into software.
- Apply time-segmentation routine to break the data file into multiple 30 to 60 second seismic records, as necessary.
- Load receiver geometry (x and y positions) for each channel in seismic record.
- Calculate the SPAC coefficients for each seismic record and average.
- Optionally, select a subset of receiver offset ranges for analysis (e.g., only select receiver pairs with multiple azimuths).

- For each frequency calculate the RMS error between the SPAC coefficients and a Bessel function of the first kind and order zero over a user defined phase velocity range and velocity step.
- Plot an image of RMS error as a function for frequency (f) and phase velocity (v).
- Identify and pick the dispersion curve as the continuous trend on the f-v image with the lowest RMS error.
- Repeat the process for all arrays and time blocks.
- Use an in-house script to convert dispersion curves to appropriate format for editing.
- Edit dispersion data, as necessary, and use in-house script to combine all dispersion data after setting maximum wavelength to about 2 times the maximum receiver spacing.
- Calculate a representative dispersion curve for the passive dispersion data from each array using a moving average polynomial curve fitting routine.

The representative dispersion curves from the active and passive surface wave data were combined and the moving average polynomial curve fitting routine in WinSASW V3 was used to generate a composite representative dispersion curve for modeling. During this process, the active and passive surface wave dispersion data were given equal weights. An equal logarithm wavelength sample rate was used for the representative dispersion curve to reflect the gradual loss in model resolution with depth.

## 5 DATA MODELING

Surface wave data were modeled using the fundamental mode Rayleigh wave routine in the WinSASW V3 software package. During this process, an initial velocity model was generated based on general characteristics of the dispersion curve and the inverse modeling routine utilized to adjust the layer  $V_S$  until an acceptable agreement with the observed data was obtained. Layer thicknesses were adjusted, and the inversion process repeated until a  $V_S$  model was developed with low RMS error between the observed and calculated dispersion curves. In many cases, once an acceptable  $V_S$  model is developed, layer thicknesses are adjusted, and the inversion process repeated to develop an ensemble of  $V_S$  models with similar RMS error to quantify non-uniqueness. Because the primary purpose of this investigation was to estimate  $V_{S100ft}$ , it was not considered necessary to develop multiple  $V_S$  models. Data inputs into the modeling software include layer thickness, S-wave velocity, P-wave velocity or Poisson's ratio, and mass density. P-wave velocity and mass density only have a very small influence (i.e., less than 10%) on the S-wave velocity model generated from a surface wave dispersion curve. However, realistic assumptions for P-wave velocity, which is significantly impacted by the location of the saturated zone, and mass density will slightly improve the accuracy of the S-wave velocity model.

Constant mass density values of 114 to 127 lb/ft<sup>3</sup> were used in the velocity profiles for subsurface soils/rock depending on P- and S-wave velocity. Within the normal range encountered in geotechnical engineering, variation in mass density has a negligible ( $\pm 2\%$ ) effect on the estimated  $V_S$  from surface wave dispersion data. During modeling of Rayleigh wave dispersion data, the compression wave velocity,  $V_P$ , for unsaturated sediments was estimated using a Poisson's ratio,  $\nu$ , of 0.3 and the relationship:

$$V_P = V_S [(2(1-\nu))/(1-2\nu)]^{0.5}$$

Poisson's ratio has a larger effect than density on the estimated  $V_S$  from Rayleigh wave dispersion data. Achenbach (1973) provides approximate relationship between Rayleigh wave velocity ( $V_R$ ),  $V_S$  and  $\nu$ :

$$V_R = V_S [(0.862 + 1.14 \nu)/(1 + \nu)]$$

Using this relationship, it can be shown that  $V_S$  derived from  $V_R$  only varies by about 10% over possible 0 to 0.5 range for Poisson's ratio where:

$$\begin{aligned} V_S &= 1.16V_R \text{ for } \nu = 0 \\ V_S &= 1.05V_R \text{ for } \nu = 0.5 \end{aligned}$$

The common range of Poisson's ratio for unsaturated sediments is about 0.25 to 0.35. Over this range,  $V_S$  derived from modeling of Rayleigh wave dispersion data will vary by about 5%. There is no evidence of the saturated zone in the upper 100 ft in the MASW seismic records. A possible water table refractor in the seismic records indicates that the saturated zone may be about 125 feet deep. Therefore, for the purpose of modeling, high Poisson's ratio saturated sediments were assumed to be present in model layers below a depth of 125 ft.

## 6 INTERPRETATION AND RESULTS

### 6.1 Arrays 1 and 2

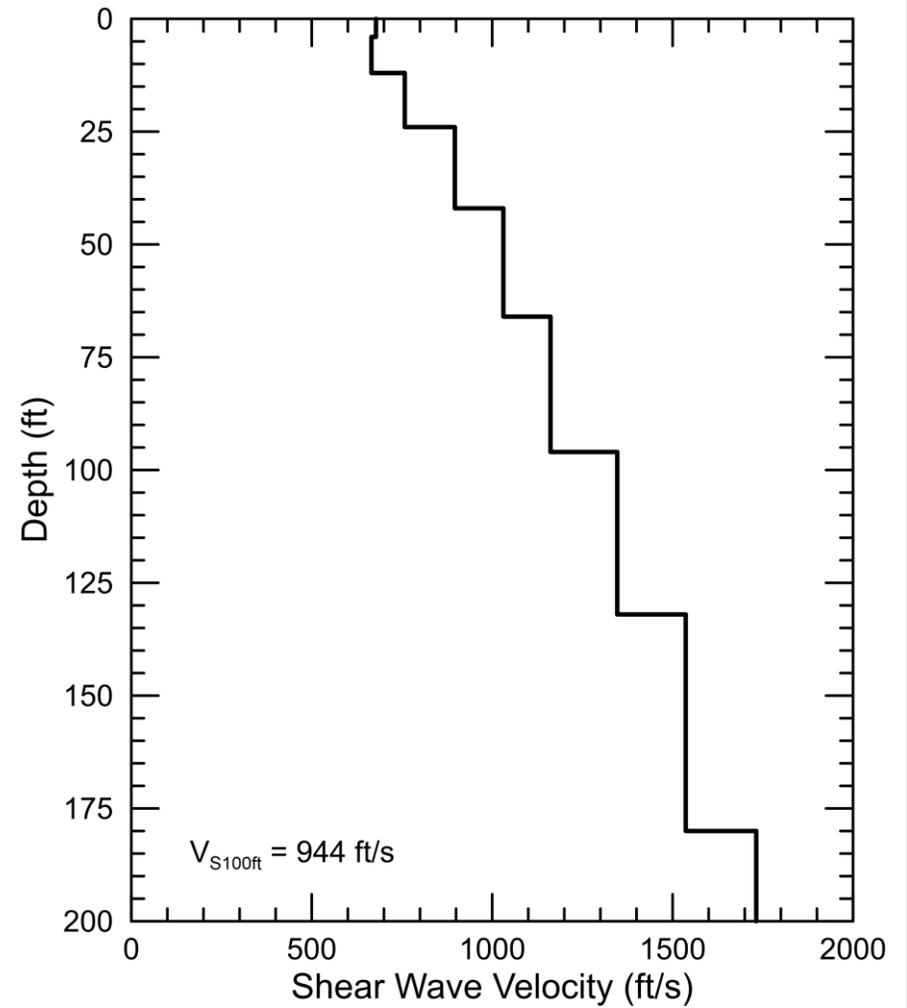
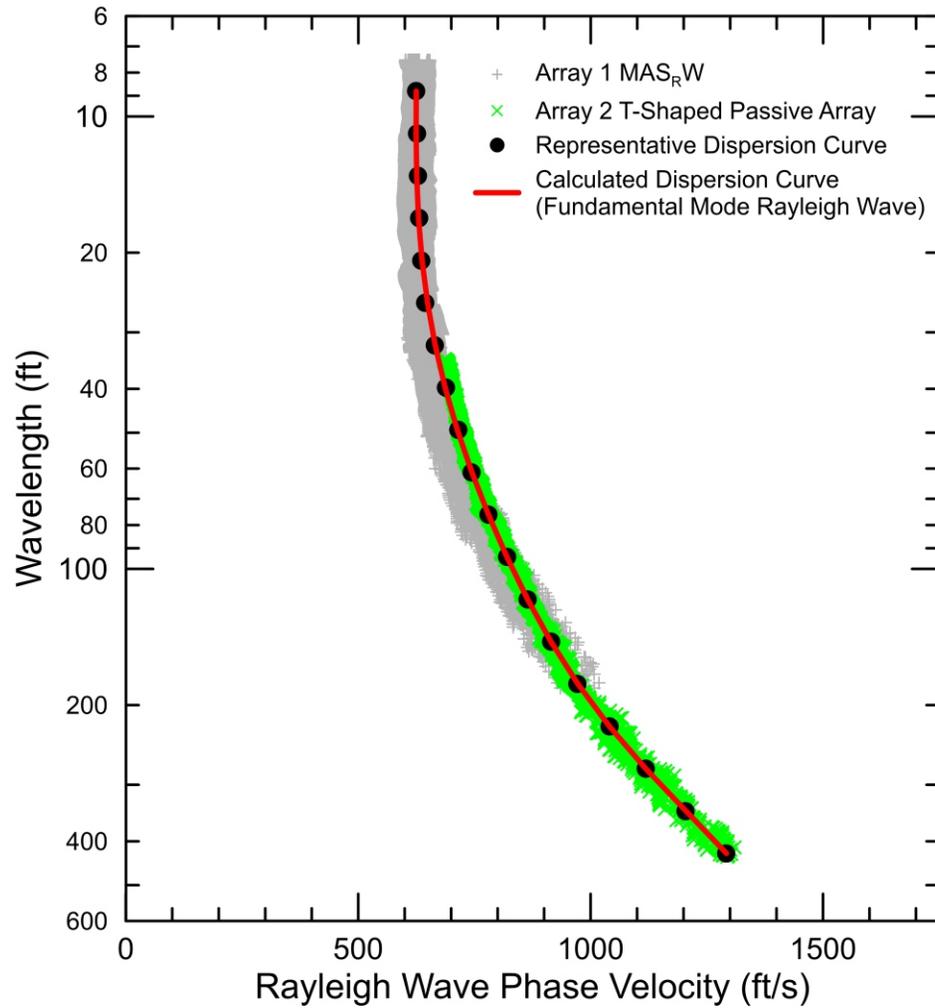
The fit of the calculated fundamental mode dispersion curve to the experimental data collected along Arrays 1 and 2 and the modeled  $V_S$  profile for the surface wave sounding is presented as Figure 2. The resolution decreases gradually with depth due to the loss of sensitivity of the dispersion curve to changes in  $V_S$  at greater depth. The  $V_S$  profile used to match the field data is provided in tabular form as Table 2.

The  $V_S$  model was developed from the Rayleigh wave dispersion data derived from MASW and array microtremor data, acquired along Arrays 1 and 2. The Rayleigh wave phase velocities from the microtremor array are in good agreement with those from the MASW data in the overlapping wavelength region. Scatter in dispersion data from the two methods are expected to be primarily associated with lateral velocity variability beneath the arrays.

The estimated depth of investigation for the combined active- and passive-source surface wave sounding is about 200 ft.  $V_S$  is about 665 to 680 ft/s in the upper 12 ft. Below a depth of 12 ft,  $V_S$  gradually increases with depth from about 760 ft/s to 1,730 ft/s at a model depth of 180 ft. The average shear wave velocity to a depth of 100 ft ( $V_{S100ft}$ ) is 944 ft/s for the  $V_S$  model. Therefore, per the criteria in the 2019 CBC and ASCE/SEI 7-16, the site is classified as Site Class D, stiff soil.

**Table 2 Arrays 1 and 2  $V_S$  Model**

<b>Depth to Top of Layer (ft)</b>	<b>Layer Thickness (ft)</b>	<b>S-Wave Velocity (ft/s)</b>	<b>Inferred P-Wave Velocity (ft/s)</b>	<b>Inferred Poisson's Ratio</b>	<b>Inferred Unit Weight (lb/ft<sup>3</sup>)</b>
0.0	4.0	678	1269	0.300	114.0
4.0	8.0	665	1245	0.300	114.0
12.0	12.0	758	1418	0.300	116.0
24.0	18.0	897	1678	0.300	119.0
42.0	24.0	1031	1929	0.300	120.5
66.0	30.0	1162	2173	0.300	122.0
96.0	36.0	1347	2519	0.300	123.5
132.0	48.0	1536	6000	0.465	125.0
180.0	Half Space	1732	6500	0.462	127.0



Project No: 22462  
 Date: Dec 20, 2022  
 Drawn By: A. Martin  
 Approved By: *Anthony Martin*

P:\IGV\Projects\2022\22462 Twining\Report\Figure 2.cdr

FIGURE 2  
 SURFACE WAVE MODEL - ARRAYS 1 AND 2

KAISER REDLANDS MEDICAL CENTER  
 REDLANDS, CALIFORNIA

PREPARED FOR  
 TWINING CONSULTING, INC.

## 6.2 Arrays 3 and 4

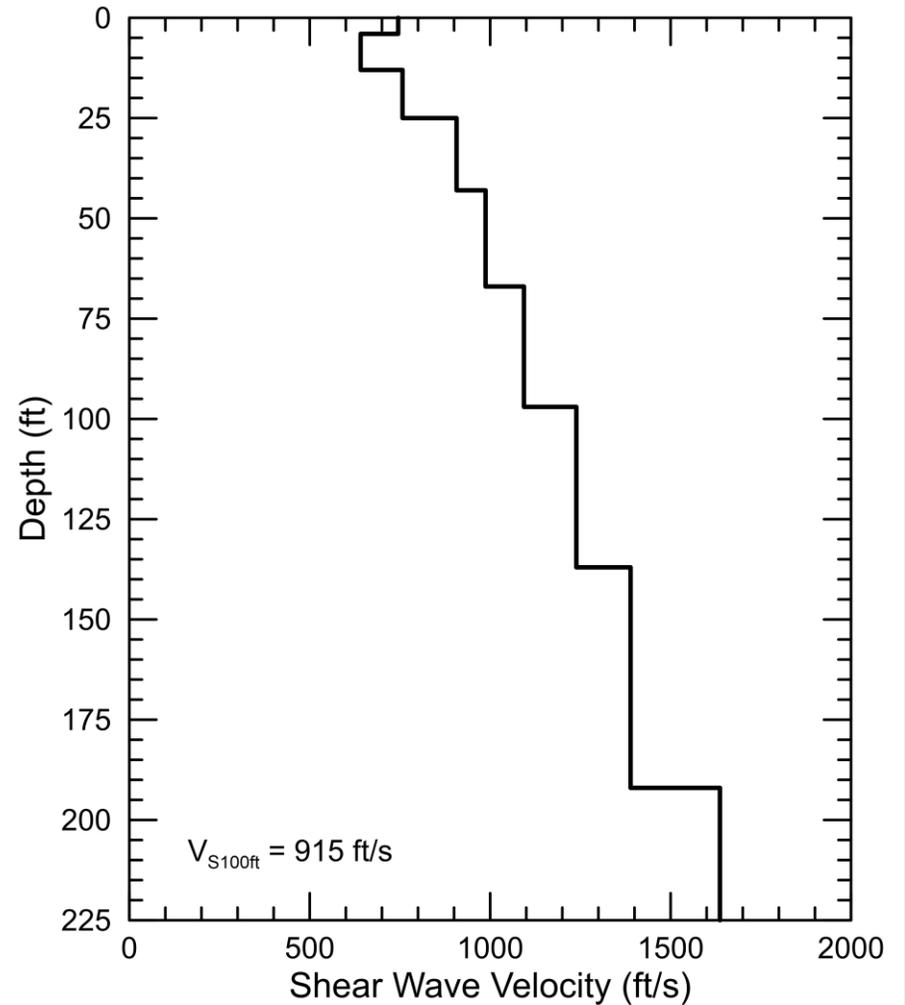
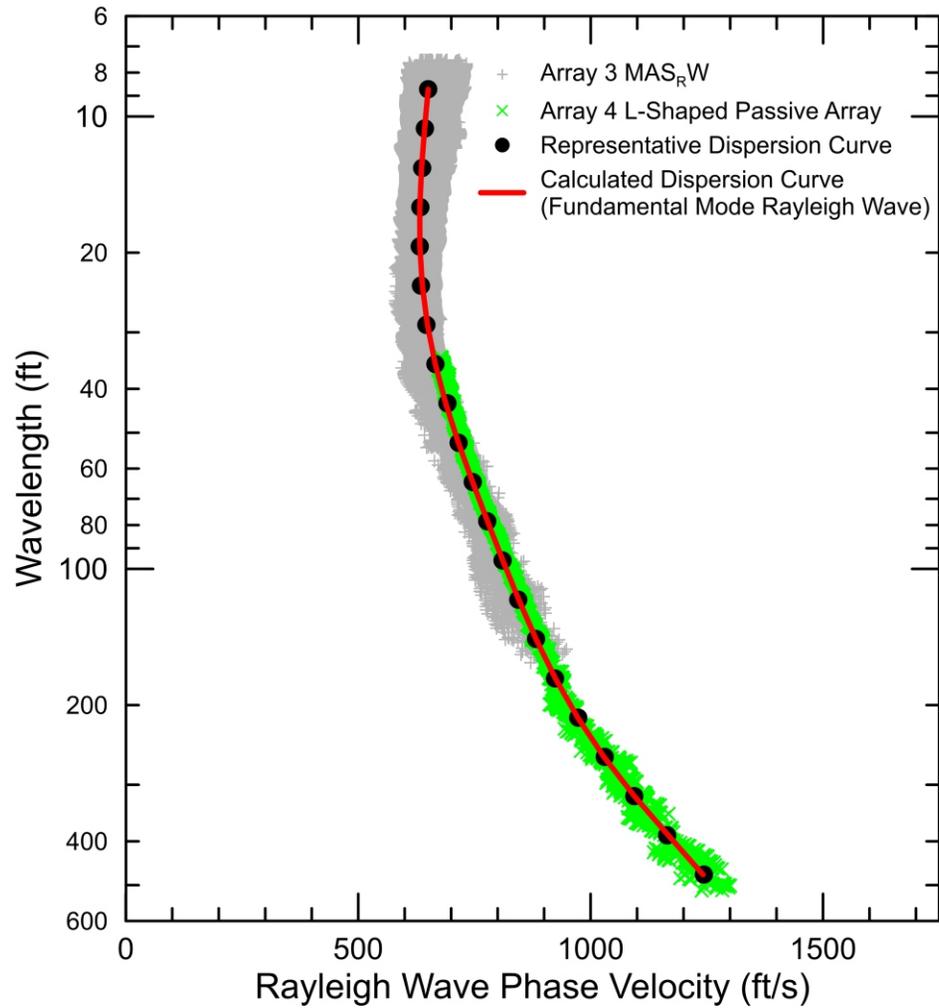
The fit of the calculated fundamental mode dispersion curve to the experimental data collected along Arrays 3 and 4 and the modeled  $V_S$  profile for the surface wave sounding is presented as Figure 3. The resolution decreases gradually with depth due to the loss of sensitivity of the dispersion curve to changes in  $V_S$  at greater depth. The  $V_S$  profile used to match the field data is provided in tabular form as Table 3.

The  $V_S$  model was developed from the Rayleigh wave dispersion data derived from MASW and array microtremor data, acquired along Arrays 3 and 4. The Rayleigh wave phase velocities from the microtremor array are in good agreement with those from the MASW data in the overlapping wavelength region. Scatter in dispersion data from the two methods are expected to be primarily associated with lateral velocity variability beneath the arrays.

The estimated depth of investigation for the combined active- and passive-source surface wave sounding is about 225 ft.  $V_S$  is about 640 to 760 ft/s in the upper 25 ft. Below a depth of 25 ft,  $V_S$  gradually increases with depth from about 910 ft/s to 1,640 ft/s at a model depth of 192 ft.  $V_{S100ft}$  is 915 ft/s for the  $V_S$  model. Therefore, per the criteria in the 2019 CBC and ASCE/SEI 7-16, the site is classified as Site Class D, stiff soil.

**Table 3 Arrays 3 and 4  $V_S$  Model**

<b>Depth to Top of Layer (ft)</b>	<b>Layer Thickness (ft)</b>	<b>S-Wave Velocity (ft/s)</b>	<b>Inferred P-Wave Velocity (ft/s)</b>	<b>Inferred Poisson's Ratio</b>	<b>Inferred Unit Weight (lb/ft<sup>3</sup>)</b>
0.0	4.0	745	1394	0.300	115.0
4.0	9.0	641	1199	0.300	114.0
13.0	12.0	757	1417	0.300	115.0
25.0	18.0	907	1697	0.300	119.0
43.0	24.0	987	1847	0.300	120.0
67.0	30.0	1094	2046	0.300	121.0
97.0	40.0	1239	2318	0.300	122.0
137.0	55.0	1389	6000	0.472	124.0
192.0	Half Space	1637	6500	0.466	126.0



Project No: 22462  
 Date: Dec 20, 2022  
 Drawn By: A. Martin  
 Approved By: *Anthony Martin*

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FIGURE 3  
 SURFACE WAVE MODEL - ARRAYS 3 AND 4

KAISER REDLANDS MEDICAL CENTER  
 REDLANDS, CALIFORNIA

PREPARED FOR  
 TWINING CONSULTING, INC.

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## 8 CERTIFICATION

All geophysical data, analysis, interpretations, conclusions, and recommendations in this document have been prepared under the supervision of and reviewed by a **GEOVision** California Professional Geophysicist.

Reviewed and approved by,



12/20/2022

---

Antony J. Martin  
California Professional Geophysicist, P. Gp.  
**GEOVision** Geophysical Services

Date

- \* This geophysical investigation was conducted under the supervision of a California Professional Geophysicist using industry standard methods and equipment. A high degree of professionalism was maintained during all aspects of the project from the field investigation and data acquisition, through data processing interpretation and reporting. All original field data files, field notes and observations, and other pertinent information are maintained in the project files and are available for the client to review for a period of at least one year.

A professional geophysicist's certification of interpreted geophysical conditions comprises a declaration of his/her professional judgment. It does not constitute a warranty or guarantee, expressed or implied, nor does it relieve any other party of its responsibility to abide by contract documents, applicable codes, standards, regulations, or ordinances.

Date: 10-26-22

Job No. 224

## DAILY FIELD SUMMARY

Sheet: \_\_\_\_\_ of \_\_\_\_\_

Client: Twining  
Sr 3000 400 mhz TRL

Location: 1301 California St  
Redlands CA

Description	Hours
0800	
<u>Arrive on Site</u>	
<u>Cleared 10 location</u>	
<u>Note B-9 man to the East 3'</u>	
<u>B-2 - B-3 &amp; B-4 No Scan</u>	
<u>Hand Auger 5' only</u>	
Total:	

GV Personnel: Chris C

Rabit

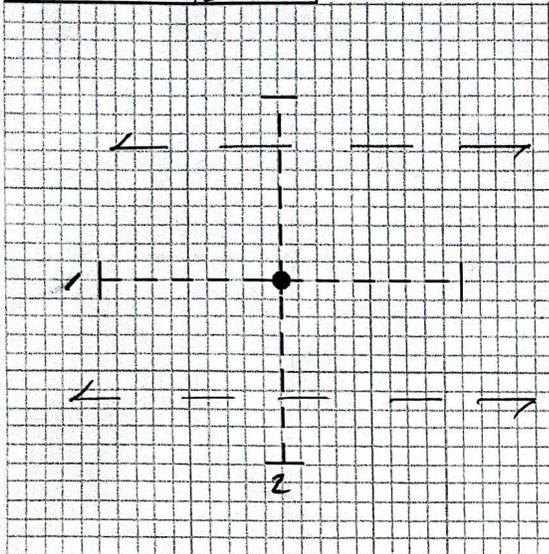
GV Field Supervisor: [Signature]

Date: 10-26-22

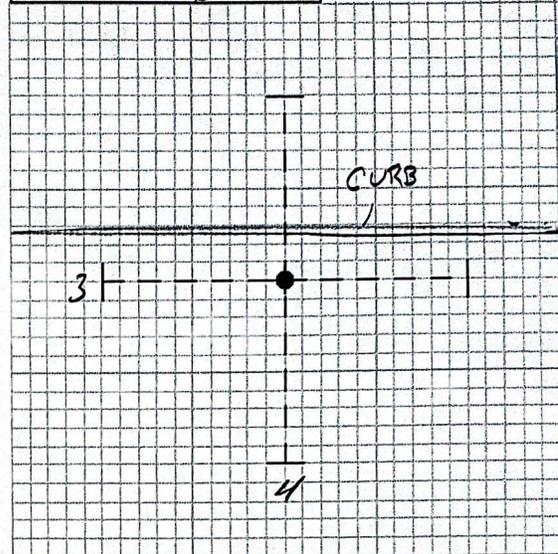
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Date: 10/26/22

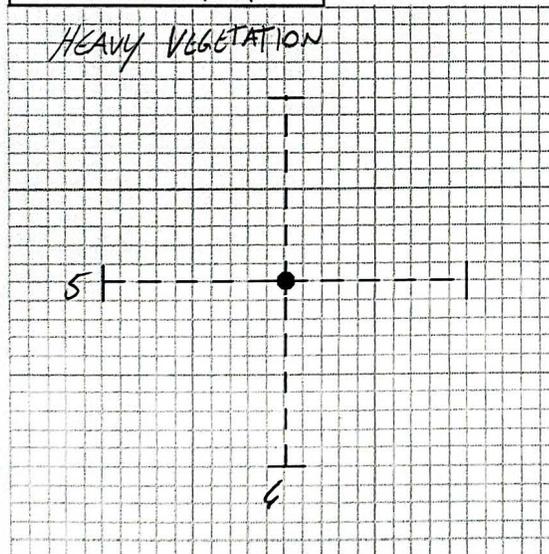
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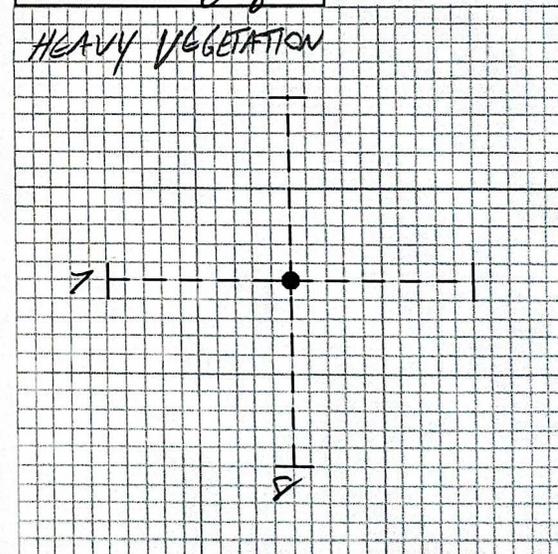
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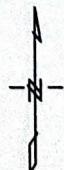
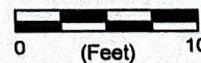


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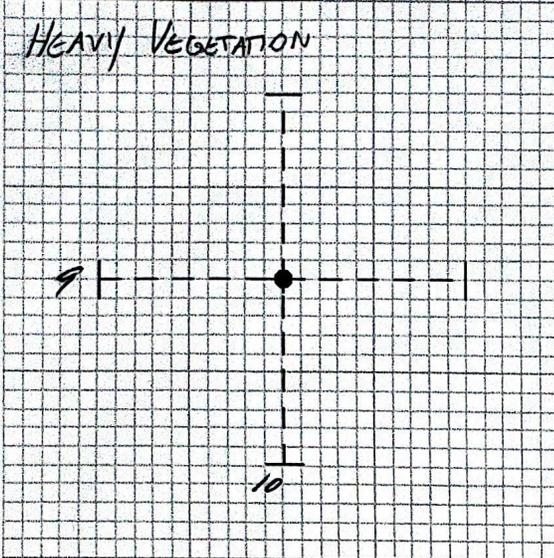
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- Proposed Boring Location
- GPR Traverse with File Number
- Utility: E= Electrical, T= Telepone, NG = Natural Gas, SS = Sanitary Sewer  
SD = Storm Drain, W= Water, C = Unknown Line

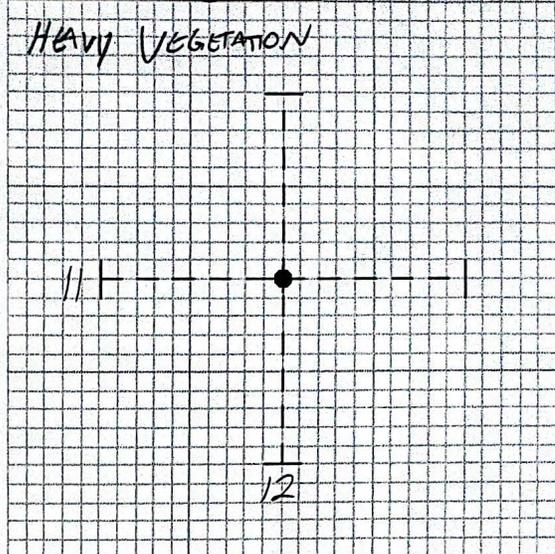


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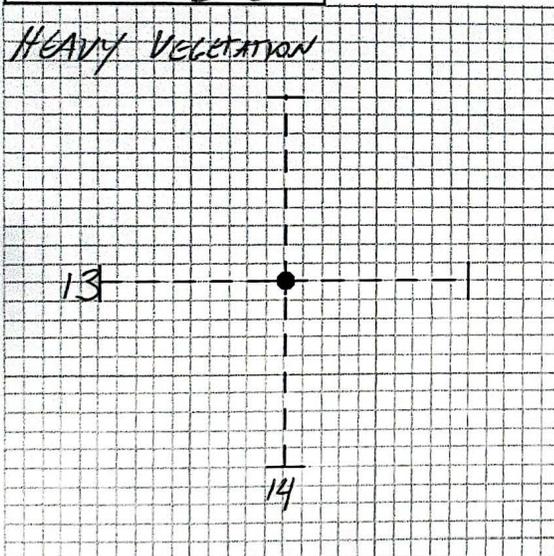
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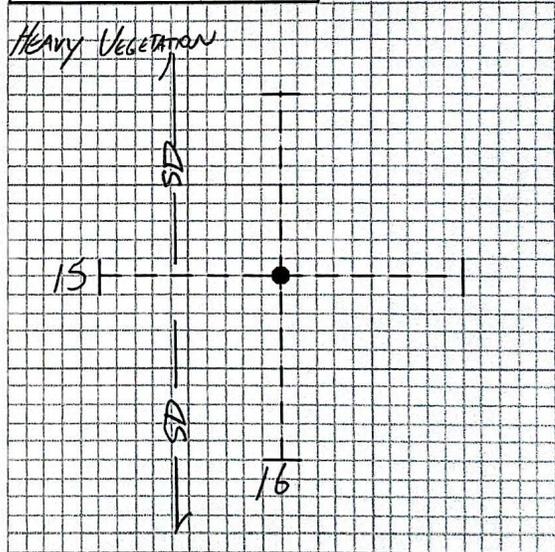
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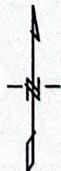
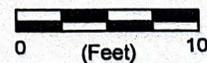


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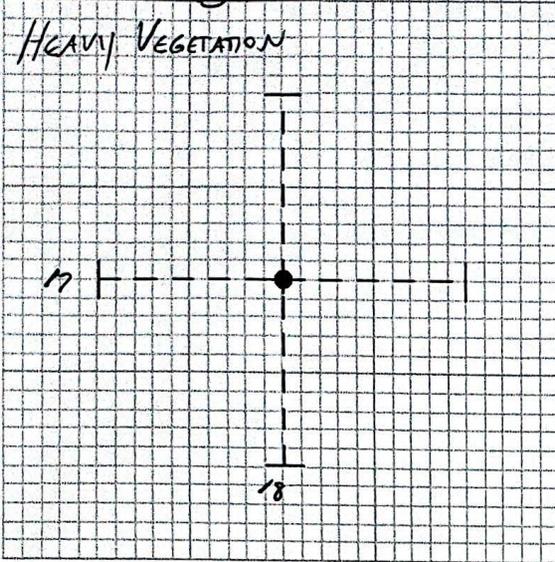
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SD = Storm Drain, W = Water, C = Unknown Line

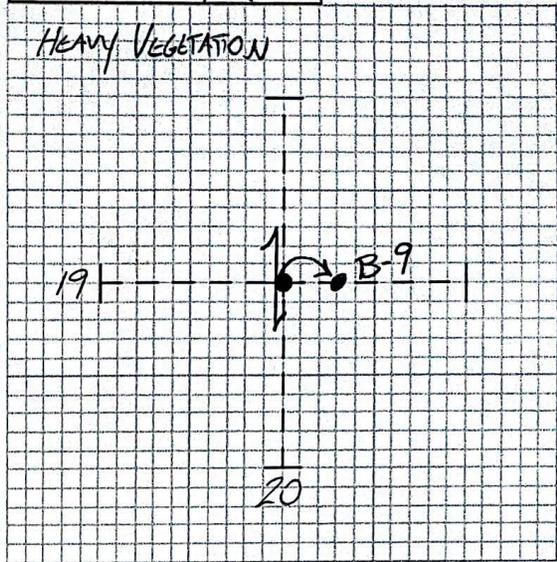


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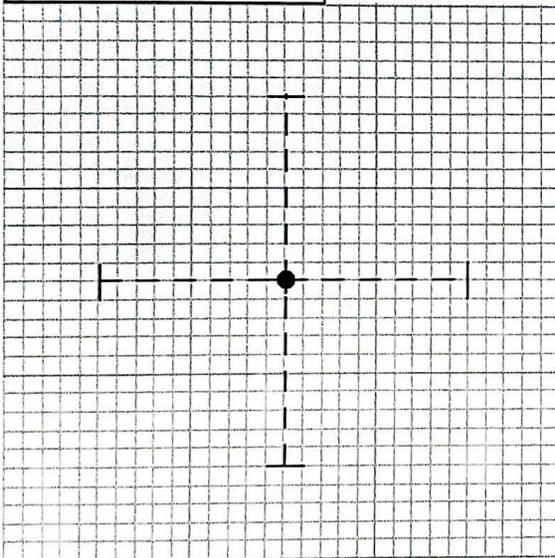
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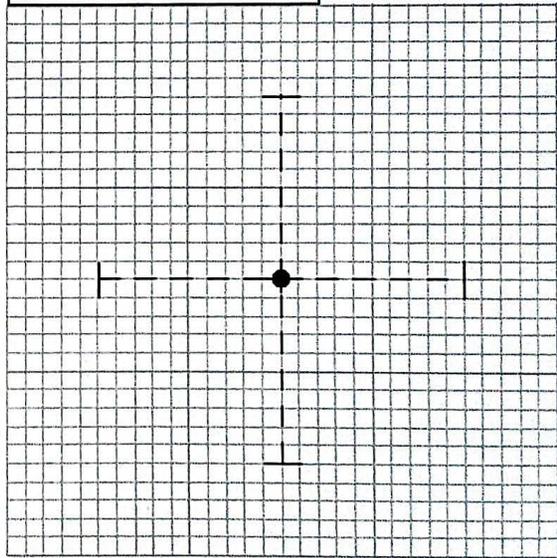
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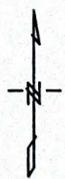
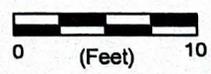


BORING NUMBER:



**LEGEND**

-  Proposed Boring Location
-  GPR Traverse with File Number
-  Utility: E= Electrical, T= Telepone, NG = Natural Gas, SS = Sanitary Sewer  
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# **APPENDIX E**

# **SEISMIC SETTLEMENT ANALYSIS**

## SPT BASED LIQUEFACTION ANALYSIS REPORT

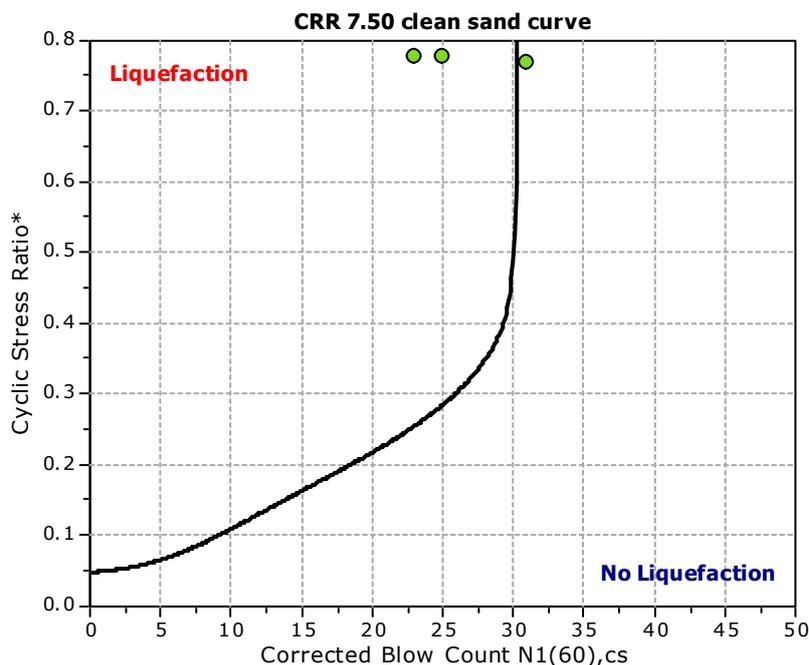
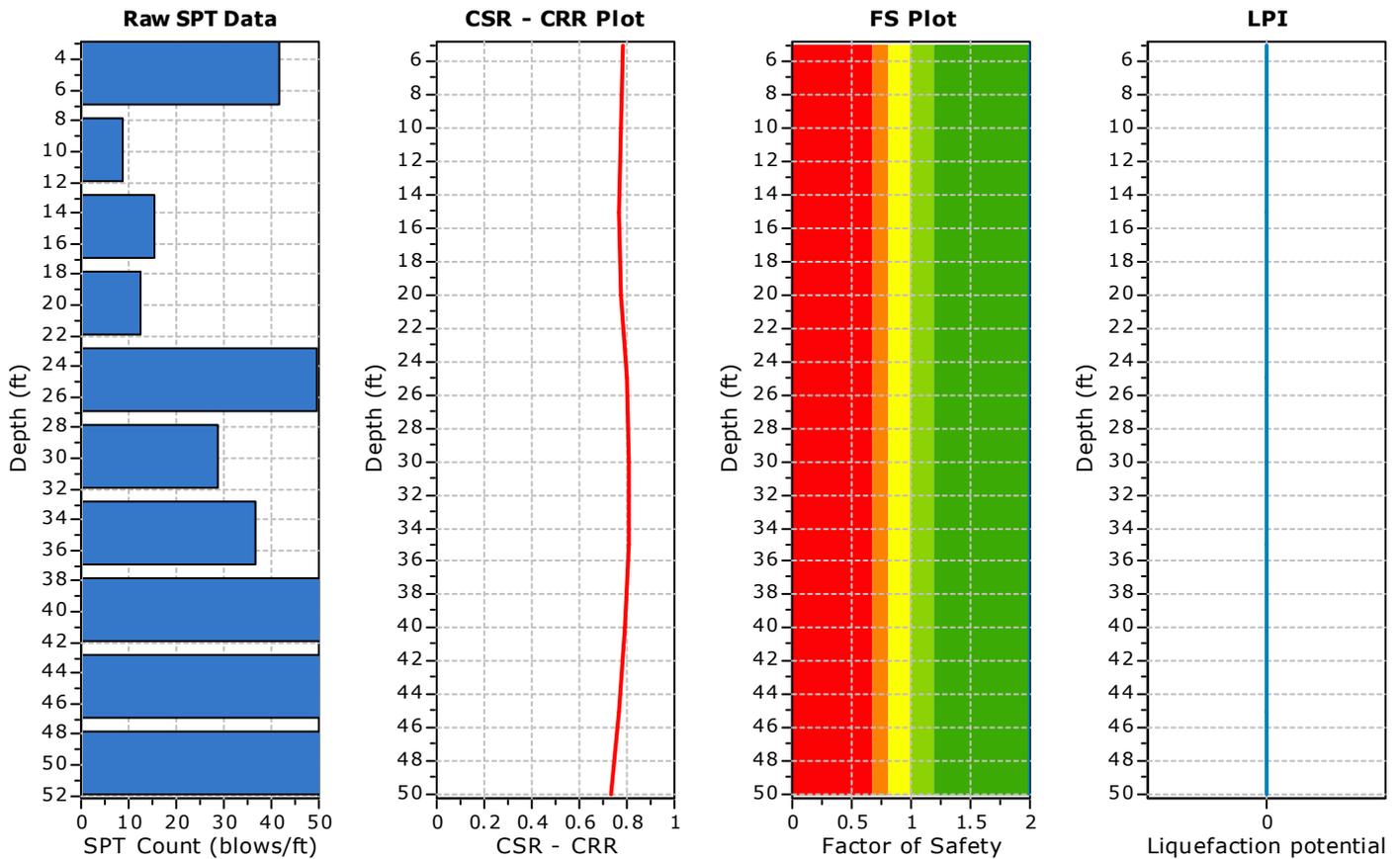
**Project title : Kaiser Redlands MOB 2**

**SPT Name: B-5**

**Location : NW corner of California St & Lugonia Ave, Redlands, CA**

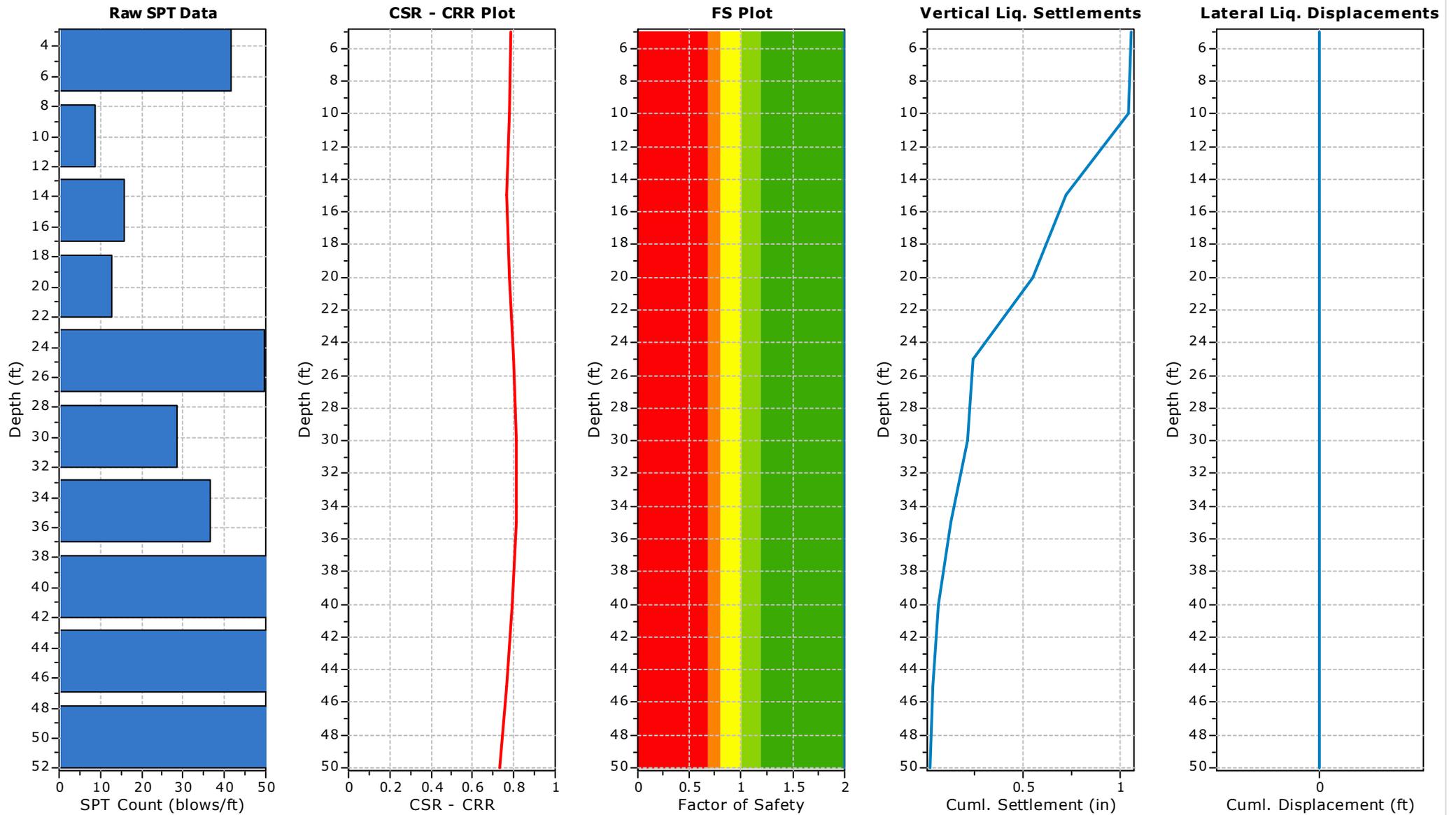
**:: Input parameters and analysis properties ::**

Analysis method:	NCEER 1998	G.W.T. (in-situ):	60.00 ft
Fines correction method:	NCEER 1998	G.W.T. (earthq.):	60.00 ft
Sampling method:	Standard Sampler	Earthquake magnitude $M_w$ :	8.10
Borehole diameter:	200mm	Peak ground acceleration:	0.91 g
Rod length:	3.28 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.30		



- F.S. color scheme**
- Almost certain it will liquefy
  - Very likely to liquefy
  - Liquefaction and no liq. are equally likely
  - Unlike to liquefy
  - Almost certain it will not liquefy
- LPI color scheme**
- Very high risk
  - High risk
  - Low risk

**:: Overall Liquefaction Assessment Analysis Plots ::**



:: Field input data ::					
Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
5.00	42	0.00	120.00	5.00	Yes
10.00	9	39.20	120.00	5.00	Yes
15.00	16	39.20	120.00	5.00	Yes
20.00	13	33.60	120.00	5.00	Yes
25.00	50	33.60	120.00	5.00	Yes
30.00	29	46.30	120.00	5.00	Yes
35.00	37	46.30	120.00	5.00	Yes
40.00	57	38.60	120.00	5.00	Yes
45.00	98	38.60	120.00	5.00	Yes
50.00	74	38.60	120.00	5.00	Yes

**Abbreviations**

Depth: Depth at which test was performed (ft)  
 SPT Field Value: Number of blows per foot  
 Fines Content: Fines content at test depth (%)  
 Unit Weight: Unit weight at test depth (pcf)  
 Infl. Thickness: Thickness of the soil layer to be considered in settlements analysis (ft)  
 Can Liquefy: User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::																
Depth (ft)	SPT Field Value	Unit Weight (pcf)	$\sigma_v$ (tsf)	$u_o$ (tsf)	$\sigma'_{vo}$ (tsf)	$C_N$	$C_E$	$C_B$	$C_R$	$C_S$	$(N_1)_{60}$	Fines Content (%)	$\alpha$	$\beta$	$(N_1)_{60cs}$	CRR <sub>7.5</sub>
5.00	42	120.00	0.30	0.00	0.30	1.70	1.30	1.15	0.75	1.00	80	0.00	0.00	1.00	80	4.000
10.00	9	120.00	0.60	0.00	0.60	1.33	1.30	1.15	0.85	1.00	15	39.20	5.00	1.20	23	4.000
15.00	16	120.00	0.90	0.00	0.90	1.08	1.30	1.15	0.85	1.00	22	39.20	5.00	1.20	31	4.000
20.00	13	120.00	1.20	0.00	1.20	0.94	1.30	1.15	0.95	1.00	17	33.60	4.91	1.18	25	4.000
25.00	50	120.00	1.50	0.00	1.50	0.84	1.30	1.15	0.95	1.00	60	33.60	4.91	1.18	76	4.000
30.00	29	120.00	1.80	0.00	1.80	0.77	1.30	1.15	1.00	1.00	33	46.30	5.00	1.20	45	4.000
35.00	37	120.00	2.10	0.00	2.10	0.71	1.30	1.15	1.00	1.00	39	46.30	5.00	1.20	52	4.000
40.00	57	120.00	2.40	0.00	2.40	0.66	1.30	1.15	1.00	1.00	57	38.60	5.00	1.20	73	4.000
45.00	98	120.00	2.70	0.00	2.70	0.63	1.30	1.15	1.00	1.00	92	38.60	5.00	1.20	115	4.000
50.00	74	120.00	3.00	0.00	3.00	0.59	1.30	1.15	1.00	1.00	66	38.60	5.00	1.20	84	4.000

**Abbreviations**

$\sigma_v$ : Total stress during SPT test (tsf)  
 $u_o$ : Water pore pressure during SPT test (tsf)  
 $\sigma'_{vo}$ : Effective overburden pressure during SPT test (tsf)  
 $C_N$ : Overburden correction factor  
 $C_E$ : Energy correction factor  
 $C_B$ : Borehole diameter correction factor  
 $C_R$ : Rod length correction factor  
 $C_S$ : Liner correction factor  
 $N_{1(60)}$ : Corrected  $N_{SPT}$  to a 60% energy ratio  
 $\alpha, \beta$ : Clean sand equivalent clean sand formula coefficients  
 $N_{1(60)cs}$ : Corrected  $N_{1(60)}$  value for fines content  
 CRR<sub>7.5</sub>: Cyclic resistance ratio for M=7.5

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::													
Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	$r_d$	$\alpha$	CSR	MSF	CSR <sub>eq, M=7.5</sub>	$K_{\sigma}$	CSR*	FS	
5.00	120.00	0.30	0.00	0.30	0.99	1.00	0.586	0.82	0.714	1.00	0.863	2.000	●
10.00	120.00	0.60	0.00	0.60	0.98	1.00	0.579	0.82	0.706	1.00	0.854	2.000	●

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::													
Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	$r_d$	$\alpha$	CSR	MSF	$CSR_{eq,M=7.5}$	$K_{\sigma}$	CSR*	FS	
15.00	120.00	0.90	0.00	0.90	0.97	1.00	0.573	0.82	0.698	1.00	0.844	2.000	●
20.00	120.00	1.20	0.00	1.20	0.96	1.00	0.566	0.82	0.690	0.98	0.856	2.000	●
25.00	120.00	1.50	0.00	1.50	0.94	1.00	0.557	0.82	0.679	0.93	0.881	2.000	●
30.00	120.00	1.80	0.00	1.80	0.92	1.00	0.545	0.82	0.663	0.90	0.893	2.000	●
35.00	120.00	2.10	0.00	2.10	0.89	1.00	0.527	0.82	0.642	0.87	0.891	2.000	●
40.00	120.00	2.40	0.00	2.40	0.85	1.00	0.503	0.82	0.613	0.85	0.874	2.000	●
45.00	120.00	2.70	0.00	2.70	0.80	1.00	0.475	0.82	0.579	0.83	0.845	2.000	●
50.00	120.00	3.00	0.00	3.00	0.75	1.00	0.445	0.82	0.542	0.81	0.808	2.000	●

**Abbreviations**

- $\sigma_{v,eq}$ : Total overburden pressure at test point, during earthquake (tsf)
- $u_{o,eq}$ : Water pressure at test point, during earthquake (tsf)
- $\sigma'_{vo,eq}$ : Effective overburden pressure, during earthquake (tsf)
- $r_d$ : Nonlinear shear mass factor
- $\alpha$ : Improvement factor due to stone columns
- CSR: Cyclic Stress Ratio (adjusted for improvement)
- MSF: Magnitude Scaling Factor
- $CSR_{eq,M=7.5}$ : CSR adjusted for M=7.5
- $K_{\sigma}$ : Effective overburden stress factor
- CSR\*: CSR fully adjusted (user FS applied)\*\*\*
- FS: Calculated factor of safety against soil liquefaction

\*\*\* User FS: 1.10

:: Liquefaction potential according to Iwasaki ::					
Depth (ft)	FS	F	wz	Thickness (ft)	$I_L$
5.00	2.000	0.00	9.24	5.00	0.00
10.00	2.000	0.00	8.48	5.00	0.00
15.00	2.000	0.00	7.71	5.00	0.00
20.00	2.000	0.00	6.95	5.00	0.00
25.00	2.000	0.00	6.19	5.00	0.00
30.00	2.000	0.00	5.43	5.00	0.00
35.00	2.000	0.00	4.67	5.00	0.00
40.00	2.000	0.00	3.90	5.00	0.00
45.00	2.000	0.00	3.14	5.00	0.00
50.00	2.000	0.00	2.38	5.00	0.00

**Overall potential  $I_L$ : 0.00**

- $I_L = 0.00$  - No liquefaction
- $I_L$  between 0.00 and 5 - Liquefaction not probable
- $I_L$  between 5 and 15 - Liquefaction probable
- $I_L > 15$  - Liquefaction certain

:: Vertical settlements estimation for dry sands ::												
Depth (ft)	$(N_1)_{60}$	$\tau_{av}$	p	$G_{max}$ (tsf)	$\alpha$	b	$\gamma$	$\epsilon_{15}$	$N_c$	$\epsilon_{Nc}$ (%)	$\Delta h$ (ft)	$\Delta S$ (in)
5.00	80	0.18	0.20	863.51	0.14	13179.75	0.00	0.00	21.37	0.01	5.00	0.014
10.00	15	0.35	0.40	805.99	0.15	8695.39	0.00	0.00	21.37	0.27	5.00	0.325
15.00	22	0.52	0.60	1090.40	0.16	6817.65	0.00	0.00	21.37	0.14	5.00	0.169
20.00	17	0.68	0.80	1171.97	0.17	5736.82	0.00	0.00	21.37	0.26	5.00	0.306
25.00	60	0.84	1.00	1898.14	0.18	5017.94	0.00	0.00	21.37	0.02	5.00	0.028

:: Vertical settlements estimation for dry sands ::												
Depth (ft)	(N <sub>1</sub> ) <sub>60</sub>	T <sub>av</sub>	p	G <sub>max</sub> (tsf)	a	b	γ	ε <sub>15</sub>	N <sub>c</sub>	ε <sub>N<sub>c</sub></sub> (%)	Δh (ft)	ΔS (in)
30.00	33	0.98	1.21	1746.03	0.19	4497.97	0.00	0.00	21.37	0.07	5.00	0.086
35.00	39	1.11	1.41	1979.05	0.21	4100.61	0.00	0.00	21.37	0.05	5.00	0.063
40.00	57	1.21	1.61	2368.96	0.22	3784.89	0.00	0.00	21.37	0.03	5.00	0.031
45.00	92	1.28	1.81	2923.65	0.23	3526.64	0.00	0.00	21.37	0.01	5.00	0.013
50.00	66	1.34	2.01	2775.44	0.24	3310.60	0.00	0.00	21.37	0.02	5.00	0.021

**Cumulative settlements: 1.056**

**Abbreviations**

- T<sub>av</sub>: Average cyclic shear stress
- p: Average stress
- G<sub>max</sub>: Maximum shear modulus (tsf)
- a, b: Shear strain formula variables
- γ: Average shear strain
- ε<sub>15</sub>: Volumetric strain after 15 cycles
- N<sub>c</sub>: Number of cycles
- ε<sub>N<sub>c</sub></sub>: Volumetric strain for number of cycles N<sub>c</sub> (%)
- Δh: Thickness of soil layer (in)
- ΔS: Settlement of soil layer (in)

:: Lateral displacements estimation for saturated sands ::						
Depth (ft)	(N <sub>1</sub> ) <sub>60</sub>	D <sub>r</sub> (%)	γ <sub>max</sub> (%)	d <sub>z</sub> (ft)	LDI	LD (ft)
5.00	80	100.00	0.00	5.00	0.000	0.00
10.00	15	54.22	0.00	5.00	0.000	0.00
15.00	22	65.67	0.00	5.00	0.000	0.00
20.00	17	57.72	0.00	5.00	0.000	0.00
25.00	60	100.00	0.00	5.00	0.000	0.00
30.00	33	80.42	0.00	5.00	0.000	0.00
35.00	39	87.43	0.00	5.00	0.000	0.00
40.00	57	100.00	0.00	5.00	0.000	0.00
45.00	92	100.00	0.00	5.00	0.000	0.00
50.00	66	100.00	0.00	5.00	0.000	0.00

**Cumulative lateral displacements: 0.00**

**Abbreviations**

- D<sub>r</sub>: Relative density (%)
- γ<sub>max</sub>: Maximum amplitude of cyclic shear strain (%)
- d<sub>z</sub>: Soil layer thickness (ft)
- LDI: Lateral displacement index (ft)
- LD: Actual estimated displacement (ft)

## SPT BASED LIQUEFACTION ANALYSIS REPORT

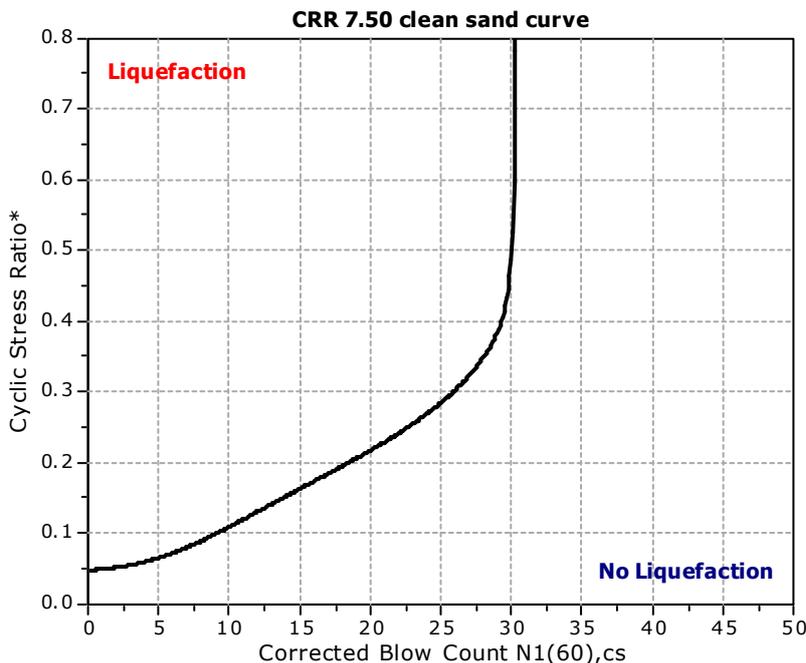
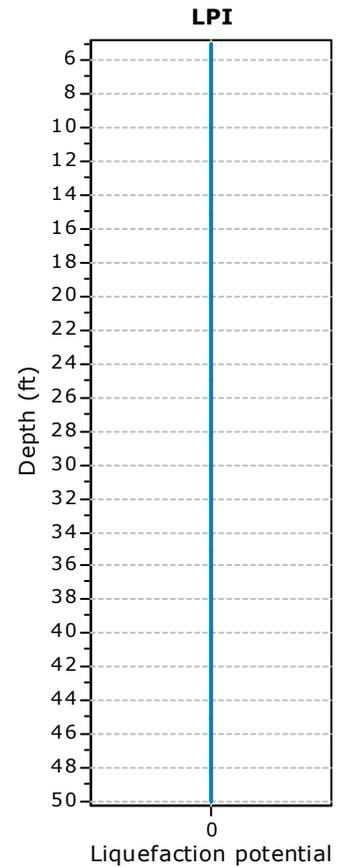
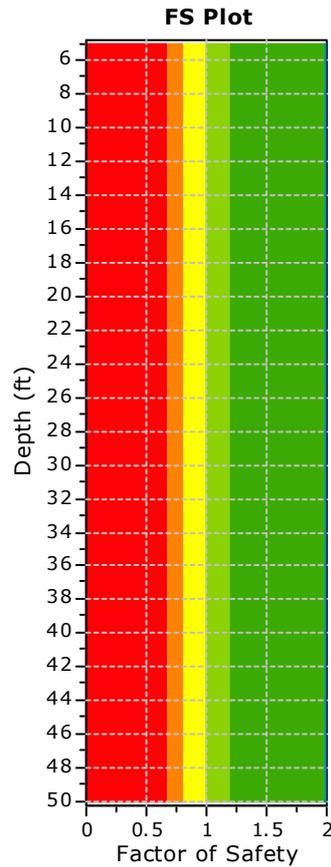
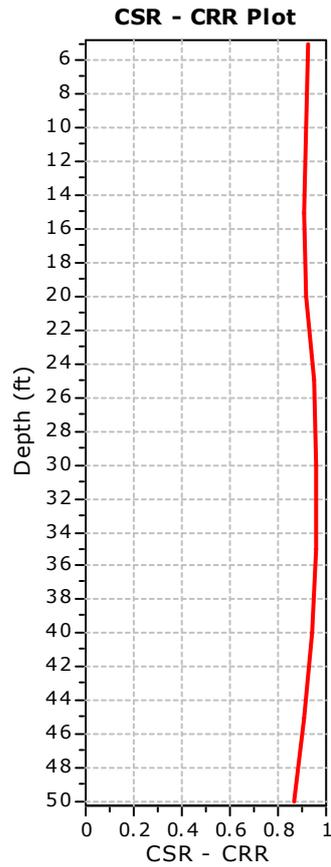
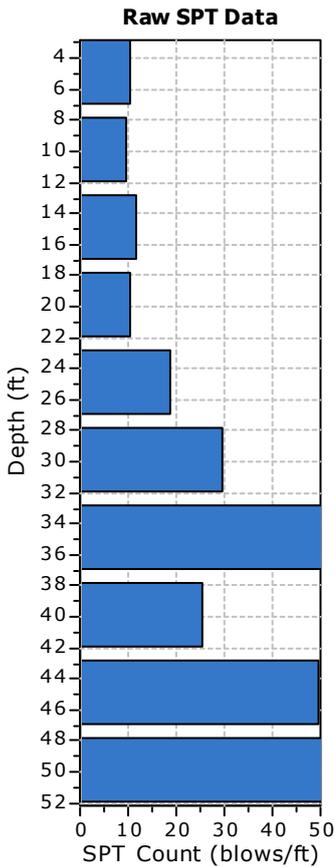
**Project title :** Kaiser Redlands MOB 2

**SPT Name:** B-10

**Location :** NW corner of California St & Lugonia Ave, Redlands, CA

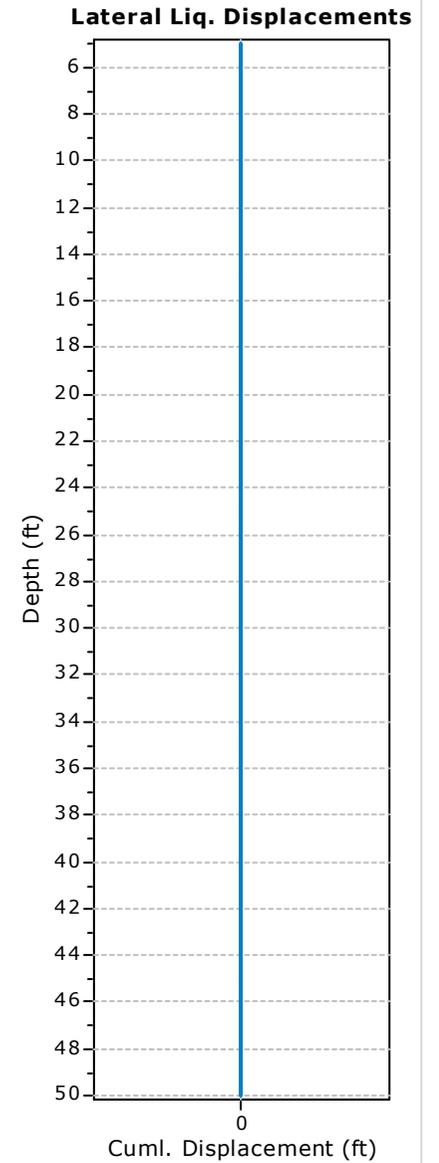
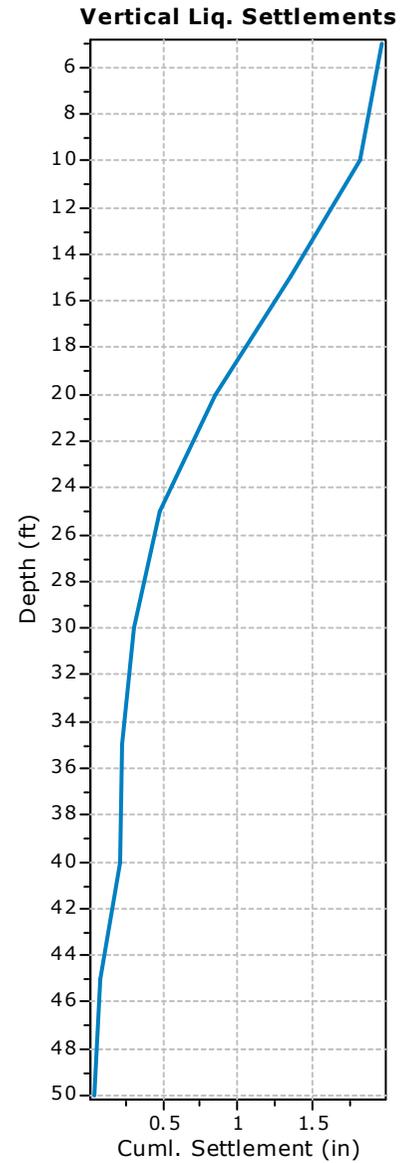
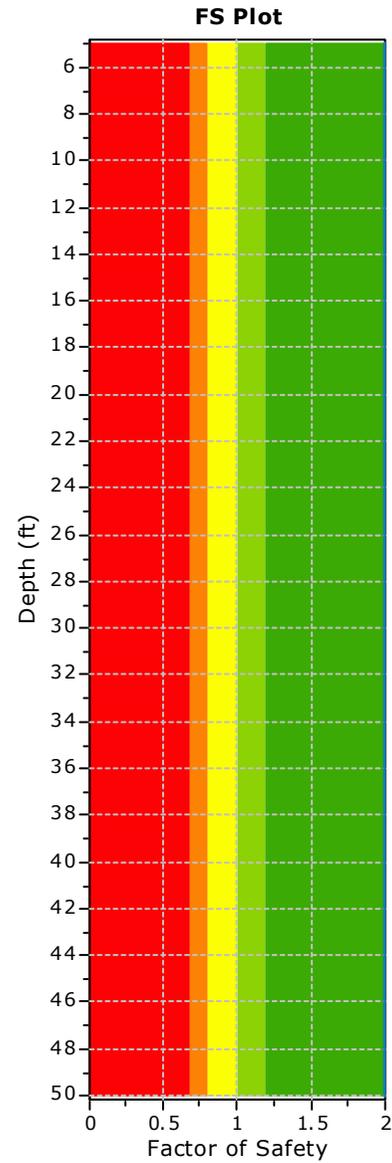
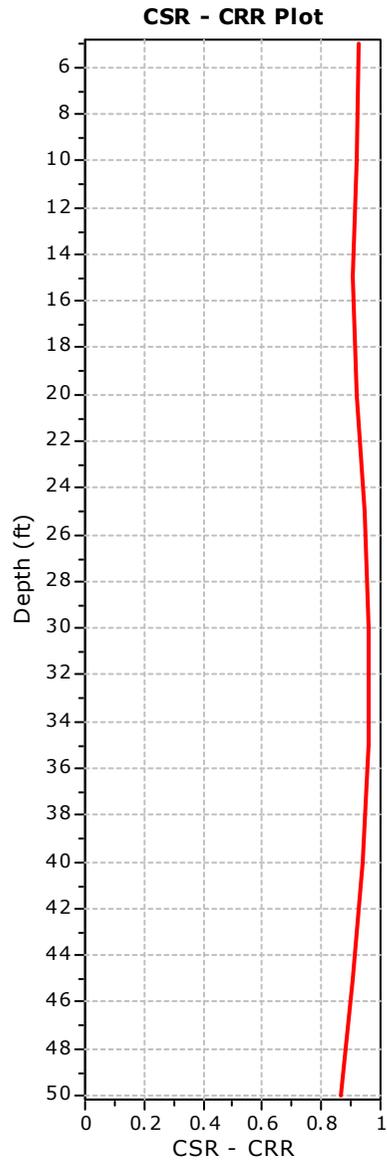
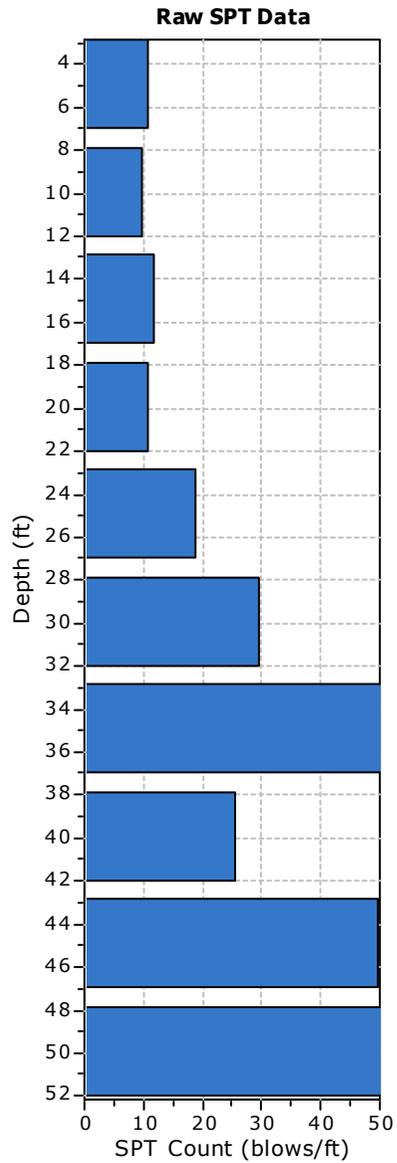
**:: Input parameters and analysis properties ::**

Analysis method:	NCEER 1998	G.W.T. (in-situ):	60.00 ft
Fines correction method:	NCEER 1998	G.W.T. (earthq.):	60.00 ft
Sampling method:	Standard Sampler	Earthquake magnitude $M_w$ :	8.10
Borehole diameter:	200mm	Peak ground acceleration:	0.91 g
Rod length:	3.28 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.30		



- F.S. color scheme**
- Almost certain it will liquefy
  - Very likely to liquefy
  - Liquefaction and no liq. are equally likely
  - Unlike to liquefy
  - Almost certain it will not liquefy
- LPI color scheme**
- Very high risk
  - High risk
  - Low risk

**:: Overall Liquefaction Assessment Analysis Plots ::**



:: Field input data ::					
Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
5.00	11	54.30	120.00	5.00	Yes
10.00	10	15.00	120.00	5.00	Yes
15.00	12	15.00	120.00	5.00	Yes
20.00	11	49.20	120.00	5.00	Yes
25.00	19	49.20	120.00	5.00	Yes
30.00	30	49.20	120.00	5.00	Yes
35.00	78	49.20	120.00	5.00	Yes
40.00	26	49.20	120.00	5.00	Yes
45.00	50	49.20	120.00	5.00	Yes
50.00	53	49.20	120.00	5.00	Yes

**Abbreviations**

Depth: Depth at which test was performed (ft)  
 SPT Field Value: Number of blows per foot  
 Fines Content: Fines content at test depth (%)  
 Unit Weight: Unit weight at test depth (pcf)  
 Infl. Thickness: Thickness of the soil layer to be considered in settlements analysis (ft)  
 Can Liquefy: User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::																
Depth (ft)	SPT Field Value	Unit Weight (pcf)	$\sigma_v$ (tsf)	$u_o$ (tsf)	$\sigma'_{vo}$ (tsf)	$C_N$	$C_E$	$C_B$	$C_R$	$C_S$	$(N_1)_{60}$	Fines Content (%)	$\alpha$	$\beta$	$(N_1)_{60cs}$	CRR <sub>7.5</sub>
5.00	11	120.00	0.30	0.00	0.30	1.70	1.30	1.15	0.75	1.00	21	54.30	5.00	1.20	30	4.000
10.00	10	120.00	0.60	0.00	0.60	1.33	1.30	1.15	0.85	1.00	17	15.00	2.50	1.05	20	4.000
15.00	12	120.00	0.90	0.00	0.90	1.08	1.30	1.15	0.85	1.00	17	15.00	2.50	1.05	20	4.000
20.00	11	120.00	1.20	0.00	1.20	0.94	1.30	1.15	0.95	1.00	15	49.20	5.00	1.20	23	4.000
25.00	19	120.00	1.50	0.00	1.50	0.84	1.30	1.15	0.95	1.00	23	49.20	5.00	1.20	33	4.000
30.00	30	120.00	1.80	0.00	1.80	0.77	1.30	1.15	1.00	1.00	34	49.20	5.00	1.20	46	4.000
35.00	78	120.00	2.10	0.00	2.10	0.71	1.30	1.15	1.00	1.00	83	49.20	5.00	1.20	105	4.000
40.00	26	120.00	2.40	0.00	2.40	0.66	1.30	1.15	1.00	1.00	26	49.20	5.00	1.20	36	4.000
45.00	50	120.00	2.70	0.00	2.70	0.63	1.30	1.15	1.00	1.00	47	49.20	5.00	1.20	61	4.000
50.00	53	120.00	3.00	0.00	3.00	0.59	1.30	1.15	1.00	1.00	47	49.20	5.00	1.20	61	4.000

**Abbreviations**

$\sigma_v$ : Total stress during SPT test (tsf)  
 $u_o$ : Water pore pressure during SPT test (tsf)  
 $\sigma'_{vo}$ : Effective overburden pressure during SPT test (tsf)  
 $C_N$ : Overburden correction factor  
 $C_E$ : Energy correction factor  
 $C_B$ : Borehole diameter correction factor  
 $C_R$ : Rod length correction factor  
 $C_S$ : Liner correction factor  
 $N_{1(60)}$ : Corrected  $N_{SPT}$  to a 60% energy ratio  
 $\alpha, \beta$ : Clean sand equivalent clean sand formula coefficients  
 $N_{1(60)cs}$ : Corrected  $N_{1(60)}$  value for fines content  
 CRR<sub>7.5</sub>: Cyclic resistance ratio for M=7.5

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::														
Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	$r_d$	$\alpha$	CSR	MSF	CSR <sub>eq, M=7.5</sub>	$K_{\sigma}$	CSR*	FS		
5.00	120.00	0.30	0.00	0.30	0.99	1.00	0.586	0.82	0.714	1.00	1.206	2.000	●	
10.00	120.00	0.60	0.00	0.60	0.98	1.00	0.579	0.82	0.706	1.00	1.192	2.000	●	

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::													
Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	$r_d$	$\alpha$	CSR	MSF	$CSR_{eq,M=7.5}$	$K_{\sigma}$	CSR*	FS	
15.00	120.00	0.90	0.00	0.90	0.97	1.00	0.573	0.82	0.698	1.00	1.179	2.000	●
20.00	120.00	1.20	0.00	1.20	0.96	1.00	0.566	0.82	0.690	0.98	1.195	2.000	●
25.00	120.00	1.50	0.00	1.50	0.94	1.00	0.557	0.82	0.679	0.93	1.230	2.000	●
30.00	120.00	1.80	0.00	1.80	0.92	1.00	0.545	0.82	0.663	0.90	1.247	2.000	●
35.00	120.00	2.10	0.00	2.10	0.89	1.00	0.527	0.82	0.642	0.87	1.244	2.000	●
40.00	120.00	2.40	0.00	2.40	0.85	1.00	0.503	0.82	0.613	0.85	1.221	2.000	●
45.00	120.00	2.70	0.00	2.70	0.80	1.00	0.475	0.82	0.579	0.83	1.180	2.000	●
50.00	120.00	3.00	0.00	3.00	0.75	1.00	0.445	0.82	0.542	0.81	1.129	2.000	●

**Abbreviations**

- $\sigma_{v,eq}$ : Total overburden pressure at test point, during earthquake (tsf)
- $u_{o,eq}$ : Water pressure at test point, during earthquake (tsf)
- $\sigma'_{vo,eq}$ : Effective overburden pressure, during earthquake (tsf)
- $r_d$ : Nonlinear shear mass factor
- $\alpha$ : Improvement factor due to stone columns
- CSR: Cyclic Stress Ratio (adjusted for improvement)
- MSF: Magnitude Scaling Factor
- $CSR_{eq,M=7.5}$ : CSR adjusted for M=7.5
- $K_{\sigma}$ : Effective overburden stress factor
- CSR\*: CSR fully adjusted (user FS applied)\*\*\*
- FS: Calculated factor of safety against soil liquefaction

\*\*\* User FS: 1.30

:: Liquefaction potential according to Iwasaki ::					
Depth (ft)	FS	F	wz	Thickness (ft)	$I_L$
5.00	2.000	0.00	9.24	5.00	0.00
10.00	2.000	0.00	8.48	5.00	0.00
15.00	2.000	0.00	7.71	5.00	0.00
20.00	2.000	0.00	6.95	5.00	0.00
25.00	2.000	0.00	6.19	5.00	0.00
30.00	2.000	0.00	5.43	5.00	0.00
35.00	2.000	0.00	4.67	5.00	0.00
40.00	2.000	0.00	3.90	5.00	0.00
45.00	2.000	0.00	3.14	5.00	0.00
50.00	2.000	0.00	2.38	5.00	0.00

**Overall potential  $I_L$ : 0.00**

- $I_L = 0.00$  - No liquefaction
- $I_L$  between 0.00 and 5 - Liquefaction not probable
- $I_L$  between 5 and 15 - Liquefaction probable
- $I_L > 15$  - Liquefaction certain

:: Vertical settlements estimation for dry sands ::												
Depth (ft)	$(N_1)_{60}$	$\tau_{av}$	p	$G_{max}$ (tsf)	$\alpha$	b	$\gamma$	$\epsilon_{15}$	$N_c$	$\epsilon_{Nc}$ (%)	$\Delta h$ (ft)	$\Delta S$ (in)
5.00	21	0.18	0.20	622.70	0.14	13179.75	0.00	0.00	21.37	0.12	5.00	0.142
10.00	17	0.35	0.40	769.30	0.15	8695.39	0.00	0.00	21.37	0.39	5.00	0.470
15.00	17	0.52	0.60	942.20	0.16	6817.65	0.00	0.00	21.37	0.42	5.00	0.507
20.00	15	0.68	0.80	1139.84	0.17	5736.82	0.00	0.00	21.37	0.31	5.00	0.376
25.00	23	0.84	1.00	1437.35	0.18	5017.94	0.00	0.00	21.37	0.14	5.00	0.166

:: Vertical settlements estimation for dry sands ::												
Depth (ft)	(N <sub>1</sub> ) <sub>60</sub>	T <sub>av</sub>	p	G <sub>max</sub> (tsf)	a	b	γ	ε <sub>15</sub>	N <sub>c</sub>	ε <sub>N<sub>c</sub></sub> (%)	Δh (ft)	ΔS (in)
30.00	34	0.98	1.21	1758.87	0.19	4497.97	0.00	0.00	21.37	0.07	5.00	0.082
35.00	83	1.11	1.41	2501.41	0.21	4100.61	0.00	0.00	21.37	0.01	5.00	0.016
40.00	26	1.21	1.61	1871.62	0.22	3784.89	0.00	0.00	21.37	0.11	5.00	0.129
45.00	47	1.28	1.81	2366.66	0.23	3526.64	0.00	0.00	21.37	0.03	5.00	0.042
50.00	47	1.34	2.01	2494.68	0.24	3310.60	0.00	0.00	21.37	0.03	5.00	0.038

**Cumulative settlements: 1.967**

**Abbreviations**

- T<sub>av</sub>: Average cyclic shear stress
- p: Average stress
- G<sub>max</sub>: Maximum shear modulus (tsf)
- a, b: Shear strain formula variables
- γ: Average shear strain
- ε<sub>15</sub>: Volumetric strain after 15 cycles
- N<sub>c</sub>: Number of cycles
- ε<sub>N<sub>c</sub></sub>: Volumetric strain for number of cycles N<sub>c</sub> (%)
- Δh: Thickness of soil layer (in)
- ΔS: Settlement of soil layer (in)

:: Lateral displacements estimation for saturated sands ::						
Depth (ft)	(N <sub>1</sub> ) <sub>60</sub>	D <sub>r</sub> (%)	γ <sub>max</sub> (%)	d <sub>z</sub> (ft)	LDI	LD (ft)
5.00	21	64.16	0.00	5.00	0.000	0.00
10.00	17	57.72	0.00	5.00	0.000	0.00
15.00	17	57.72	0.00	5.00	0.000	0.00
20.00	15	54.22	0.00	5.00	0.000	0.00
25.00	23	67.14	0.00	5.00	0.000	0.00
30.00	34	81.63	0.00	5.00	0.000	0.00
35.00	83	100.00	0.00	5.00	0.000	0.00
40.00	26	71.39	0.00	5.00	0.000	0.00
45.00	47	100.00	0.00	5.00	0.000	0.00
50.00	47	100.00	0.00	5.00	0.000	0.00

**Cumulative lateral displacements: 0.00**

**Abbreviations**

- D<sub>r</sub>: Relative density (%)
- γ<sub>max</sub>: Maximum amplitude of cyclic shear strain (%)
- d<sub>z</sub>: Soil layer thickness (ft)
- LDI: Lateral displacement index (ft)
- LD: Actual estimated displacement (ft)

## SPT BASED LIQUEFACTION ANALYSIS REPORT

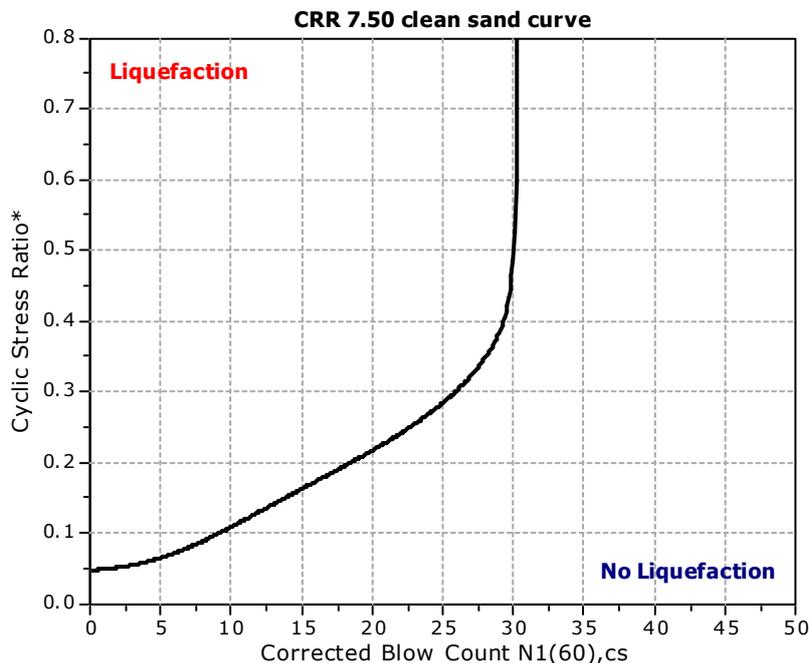
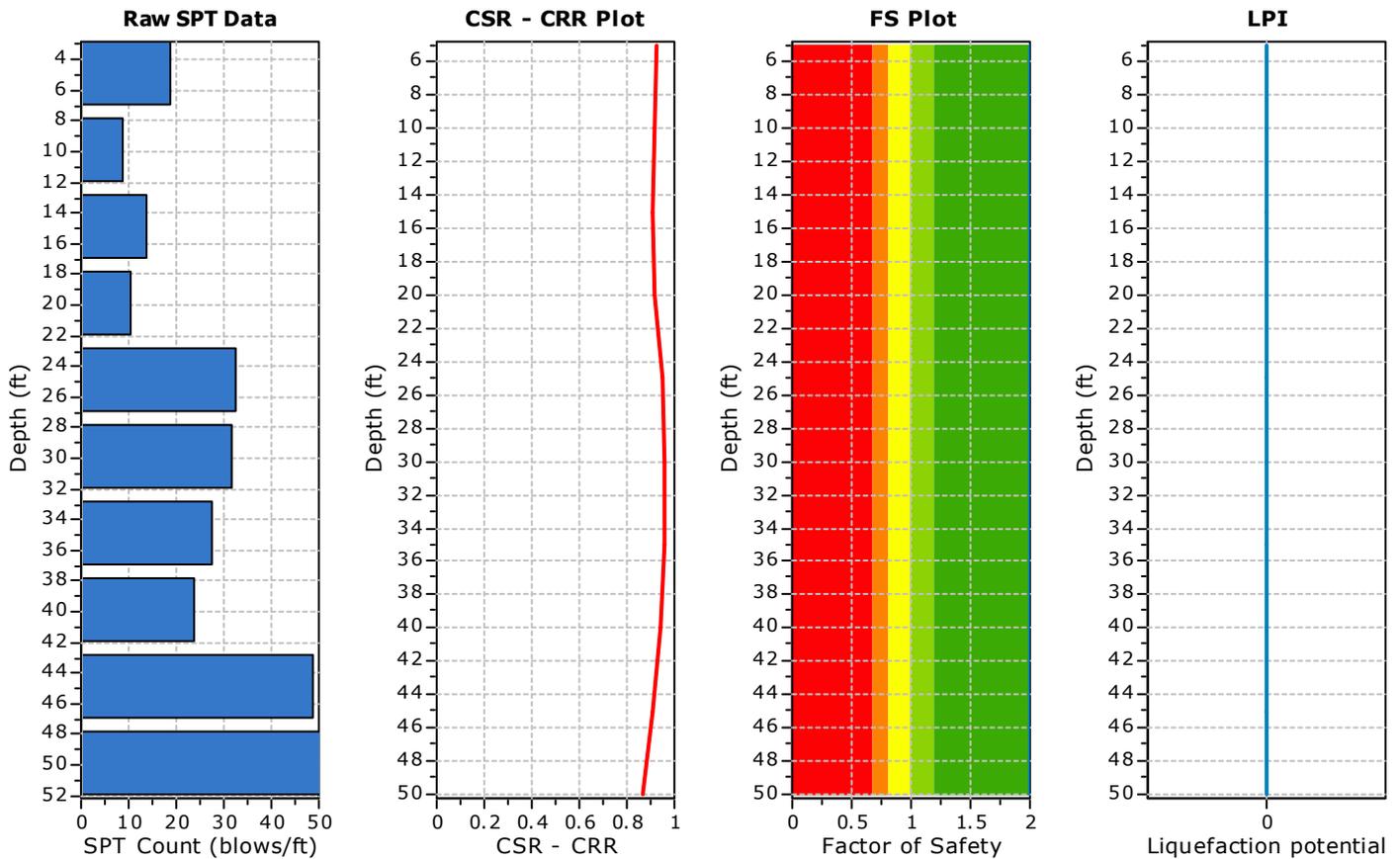
**Project title : Kaiser Redlands MOB 2**

**SPT Name: B-12**

**Location : NW corner of California St & Lugonia Ave, Redlands, CA**

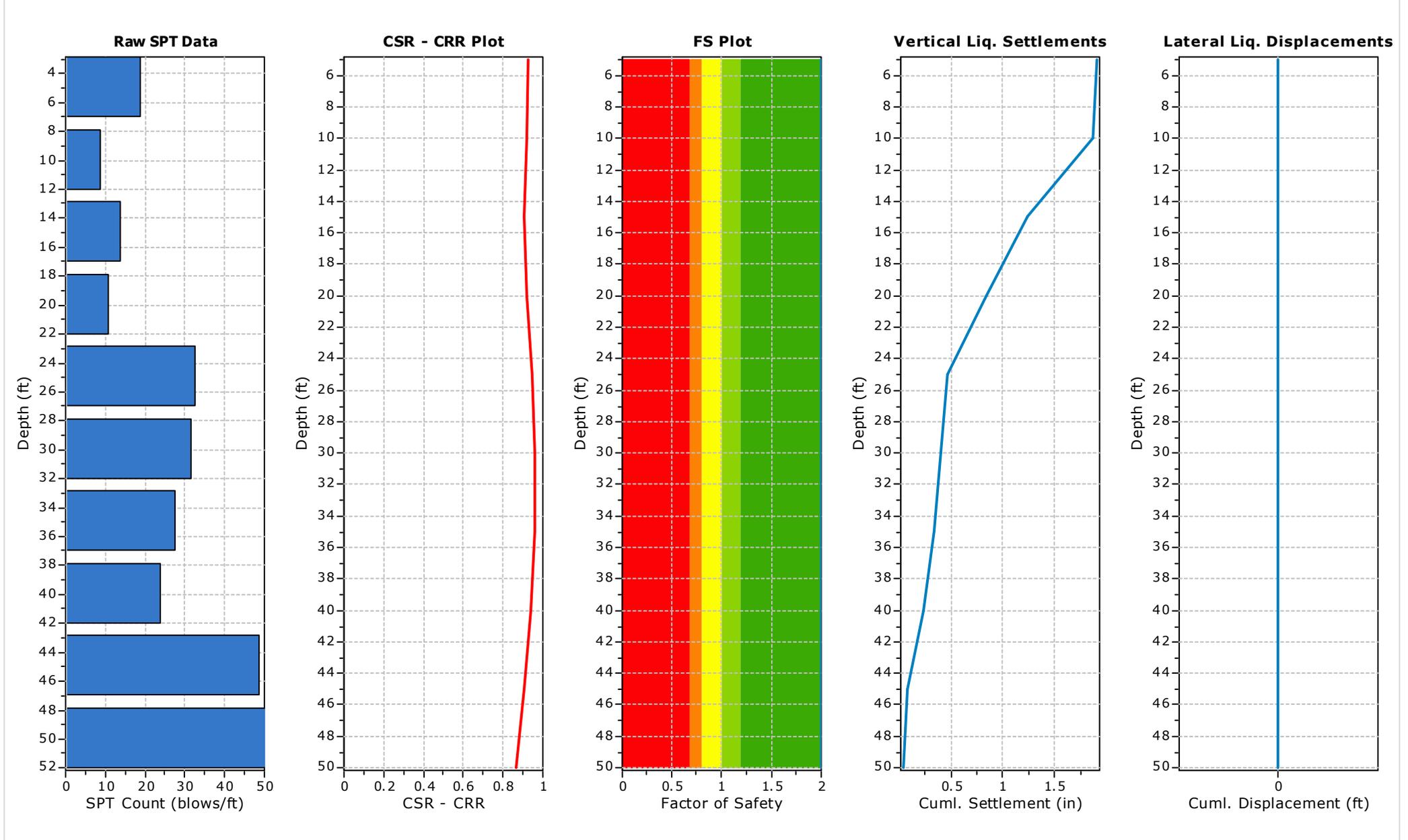
**:: Input parameters and analysis properties ::**

Analysis method:	NCEER 1998	G.W.T. (in-situ):	60.00 ft
Fines correction method:	NCEER 1998	G.W.T. (earthq.):	60.00 ft
Sampling method:	Standard Sampler	Earthquake magnitude $M_w$ :	8.10
Borehole diameter:	200mm	Peak ground acceleration:	0.91 g
Rod length:	3.28 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.30		



- F.S. color scheme**
- Almost certain it will liquefy
  - Very likely to liquefy
  - Liquefaction and no liq. are equally likely
  - Unlike to liquefy
  - Almost certain it will not liquefy
- LPI color scheme**
- Very high risk
  - High risk
  - Low risk

**:: Overall Liquefaction Assessment Analysis Plots ::**



:: Field input data ::					
Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
5.00	19	48.80	120.00	5.00	Yes
10.00	9	13.70	120.00	5.00	Yes
15.00	14	13.70	120.00	5.00	Yes
20.00	11	75.60	120.00	5.00	Yes
25.00	33	75.60	120.00	5.00	Yes
30.00	32	75.60	120.00	5.00	Yes
35.00	28	75.60	120.00	5.00	Yes
40.00	24	47.60	120.00	5.00	Yes
45.00	49	55.90	120.00	5.00	Yes
50.00	61	15.60	120.00	5.00	Yes

**Abbreviations**

Depth: Depth at which test was performed (ft)  
 SPT Field Value: Number of blows per foot  
 Fines Content: Fines content at test depth (%)  
 Unit Weight: Unit weight at test depth (pcf)  
 Infl. Thickness: Thickness of the soil layer to be considered in settlements analysis (ft)  
 Can Liquefy: User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::																
Depth (ft)	SPT Field Value	Unit Weight (pcf)	$\sigma_v$ (tsf)	$u_o$ (tsf)	$\sigma'_{vo}$ (tsf)	$C_N$	$C_E$	$C_B$	$C_R$	$C_S$	$(N_1)_{60}$	Fines Content (%)	$\alpha$	$\beta$	$(N_1)_{60cs}$	CRR <sub>7.5</sub>
5.00	19	120.00	0.30	0.00	0.30	1.70	1.30	1.15	0.75	1.00	36	48.80	5.00	1.20	48	4.000
10.00	9	120.00	0.60	0.00	0.60	1.33	1.30	1.15	0.85	1.00	15	13.70	2.11	1.04	18	4.000
15.00	14	120.00	0.90	0.00	0.90	1.08	1.30	1.15	0.85	1.00	19	13.70	2.11	1.04	22	4.000
20.00	11	120.00	1.20	0.00	1.20	0.94	1.30	1.15	0.95	1.00	15	75.60	5.00	1.20	23	4.000
25.00	33	120.00	1.50	0.00	1.50	0.84	1.30	1.15	0.95	1.00	39	75.60	5.00	1.20	52	4.000
30.00	32	120.00	1.80	0.00	1.80	0.77	1.30	1.15	1.00	1.00	37	75.60	5.00	1.20	49	4.000
35.00	28	120.00	2.10	0.00	2.10	0.71	1.30	1.15	1.00	1.00	30	75.60	5.00	1.20	41	4.000
40.00	24	120.00	2.40	0.00	2.40	0.66	1.30	1.15	1.00	1.00	24	47.60	5.00	1.20	34	4.000
45.00	49	120.00	2.70	0.00	2.70	0.63	1.30	1.15	1.00	1.00	46	55.90	5.00	1.20	60	4.000
50.00	61	120.00	3.00	0.00	3.00	0.59	1.30	1.15	1.00	1.00	54	15.60	2.66	1.05	59	4.000

**Abbreviations**

$\sigma_v$ : Total stress during SPT test (tsf)  
 $u_o$ : Water pore pressure during SPT test (tsf)  
 $\sigma'_{vo}$ : Effective overburden pressure during SPT test (tsf)  
 $C_N$ : Overburden correction factor  
 $C_E$ : Energy correction factor  
 $C_B$ : Borehole diameter correction factor  
 $C_R$ : Rod length correction factor  
 $C_S$ : Liner correction factor  
 $N_{1(60)}$ : Corrected  $N_{SPT}$  to a 60% energy ratio  
 $\alpha, \beta$ : Clean sand equivalent clean sand formula coefficients  
 $N_{1(60)cs}$ : Corrected  $N_{1(60)}$  value for fines content  
 CRR<sub>7.5</sub>: Cyclic resistance ratio for M=7.5

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::													
Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	$r_d$	$\alpha$	CSR	MSF	CSR <sub>eq, M=7.5</sub>	$K_{\sigma}$	CSR*	FS	
5.00	120.00	0.30	0.00	0.30	0.99	1.00	0.586	0.82	0.714	1.00	1.206	2.000	●
10.00	120.00	0.60	0.00	0.60	0.98	1.00	0.579	0.82	0.706	1.00	1.192	2.000	●

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::													
Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	$r_d$	$\alpha$	CSR	MSF	$CSR_{eq,M=7.5}$	$K_{\sigma}$	CSR*	FS	
15.00	120.00	0.90	0.00	0.90	0.97	1.00	0.573	0.82	0.698	1.00	1.179	2.000	●
20.00	120.00	1.20	0.00	1.20	0.96	1.00	0.566	0.82	0.690	0.98	1.195	2.000	●
25.00	120.00	1.50	0.00	1.50	0.94	1.00	0.557	0.82	0.679	0.93	1.230	2.000	●
30.00	120.00	1.80	0.00	1.80	0.92	1.00	0.545	0.82	0.663	0.90	1.247	2.000	●
35.00	120.00	2.10	0.00	2.10	0.89	1.00	0.527	0.82	0.642	0.87	1.244	2.000	●
40.00	120.00	2.40	0.00	2.40	0.85	1.00	0.503	0.82	0.613	0.85	1.221	2.000	●
45.00	120.00	2.70	0.00	2.70	0.80	1.00	0.475	0.82	0.579	0.83	1.180	2.000	●
50.00	120.00	3.00	0.00	3.00	0.75	1.00	0.445	0.82	0.542	0.81	1.129	2.000	●

**Abbreviations**

- $\sigma_{v,eq}$ : Total overburden pressure at test point, during earthquake (tsf)
- $u_{o,eq}$ : Water pressure at test point, during earthquake (tsf)
- $\sigma'_{vo,eq}$ : Effective overburden pressure, during earthquake (tsf)
- $r_d$ : Nonlinear shear mass factor
- $\alpha$ : Improvement factor due to stone columns
- CSR: Cyclic Stress Ratio (adjusted for improvement)
- MSF: Magnitude Scaling Factor
- $CSR_{eq,M=7.5}$ : CSR adjusted for M=7.5
- $K_{\sigma}$ : Effective overburden stress factor
- CSR\*: CSR fully adjusted (user FS applied)\*\*\*
- FS: Calculated factor of safety against soil liquefaction

\*\*\* User FS: 1.30

:: Liquefaction potential according to Iwasaki ::					
Depth (ft)	FS	F	wz	Thickness (ft)	$I_L$
5.00	2.000	0.00	9.24	5.00	0.00
10.00	2.000	0.00	8.48	5.00	0.00
15.00	2.000	0.00	7.71	5.00	0.00
20.00	2.000	0.00	6.95	5.00	0.00
25.00	2.000	0.00	6.19	5.00	0.00
30.00	2.000	0.00	5.43	5.00	0.00
35.00	2.000	0.00	4.67	5.00	0.00
40.00	2.000	0.00	3.90	5.00	0.00
45.00	2.000	0.00	3.14	5.00	0.00
50.00	2.000	0.00	2.38	5.00	0.00

**Overall potential  $I_L$ : 0.00**

- $I_L = 0.00$  - No liquefaction
- $I_L$  between 0.00 and 5 - Liquefaction not probable
- $I_L$  between 5 and 15 - Liquefaction probable
- $I_L > 15$  - Liquefaction certain

:: Vertical settlements estimation for dry sands ::												
Depth (ft)	$(N_1)_{60}$	$\tau_{av}$	p	$G_{max}$ (tsf)	$\alpha$	b	$\gamma$	$\epsilon_{15}$	$N_c$	$\epsilon_{Nc}$ (%)	$\Delta h$ (ft)	$\Delta S$ (in)
5.00	36	0.18	0.20	728.32	0.14	13179.75	0.00	0.00	21.37	0.04	5.00	0.045
10.00	15	0.35	0.40	742.75	0.15	8695.39	0.00	0.00	21.37	0.52	5.00	0.626
15.00	19	0.52	0.60	972.61	0.16	6817.65	0.00	0.00	21.37	0.33	5.00	0.396
20.00	15	0.68	0.80	1139.84	0.17	5736.82	0.00	0.00	21.37	0.31	5.00	0.376
25.00	39	0.84	1.00	1672.60	0.18	5017.94	0.00	0.00	21.37	0.05	5.00	0.061

:: Vertical settlements estimation for dry sands ::												
Depth (ft)	(N <sub>1</sub> ) <sub>60</sub>	T <sub>av</sub>	p	G <sub>max</sub> (tsf)	a	b	γ	ε <sub>15</sub>	N <sub>c</sub>	ε <sub>N<sub>c</sub></sub> (%)	Δh (ft)	ΔS (in)
30.00	37	0.98	1.21	1796.30	0.19	4497.97	0.00	0.00	21.37	0.06	5.00	0.071
35.00	30	1.11	1.41	1828.31	0.21	4100.61	0.00	0.00	21.37	0.09	5.00	0.103
40.00	24	1.21	1.61	1836.30	0.22	3784.89	0.00	0.00	21.37	0.12	5.00	0.146
45.00	46	1.28	1.81	2353.66	0.23	3526.64	0.00	0.00	21.37	0.04	5.00	0.043
50.00	54	1.34	2.01	2467.12	0.24	3310.60	0.00	0.00	21.37	0.03	5.00	0.041

**Cumulative settlements: 1.908**

**Abbreviations**

- T<sub>av</sub>: Average cyclic shear stress
- p: Average stress
- G<sub>max</sub>: Maximum shear modulus (tsf)
- a, b: Shear strain formula variables
- γ: Average shear strain
- ε<sub>15</sub>: Volumetric strain after 15 cycles
- N<sub>c</sub>: Number of cycles
- ε<sub>N<sub>c</sub></sub>: Volumetric strain for number of cycles N<sub>c</sub> (%)
- Δh: Thickness of soil layer (in)
- ΔS: Settlement of soil layer (in)

:: Lateral displacements estimation for saturated sands ::						
Depth (ft)	(N <sub>1</sub> ) <sub>60</sub>	D <sub>r</sub> (%)	γ <sub>max</sub> (%)	d <sub>z</sub> (ft)	LDI	LD (ft)
5.00	36	84.00	0.00	5.00	0.000	0.00
10.00	15	54.22	0.00	5.00	0.000	0.00
15.00	19	61.02	0.00	5.00	0.000	0.00
20.00	15	54.22	0.00	5.00	0.000	0.00
25.00	39	87.43	0.00	5.00	0.000	0.00
30.00	37	85.16	0.00	5.00	0.000	0.00
35.00	30	76.68	0.00	5.00	0.000	0.00
40.00	24	68.59	0.00	5.00	0.000	0.00
45.00	46	100.00	0.00	5.00	0.000	0.00
50.00	54	100.00	0.00	5.00	0.000	0.00

**Cumulative lateral displacements: 0.00**

**Abbreviations**

- D<sub>r</sub>: Relative density (%)
- γ<sub>max</sub>: Maximum amplitude of cyclic shear strain (%)
- d<sub>z</sub>: Soil layer thickness (ft)
- LDI: Lateral displacement index (ft)
- LD: Actual estimated displacement (ft)

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