ROCKRIDGE
GEOTECHNICAL

Prepared for **AE3 Architecture**

GEOTECHNICAL INVESTIGATION
PROPOSED TANK REMOVAL AND REPLACEMENT
WILEY W. MANUEL COURTHOUSE
661 WASHINGTON STREET
OAKLAND, CALIFORNIA

UNAUTHORIZED USE OR COPYING OF THIS DOCUMENT IS STRICTLY PROHIBITED BY ANYONE OTHER THAN THE CLIENT FOR THE SPECIFIC PROJECT

July 15, 2024
Project No. 24-2647

ROCKRIDGE
GEOTECHNICAL

July 15, 2024
Project No. 24-2647

Anna Dunnigan
Senior Project Manager/Architect
AE3 Architecture
315 Montgomery Street #1000
San Francisco, California 94104

Subject: Geotechnical Investigation
Proposed Tank Removal and Replacement
Wiley W. Manuel Courthouse
661 Washington Street
Oakland, California

Dear Ms. Dunnigan:

We are pleased to present the results of our geotechnical investigation for the proposed underground storage tank removal and below-grade storage tank installation at the Wiley W. Manuel Courthouse located at 661 Washington Street in Oakland, California. Our investigation was performed in accordance with our contract dated May 22, 2024.

The existing tanks are both 10,000-gallon underground storage tanks (USTs) adjacent to mechanical shafts in a landscaped area on the western side of the courthouse. The mechanical shafts extend to a depth of about 15 feet below sidewalk grade and are at the same elevation as the courthouse basement finished floor.

Plans are to remove the two existing USTs, backfill the excavation, and then install a fuel storage tank that will be in a concrete vault with a bottom-of-mat elevation about 6 feet below sidewalk grade and 9 feet above the mechanical shaft floor slab. The vault will have plan dimensions of 16.5 by 45.7 feet and will be accessed by new stairs on the southern side of the vault. The tank will weigh about 128,000 pounds when full.

Based on the results of our geotechnical investigation, we conclude there are no major geotechnical issues that would preclude development of the site as proposed. The primary geotechnical concerns at the site are: 1) the presence of loose to medium dense sand beneath the proposed tank location that is susceptible to cyclic densification above the groundwater table and liquefaction below the groundwater table, and 2) providing adequate vertical and lateral support for the proposed tank and excavation. We conclude the new tank can be supported on a mat foundation bearing on engineered fill that extend to a depth of 20 feet below existing grade.

The recommendations contained in our report are based on limited subsurface exploration. Consequently, variations between expected and actual subsurface conditions may be encountered during construction. Therefore, we should be engaged to observe site preparation, shoring installation, and foundation installation, during which time we may make changes to our recommendations if deemed necessary.

We appreciate the opportunity to provide our services to you on this project. Should you have any questions, please call.

Sincerely,
ROCKRIDGE GEOTECHNICAL, INC.

REGISTERED PROFESSIONAL ENGINEER
ALEXANDRA D. LIMPERT
C 93481
CIVIL
STATE OF CALIFORNIA
Date: 7/15/2024

REGISTERED PROFESSIONAL ENGINEER
CRAIG S. SHIELDS
GE 2116
GEOTECHNICAL
STATE OF CALIFORNIA
Date: 7/15/2024

Alex D. Limpert, P.E.
Project Engineer

Craig S. Shields, P.E., G.E.
Principal Engineer

Enclosure

QUALITY CONTROL REVIEWER

Linda H. J. Liang, P.E., G.E.
Principal Engineer

TABLE OF CONTENTS

LIST OF FIGURES

APPENDIX A

APPENDIX B

GEOTECHNICAL INVESTIGATION
PROPOSED TANK REMOVAL AND REPLACEMENT
WILEY W. MANUEL COURTHOUSE
661 WASHINGTON STREET
OAKLAND, CALIFORNIA

1.0 INTRODUCTION

This report presents the results of the geotechnical investigation performed by Rockridge Geotechnical, Inc. for the proposed removal of two underground storage tanks (USTs) and installation of a new below-grade storage tank adjacent to the Wiley W. Manuel Courthouse at 661 Washington Street in Oakland, California. The existing USTs are located in a courtyard between the courthouse and detention facility along 6th Street, as shown in the Site Location Map (Figure 1).

Based on review of the 80% Design Development drawings[1] for the project, the existing tanks are 10,000-gallon USTs adjacent to mechanical shafts in a landscaped area on the western side of the courthouse, as shown in the Site Plan (Figure 2). The mechanical shafts extend to a depth of about 15 feet below sidewalk grade and are at the same elevation as the courthouse basement finished floor.

Plans are to remove the two USTs, backfill the excavation, and then install a fuel storage tank that will be in a concrete vault with a bottom-of-mat elevation about 6 feet below sidewalk grade and 9 feet above the mechanical shaft floor slab. The vault will have plan dimensions of 16.5 feet by 45.7 feet and will be accessed by new stairs on the southern side of the vault. The tank will weigh about 128,000 pounds when full.

2.0 SCOPE OF SERVICES

Our investigation was performed in accordance with our contract dated May 24, 2024. Our scope of services consisted of exploring subsurface conditions at the site and performing engineering analyses to develop conclusions and recommendations regarding:

- soil and groundwater conditions at the site

---

[1] Drawings prepared by AE3 Architecture and dated April 26, 2024.

- site seismicity and seismic hazards
- the most appropriate foundation type(s) for the proposed new storage tank
- design criteria for the recommended foundation type, including vertical and lateral capacities
- estimates of foundation settlement
- lateral earth pressures for design of the below-grade vault walls
- subgrade preparation for new stairs and exterior concrete slabs-on-grade
- site grading and excavation, including criteria for fill quality and compaction, including criteria for backfill type and compaction for fill placed in the excavations following removal of the existing tanks
- 2022 California Building Code (CBC) site class and design spectral response acceleration parameters
- shoring design parameters
- corrosivity of the near-surface soil and the potential effects on buried concrete and metal structures and foundations
- construction considerations.

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

We explored the subsurface conditions at the site by drilling two test borings. Before performing the field investigation, we obtained a drilling permit from Alameda County Public Works Agency (ACPWA). We also contacted Underground Services Alert (USA) to notify them of our work, as required by law, and retained C. Cruz Sub-Surface Locators of Milpitas, California, a private utility locator, to check the boring locations to reduce the potential for encountering buried utilities during our field investigation. Details of our field investigation and laboratory testing are presented in this section.

3.1 Test Borings

Two test borings, designated as B-1 and B-2, were drilled on June 7, 2024 by Stapleton Engineering and Exploration of Santa Rosa, California at the approximate locations shown in Figure 2. The borings were drilled using hydraulic drilling equipment mounted on a skid steer equipped with 4-inch-diameter solid-stem flight augers. The borings were each drilled to a depth of about 21.5 feet below the ground surface (bgs).

During drilling, our field engineer logged the soil encountered and obtained representative samples for visual classification and laboratory testing. The boring logs are presented in Figures A-1 and A-2 in Appendix A. The soil encountered in the borings was classified in accordance with the classification system presented in Figure A-3.

Soil samples were obtained using:

- Modified California (MC) split-barrel sampler with a 3.0-inch outside diameter and 2.5-inch inside diameter, lined with 2.43-inch inside diameter stainless steel tubes.
- Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside and 1.5-inch inside diameter; the sampler was designed to accommodate liners, but liners were not used.

The samplers were driven with a 140-pound rope-and-cathead safety hammer falling about 30 inches per drop. The samplers were driven up to 18 inches, and the hammer blows required to drive the samplers were recorded every 6 inches and are presented in the boring logs. A “blow count” is defined as the number of hammer blows per 6 inches of penetration or 50 blows for 6 inches or less of penetration. The blow counts required to drive the MC and SPT samplers were converted to approximate SPT N-values using factors of 0.7 and 1.2, respectively, to account for sampler type, approximate hammer energy, and the fact that the SPT sampler was designed to accommodate liners, but liners were not used. The blow counts used for this conversion were the last two blow counts. The converted SPT N-values are presented in the boring logs.

Upon completion of drilling, the boreholes were backfilled with neat cement grout in accordance with ACPWA requirements. The soil cuttings were removed from the site and disposed of as clean fill.

3.2 Laboratory Testing

We re-examined the soil samples obtained from our boring to confirm the field classifications and selected representative samples for laboratory testing. Laboratory tests were performed by B. Hillebrandt Soils Testing, Inc. of Alamo, California to measure moisture content, plasticity (Atterberg limits), and particle size distribution. Laboratory tests were performed by Project X Corrosion Engineering of Murrieta, California on one soil sample to provide data for evaluating the soil corrosivity. The results of the laboratory tests are presented in the boring logs and in Appendix B.

4.0 SUBSURFACE CONDITIONS

Regional geologic information (Figure 3) indicates the site is underlain by Holocene- and Pleistocene-age Merritt sand (Qms). Results from our borings indicate the site is underlain by clayey sand that extends to the maximum depth explored of 21.5 feet bgs. The clayey sand is loose to medium dense to a depth of about 17 feet bgs and contains traces of gravel, brick debris, and roots. Based on the SPT N-values, we believe the soil above depths of 14.5 feet and 17 feet bgs in Borings B-1 and B-2, respectively, is fill placed during the construction of the mechanical shafts and basement for the adjacent courthouse building. The fill is underlain by loose to medium dense clayey sand that extends to a depth of 20 feet bgs. Below a depth of 20 feet, the clayey sand becomes very dense.

Groundwater was initially encountered in both borings at a depth of 21 feet bgs. The boreholes were left open for about 10 to 15 minutes after drilling and the groundwater rose to 20 feet bgs before grouting. Available historic groundwater information presented in the *Seismic Hazard Zone Report for the Oakland West 7.5-Minute Quadrangle* prepared by the California Geological Survey (CGS, 2003) indicates a historic high groundwater level at the site vicinity of about 15 feet bgs.

The depth to groundwater is expected to vary several feet seasonally, depending on rainfall amounts. Based on available groundwater information presented in this section, we recommend a design groundwater table of 15 feet bgs be used for this project.

5.0 SEISMIC CONSIDERATIONS

5.1 Regional Seismicity

The site is located within the Coast Ranges Geomorphic Province of California, which is characterized by northwest-trending valleys and ridges. These topographic features are controlled by folds and faults that resulted from the collision of the Farallon and North American plates and subsequent strike-slip faulting along the San Andreas Fault system. The San Andreas Fault is more than 600 miles long and extends from Point Arena in the north to the Gulf of California in the south. The Coast Ranges Geomorphic Province is bounded on the east by the Great Valley and on the west by the Pacific Ocean.

The major active faults in the area are the Hayward, San Andreas, and Calaveras faults. These and other faults in the region are shown in Figure 4. For these and other active faults within a 50-kilometer radius of the site, the distance from the site and estimated characteristic moment magnitude[2] [Petersen et al. (2014) & Thompson et al. (2016)] are summarized in Table 1. These references are based on the Third Uniform California Earthquake Rupture Forecast (UCERF3), prepared by Field et al. (2013).

TABLE 1
Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Characteristic Moment Magnitude
Total Hayward + Rodgers Creek (RC+HN+HS+HE)	6.4	East	7.58
Hayward (North, HN)	6.4	East	6.90
Hayward (South, HS)	8.9	East	7.00
Total Calaveras (CN+CC+CS+CE)	21	East	7.43
Calaveras (North, CN)	21	East	6.86
Mount Diablo Thrust	23	East	6.67
Mount Diablo Thrust North CFM	23	East	6.72
Total North San Andreas (SAO+SAN+SAP+SAS)	24	Southwest	8.04
North San Andreas (Peninsula, SAP)	24	Southwest	7.38
Concord	28	East	6.45
San Gregorio (North)	30	West	7.44
Green Valley	32	Northeast	6.30
Mount Diablo Thrust South	33	East	6.50
Clayton	34	Northeast	6.57
North San Andreas (North Coast, SAN)	36	West	7.52

---

[2] Moment magnitude (M_w) is an energy-based scale and provides a physically meaningful measure of the size of a faulting event. Moment magnitude is directly related to average slip and fault rupture area.

Fault Segment	**Approximate Distance from Site (km)**	**Direction from Site**	**Characteristic Moment Magnitude**
Monte Vista - Shannon	36	South	7.14
Greenville (North)	36	East	6.86
West Napa	41	North	6.97
Great Valley 05 (Pittsburg - Kirby Hills alt1)	42	Northeast	6.60
Rodgers Creek - Healdsburg	43	North	7.19
Great Valley 05 (Pittsburg - Kirby Hills alt2)	45	East	6.66
Las Positas	46	East	6.50

Damaging earthquakes have occurred along many of these faults in recorded history, as depicted in Figure 4 (USGS, 2021). Notable historic earthquakes which have impacted the Bay Area in recorded history include:

- 1838 San Andreas Earthquake, $M_w = 7.4$ (estimated)
- 1865 San Andreas Earthquake, $M_w = 6.5$ (estimated)
- 1868 Hayward Earthquake, $M_w = 7.0$ (estimated)
- 1906 Great San Francisco Earthquake (San Andreas Fault), $M_w = 7.9$ (estimated)
- 1989 Loma Prieta Earthquake (San Andreas Fault), $M_w = 6.9$
- 2014 West Napa Earthquake, $M_w = 6.0$

As a part of the UCERF3 project, researchers estimated that the probability of at least one $M_w \geq 6.7$ earthquake occurring in the greater San Francisco Bay Area during a 30-year period (starting in 2014) is 72 percent. The highest probabilities are assigned to sections of the Hayward (South), Calaveras (Central), and San Andreas (Santa Cruz Mountains) faults. The respective probabilities are approximately 25, 21, and 17 percent.

5.2 Geologic Hazards

Because the site is in a seismically active region, we evaluated the potential for earthquake-induced geologic hazards including ground shaking, ground surface rupture, liquefaction,[3] lateral spreading,[4] and cyclic densification.[5] We used the results of our investigation to evaluate the potential of these phenomena occurring at the site.

5.2.1 Ground Shaking

The seismicity of the site is governed by the activity of the Hayward Fault, although ground shaking from future earthquakes on other faults, including the Calaveras and San Andreas faults, will also be felt at the site. The intensity of earthquake ground motion at the site will depend upon the characteristics of the generating fault, distance to the earthquake epicenter, and magnitude and duration of the earthquake. We judge strong to very strong ground shaking could occur at the site during a large earthquake on one of the nearby faults.

5.2.2 Ground Surface Rupture

Historically, ground surface displacements closely follow the trace of geologically young faults. The site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act, and no known active or potentially active faults exist on the site. Therefore, we conclude the probability of fault offset at the site from a known active fault to be very low. In a seismically active area, the remote possibility exists for future faulting in areas where no faults previously existed; however, we conclude the probability of surface faulting, and consequently secondary ground failure from previously unknown faults, is very low.

---

[3] Liquefaction is a phenomenon where loose, saturated, cohesionless soil experiences temporary reduction in strength during cyclic loading such as that produced by earthquakes.

[4] Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported downslope or in the direction of a free face by earthquake and gravitational forces.

[5] Cyclic densification is a phenomenon in which non-saturated, cohesionless soil is compacted by earthquake vibrations, causing ground-surface settlement.

5.2.3 Liquefaction and Liquefaction-Induced Settlement

When saturated, cohesionless soil liquefies, it experiences a temporary loss of shear strength created by a transient rise in excess pore pressure generated by a strong ground motion. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits. Flow failure, lateral spreading, differential settlement, loss of bearing strength, ground fissures, and sand boils are evidence of excess pore pressure generation and liquefaction.

The site has **not** been mapped within a zone of liquefaction potential as shown on the map titled *State of California Earthquake Zones of Required Investigation, Oakland West Quadrangle, California Geological Survey, Official Map*, dated February 14, 2003, as shown in Figure 5. We encountered loose to medium dense clayey sand in our borings, likely from excavation of the basement for the adjacent courthouse building. We evaluated the liquefaction potential of soil encountered below the groundwater table at the site using data collected from our boring and the methodology proposed by Youd et al. (2001).

Our liquefaction analyses were performed using a high groundwater depth of 15 feet bgs. In accordance with the 2022 CBC, we used a peak ground acceleration of 0.76 times gravity (g) in our liquefaction evaluation; this peak ground acceleration is consistent with the Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration adjusted for site effects (PGA_M). We also used a moment magnitude 7.58 earthquake, which is consistent with the characteristic moment magnitude for the Hayward Fault, as presented in Table 1.

Our liquefaction analyses indicate the loose to medium dense clayey sand present between depths of about 15 to 20 feet bgs is susceptible to liquefaction during a major earthquake. Based on the results of our analyses, we estimate total “free-field” ground settlement associated with liquefaction after an MCE event generating a PGA_M of 0.76g will be up to about 1-1/2 inches. We estimate liquefaction-induced differential settlement will be less than 3/4 inch over a horizontal distance of 30 feet. To mitigate the potential for liquefaction, it would be necessary to overexcavate and recompact the soil to a depth of up to 20 feet bgs.

5.2.4 Cyclic Densification

Cyclic densification (also referred to as differential compaction) of non-saturated sand (sand above the groundwater table) can occur during an earthquake, resulting in settlement of the ground surface and overlying improvements. The site is underlain by loose to medium dense clayey sand, which is likely fill placed during construction of the basement for the adjacent courthouse building.

We evaluated the cyclic densification potential of soil encountered at the site using data collected from our borings using the methodology developed by Pradel (1998). Using the earthquake parameters as discussed in Section 5.2.3, we estimate settlement due to cyclic densification of the loose to medium dense clayey sand between the bottom of the proposed tank foundation and the design groundwater table could be up to 1-1/2 inches. The potential for cyclic densification can be mitigated by overexcavating and recompacting the soil below the proposed tank mat foundation to a depth of 15 feet below existing grade.

6.0 DISCUSSION AND CONCLUSIONS

From a geotechnical standpoint, we conclude the site can be developed as planned, provided the recommendations presented in this report are incorporated into the project plans and specifications and implemented during construction. The primary geotechnical concerns at the site are: 1) the presence of loose to medium dense sand beneath the proposed tank location that is susceptible to cyclic densification and liquefaction, and 2) providing adequate vertical and lateral support for the proposed tank and excavation. These and other geotechnical issues are discussed in this section.

6.1 Foundations and Settlement

The factors influencing the selection of a safe, economical foundation system are providing an adequate factor of safety against bearing capacity failure, limiting differential settlement to an amount that can be tolerated by the improvements above, constructability, and cost. The tank location is underlain by soil that is susceptible to cyclic densification above the design groundwater table (15 feet bgs) and soil below the design groundwater table that is susceptible to liquefaction. We estimate seismically induced settlements resulting from a combination of cyclic densification and liquefaction during an MCE event will be on the order of 2 inches under current conditions. We conclude the new tank may be supported on a mat foundation provided the potential settlement from a combination of static and seismic loading is reduced to a value that can be tolerated by the mat foundation. In addition, the potential differential settlement between the tank and the building following a major earthquake should be acceptable to the design team.

To address these issues, we recommend the soil below the tank, extending to 15 feet below existing grade, be overexcavated and recompacted. This depth of overexcavation will reduce to static settlement of the tank to less than 1/4 inch and potentially seismically induced settlement at the Boring B-1 location (northern side of proposed tank) to less than 1/4 inch; however, because potentially liquefiable soil was encountered to a depth of about 20 feet bgs at the Boring B-2 location, up to 1-1/2 inches of seismically induced settlement could occur beneath the southern end of the tank following a major earthquake. Mitigation of the potential for seismically induced settlement beneath the entire tank footprint would require an excavation to 20 feet below existing grade.

If the proposed tank will be supported on a mat bearing on engineered fill that extend to a depth of 20 feet below the existing grade and prepared following the recommendations presented in Section 7.1.1, we estimate total and differential static settlement of the mat will be less than 3/4 inch and 1/2 inch across a horizontal distance of 30 feet, respectively. We also estimate seismically induced settlement will be less than 1/4 inch.

6.2 Excavation and Construction Considerations

The soil to be excavated to remove the existing tanks and associated utilities is expected to consist predominantly of clayey sand which can be excavated with conventional earth-moving equipment such as backhoes. Excavations that will be entered by workers should be sloped or shored in accordance with the Occupational Safety and Health Administration (OSHA) standards (29 CFR Part 1926). Where space permits, the sides of the temporary excavation can be sloped. Where space does not permit sloping of the excavation perimeter, a shoring system will be required to support the sides of the proposed excavation. Considering the depth of the excavation, we believe a soldier-pile-and-lagging shoring system with internal bracing would be a suitable shoring system to retain the sides of the proposed excavation. The contractor should be responsible for the construction and safety of temporary slopes. The shoring designer should be responsible for the shoring design.

6.3 Soil Corrosivity

Corrosivity tests were performed by Project X Corrosion Engineering of Murrieta, California on a soil sample obtained from B-2 at a depth of 11 feet bgs. The corrosivity test results are presented in Appendix B.

Many factors can affect the corrosion potential of soil including, but not limited to, resistivity, pH, and chloride and sulfate concentrations. Based on the minimum soil resistivity measurements of 5,762 ohm-cm, we conclude the soil is “moderately corrosive” to buried metal (Roberge, 2018). Accordingly, all buried iron, steel, cast iron, galvanized steel, and dielectric-coated steel or iron should be protected against corrosion depending upon the critical nature of the structure. If it is necessary to have metal in contact with soil, a corrosion engineer should be consulted to provide recommendations for corrosion protection.

The results of the pH test (7.6) indicate the near-surface soil is “negligibly corrosive” to buried metallic and concrete structures. The chloride ion concentration (60.3 mg/kg) indicates the chlorides in the near-surface soil are “negligibly corrosive” to buried metallic structures and reinforcing steel in concrete structures below ground. The results also indicate the sulfate ion concentration (65.3 mg/kg) is sufficiently low such that sulfates do not pose a threat to buried concrete and mortars.

7.0 RECOMMENDATIONS

Our recommendations for site preparation and grading, foundation design, temporary cuts and shoring, and other geotechnical aspects of the project are presented in this section.

7.1 Site Preparation, Grading, and Fill Placement

Site demolition should include the removal of existing foundations, tanks, and underground utilities, if any, where the new tank is proposed. In general, abandoned underground utilities should be removed to the property line or service connections and properly capped or plugged with concrete. Where existing utility lines are outside the proposed tank footprint and will not interfere with the proposed construction, they may be abandoned in place provided the lines are filled with lean concrete or cement grout to the property line. Voids resulting from demolition activities should be properly backfilled with compacted fill following the recommendations provided later in this section.

7.1.1 Subgrade Preparation

Tank Pad

After demolition of the existing tanks, the tank pad footprint should be overexcavated to a depth of 20 feet below existing grade. The overexcavation should extend at least 3 feet beyond the perimeter of the proposed tank except where constrained by existing utilities or adjacent to the courthouse. The excavation subgrade should be compacted with 2 to 3 passes prior to placing backfill. The compaction effort should cease if excessive deflection of the subgrade is observed under compaction equipment by our field engineer. If the excavation extends below or within 2 feet of the groundwater table at the time of excavation, mitigation measures, such as placement of geogrid, drain rock, or controlled low-strength material (CLSM), may be required to provide a stable subgrade on which to place and compact fill.

After subgrade preparation is completed, a layer of woven geotextile (Mirafi 500X or equivalent) should be placed on the prepared subgrade and extend up the sides of the excavation. The excavated material should then be placed in lifts not exceeding 8 inches in loose thickness, moisture-conditioned to near optimum moisture content and compacted to at least 95 percent relative compaction.[6]. After placement of 2 feet of properly compacted fill, the geotextile extending up the sides of the excavation should be wrapped over the surface of the compacted

---

[6] Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557 laboratory compaction procedure.

fill. This process should be repeated to within one foot of the mat subgrade. The purpose of the geotextile layers is to retain the sides of the engineered fill to prevent the fill from settling during an earthquake if the soil adjacent to the fill liquefies and loses strength during an earthquake.

Exterior Concrete Flatwork

We recommend exterior concrete flatwork be underlain by at least 4 inches of aggregate base compacted to at least 90 percent relative compaction. Prior to placing the aggregate base, the upper 8 inches of soil subgrade should be scarified, moisture-conditioned to near optimum moisture content, and compacted to at least 90 percent relative compaction (95 percent relative compaction for clean sand or gravel).

7.1.2 Fill Materials and Compaction

Fill should consist of on-site soil or imported (select) fill that is free of organic matter, contains no rocks or lumps larger than 3 inches in greatest dimension, and be approved by the Geotechnical Engineer. Imported select fill should also have a liquid limit less than 40 and plasticity index less than 12 and be approved by the Geotechnical Engineer. Samples of proposed imported fill material should be submitted to the Geotechnical Engineer at least three business days prior to use at the site. The grading contractor should provide analytical test results or other suitable environmental documentation indicating the imported fill is free of hazardous materials at least three days before use at the site. If this data is not provided, a minimum of two weeks will be required to perform any necessary analytical testing.

Fill should be placed in horizontal lifts not exceeding 8 inches in uncompacted thickness, moisture-conditioned to near optimum moisture content, and compacted to at least 95 percent relative compaction below the tank mat foundation and 90 percent elsewhere. Fill consisting of clean sand or gravel (defined as poorly graded soil with less than 5 percent fines by weight) or greater than 5 feet in thickness should be compacted to at least 95 percent relative compaction. Fill placement within the upper 12 inches of vehicular pavement soil subgrade should also be compacted to at least 95 percent relative compaction and be non-yielding.

7.1.3 Utility Trench Backfill and Flexible Connection

The thickness and type of bedding material required for utilities will depend on the soil conditions at the utility trench bottom. As a minimum, bedding should extend at least D/4 (with D equal to the outside pipe diameter) below the bottom of the pipe; however, the bedding should be at least 4 inches thick. This minimum bedding thickness and either clean sand, rod mill or pea gravel bedding material is adequate for shallow trenches above the groundwater level.

Backfill for utility trenches should be compacted according to the recommendations presented for general site fill. Jetting of trench backfill should not be permitted. If sand or gravel with less than 5 percent fines (particles passing the No. 200 sieve) is used, it should be compacted to at least 95 percent relative compaction. Pea gravel, drain rock, and rod mill should be mechanically tamped in 12-inch lifts where placed beneath pavements. Special care should be taken when backfilling utility trenches in pavement areas. Poor compaction may cause excessive settlements, resulting in damage to the pavement section.

Considering the ground surface beyond the limits of the overexcavation and recompaction is expected to settle differentially from the proposed tank due to cyclic densification and/or liquefaction during a major earthquake (see Sections 5.2.3 and 5.2.4), we recommend utilities connected to the tank designed to tolerate 2 inches of abrupt differential settlement.

7.1.4 Surface Drainage

Positive surface drainage should be provided around the tank to direct surface water away from the foundations. Grades around the tank should be designed by the Civil Engineer and conform to the requirements of the 2022 CBC, which will help minimize stormwater accumulation adjacent to foundations.

7.2 Mat Foundation

As discussed in Section 6.1, the proposed tank may be supported on a mat bearing on engineered fill that extends to a depth of 20 feet below existing grade and prepared following the recommendations presented in Section 7.1.1. For design of the mat, we recommend a modulus of vertical subgrade reaction of 30 pounds per cubic inch (pci) be used; this value has been reduced to account for the size of the mat and may be increased by one-third for total load conditions. Considering the area of the mat, we expect the average bearing stress under the mat to be low; however, concentrated stresses will occur at the edges of the mat. The mat should be designed to impose a maximum dead-plus-live bearing pressure of 3,000 pounds per square foot (psf) on the foundation subgrade soil. This pressure may be increased by one-third for total load conditions. Once the Structural Engineer estimates the distribution of bearing stress on the bottom of the mat, we should review the distribution and revise the modulus of subgrade reaction, if appropriate.

Lateral loads can be resisted by a combination of passive pressure on the outside edges of the mat foundation and friction along the bottom of the mat. Passive resistance may be calculated using an equivalent fluid weight of 250 pounds per cubic foot (pcf). Passive resistance provided by the upper foot of soil should be ignored unless it is confined by a slab or pavement. Frictional resistance should be computed using a base friction coefficient of 0.35 where the mat is in contact with soil. This value includes a factor of safety of at least 1.5 and may be used in combination without further reduction.

The mat subgrade should be free of loose, weak, or disturbed material. We should check the subgrade prior to placement of reinforcing steel to confirm the condition of the subgrade is acceptable.

7.3 Permanent Below-Grade Walls

Permanent below-grade walls should be designed to resist lateral earth pressures imposed by the retained soil, as well as a surcharge pressure from nearby foundations and vehicles where appropriate. In addition, because the site is in a seismically active area, retaining walls that retain more than 6 feet of soil should be designed for the more critical of static or seismic conditions.

For static conditions, we recommend restrained and unrestrained walls be designed for the following lateral earth pressures:

- Restrained Wall - At-rest earth pressure using an equivalent fluid weight of 54 pcf for drained conditions and 88 pcf for undrained conditions.

- Unrestrained Wall - Active earth pressure using an equivalent fluid weight of 35 pcf and 79 pcf for undrained conditions.

Walls that will retain more than 6 feet of soil will need to be designed for the more critical of static (presented above) or the following seismic conditions.

- Restrained Wall - Active earth pressure using an equivalent fluid weight of 35 pcf plus a seismic increment of 33 pcf for drained conditions; and 79 pcf plus a seismic increment of 15 pcf for undrained conditions.
- Unrestrained Wall - Active earth pressure using an equivalent fluid weight of 35 pcf plus a seismic increment of 15 pcf for drained conditions; and 79 pcf plus a seismic increment of 7 pcf for undrained conditions.

Where there will be vehicular loading behind the top of a permanent wall within a horizontal distance equal to 1.5 times the height of the wall, the wall should be designed for vehicular surcharge of 50 psf over the upper 10 feet of the wall.

If the “drained” earth pressures presented above are used to design the walls, they will need to incorporate a drainage system. Alternatively, the walls may be designed for the recommended “undrained” earth pressures presented above over their entire height, in which case the drainage system may be omitted. Where the wall extends below the design groundwater level, the wall should be designed using “undrained” earth pressures presented above.

One acceptable method for backdraining below-grade walls is to place a prefabricated drainage panel against the back of the wall. The drainage panel should extend down to a perforated PVC collector pipe at the base of the wall. The pipe should be surrounded on all sides by at least 4 inches of Caltrans Class 2 permeable material or 3/4-inch drain rock wrapped in filter fabric (Mirafi NC or equivalent). A proprietary, prefabricated collector drain system, such as Tremdrain Total Drain or Hydroduct Coil (or equivalent), designed to work in conjunction with the drainage panel may be used in lieu of the perforated pipe surrounded by gravel described above. The pipe should be connected to a suitable discharge point; a sump and pump system may be required to drain the collector pipes.

To protect against moisture migration, below-grade retaining walls should be waterproofed and water stops should be placed at all construction joints. If backfill is required behind below-grade walls, the walls should be braced, or hand compaction equipment used, to prevent unacceptable surcharges on walls (as determined by the Structural Engineer).

7.4 Temporary Cut Slopes and Shoring

We anticipate an excavation to a depth of roughly 20 feet bgs will be needed to install the proposed tank and overexcavate and re-compact the loose soil below the tank foundation. All excavations should conform to the current CAL-OSHA requirements. Where feasible and above the groundwater, excavations may be slope cut. The soil encountered at the site can be considered OSHA Type C soil, which should be slope cut at a maximum inclination of 1.5:1 (horizontal to vertical). The contractor should be responsible for the construction and safety of temporary slopes and shoring. Where there is inadequate space to slope cut the excavation or where groundwater is present, temporary shoring will be required.

We judge a soldier pile and lagging shoring system with internal bracing is appropriate for support of excavations. A soldier pile and lagging shoring system usually consists of steel H-beams and concrete placed in predrilled holes extending below the bottom of the excavation. Wood lagging is placed between the piles as the excavation proceeds from the top down. Where the required cut is less than about 12 feet, a soldier pile and lagging system can typically provide economical shoring without bracing. Where cuts exceed about 12 feet in height, soldier pile and lagging systems are typically more economical if they include bracing.

A Structural/Civil Engineer knowledgeable in this type of construction should be retained for shoring design. The shoring design should design the shoring system for lateral deformation of less than an inch at any location on the shoring. We should review the final shoring plans and calculations to check that they are consistent with the recommendations presented in this report.

7.4.1 Soldier Pile-and-Lagging Shoring System

We recommend a cantilevered, soldier pile-and-lagging shoring system be designed to resist an active equivalent fluid weight of 35 pcf. In locations where minimizing lateral deflections is critical, such as near adjacent pavements or near sensitive underground utilities, the shoring system should be designed to resist an at-rest equivalent fluid weight of 54 pcf. For a braced shoring system, we recommend using the pressures shown in Figure 6.

In calculating these design pressures, we assumed drained conditions with no hydrostatic pressure acting on the soldier pile-and-lagging shoring. Where traffic loads (including truck traffic) are expected within 10 feet of the shoring walls, an additional design load of 50 psf should be applied to the upper 10 feet of the wall. Where construction equipment will be working behind the walls within a horizontal distance equal to the wall height, the design should include a surcharge pressure of 250 psf. The above pressures should be assumed to act over the entire width of the lagging installed above the excavation.

Passive resistance at the toe of the soldier piles should be computed using an equivalent fluid weight of 285 pcf above the groundwater level and 135 pcf below the groundwater. Passive pressure can be assumed to act over an area of three pile widths assuming the toe of the soldier pile is filled with structural concrete. These passive pressure values include a factor of safety of at least 1.5.

7.5 Seismic Design

The latitude and longitude of the site are 37.7999° and -122.2769°, respectively. In accordance with the 2022 CBC, we recommend the following:

- Site Class D (stiff soil, non-default)
- $S_S = 1.646g$, $S_1 = 0.622g$

The 2022 SFBC is based on the guidelines contained within ASCE 7-16 (Supplement 3 revision), which stipulates that where S_1 is greater than 0.2 times gravity (g) for Site Class D, a ground motion hazard analysis is required unless the long-period spectral design parameters (S_{M1}, S_{D1}) are increased by 50%. Therefore, we recommend the following seismic design parameters, which include the 50% increase as designated by an asterisk:

- $F_a = 1.0$, $F_v = 1.7$
- $S_{MS} = 1.646g$, $S_{M1}* = 1.586g$

- S_{DS} = 1.097g, S_{D1}* = 1.057g
- Seismic Design Category D for Risk Factors I, II, and III

8.0 ADDITIONAL GEOTECHNICAL SERVICES

Prior to construction, we should review the project plans and specifications to check that they conform to the intent of our recommendations. During construction, our field engineer should observe shoring installation, perform field density testing to check subgrade preparation and fill compaction, and check mat foundation subgrade preparation. These observations will allow us to compare actual with anticipated soil conditions and to check the contractor's work conforms to the geotechnical aspects of the plans and specifications.

9.0 LIMITATIONS

This geotechnical investigation has been conducted in accordance with the standard of care commonly used as state-of-practice in the profession. No other warranties are either expressed or implied. The recommendations made in this report are based on the assumption that the subsurface conditions do not deviate appreciably from those disclosed in our field investigation. If any variations or undesirable conditions are encountered during construction, we should be notified so additional recommendations can be made. The foundation recommendations presented in this report are developed exclusively for the proposed development described in this report and are not valid for other locations and construction in the site vicinity.

REFERENCES

American Society of Civil Engineers (ASCE). (2017). *ASCE/SEI 7-16: Minimum Design Loads and Associated Criteria for Buildings and Other Structures*. ASCE Publications. http://doi.org/10.1061/9780784414248

California Department of General Services. (2022). *California Building Code*. State Government of California. https://www.dgs.ca.gov/BSC/Codes

California Department of Transportation (Caltrans). (2022). *Caltrans Standard Specifications*. State Government of California. https://dot.ca.gov/-/media/dot-media/programs/design/documents/2022_stdspecs-a11y.pdf

California Geological Survey. (2003, February 14). *Earthquake Zones of Required Investigation, Oakland West Quadrangle, Revised Official Map*. California Department of Conservation.

California Geological Survey. (2005). *Seismic Hazard Zone Report for the Oakland West 7.5-Minute Quadrangle, Alameda County, California*. California Department of Conservation.

Field, E.H., Biasi, G.P., Bird, P., Dawson, T.E., Felzer, K.R., Jackson, D.D., Johnson, K.M., Jordan, T.H., Madden, C., Michael, A.J., Milner, K.R., Page, M.T., Parsons, T., Powers, P.M., Shaw, B.E., Thatcher, W.R., Weldon, R.J., & Zeng, Y. (2013). *Uniform California Earthquake Rupture Forecast, Version 3 (UCERF3)—The Time-Independent Model* (Open-File Report 2013-1165). United States Geological Survey. http://doi.org/10.3133/ofr20131165

Graymer, R.W. (2000). *Geologic Map and Map Database of the Oakland Metropolitan Area, Alameda, Contra Costa and San Francisco Bay Counties, California*, U.S. Geological Survey Miscellaneous Field Studies MF-2342.

Petersen, M.D., Moschetti, M.P., Powers, P.M., Mueller, C.S., Haller, K.M., Frankel, A.D., Zeng, Y., Rezaeian, S., Harmsen, S.C., Boyd, O.S., Field, N., Chen, R., Rukstales, K.S., Luco, N., Wheeler, R.L., Williams, R.A., & Olsen, A.H. (2014). *Documentation for the 2014 Update of the United States National Seismic Hazard Maps* (Open-File Report 2014-1091). United States Geological Survey. https://pubs.usgs.gov/of/2014/1091/

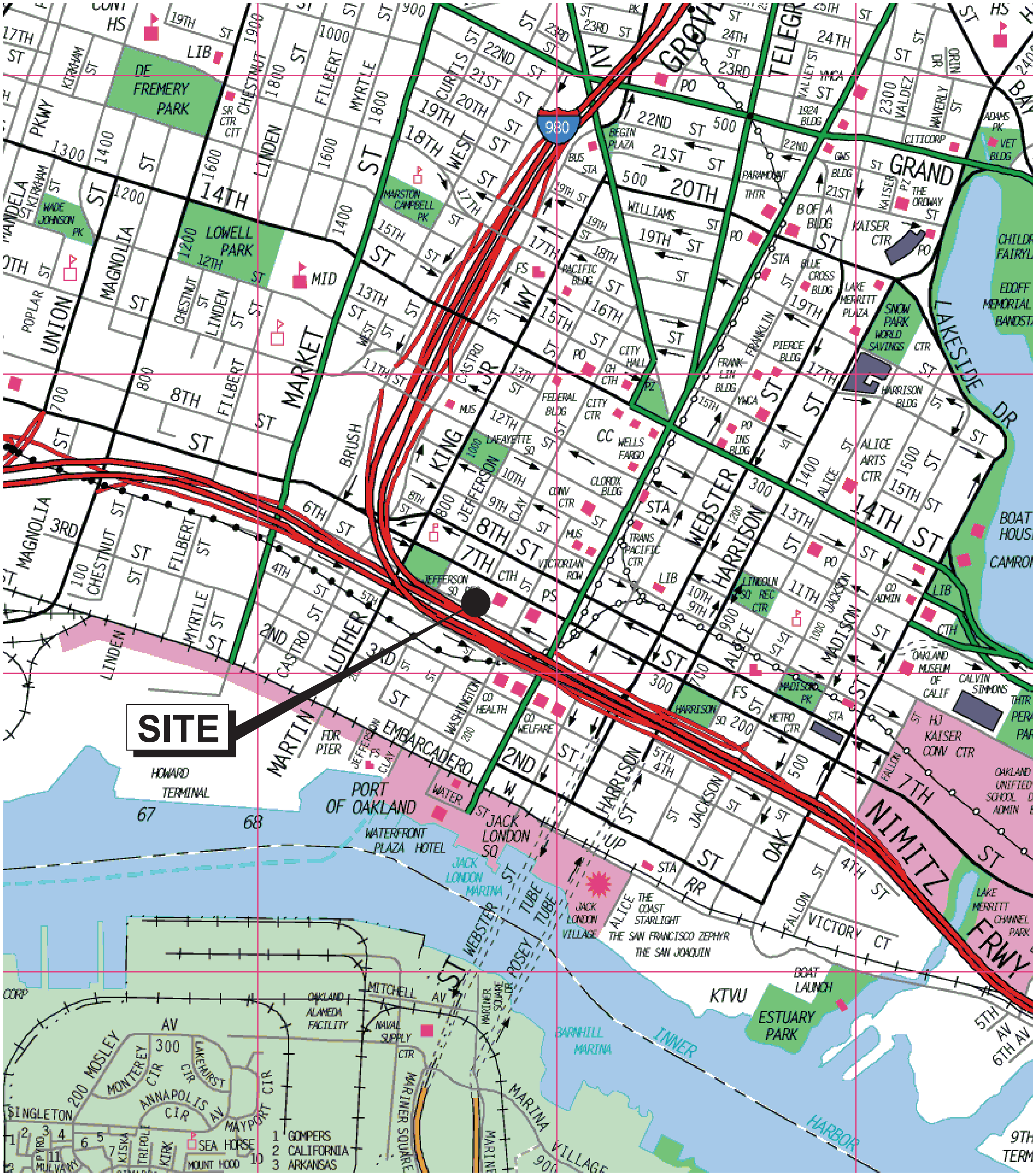
Pradel, D. (1998). Procedure to Evaluate Earthquake-Induced Settlement in Dry Sandy Soils. *Journal of Geotechnical and Geoenvironmental Engineering, 124*(4). https://doi.org/10.1061/(ASCE)1090-0241(1998)124:4(364)

Thompson, E.M., Wald, D.J., Worden, B., Field, E.H., Luco, N., Petersen, M.D., Powers, P.M., Badie, R. (2016). *Shakemap earthquake scenario: Building Seismic Safety Council 2014 Event Set (BSSC2014)*. United States Geological Survey. https://doi.org/10.5066/F7V122XD

United States Geological Survey. (2021). *Earthquake Hazards Program: Advanced National Seismic System (ANSS) Comprehensive Catalog of Earthquake Events and Products*. United States Department of the Interior. https://earthquake.usgs.gov/earthquakes/search/

Youd, T.L., Idriss, I.M., Andrus, R.D., Arango, I., Castro, G., Christian, J.T., Dobry, R., Liam Finn, W.D., Harder, L.F., Hynes, M.E., Ishihara, K., Koester, J.P., Liao, S.C., Marcuson, W.F., Martin, G.R., Mitchell, J.K., Moriwaki, Y., Power, M.S., Robertson, P.K., Seed, R.B., & Stokoe, K.H. (2001). Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils. *Journal of Geotechnical and Geoenvironmental Engineering, 127*(4), 297-313. https://doi.org/10.1061/(ASCE)1090-0241(2001)127:10(817)

FIGURES



Base map: The Thomas Guide
Alameda County
2002

0 1/4 1/2 Mile

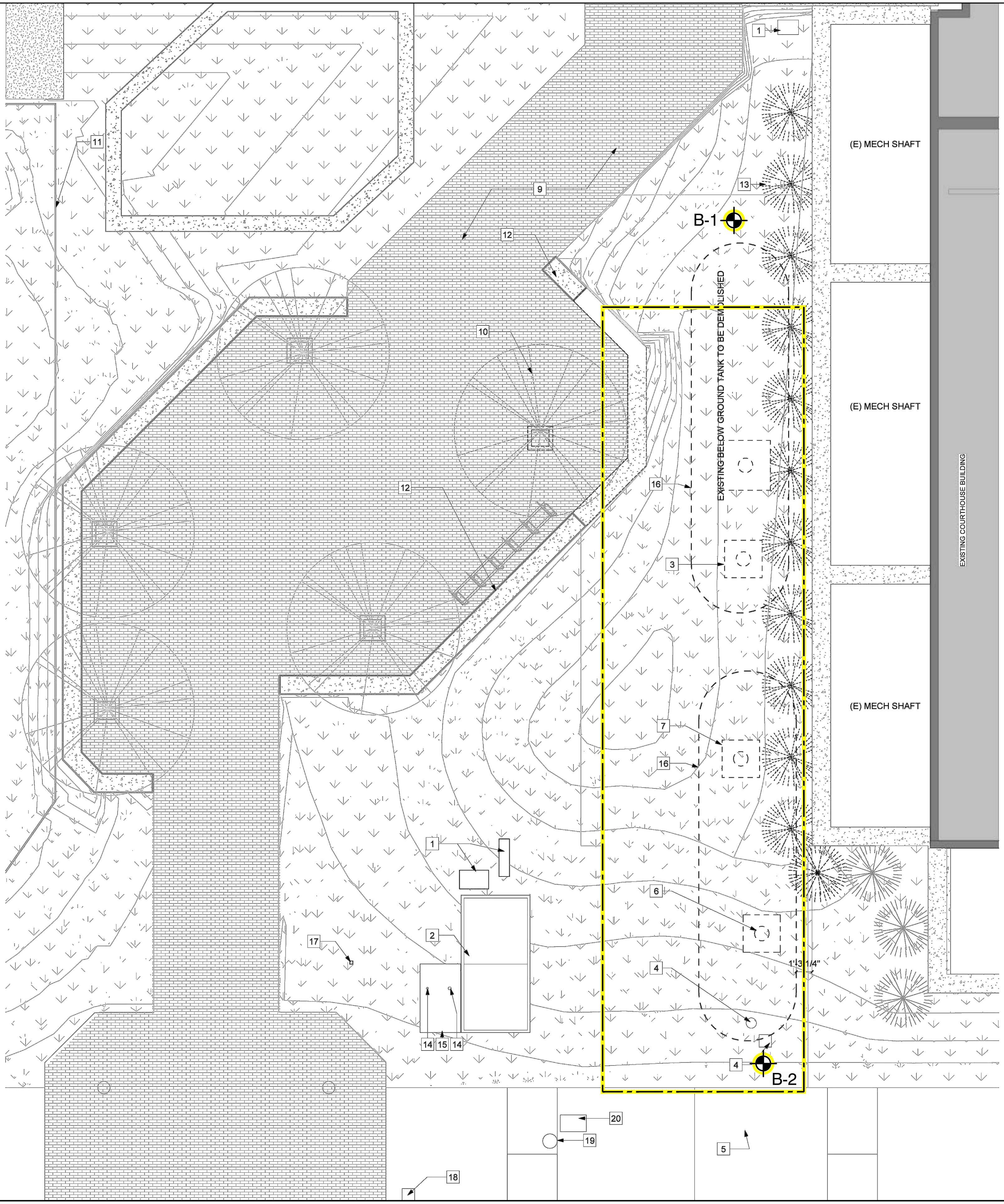
Approximate scale

WILEY W. MANUEL COURTHOUSE
TANK REMOVAL AND REPLACEMENT
Oakland, California

ROCKRIDGE
GEOTECHNICAL

SITE LOCATION MAP

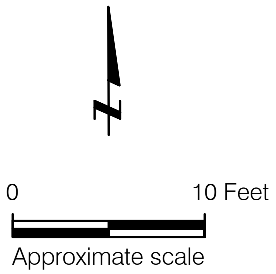
Date 06/12/24	Project No. 24-2647	Figure 1



EXPLANATION

B-1 ⊕ Approximate location of boring by Rockridge Geotechnical, Inc., June 7, 2024

— · — Project limits



Reference: Base map from a drawing titled "Enlarged Site Demolition Plan", by AE3 Architecture, dated April 23, 2024.

WILEY W. MANUEL COURTHOUSE
TANK REMOVAL AND REPLACEMENT
Oakland, California

SITE PLAN

Date 06/20/24 | Project No. 24-2647 | Figure 2

ROCKRIDGE GEOTECHNICAL

Base map: USGS MF 2342, Geologic Map and Map Database of the Oakland Metropolitan Area, Alameda, Contra Costa, and San Francisco Counties, California (Graymer, 2000).

EXPLANATION

Contact - Depositional or intrusive contact, dashed where approximately located, dotted where concealed

Fault - Dashed where approximately located, small dashed where inferred, dotted where concealed, queried where locations is uncertain

Reverse or thrust fault - Dotted where concealed

Anticline -Shows fold axis, dotted where concealed

Syncline

35 Strike and dip of bedding

Overturned bedding

Flat bedding

Vertical bedding

35 Strike and dip of foliation

Vertical foliation

35 Strike and dip of joints in plutonic rocks

Vertical joint

af	Artificial fill (Historic)
Qpaf	Alluvial fan and fluvial deposits
Qms	Merritt sand (Holocene and Pleistocene)
Qmt	Marine terrace deposits (Pheistocene)

0 1,000 2,000 Feet

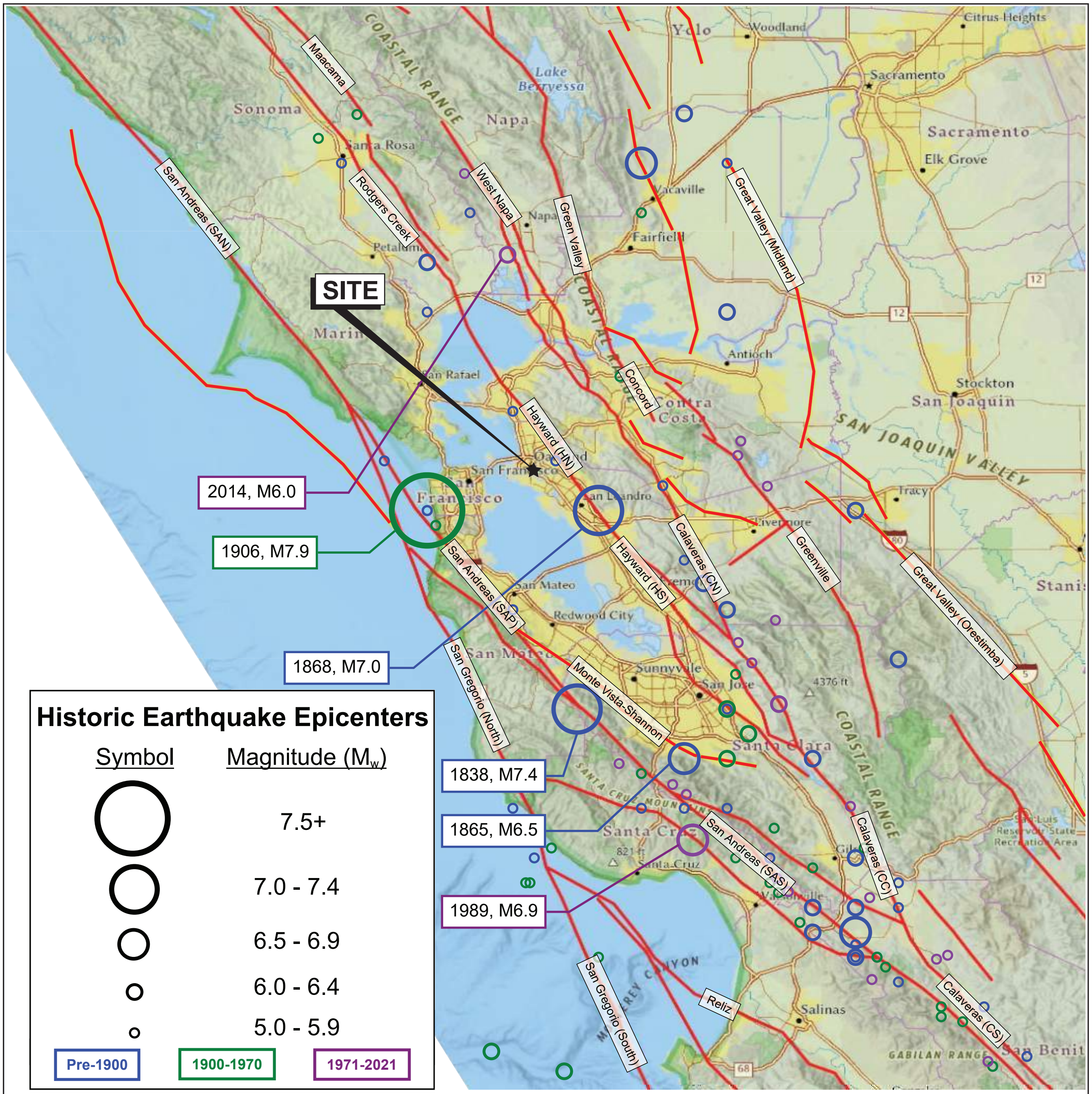
Approximate scale

WILEY W. MANUEL COURTHOUSE
TANK REMOVAL AND REPLACEMENT
Oakland, California

ROCKRIDGE
GEOTECHNICAL

REGIONAL GEOLOGIC MAP

Date 06/12/24 | Project No. 24-2647 | Figure 3



EXPLANATION

Faults (National Seismic Hazard Model)

0 10 20 Miles

0 16 32 Kilometers

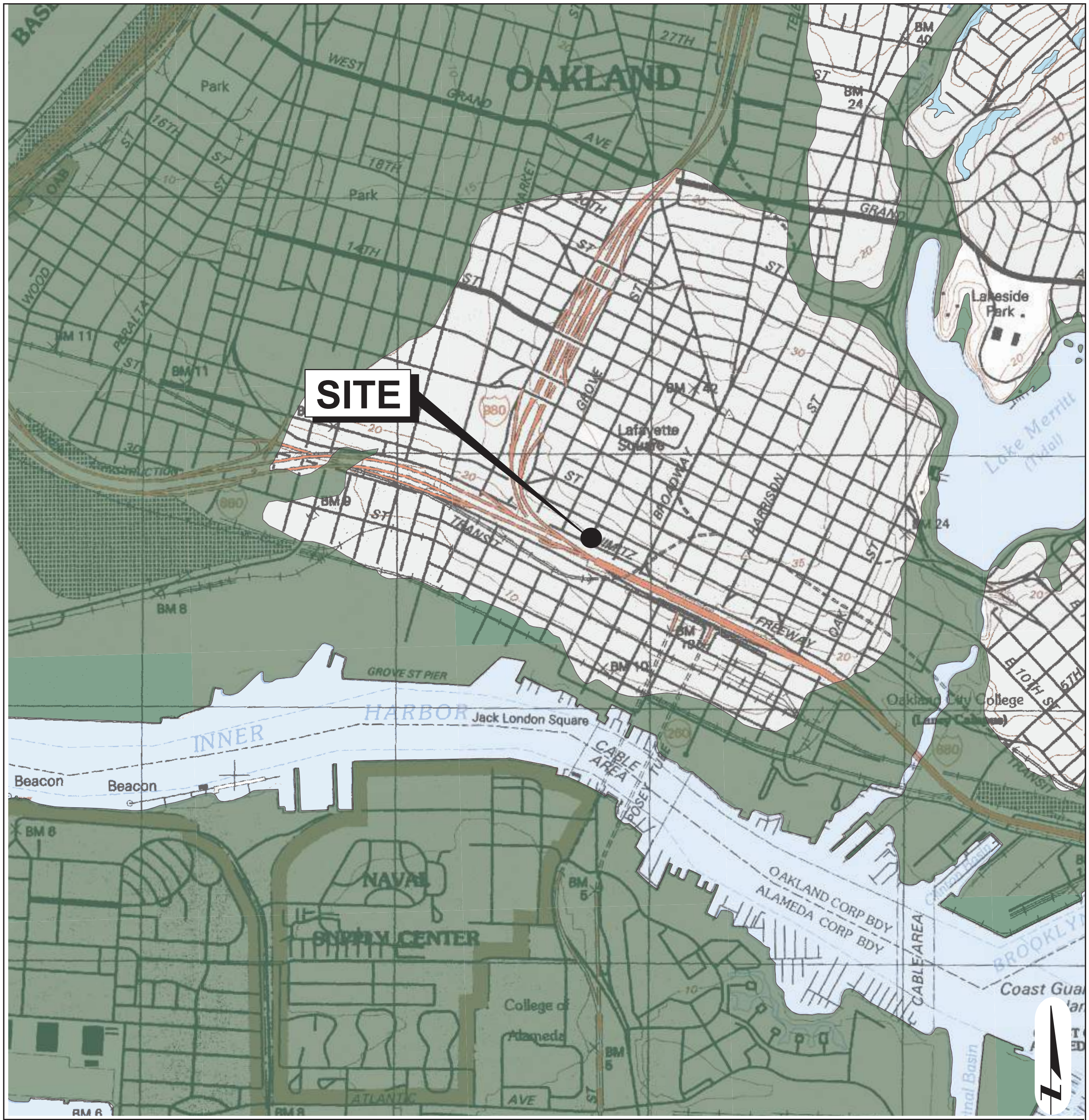
Approximate scale

WILEY W. MANUEL COURTHOUSE
TANK REMOVAL AND REPLACEMENT
Oakland, California

ROCKRIDGE
GEOTECHNICAL

REGIONAL FAULT AND HISTORIC SEISMICITY MAP

Date 06/12/24	Project No. 24-2647	Figure 4



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

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

Reference:
Earthquake Zones of Required Investigation
Oakland West Quadrangle
California Geological Survey
Released February 14, 2003

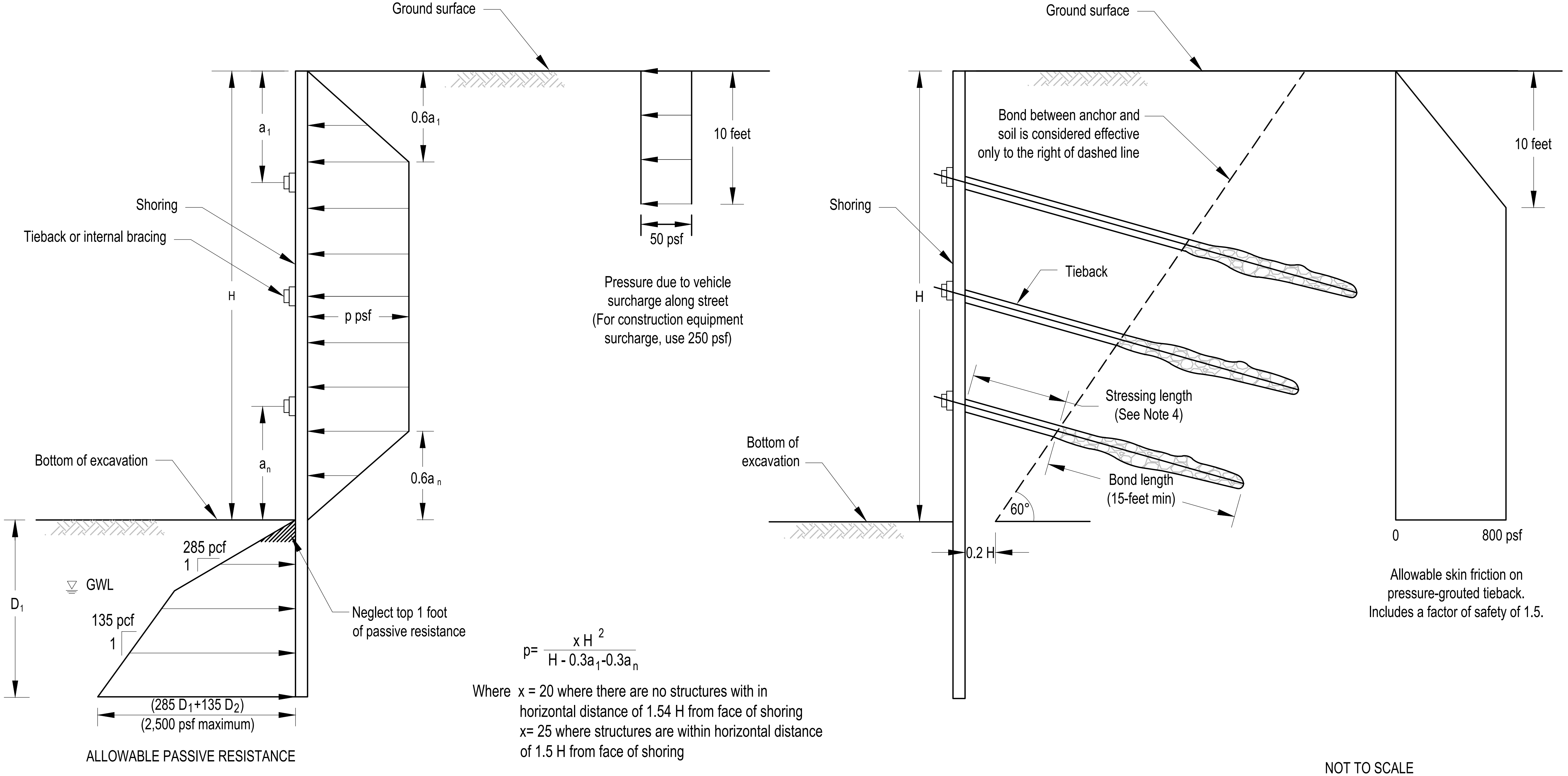
0 2,000 4,000 Feet
Approximate scale

WILEY W. MANUEL COURTHOUSE
TANK REMOVAL AND REPLACEMENT
Oakland, California

ROCKRIDGE
GEOTECHNICAL

EARTHQUAKE ZONES OF REQUIRED INVESTIGATION MAP

Date 06/12/24	Project No. 24-2647	Figure 5



$$p = \frac{x\,H^2}{H - 0.3a_1 - 0.3a_n}$$

Where x = 20 where there are no structures with in horizontal distance of 1.54 H from face of shoring
x= 25 where structures are within horizontal distance of 1.5 H from face of shoring

Notes:
1. Passive pressures include a factor of safety of about 1.5.
2. For soldier piles spaced at more than three times the soldier pile diameter, the passive pressure should be assumed to act over three diameters, provided the concrete is sufficiently strong to accommodate the corresponding stresses (shoring designer should confirm).
3. Stressing length: minimum 15 feet for strands, minimum 10 feet for bars.

WILEY W. MANUEL COURTHOUSE
TANK REMOVAL AND REPLACEMENT
Oakland, California

DESIGN PARAMETERS FOR TIED-BACK OR INTERNALLY BRACED SOLDIER-PILE-AND-LAGGING TEMPORARY SHORING SYSTEM

Date 07/12/24	Project No. 24-2647	Figure 6

ROCKRIDGE GEOTECHNICAL

APPENDIX A
Logs of Borings

PROJECT: **WILEY W. MANUEL COURTHOUSE**
TANK REMOVAL AND REPLACEMENT
Oakland, California

Log of Boring B-1

PAGE 1 OF 1

Boring location:	See Site Plan, Figure 2	Logged by:	J. Graham
Date started: 06/07/2024	Date finished: 06/07/2024	Drilled by:	Stapleton Engineering & Exploration
Drilling method:	4-inch-diameter solid-stem auger	Rig:	Portable Bobcat Rig
Hammer weight/drop: 140 lbs./30 inches	Hammer type: Rope & cathead safety hammer		
Sampler:	Grab, Modified California (MC), Standard Penetration Test (SPT)		

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION		LABORATORY TEST DATA Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content,%	Dry Density Lbs/Cu Ft
1						**CLAYEY SAND (SC)** yellow-brown, moist, fine to medium sand, trace fine subangular to subrounded gravel	↑						
2	GRAB	☒											
4	GRAB	☒				yellow-brown and gray							
6	GRAB	☒				no gravel medium dense							
7	MC	■	11 11 14	18	SC		FILL						
10	SPT	◩	6 7 8	18		trace fine subrounded gravel, trace brick debris Particle Size Distribution; see Appendix B					20		
13	SPT	◩	6 3 3	7		loose, no brick debris Particle Size Distribution; see Appendix B	↓				18		
15						**CLAYEY SAND (SC)** yellow-brown, medium dense, moist, fine sand							
16	SPT	◩	7 9 10	23									
18					SC								
20	SPT	◩	20 32 40	86		▼ (06/12/2024; 11:15 AM) light brown, very dense, wet, fine sand ▽ (06/12/2024; 11:00 AM)							

Boring terminated at a depth of 21.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 21 feet and 20 feet before backfilling

[1] MC and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively, to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 24-2647

Figure: A-1

PROJECT: **WILEY W. MANUEL COURTHOUSE**
TANK REMOVAL AND REPLACEMENT
Oakland, California

Log of Boring B-2

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2
Date started: 06/07/2024
Date finished: 06/07/2024
Drilling method: 4-inch-diameter solid-stem auger
Logged by: J. Graham
Drilled by: Stapleton Engineering & Exploration
Rig: Portable Bobcat Rig
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Rope & cathead safety hammer
Sampler: Grab, Standard Penetration Test (SPT)

DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION		LABORATORY TEST DATA					
	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]				Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content,%	Dry Density Lbs/Cu Ft
1	GRAB	☒				CLAYEY SAND (SC) brown, moist, fine to medium sand, trace fine to coarse sand, trace fine to coarse subangular to subrounded gravel							
2													
3	GRAB	☒				yellow-brown, trace fine subrounded gravel							
4													
5	GRAB	☒				no gravel							
6													
7	SPT	◪	9 12 12	29		yellow-brown and gray, medium dense, trace fine subangular gravel, trace brick debris							
8													
9					SC		FILL						
10	SPT	◪	7 5 3	10		loose to medium dense, trace rootlets, no gravel or brick debris Particle Size Distribution; see Appendix B					21		
11													
12	SPT	◪	11 12 9	25		medium dense Soil Corrosivity Test; see Appendix B							
13													
14													
15													
16	SPT	◪	2 2 2	5		loose, trace rootlets Particle Size Distribution; see Appendix B					20		
17													
18	SPT	◪	2 3 4	8	SC-SM	SILTY CLAYEY SAND (SC-SM) light brown, gray, and yellow, loose, moist, fine sand, rootlets LL = 17, PI = 4; see Appendix B Particle Size Distribution; see Appendix B					20	17.9	
19													
20						▼ (06/12/2024; 02:00 PM)							
21	SPT	◪	12 12 20	38	SC	CLAYEY SAND (SC) light brown, gray, and yellow, dense, wet, fine sand ▽ (06/12/2024; 1:45 PM)							
22													
23													
24													
25													
26													
27													
28													
29													
30													

Boring terminated at a depth of 21.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 21 feet and 20 feet before backfilling

[1] MC and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively, to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 24-2647
Figure: A-2

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions		Symbols	Typical Names
Coarse-Grained Soils (more than half of soil > no. 200 sieve size)	Gravels (More than half of coarse fraction > no. 4 sieve size)	**GW**	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	Sands (More than half of coarse fraction < no. 4 sieve size)	**SW**	Well-graded sands or gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
Fine -Grained Soils (more than half of soil < no. 200 sieve size)	Silts and Clays LL = < 50	**ML**	Inorganic silts and clayey silts of low plasticity, sandy silts, gravelly silts
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	Silts and Clays LL = > 50	**MH**	Inorganic silts of high plasticity
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic silts and clays of high plasticity
Highly Organic Soils		**PT**	Peat and other highly organic soils

GRAIN SIZE CHART

Classification	Range of Grain Sizes	
	U.S. Standard Sieve Size	Grain Size in Millimeters
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel coarse fine	3" to No. 4 3" to 3/4" 3/4" to No. 4	76.2 to 4.76 76.2 to 19.1 19.1 to 4.76
Sand coarse medium fine	No. 4 to No. 200 No. 4 to No. 10 No. 10 to No. 40 No. 40 to No. 200	4.76 to 0.075 4.76 to 2.00 2.00 to 0.420 0.420 to 0.075
Silt and Clay	Below No. 200	Below 0.075

▽ Unstabilized groundwater level

▼ Stabilized groundwater level

SAMPLE DESIGNATIONS/SYMBOLS

⬒	Sample taken with California or Modified California split-barrel sampler. Darkened area indicates soil recovered
◩	Classification sample taken with Standard Penetration Test sampler
◨	Undisturbed sample taken with thin-walled tube
⊠	Disturbed sample
⊙	Sampling attempted with no recovery
◫	Core sample
●	Analytical laboratory sample
▥	Sample taken with Direct Push sampler
▥	Sonic

SAMPLER TYPE

C	Core barrel
CA	California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter
D&M	Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube
O	Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube
PT	Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube
MC	Modified California sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter
SPT	Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.38- or 1.5-inch inside diameter (refer to text)
ST	Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

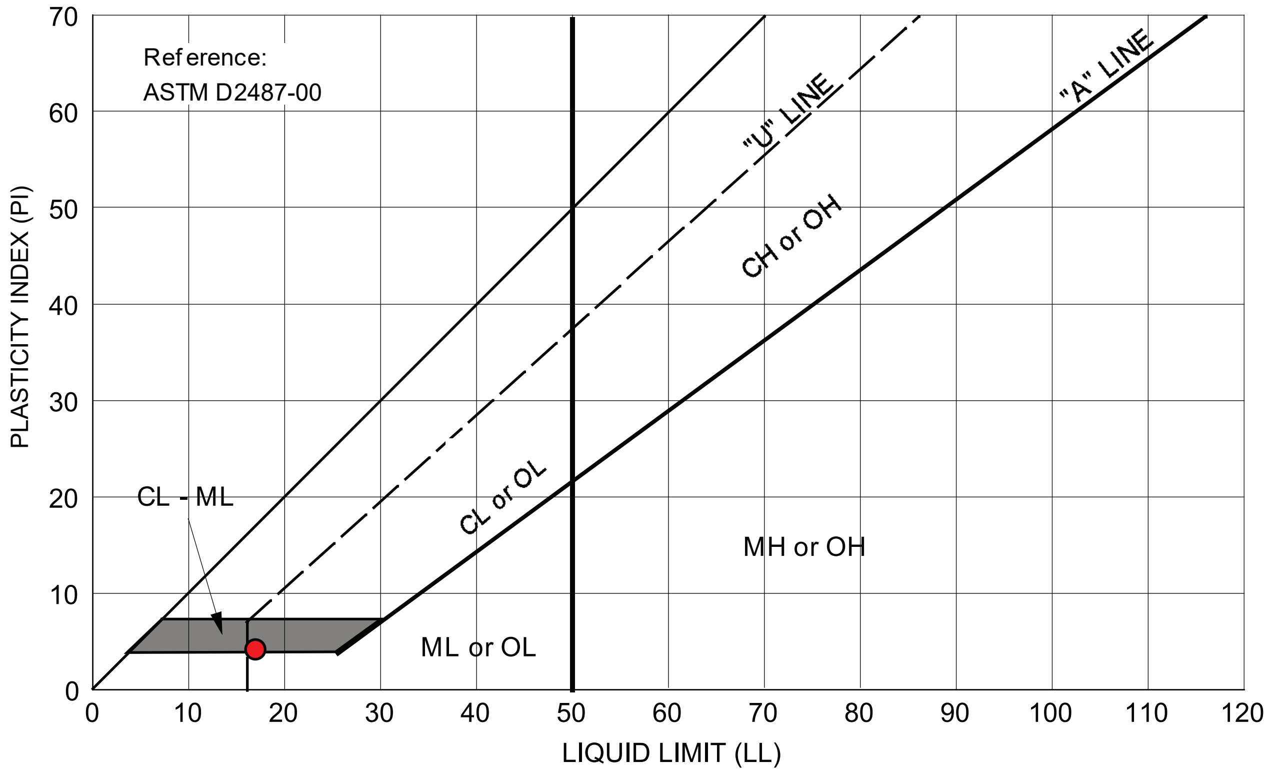
WILEY W. MANUEL COURTHOUSE
TANK REMOVAL AND REPLACEMENT
Oakland, California

ROCKRIDGE
GEOTECHNICAL

CLASSIFICATION CHART

Date 06/12/24 | Project No. 24-2647 | Figure A-3

APPENDIX B
Laboratory Test Results



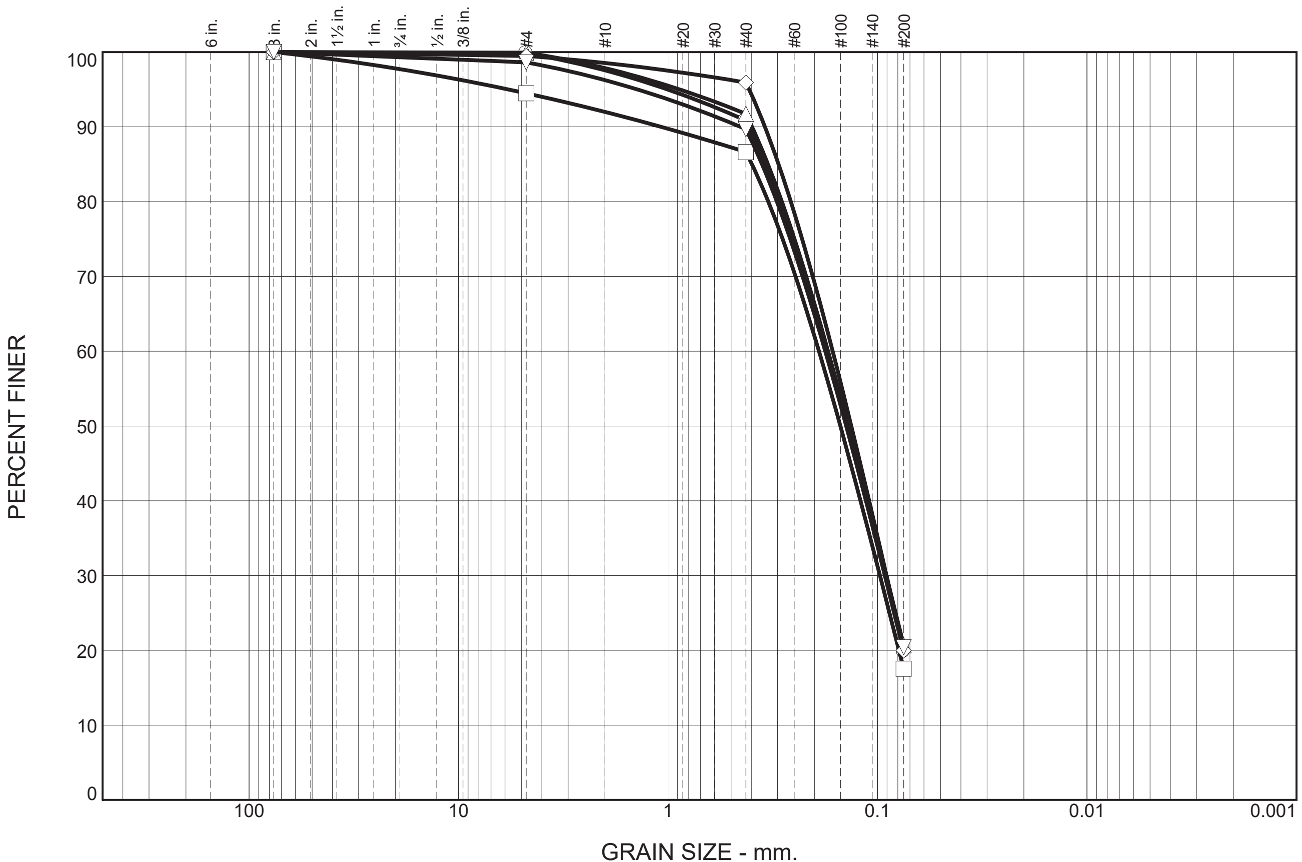
Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-2 at 17.0 feet	SILTY CLAYEY SAND (SC-SM), light brown, gray, and yellow	17.9	17	4	20

WILEY W. MANUEL COURTHOUSE
TANK REMOVAL AND REPLACEMENT
Oakland, California

ROCKRIDGE
GEOTECHNICAL

PLASTICITY CHART

Date 07/03/24 | Project No. 24-2647 | Figure B-1



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	.2		79.9			19.9	
□	0.0	5.5		77.0			17.5	
△	0.0	.1		79.2			20.7	
◇	0.0	.5		79.5			20.0	
▽	0.0	1.4		78.2			20.4	

SOIL DATA

SYMBOL	SOURCE	DEPTH (ft.)	Material Description	USCS
○	B-1	10.0'	CLAYEY SAND, yellow-brown and gray	SC
□	B-1	12.5'	CLAYEY SAND, yellow-brown and gray	SC
△	B-2	10.0'	CLAYEY SAND, yellow-brown, light gray, and yellow	SC
◇	B-2	15.0'	CLAYEY SAND, yellow-brown, light gray, and yellow	SC
▽	B-2	17.0'	SILTY CLAYEY SAND, light brown, gray, and yellow	SC-SM

WILEY W. MANUEL COURTHOUSE
TANK REMOVAL AND REPLACEMENT
Oakland, California

ROCKRIDGE
GEOTECHNICAL

PARTICLE SIZE DISTRIBUTION REPORT

Date 07/15/24 | Project No. 24-2647 | Figure B-2

Project X
Corrosion Engineering
Corrosion Control – Soil, Water, Metallurgy Testing Lab

REPORT S240613T

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM G51	ASTM G200	SM 4500-D	ASTM D4327	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
Bore# / Description	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity** As Rec'd	Minimum	**pH**	**Redox**	**Sulfide** S^{2-}	**Nitrate** NO_3^-	**Ammonium** NH_4^+	**Lithium** Li^+	**Sodium** Na^+	**Potassium** K^+	**Magnesium** Mg^{2+}	**Calcium** Ca^{2+}	**Fluoride** F_2^{2-}	**Phosphate** PO_4^{3-}
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ω-cm)	(Ω-cm)		(mV)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)
B-2: CLAYEY SAND (SC), yellow-brown, light gray, and yellow	11.5	65.3	0.0065	60.3	0.0060	7,370	5,762	7.6	139	4.5	2.0	3.8	ND	38.2	7.8	34.8	140.5	3.0	1.5

Cations and Anions, except Sulfide and Bicarbonate, tested with Ion Chromatography
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
ND = 0 = Not Detected | NT = Not Tested | Unk = Unknown
Chemical Analysis performed on 1:3 Soil-To-Water extract
PPM = mg/kg (soil) = mg/L (Liquid)

Note: Sometimes a bad sulfate hit is a contaminated spot. Typical fertilizers are Potassium chloride, ammonium sulfate or ammonium sulfate nitrate (ASN). So this is another reason why testing full corrosion series is good because we then have the data to see if those other ingredients are present meaning the soil sample is just fertilizer-contaminated soil. This can happen often when the soil samples collected are simply surface scoops. This is why it's best to dig in a foot, throw away the top and test the deeper stuff. Dairy farms are also notorious for these items.

If one sample pops up much more corrosive than all others, we would recommend collecting more samples surrounding the problem sample location to determine if the peak is isolated to it. This allows us to conclude it was a contaminated sample and able to declare it an outlier.

WILEY W. MANUEL COURTHOUSE
TANK REMOVAL AND REPLACEMENT
Oakland, California

ROCKRIDGE
GEOTECHNICAL

SOIL CORROSIVITY
TEST RESULTS

Date 07/03/24 | Project No. 24-2647 | Figure B-3