ROCKRIDGE
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Prepared for **Quarterra Multifamily Communities, LLC**

PRELIMINARY GEOTECHNICAL INVESTIGATION
TO SUPPORT DUE DILIGENCE EVALUATION
PROPOSED RESIDENTIAL DEVELOPMENT
WILLOW AVENUE APARTMENTS
APN 406-070-042
HERCULES, CALIFORNIA

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February 9, 2025
Project No. 24-2751

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February 9, 2025
Project No. 24-2751

Mr. Tyler Wood
Vice President, Development
Quarterra Multifamily Communities
492 9th Street, Suite 300
Oakland, California 94607

Subject: Preliminary Geotechnical Investigation
to Support Due Diligence Evaluation
Proposed Residential Development
Willow Avenue Apartments
APN 406-070-042
Hercules, California

Dear Mr. Wood:

We are pleased to present the results of our preliminary geotechnical investigation in support of the due diligence evaluation of the property located at Assessor Parcel Number (APN) 406-070-042, California. Our preliminary investigation was performed in accordance with our contract dated December 18, 2024.

The project site consists of a 6.65-acre irregularly shaped property located at the southeastern corner of the intersections of Highway 80 and Highway 4 on the north side of Willow Avenue. The site is bounded by Highway 80 to the west, Highway 4 to the north, the Highway 4 off-ramp to the east, and Willow Avenue to the south. The subject property has multiple small hills throughout the site and a small creek at the northern end of the site. The site is currently vacant and covered with grass and some brush.

Based on our review of the conceptual drawings prepared by LPAS Architects dated October 18, 2024, we understand the proposed development being considered for the site consists of significant cuts and fills to create a more level site, construction of six 3-story residential buildings, designated Building 1 through Building 6, and construction of a 2-story clubhouse building. Proposed cuts will be as much as about 35 feet (southeast corner of site) and proposed fills will be as thick as about 17 feet (northeast corner of the site). Other improvements include asphalt-paved parking lots and drive aisles, site retaining walls, and landscaping. The proposed construction will be set back from the creek and associated FEMA flood zone in the northern portion of the site.

Based on the results of our preliminary geotechnical investigation, we conclude there are no major geotechnical issues that would preclude development of the site as proposed. The primary geotechnical issues affecting the proposed development include: 1) the presence of highly to very highly expansive soil and bedrock and 2) the proposed differential fill thicknesses below some of the building pads, which may result in differential settlement. For preliminary planning and budgeting purposes, based on previous experience, we preliminarily recommend limiting differential fill thicknesses to about 7 feet below individual building pads. Even with 7 feet of differential fill thickness, there will likely be differential settlement across the pads, although it is difficult to predict the magnitude and timeline for this settlement to occur. This will likely require overexcavation of existing soils/bedrock in some areas to reduce the differential fill thickness. If it is determined that limiting differential fill thicknesses to 7 feet below individual building pads is not cost effective, other means of controlling differential settlement may be considered, including: 1) installing aggregate piers beneath the zones of proposed fill, or 2) chemically treating the fill with lime to reduce its compressibility. We preliminarily conclude the proposed buildings should be supported on stiffened mat foundations or post-tensioned (P-T) slabs.

Our preliminary geotechnical investigation consisted of a limited subsurface exploration program. Prior to final design, a final geotechnical investigation should be performed to fill in data gaps of subsurface conditions and provide final conclusions and recommendations regarding the geotechnical aspects of the project.

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

Sincerely,
ROCKRIDGE GEOTECHNICAL, INC.

Alex D. Limpert, P.E. Project Engineer	Logan D. Medeiros, P.E., G.E. Principal Engineer

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WILLOW AVENUE APARTMENTS
HERCULES, CALIFORNIA

1.0 INTRODUCTION

This report presents the results of the preliminary geotechnical investigation performed by Rockridge Geotechnical, Inc. to support the due diligence evaluation for the proposed residential development, referred to as Willow Avenue Apartments, to be constructed at APN 406-070-042 in Hercules, California. The site is located on the southeastern corner of the intersection of Highway 80 and Highway 4, on the north side of Willow Avenue, as shown in the Site Location Map (Figure 1).

The project site consists of a 6.65-acre irregularly shaped property accessed from Willow Avenue. The site is bounded by Highway 80 to the west, Highway 4 to the north, the Highway 4 off-ramp to the east, and Willow Avenue to the south. The subject property has multiple small hills throughout the site and a small creek at the northern end of the site. The site is currently vacant and covered with grass and brush.

Based on our review of the conceptual drawings for the project,[1] we understand the proposed development being considered for the site consists of significant cuts and fills to create a more level site, construction of six 3-story residential buildings, designated Building 1 through Building 6, and construction of a 2-story clubhouse building. The approximate locations of the proposed buildings are presented on Figures 2 and 3. The approximate magnitudes of proposed cuts and fills throughout the site are shown on Figure 3. Proposed cuts will be as much as about 35 feet[2] (southeast corner of site) and proposed fills will be as thick as about 17 feet (northeast corner of the site). Other improvements include asphalt-paved parking lots and drive aisles, site

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[1] Plan set titled *Pre-App Design Package*, dated October 18, 2024 and prepared by LPAS Architecture + Design

[2] Drawing titled *Rough Grading Earthwork Cut/Fill Map, Willow Ave. Apartments*, dated November 1, 2024 and prepared by CBG Civil Engineers

retaining walls, and landscaping. The proposed construction will be set back from the creek and associated FEMA flood zone in the northern portion of the site, as shown on Figures 2 and 3.

2.0 SCOPE OF SERVICES

Our preliminary investigation was performed in accordance with our contract dated December 18, 2024. Our scope of services consisted of exploring subsurface conditions at the site by drilling about six test borings, performing limited laboratory testing on select soil samples, and performing engineering analyses to develop preliminary conclusions and recommendations regarding:

- soil and groundwater conditions at the site
- site seismicity and seismic hazards, including the potential for liquefaction and liquefaction-induced ground failure
- the most appropriate foundation type(s) for the proposed buildings
- preliminary design criteria for the recommended foundation type(s), including vertical and lateral capacities
- preliminary estimates of foundation settlement
- 2022 California Building Code (CBC) site class and mapped design spectral response acceleration parameters
- construction considerations.

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

Our preliminary field investigation consisted of drilling four test borings and performing limited laboratory testing on select soil samples. Although we planned to drill as many as six test borings, soft ground conditions at the time of our field work resulted in equipment access issues and two of the planned borings were not drilled. Before performing the field investigation, we obtained a drilling permit from Contra Costa County Environmental Health Department (CCCEHD). We also contacted Underground Services Alert (USA) to notify them of our work, as required by law, and retained Subtronic Corporation of Martinez, 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

The test borings, designated as Boring B-1, B-3, B-5, and B-6, were drilled on January 7, 2025, by Taber Drilling of Sacramento, California, at the approximate locations shown in Figure 2 and Figure 3. Planned Borings B-2 and B-4 were not drilled due to access and time constraints associated with soft ground conditions but are shown in Figure 2. The borings were drilled to depths of 25 to 35 feet below the existing ground surface (bgs) using a CME-55 track-mounted drill rig equipped with 6-inch outside diameter hollow-stem flight augers. During drilling, our field engineer logged the soil and bedrock encountered and obtained representative samples for visual classification and laboratory testing. The boring logs are presented in Figures A-1 through A-4 in Appendix A. The soil and bedrock encountered in the boring were classified in accordance with the classification systems presented in Figures A-5 and A-6, respectively.

Soil samples were obtained using a 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. The sampler was driven with a 140-pound automatic 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 sampler were converted to approximate SPT N-values using a factor of 1.0 to account for sampler type and approximate hammer energy. 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 CCCEHD requirements. Soil cuttings generated during the drilling of the borings were spread onsite near the boreholes.

3.2 Laboratory Testing

We re-examined the soil samples obtained from our borings 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, dry density, particle size distribution, and plasticity (Atterberg limits). The results of the laboratory tests are presented in the boring logs and in Appendix B.

4.0 SUBSURFACE CONDITIONS

Regional geologic information (Figure 4) indicates the site is underlain by Holocene-age alluvium (Qha), although the contact for Miocene-age sedimentary rocks (Tms) is within about 50 feet of the southern boundary of the site and mapped artificial fill (af) associated with the highway embankments is located just west and north of the site. The results of our field investigation indicate the site is underlain by alluvium and bedrock. Where encountered, the alluvium consists of very stiff to hard high-plasticity clay with variable sand content, except for the sample at 1 foot at B-3, which was stiff. We encountered bedrock below the alluvium at borings B-1, B-5, and B-6 at depths between about 6 and 21 feet bgs. Where encountered, the bedrock consists of siltstone and claystone. The bedrock is soft to low hardness, plastic to weak, and deeply to completely weathered. We did not encounter bedrock in B-3, which was drilled to the maximum depth explored of 35 feet bgs.

Atterberg limits tests performed on samples of the clay and claystone encountered near the proposed finished floor elevation of four proposed building locations indicate the clay is highly to very highly expansive[4] with plasticity indices (PIs) of 36 to 46.

4.1 Groundwater

We did not encounter groundwater in any of our test borings. Based on the low permeability of the soil at the site, we judge the groundwater levels may not have been able to stabilize during our field investigation. Available historic high groundwater information presented in the *Seismic Hazard Zone Report for the Richmond, Mare Island, and San Quentin 7.5-Minute Quadrangle* prepared by the California Geological Survey (CGS, 2024) indicates a historic high groundwater level in the site vicinity to be roughly 10 feet bgs, however, given the significant amount of variability in existing ground surface elevation at the site, we conclude this mapped groundwater depth contour is not particularly meaningful. The creek at the site is at about elevation 35, which

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[4] Expansive soil undergoes large volume changes with changes in moisture content (i.e., it shrinks when dried and swells when wetted).

is about 20 to 25 feet below the proposed finished grades of the site. We preliminarily recommend assuming a design groundwater depth to match the elevation of the creek. The depth to groundwater is expected to fluctuate several feet seasonally, depending on the amount of rainfall.

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 5. For these and other active faults within a 50-kilometer radius of the site, the distance from the site and estimated characteristic moment magnitude[5] [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).

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[5] 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.

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)	7.4	Southwest	7.58
Hayward (North, HN)	7.4	Southwest	6.90
Green Valley	16	East	6.30
Concord	16	East	6.45
West Napa	17	North	6.97
Rodgers Creek - Healdsburg	23	Northwest	7.19
Mount Diablo Thrust North CFM	23	Southeast	6.72
Mount Diablo Thrust	25	Southeast	6.67
Hayward (South, HS)	27	Southeast	7.00
Clayton	27	East	6.57
Total Calaveras (CN+CC+CS+CE)	27	Southeast	7.43
Calaveras (North, CN)	27	Southeast	6.86
Great Valley 05 (Pittsburg - Kirby Hills alt1)	32	East	6.60
Great Valley 05 (Pittsburg - Kirby Hills alt2)	35	East	6.66
Total North San Andreas (SAO+SAN+SAP+SAS)	37	West	8.04
North San Andreas (Peninsula, SAP)	37	West	7.38
North San Andreas (North Coast, SAN)	37	West	7.52
Greenville (North)	38	East	6.86
San Gregorio (North)	39	West	7.44
Great Valley 04b (Gordon Valley)	42	Northeast	6.77
Mount Diablo Thrust South	42	Southeast	6.50
Hunting Creek (Berryessa)	45	North	6.69

Damaging earthquakes have occurred along many of these faults in recorded history, as depicted in Figure 5 (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 project site is in a seismically active region, we preliminarily evaluated the potential for earthquake-induced geologic hazards including ground shaking, ground surface rupture, liquefaction,[6] lateral spreading,[7] and cyclic densification.[8] We used the results of our borings to preliminarily evaluate the potential of these phenomena occurring at the site.

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

[7] 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.

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

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 San Andreas, Calaveras, and Rodges Creek 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 preliminarily 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 preliminarily conclude the probability of surface faulting, and consequently secondary ground failure from previously unknown faults, is very low.

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 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 majority of the site is not within a zone of liquefaction potential as shown on the map titled *State of California Seismic Hazard Zones, Mare Island Quadrangle, Official Map*, dated February 22, 2024 (Figure 6). Only the northern portion of the site, within the existing creek and outside the proposed development footprint, is mapped as a zone of liquefaction potential.

The soil at the site generally consists of very stiff to hard cohesive soil that is not susceptible to liquefaction due to its cohesion. The soil is also underlain by bedrock, which is not susceptible to liquefaction. Therefore, we preliminarily conclude that the potential for liquefaction and liquefaction-induced ground failures, such as lateral spreading, to occur at the site during a major earthquake is very low.

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 results of our borings indicate the soil above the groundwater at the site is not susceptible to cyclic densification due to its cohesion. Therefore, we preliminarily conclude that the potential for ground surface settlement resulting from cyclic densification at the site is very low.

6.0 PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

From a geotechnical standpoint, we preliminarily conclude there are no major geotechnical issues that would preclude development of the site as proposed. The primary geotechnical issues affecting the proposed development include: 1) the presence of highly to very highly expansive soil and bedrock and 2) the proposed differential fill thicknesses below some of the building pads, which may result in differential settlement. Our preliminary conclusions and recommendations regarding these issues are presented in the following sections.

6.1 Expansive Soil

Atterberg limits tests performed on samples of the onsite clay at depths of 1.5 to 19.5 feet bgs (to account for future cuts within the building pads during mass grading) indicate the onsite clay and claystone is highly to very highly expansive. Expansive near-surface soil is subject to volume changes during seasonal fluctuations in moisture content. These volume changes can cause movement and cracking of foundations and slabs. Therefore, foundations and slabs should be designed and constructed to mitigate the effects of the expansive soil. In general, the effects of expansive soil can be mitigated by moisture-conditioning the expansive soil, providing non-expansive soil below slabs, and either supporting foundations below the zone of severe moisture change or by providing a stiff, shallow foundation that can limit deformation of the superstructure as the underlying soil shrinks and swells.

We preliminarily recommend the upper 18 inches of soil subgrade beneath the proposed buildings be replaced with non-expansive fill. Similarly, the upper 12 inches of soil subgrade beneath exterior concrete flatwork should be replaced with non-expansive fill. The non-expansive fill may consist of lime-treated, on-site clay or select fill. Select fill should consist of imported or on-site soil that is free of organic matter, contain no rocks or lumps larger than 3 inches in greatest dimension, have a liquid limit less than 40 and plasticity index less than 12, and be approved by the Geotechnical Engineer.

Even with proper moisture conditioning and 12 inches of non-expansive fill, exterior slabs may experience some cracking due to shrinking and swelling of the underlying expansive soil. Thickening the slab edges and adding additional reinforcement will control this cracking to some degree. In addition, where slabs provide access to buildings, it would be prudent to dowel the entrance to the building to permit rotation of the slab as the exterior ground shrinks and swells and to prevent a vertical offset at the entries.

At expansive soil sites, it is critical to manage surface and subsurface drainage to prevent water from collecting beneath pavements and slabs, where it can lead to swelling and shrinking of the subgrade soil and can cause subgrade instability under vehicular loads. If permeable pavements, tree wells, irrigated landscaped zones, and stormwater infiltration basins will be constructed close to the proposed buildings, they should incorporate design elements that prevent saturation of the soil adjacent to and below building foundations. While the objective of permeable pavement systems and infiltration basins is to allow for water storage and infiltration, we preliminarily conclude infiltration into the subgrade soil is not feasible at this site due to the low permeability of the highly expansive clay and bedrock, where encountered. Furthermore, from a geotechnical standpoint, water should not be allowed to collect alongside or beneath the building foundations, pavements, and flatwork. This can be achieved by providing subdrain systems and impermeable liners beneath permeable surfaces and installing vertical barriers between permeable surfaces underlain by subdrains and non-permeable surfaces underlain by conventional aggregate base.

6.2 Foundations, Settlement, and Grading

As discussed in Section 4.0, the site is primarily underlain by very stiff to hard clay and bedrock, which are generally capable of supporting the proposed relatively light weight buildings without excessive settlement under static load conditions. However, the proposed buildings along the northern edge of the property are currently shown to be constructed at the edge of the existing slope and will be constructed on as much as about 15 feet of fill (northeast corner of Building 4). Engineered fill will undergo settlement during placement and continue to settle for a period of time after grading, the duration of which is difficult to predict. Buildings 2, 3, and 4 will transition from cut to fill conditions, which will likely result in differential settlement across the building footprints. The settlement of compacted fills can be controlled to some degree by the moisture content and level of compaction achieved during grading, however, we conclude the settlement cannot be completely mitigated through these measures. The potential for compression of fills, as well as swelling potential due to the very highly expansive nature of the site soils, should be further evaluated in the laboratory during the final geotechnical investigation for the project.

For preliminary planning and budgeting purposes, based on previous experience, we preliminarily recommend limiting differential fill thicknesses to about 7 feet below individual building pads. Even with 7 feet of differential fill thickness, there will likely be differential settlement across the pads, although it is difficult to predict the magnitude and timeline for this settlement to occur. This will likely require overexcavation of existing soils/bedrock in some areas to reduce the differential fill thickness.

If it is determined that limiting differential fill thicknesses to 7 feet below individual building pads is not cost effective, other means of controlling differential settlement may be considered, including:

- Installing aggregate piers beneath the zones of proposed fill
- Chemically treating the fill with lime to reduce its compressibility

We preliminarily recommend the buildings be supported on either stiffened mat foundations or post-tensioned (P-T) slabs using one of the mitigation measures discussed above to address differential fill thickness. Once a final investigation has been performed, detailed recommendations for design of mats or P-T slabs can be determined.

6.2.1 Site Preparation and Grading

Site demolition should include the removal of underground utilities, if any, where the new structures are 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 building 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 under the observation of the Geotechnical Engineer.

Detailed recommendations for grading and fill placement should be included in a final, design-level geotechnical investigation report once the project scope and design have been further developed.

In areas where fill wedges are planned on existing slopes, we preliminarily recommend the fills be benched into the existing slope with subdrains installed along the benches. During a final, design-level geotechnical investigation, additional borings and/or cone penetration tests (CPTs) should be performed in the proposed building footprints to better characterize depth to bedrock as it relates to proposed finished floor elevations.

6.2.2 Lime Treatment

The on-site soils may be chemically treated with lime in order to: 1) create the recommended non-expansive fill layer beneath the proposed foundations, 2) reduce the compressibility of thick fills beneath portions of the buildings, and/or 3) winterize the site and provide more stable working conditions, if grading work is performed during the rainy season. Lime treatment of fine-grained soils generally includes site preparation, application of lime, mixing, compaction, and curing of the lime-treated soil. Field quality control measures should include checking the depth of lime treatment, degree of pulverization, lime spread rate measurement, and moisture content and density measurements, and mixing efficiency.

The lime treatment process should be designed by a contractor specializing in its use and who is experienced in the application of lime in similar soil conditions. Based on our experience with lime treatment, we judge that the specialty contractor should be able to treat the highly to very highly expansive on-site material to produce a non-expansive fill for the general site fill, building pad subgrades and, if desired, for exterior flatwork and pavement subgrades. For planning purposes, we preliminarily recommend assuming the lime treatment will consist of at least 5 percent of Quicklime by dry weight of soil. An average dry unit weight of 90 pounds per cubic foot (pcf) should be preliminarily assumed for design purposes. The specialty contractor should: 1) perform a lime demand test prior to treatment to determine the percentage of Quicklime required to achieve a pH of 12.4 or higher in the treated soil, 2) perform an Atterberg limits test to confirm the proposed percentage of Quicklime will reduce the plasticity index of the treated soil to 12 or less, and 3) prepare a treatment specification for our review prior to construction.

6.3 Permanent Retaining Walls

Permanent retaining 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 preliminarily recommend both restrained and unrestrained walls be designed using an equivalent fluid weight of 61 pcf for drained conditions and 93 pcf for undrained conditions, assuming the walls are backfilled with the on-site expansive soil. These pressures may be reduced if the walls are backfilled with lime-treated on-site soil.

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 40 pcf plus a seismic increment of 32 pcf for drained conditions; and 82 pcf plus a seismic increment of 16 pcf for undrained conditions.

- Unrestrained Wall - Active earth pressure using an equivalent fluid weight of 40 pcf plus a seismic increment of 13 pcf for drained conditions; and 82 pcf plus a seismic increment of 6 pcf for undrained conditions.

Where there will be vehicle loading within 10 feet behind the wall, the wall should be designed for a vehicular surcharge of 100 psf over the upper 10 feet of the wall. If the traffic loading is limited to passenger vehicles only, the vehicular surcharge may be reduced to 50 psf. Where foundations will be supported above a “zone-of-influence” line extending up from the bottom of wall at an inclination of 1.5:1 (horizontal to vertical), the wall should be designed for a surcharge pressure. The magnitude of the surcharge pressure will need to be evaluated on a case-by-case basis.

To protect against moisture migration, all below-grade walls should be waterproofed and water stops should be placed at all construction joints. Although walls will be above the design groundwater level, water can accumulate behind the walls from other sources, such as rainfall, irrigation, broken water lines, etc. If the “drained” earth pressures presented above are used to design the walls, the walls will need to incorporate a drainage system and collector pipe as described below. 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.

One acceptable method for backdraining a retaining wall 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 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.

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).

6.4 Construction Considerations

Significant grading and cuts of up to about 15 feet below existing grades within building pads and up to about 35 feet elsewhere onsite are currently anticipated. Analytical testing should be performed on all soil to be excavated and disposed of offsite to provide data to site(s) and/or the landfill to which the soil will be transported.

If site grading is performed during the rainy season, the soil will likely be wet and will have to be dried before compaction can be achieved. Heavy rubber-tired equipment, such as scrapers and vibratory rollers, could cause excessive deflection (pumping) of the wet soil and, therefore, should be avoided. If the project schedule or weather conditions do not permit sufficient time for drying of the soil by aeration, the subgrade can be treated with lime prior to compaction, or imported granular fill can be used. The appropriate amount of lime should be determined during construction based on a visual examination and, if necessary, laboratory testing of the soil to be treated. If the grading work is performed during the dry season, moisture-conditioning may be required.

Excavations that will be entered by workers should be sloped or shored in accordance with CAL-OSHA standards (29 CFR Part 1926). Where there is sufficient clearance from the property line, the excavation sides may be slope cut at a maximum inclination of 1:1 (horizontal: vertical), which is consistent with OSHA Type B soil. Cuts in portions of the site may potentially be sloped consistent with OSHA Type A soil; however, this should be evaluated on a case-by-case basis. The contractor should be responsible for the construction and safety of temporary slopes. Where there is insufficient space to slope-cut the excavations, shoring may be required. The selection, design, construction, and performance of the shoring system (if needed) should be the responsibility of the contractor.

6.5 Excavation in Rock

With current proposed cuts of up to about 35 feet below existing grades, excavations in rock will likely be required for construction. Where encountered in our borings, the bedrock is generally plastic to weak, with soft to low hardness, and deep to complete weathering, but grades to moderately strong and moderately weathered with depth. Note that reported SPT N-values within the rock, where explored, ranged from 24 to 71 blows for 12 inches of penetration and therefore, the rock varies substantially in hardness and degree of weathering. We anticipate highly weathered rock can be excavated with conventional excavators; and harder rock at depth may require the use of hydraulic breaking equipment (i.e., a hoe ram). Furthermore, because the bedrock was characterized by discrete, widely spaced borings during our preliminary investigation, it is possible harder rock and difficult excavation may be encountered. Therefore, the contractor should be prepared to excavate hard rock, including the possible use of hydraulic breaking equipment, and should bid the project accordingly. The material descriptions and SPT N-values presented in the boring logs should be evaluated by the contractor when bidding on the project and selecting appropriate equipment.

6.6 Seismic Design

The latitude and longitude of the site are 38.0132° and -122.2692°, respectively. For preliminary design in accordance with the 2022 CBC, we recommend the following:

- Site Class D (stiff soil, non-default)
- $S_S = 1.66g$, $S_1 = 0.63g$

The 2022 CBC is based on the guidelines contained within ASCE 7-16 (Supplement 3 revision), which stipulate that if 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 indicated by an asterisk:

- $F_a = 1.0$, $F_v = 1.7$
- $S_{MS} = 1.66g$, $S_{M1}* = 1.60g$
- $S_{DS} = 1.11g$, $S_{D1}* = 1.06g$
- Seismic Design Category D for Risk Categories I, II, and III

During the final geotechnical investigation, we recommend performing a geophysical survey to confirm the average shear wave velocity in the upper 100 feet, which will allow for a more accurate determination of Site Class. Due to the presence of shallow rock in portions of the site, a Site Class C designation may be more appropriate for some of the buildings.

7.0 ADDITIONAL GEOTECHNICAL SERVICES

The preliminary conclusions and recommendations presented within are based on a preliminary field investigation and not intended for final design. Prior to final design, we should be retained to provide a final geotechnical report based on a supplemental field investigation and the final proposed development. Additional borings and/or CPTs will be required to further evaluate the subsurface conditions beneath the site. Once our final report has been completed, the design team has selected a foundation system, and prior to construction, we should review the project plans and specifications to check their conformance with the intent of our final recommendations. During construction, we should observe site grading, foundation installation, and the placement and compaction of fill. These observations will allow us to compare the actual with the anticipated soil conditions and to check if the contractors' work conforms with the geotechnical aspects of the plans and specifications.

8.0 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 preliminary 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.

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FIGURES

Base map: Google Maps, 2024

0 1,200 2,400 Feet

Approximate scale

WILLOW AVENUE APARTMENTS
HIGHWAY 80 & WILLOW AVENUE
Hercules, California

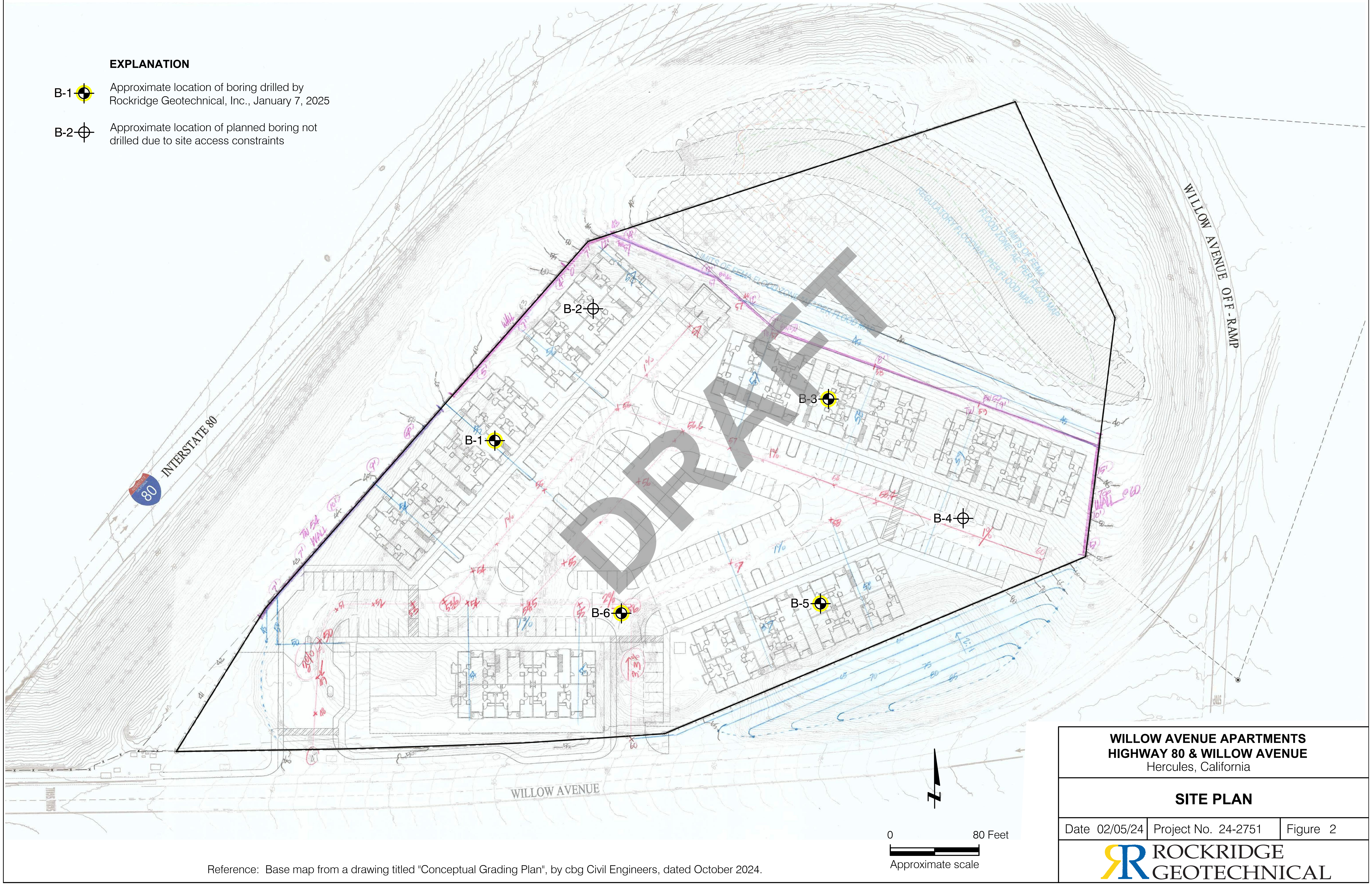
ROCKRIDGE GEOTECHNICAL

SITE LOCATION MAP

Date 12/30/24	Project No. 24-2751	Figure 1

EXPLANATION

B-1 ⊕	Approximate location of boring drilled by Rockridge Geotechnical, Inc., January 7, 2025
B-2 ⊕	Approximate location of planned boring not drilled due to site access constraints



Reference: Base map from a drawing titled "Conceptual Grading Plan", by cbg Civil Engineers, dated October 2024.

WILLOW AVENUE APARTMENTS
HIGHWAY 80 & WILLOW AVENUE
Hercules, California

SITE PLAN

Date 02/05/24	Project No. 24-2751	Figure 2

ROCKRIDGE GEOTECHNICAL

ELEVATIONS TABLE

NUMBER	MINIMUM ELEVATION	MAXIMUM ELEVATION	AREA (AC)	COLOR
1	−40.00	−35.00	0.0±	🟥
2	−35.00	−30.00	0.1±	🟥
3	−30.00	−25.00	0.1±	🟥
4	−25.00	−20.00	0.3±	🟥
5	−20.00	−15.00	0.4±	🟥
6	−15.00	−10.00	0.7±	🟥
7	−10.00	−5.00	0.9±	🟥
8	−5.00	0.00	1.6±	🟥
9	0.00	5.00	1.3±	🟩
10	5.00	10.00	0.2±	🟩
11	10.00	15.00	0.1±	🟩
12	15.00	20.00	0.0±	🟩

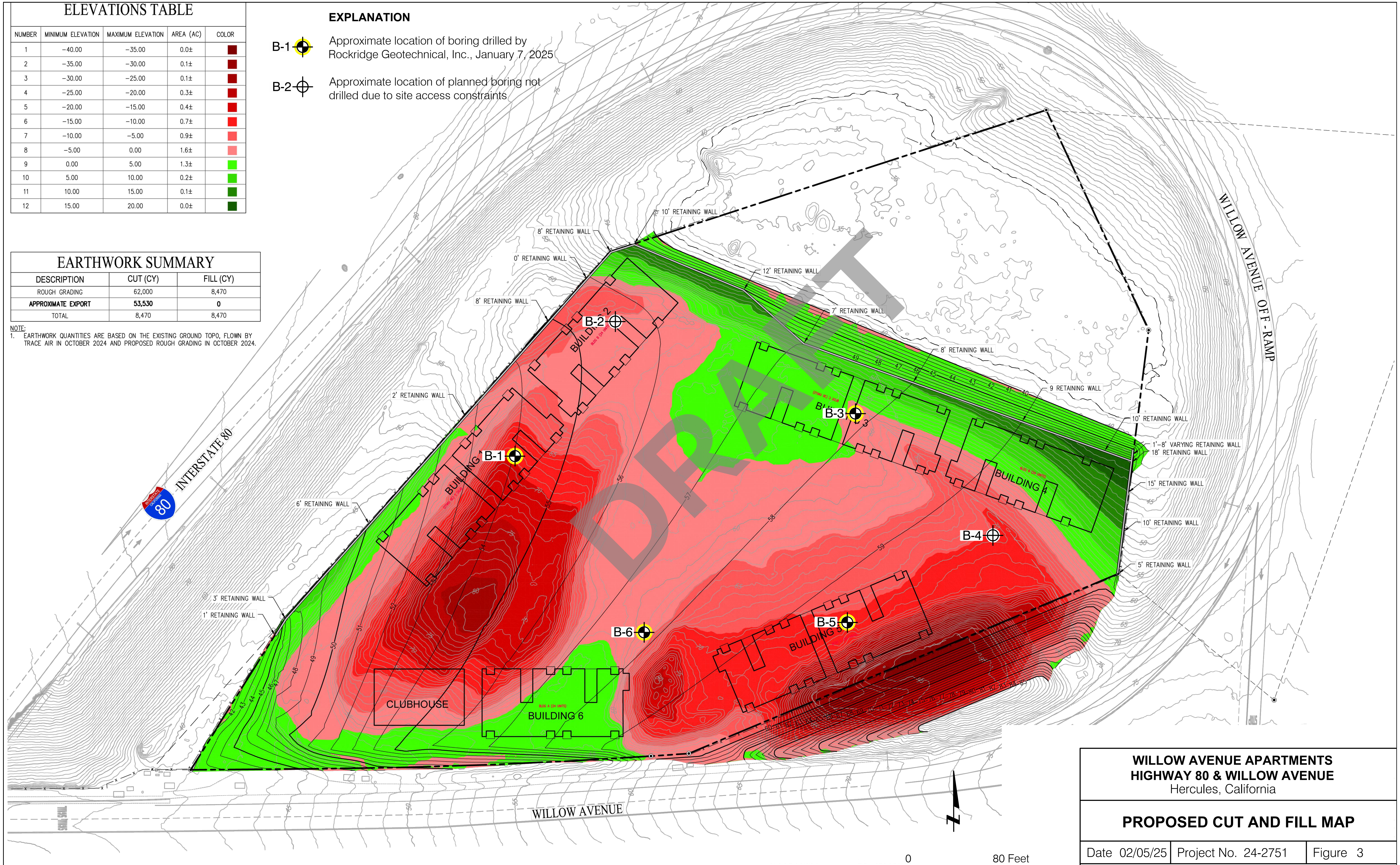
EXPLANATION

B-1 ⊕	Approximate location of boring drilled by Rockridge Geotechnical, Inc., January 7, 2025
B-2 ⊕	Approximate location of planned boring not drilled due to site access constraints

EARTHWORK SUMMARY

DESCRIPTION	CUT (CY)	FILL (CY)
ROUGH GRADING	62,000	8,470
APPROXIMATE EXPORT	**53,530**	**0**
TOTAL	8,470	8,470

NOTE:
1. EARTHWORK QUANTITIES ARE BASED ON THE EXISTING GROUND TOPO, FLOWN BY TRACE AIR IN OCTOBER 2024 AND PROPOSED ROUGH GRADING IN OCTOBER 2024.



Reference: Base map from a drawing titled "Rough Grading Earthwork Cut/Fill Map", by cbg Civil Engineers, dated November 1, 2024.

WILLOW AVENUE APARTMENTS
HIGHWAY 80 & WILLOW AVENUE
Hercules, California

PROPOSED CUT AND FILL MAP

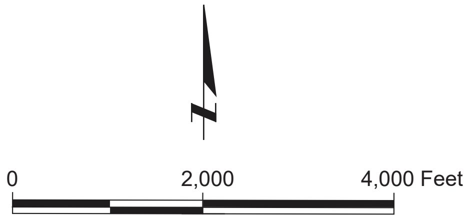
Date 02/05/25	Project No. 24-2751	Figure 3

ROCKRIDGE GEOTECHNICAL

Base map: Google Earth with U.S. Geological Survey (USGS), Contra Costa County, 2024

EXPLANATION

af	Artificial Fill
Qha	Alluvium (Holocene)
Qpa	Alluvium (Pleistocene)
Tms	Sedimentary rocks (Miocene)
⁓	Geologic contact: dashed where approximate and dotted where concealed, queried where uncertain



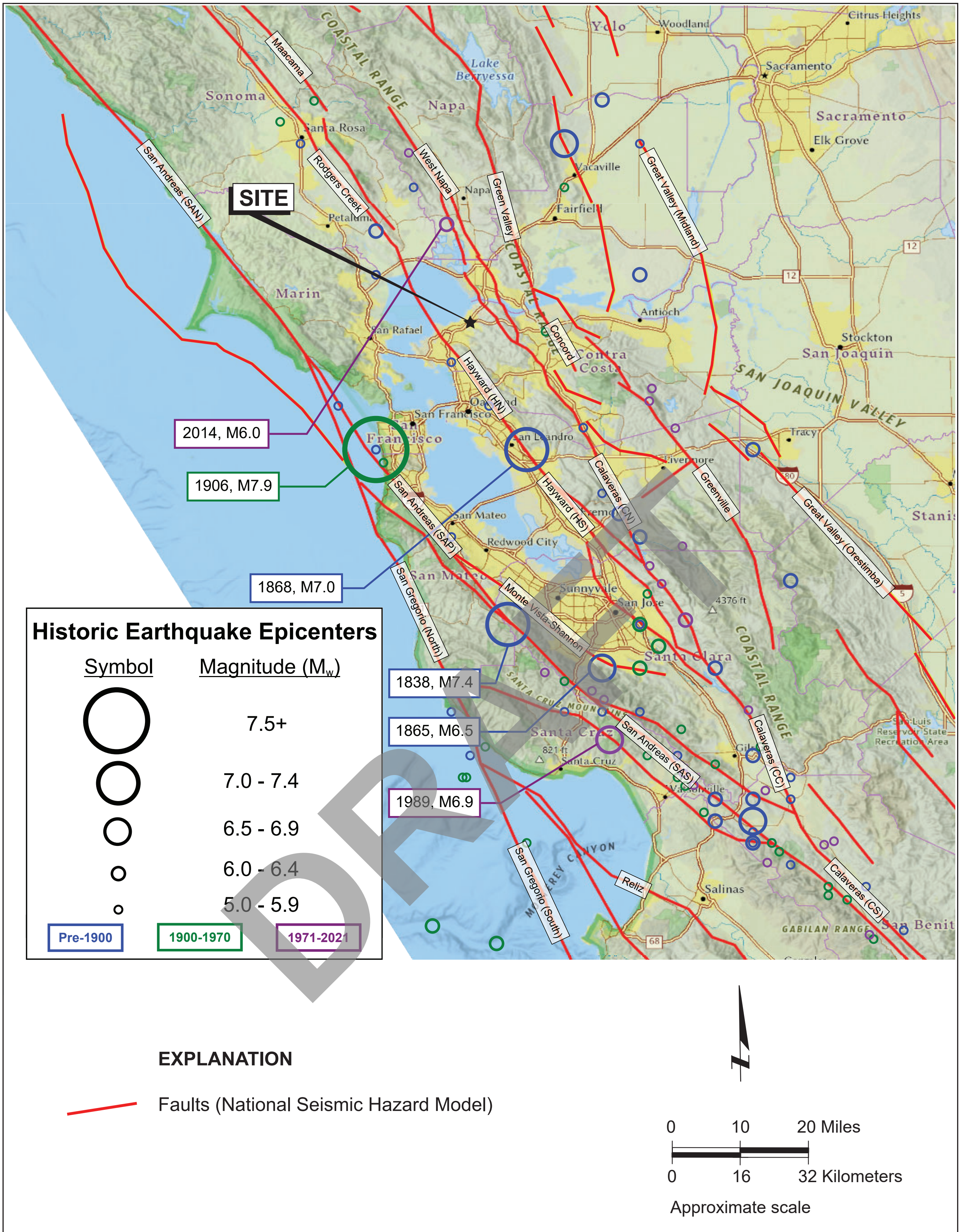
Approximate scale

WILLOW AVENUE APARTMENTS
HIGHWAY 80 & WILLOW AVENUE
Hercules, California

ROCKRIDGE
GEOTECHNICAL

REGIONAL GEOLOGIC MAP

Date 02/04/25 | Project No. 24-2751 | Figure 4

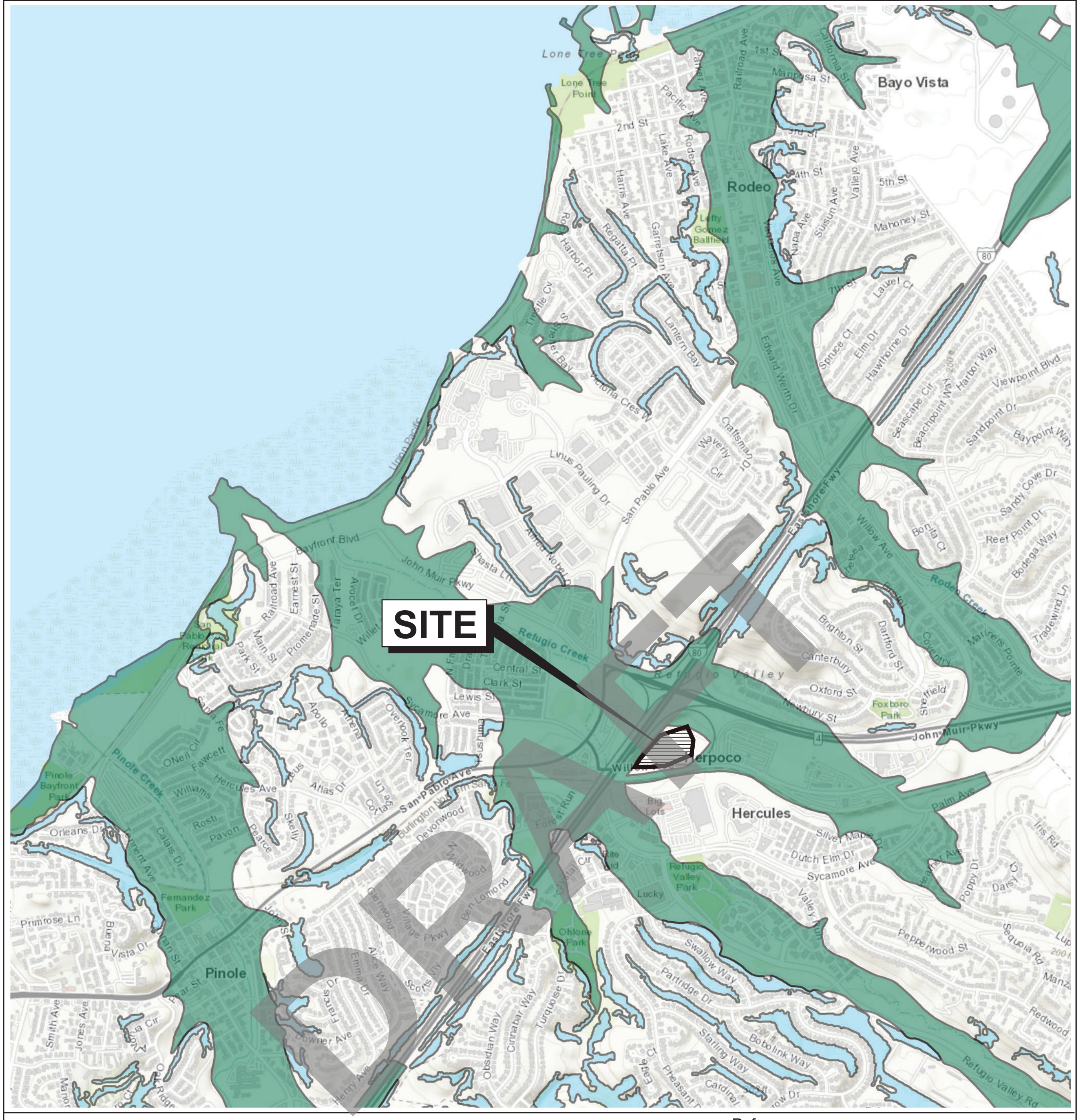


WILLOW AVENUE APARTMENTS
HIGHWAY 80 & WILLOW AVENUE
Hercules, California

ROCKRIDGE
GEOTECHNICAL

REGIONAL FAULT AND HISTORIC SEISMICITY MAP

Date 02/04/25 | Project No. 24-2751 | Figure 5



🟩 **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
Mare Island Quadrangle
California Geological Survey
Released February 22, 2024

0 2,000 4,000 Feet
Approximate scale

WILLOW AVENUE APARTMENTS
HIGHWAY 80 & WILLOW AVENUE
Hercules, California

ROCKRIDGE
GEOTECHNICAL

EARTHQUAKE ZONES OF REQUIRED INVESTIGATION MAP

Date 02/04/25	Project No. 24-2751	Figure 6

APPENDIX A
Logs of Borings

PROJECT: **WILLOW AVENUE APARTMENTS**
HIGHWAY 80 & WILLOW AVENUE
Hercules, California

Log of Boring B-1

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2
Date started: 01/07/2025
Date finished: 01/07/2025
Drilling method: 6-inch-diameter hollow-stem auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Automatic Safety Hammer
Sampler: Modified California (MC)

Logged by: D. Fernandez
Drilled by: Taber Drilling
Rig: CME-55 Track Rig

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
0–1						SANDY CLAY (CH) olive-brown to dark brown, very stiff, moist, fine sand						
5–6	MC	■	4 7 10	17								
8					CH							
10–11	MC	■	9 14 20	34		olive-gray, hard, dry to moist						
15–16	MC	■	8 12 17	29		very stiff						
17.5–19					CH	CLAY with SAND (CH) olive-gray, hard, moist, fine sand, trace fine subangular gravel						
19–20	MC	■	8 15 16	31		LL = 58, PI = 36; see Appendix B Particle Size Distribution; see Appendix B				73	21.3	88
21–22						SILTSTONE olive-gray, moderately hard, weak to moderately strong						
24–25	MC	■	11 12 19	31								
27												

Boring terminated at a depth of 27 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

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[1] MC blow counts for the last two increments were converted to SPT N-Values using a factor of 1.0 to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 24-2751

Figure: A-1

PROJECT:

WILLOW AVENUE APARTMENTS
HIGHWAY 80 & WILLOW AVENUE
Hercules, California

Log of Boring B-3

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2
Date started: 01/07/2025
Date finished: 01/07/2025
Drilling method: 6-inch-diameter hollow-stem auger
Logged by: D. Fernandez
Drilled by: Taber Drilling
Rig: CME-55 Track Rig
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Automatic Safety Hammer
Sampler: Modified California (MC)

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						CLAY (CH) olive-gray, stiff, moist, trace fine sand, trace rootlets LL = 69, PI = 46; see Appendix B						
2	MC	▨ ☐	0 4 6	10		Particle Size Distribution; see Appendix B				91	38.0	82
3					CH	very stiff						
4	MC	⊡	5 10 16	26								
5												
6												
7												
8						SANDY CLAY (CH) olive-brown, very stiff, moist, fine to medium sand						
9	MC	▨	4 7 10	17								
10												
11												
12					CH							
13												
14	MC	▨	5 10 16	26		dark brown and olive-brown						
15												
16												
17						CLAY (CL) olive-brown, very stiff, moist, trace fine sand						
18					CL							
19	MC	▨	4 12 15	27								
20					CL	SANDY CLAY (CL) olive-brown, very stiff, moist, fine sand						
21												
22						CLAY with SAND (CL) olive-brown, very stiff, moist, fine to medium sand						
23												
24	MC	▨	9 11 11	22	CL							
25												
26												
27						SANDY CLAY (CL) dark brown, hard, moist, fine sand						
28					CL							
29	MC	▨	9 13 19	32								
30												

ROCKRIDGE GEOTECHNICAL

Project No.: 24-2751
Figure: A-2a

PROJECT: **WILLOW AVENUE APARTMENTS**
HIGHWAY 80 & WILLOW AVENUE
Hercules, California

Log of Boring B-3

PAGE 2 OF 2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
						SANDY CLAY (CL) (continued)						
31												
32												
33					CL	gray						
34	MC	■	12 14 16	30								
35												
36												
37												
38												
39												
40												
41												
42												
43												
44												
45												
46												
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 35 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] MC blow counts for the last two increments were converted to SPT N-Values using a factor of 1.0 to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 24-2751

Figure: A-2b

PROJECT: **WILLOW AVENUE APARTMENTS**
HIGHWAY 80 & WILLOW AVENUE
Hercules, California

Log of Boring B-5

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2
Date started: 01/07/2025
Date finished: 01/07/2025
Drilling method: 6-inch-diameter hollow-stem auger
Logged by: D. Fernandez
Drilled by: Taber Drilling
Rig: CME-55 Track Rig
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Automatic Safety Hammer
Sampler: Modified California (MC)

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
						SANDY CLAY (CL) light olive-gray, very stiff, dry to moist, fine sand						
1												
2												
3												
4												
5		■			CL							
6	MC	■	4 9 20	29								
7												
8												
9												
10		■				red-gray						
11	MC	■	4 9 13	22								
12												
13						CLAYSTONE light brown, gray, and red, soft, plastic to friable						
14	MC	■	5 14 17	31		LL = 66, PI = 44; see Appendix B Particle Size Distribution; see Appendix B				98	31.7	86
15												
16												
17												
18												
19	MC	■	8 12 16	28		olive-gray, deeply to completely weathered						
20												
21												
22												
23												
24	MC	■	14 25 46	71		olive-brown, soft to low hardness, friable						
25												
26												
27												
28												
29	MC	■	14 25 29	54		gray, low hardness, friable to weak						
30												

Boring terminated at a depth of 30 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] MC blow counts for the last two increments were converted to SPT N-Values using a factor of 1.0 to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 24-2751
Figure: A-3

PROJECT: **WILLOW AVENUE APARTMENTS**
HIGHWAY 80 & WILLOW AVENUE
Hercules, California

Log of Boring B-6

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2
Date started: 01/07/2025
Date finished: 01/07/2025
Drilling method: 6-inch-diameter hollow-stem auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Automatic Safety Hammer
Sampler: Modified California (MC)

Logged by: D. Fernandez
Drilled by: Taber Drilling
Rig: CME-55 Track Rig

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		▨				SANDY CLAY (CH) brown and olive-brown, very stiff, moist, fine to medium stiff, trace subangular to subrounded gravel						
2	MC	▨	5 8 8	16	CH							
3												
4	MC	▨	5 9 15	24	CH	CLAY (CH) dark brown and black with orange, very stiff, moist, fine to medium sand LL = 66, PI = 42; see Appendix B Particle Size Distribution; see Appendix B				87	32.6	85
5												
6	MC	▨	9 22 39	61		CLAYSTONE red-yellow, low hardness, friable weak						
7												
8												
9	MC	▨	4 9 15	24		yellow, soft to low hardness, plastic						
10												
11												
12												
13												
14	MC	▨	17 26 43	69		red-yellow, low hardness, friable						
15												
16												
17												
18												
19	MC	▨	12 16 22	38		plastic to friable						
20												
21												
22												
23												
24	MC	▨	19 20 28	48		black						
25												
26												
27												
28												
29												
30												

Boring terminated at a depth of 25 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] MC blow counts for the last two increments were converted to SPT N-Values using a factor of 1.0 to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 24-2751
Figure: A-4

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

WILLOW AVENUE APARTMENTS
HIGHWAY 80 & WILLOW AVENUE
Hercules, California

ROCKRIDGE
GEOTECHNICAL

CLASSIFICATION CHART

Date 01/27/25 | Project No. 24-2751 | Figure A-5

I FRACTURING

Intensity	Size of Pieces in Feet
Very little fractured	Greater than 4.0
Occasionally fractured	1.0 to 4.0
Moderately fractured	0.5 to 1.0
Closely fractured	0.1 to 0.5
Intensely fractured	0.05 to 0.1
Crushed	Less than 0.05

II HARDNESS

1. **Soft** - reserved for plastic material alone.
2. **Low hardness** - can be gouged deeply or carved easily with a knife blade.
3. **Moderately hard** - can be readily scratched by a knife blade; scratch leaves a heavy trace of dust and is readily visible after the powder has been blown away.
4. **Hard** - can be scratched with difficulty; scratch produced a little powder and is often faintly visible.
5. **Very hard** - cannot be scratched with knife blade; leaves a metallic streak.

III STRENGTH

1. **Plastic** or very low strength.
2. **Friable** - crumbles easily by rubbing with fingers.
3. **Weak** - an unfractured specimen of such material will crumble under light hammer blows.
4. **Moderately strong** - specimen will withstand a few heavy hammer blows before breaking.
5. **Strong** - specimen will withstand a few heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.
6. **Very strong** - specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.

IV **WEATHERING -** The physical and chemical disintegration and decomposition of rocks and minerals by natural processes such as oxidation, reduction, hydration, solution, carbonation, and freezing and thawing.

D. Deep - moderate to complete mineral decomposition; extensive disintegration; deep and thorough discoloration; many fractures, all extensively coated or filled with oxides, carbonates and/or clay or silt.

M. Moderate - slight change or partial decomposition of minerals; little disintegration; cementation little to unaffected. Moderate to occasionally intense discoloration. Moderately coated fractures.

L. Little - no megascopic decomposition of minerals; little of no effect on normal cementation. Slight and intermittent, or localized discoloration. Few stains on fracture surfaces.

F. Fresh - unaffected by weathering agents. No disintegration of discoloration. Fractures usually less numerous than joints.

ADDITIONAL COMMENTS:

V **CONSOLIDATION OF SEDIMENTARY ROCKS:** usually determined from unweathered samples. Largely dependent on cementation.

U = unconsolidated
P = poorly consolidated
M = moderately consolidated
W = well consolidated

VI BEDDING OF SEDIMENTARY ROCKS

Splitting Property	Thickness	Stratification
Massive	Greater than 4.0 ft.	very thick-bedded
Blocky	2.0 to 4.0 ft.	thick bedded
Slabby	0.2 to 2.0 ft.	thin bedded
Flaggy	0.05 to 0.2 ft.	very thin-bedded
Shaly or platy	0.01 to 0.05 ft.	laminated
Papery	less than 0.01	thinly laminated

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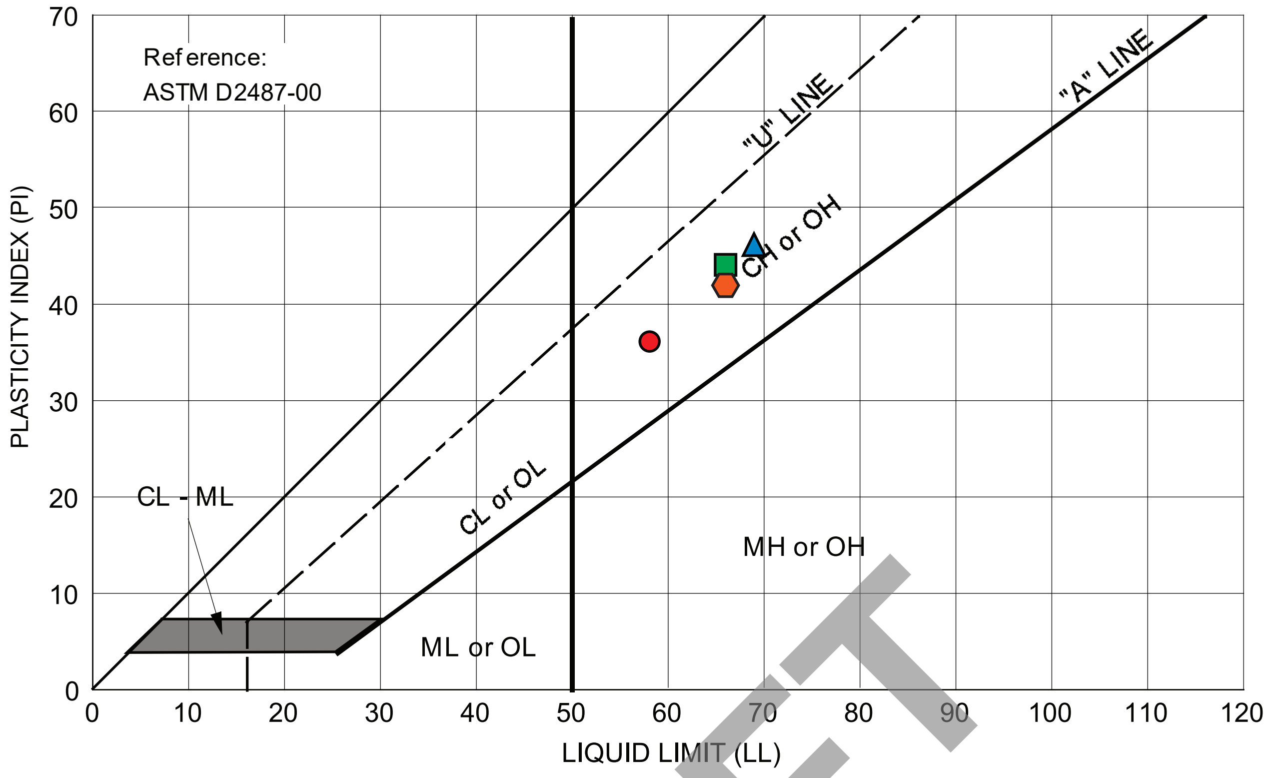
PHYSICAL PROPERTIES CRITERIA FOR ROCK DESCRIPTIONS

Date 01/27/25 | Project No. 24-2751 | Figure A-6

APPENDIX B
Laboratory Test Results



Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-1 at 19.5 feet	CLAY with SAND (CH), olive-gray	21.3	58	36	73
▲	B-3 at 1.5 feet	CLAY (CH), olive-gray	38.0	69	46	91
■	B-5 at 14.0 feet	CLAYSTONE, light brown, gray, and red	31.7	66	44	98
⬢	B-6 at 3.5 feet	CLAY (CH), dark brown and black with orange	32.6	66	42	87

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PLASTICITY CHART

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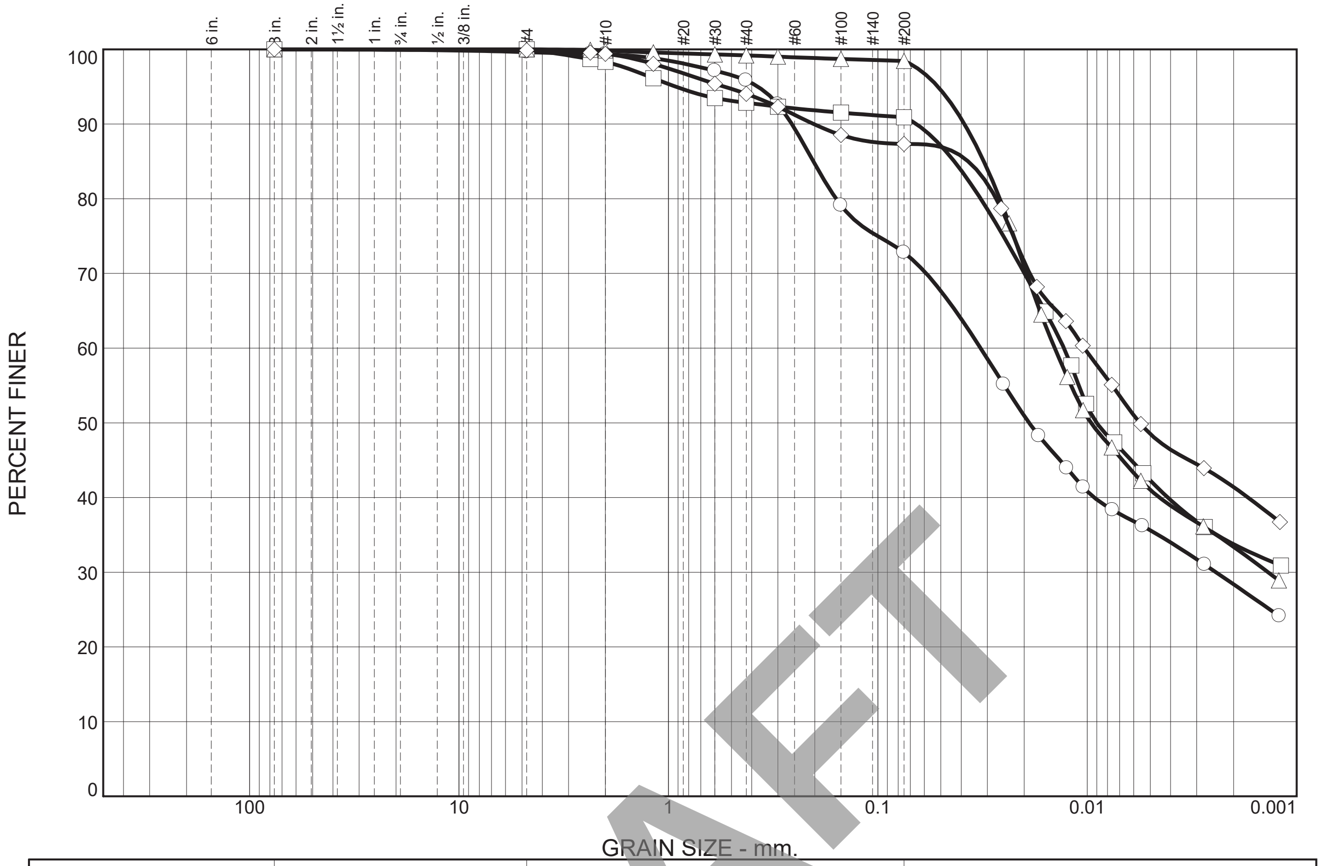

	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.1	0.3	0.3	3.5	23.0	37.2	35.6
□	0.0	0.0	0.0	1.7	5.4	2.0	48.6	42.3
△	0.0	0.0	0.0	0.2	0.6	0.8	57.4	41.0
◇	0.0	0.1	0.0	0.5	5.4	6.7	38.8	48.5

SOIL DATA

SYMBOL	SOURCE	DEPTH (ft.)	Material Description	USCS
○	B-1	19.5'	CLAY with SAND, olive-gray	CH
□	B-3	1.5'	CLAY, olive-gray	CH
△	B-5	14.0'	CLAYSTONE, light brown, gray, and red	CH
◇	B-6	3.5'	CLAY, dark brown and black with orange	CH

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PARTICLE SIZE DISTRIBUTION REPORT

Date 01/27/25 | Project No. 24-2751 | Figure B-2