ROCKRIDGE
GEOTECHNICAL

Prepared for **Greystar**

FINAL GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
749 W. EL CAMINO REAL
MOUNTAIN VIEW, CALIFORNIA

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April 8, 2022
Project No. 20-1817

ROCKRIDGE
GEOTECHNICAL

April 8, 2022
Project No. 20-1817

Mr. Tyler Evie
Development Director
Greystar
450 Sansome Street, Suite 500
San Francisco, California 94111

Subject: Final Geotechnical Investigation
Proposed Mixed-Use Development
749 W. El Camino Real
Mountain View, California

Dear Mr. Evje,

We are pleased to present the results of our final geotechnical investigation for the proposed mixed-use development at 749 W. El Camino Real in Mountain View, California. Our investigation was performed in accordance with our proposal dated January 14, 2022.

The subject property is located on the southern corner of the intersection of W. El Camino Real and Castro Street. The site is L-shaped with maximum plan dimensions of about 340 feet by 540 feet. The site is currently occupied by a two-story commercial building (Chase Bank), a single-story restaurant, paved parking lots, and a vacant undeveloped lot.

Plans are to demolish the existing improvements and construct a two-story commercial building (new Chase Bank building) and plaza at the northern corner of the site fronting W. El Camino Real and Castro Street, and a new mixed-use building that will occupy the remainder of the site. The proposed commercial building will be constructed at-grade. The proposed mixed-use building will consist of five levels of Type IIIA wood frame construction over three levels of Type IA concrete construction. The proposed mixed-use building will be five levels of residential units atop ground level parking garage, retail and residential space. The proposed mixed-use building will also have two levels of below-grade parking with finished floor of the lower parking level at about 21 feet below grade.

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 for this project are the following:

- the need for an adequate shoring system to support the proposed excavation for the two basement parking levels of the proposed mixed-use building;
- providing adequate foundation support for the proposed buildings.

We conclude the proposed buildings may be supported on shallow foundations, such as spread footings or mats bearing on firm native soil.

Our report contains specific recommendations regarding shoring design, foundation design, fill placement and compaction, and other geotechnical aspects pertaining to the project. The recommendations contained in our report are based on limited subsurface exploration. Consequently, variations between expected and actual soil conditions may be found in localized areas during construction. Therefore, we should be engaged to observe installation of temporary shoring, new foundations and fill placement and compaction, during which time we may make changes in our recommendations, if deemed necessary.

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

Sincerely,
ROCKRIDGE GEOTECHNICAL, INC

KRYSTIAN SAMLIK
C 85202
EXP. 6/30/22
CIVIL

HUI JUN LIANG
GE2663
Exp. 6/30/2022
GEOTECHNICAL

Krystian P. Samlik, P.E.
Senior Project Engineer

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

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FINAL GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
749 W. EL CAMINO REAL
Mountain View, California

1.0 INTRODUCTION

This report presents the results of the final geotechnical investigation performed by Rockridge Geotechnical, Inc. for the proposed mixed-use development at 749 W. El Camino Real in Mountain View, California. The subject property is located on the southern corner of the intersection of W. El Camino Real and Castro Street, as shown on the Site Location Map, Figure 1.

The site is L-shaped with maximum plan dimensions of about 340 feet by 540 feet, as shown on the Site Plan, Figure 2. The site encompasses an area of about 3.05 acres and is relatively level. The site is currently occupied by a two-story commercial building (Chase Bank), a single-story restaurant, paved parking lots, and a vacant undeveloped lot.

Plans are to demolish the existing improvements and construct a two-story commercial building (new Chase Bank building) and plaza at the northern corner of the site fronting W. El Camino Real and Castro Street, and a new mixed-use building that will occupy the remainder of the site. The proposed commercial building will be constructed at-grade. The proposed mixed-use building will consist of five levels of Type IIIA wood frame construction over three levels of Type IA concrete construction. The proposed mixed-use building will be five levels of residential units atop ground level parking garage, retail and residential space. The proposed mixed-use building will also have two levels of below-grade parking with finished floor of the lower parking level at about 21 feet below grade.

2.0 SCOPE OF SERVICES

Our services were performed in accordance with our proposal dated January 14, 2022. Our scope of services consisted of evaluating subsurface conditions at the site by drilling three test borings, performing eight cone penetration tests (CPTs), performing laboratory tests on selected soil samples, and performing engineering analyses to develop conclusions and recommendations regarding:

- subsurface conditions
- design groundwater level
- site seismicity and seismic hazards, including the potential for liquefaction and liquefaction-induced ground failure and cyclic densification
- the most appropriate foundation type(s) for the proposed building
- design criteria for the recommended foundation type(s), including vertical and lateral capacities
- estimates of foundation settlement
- lateral earth pressures for design of permanent below-grade walls
- temporary cut slopes and shoring
- subgrade preparation for the concrete slab-on-grade floor for the building and exterior concrete flatwork
- site grading and excavation, including criteria for fill quality and compaction
- 2019 and 2022 California Building Code (CBC) site class and design spectral response acceleration parameters
- corrosivity of the near-surface soil and the potential effects on buried concrete and metal structures and foundations
- permeable and non-permeable non-vehicular pavers
- construction considerations.

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

Subsurface conditions at the site were explored by drilling three borings, performing eight CPTs, and performing laboratory testing on selected soil samples. Prior to performing the field exploration, we obtained a drilling permit from the Santa Clara Valley Water District (SCVWD). In addition, we contacted Underground Service Alert (USA) to notify them of our work, as required by law, and retained C. Cruz Sub-Surface Locators, a private utility locator, to check for buried utilities at the boring and CPT locations to reduce the potential for encountering buried utilities during drilling. Details of the field investigation and laboratory testing are described in this section.

3.1 Test Borings

Three test borings, designated as Borings B-1 through B-3, were drilled on February 23, 2022 by Exploration Geoservices of San Jose, California, at the approximate location shown on Figure 2. All three borings were drilled to a depth of 40 feet below the existing ground surface (bgs) using a truck-mounted drill rig equipped with eight-inch diameter hollow-stem flight augers. During drilling, our field engineer logged the soil encountered and obtained representative samples for visual classification and laboratory testing. The logs of the borings are presented on Figures A-1 through A-3 in Appendix A. The soil encountered in the borings was classified in accordance with the classification chart presented on Figure A-4.

Soil samples were obtained using the following samplers:

- 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 type of sampler used was selected based on soil type and the desired sample quality for laboratory testing. In general, the MC sampler was used to obtain samples in stiff cohesive soil and the SPT sampler was used to evaluate the relative density of granular soils and to obtain samples in hard cohesive soil. The samplers were driven with a 140-pound downhole 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 six inches and are presented on the boring logs. A “blow count” is defined as the number of hammer blows per six inches of penetration or 50 blows for six 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 0.63 and 1.08, 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: (1) the last two blow counts if the sampler was driven more than 12 inches, (2) the last one blow count if the sampler was driven more than six inches but less than 12 inches, and (3) the only blow count if the sampler was driven six inches or less. The converted SPT N-values are presented on the boring logs.

Upon completion, the boreholes were backfilled with neat cement grout in accordance with SCVWD requirements. The soil cuttings from Boring B-1 were spread on the ground near the borehole and the soil cuttings from borings B-2 and B-3 were off hauled for disposal.

3.2 Cone Penetration Tests

Eight CPTs, designated as CPT-1 through CPT-8, were performed on March 3, 2022 by Middle Earth Geo Testing, Inc. of Orange, California at the approximate locations shown on Figure 2. All CPTs were advanced to a depth of about 60-1/2 feet bgs. The CPTs were performed by hydraulically pushing a 1.7-inch-diameter cone-tipped probe with a projected area of 15 square centimeters into the ground. The cone-tipped probe measured tip resistance and the friction sleeve behind the cone tip measured frictional resistance. Electrical strain gauges within the cone continuously measured soil parameters for the entire depth advanced. Soil data, including tip resistance, frictional resistance, and pore water pressure were recorded by a computer while the test was conducted. Accumulated data were processed by computer to provide engineering information such as the soil behavior types and approximate strength characteristics of the soil encountered.

The CPT logs showing tip resistance and friction ratio, as well as interpreted soil behavior type, are presented in Appendix A on Figures A-5 through A-12. Upon completion, the CPT holes were backfilled with cement grout in accordance with SCVWD requirements.

4.0 SUBSURFACE CONDITIONS

The regional geology map prepared by U.S. Geological Survey (USGS), a portion of which is shown on Figure 3, indicates the site is underlain by Holocene-age alluvium (Qha). Alluvial deposits generally consist of a mixture of fine-grained and coarse-grained deposits and are deposited by rivers and streams. Where explored, the alluvium consists of predominately clay with varying sand and gravel content interbedded with sand and gravel with varying clay and silt content that extends to the maximum depth explored of 60-1/2 feet bgs. The clay is stiff to hard and the sand and gravel layers are medium dense to very dense.

Atterberg limits tests performed on samples of the near-surface clay obtained from borings B-1 and B-2 indicate the near surface clay has plasticity indices of 18 and 23 and, therefore, has moderate expansion potential[1].

4.1 Groundwater

Groundwater was not encountered in our borings during and at the end of drilling. Groundwater was measured in CPT-2, 4, and -5 at depths of 49, 45, and 49 feet bgs, respectively, using a weighted tape prior to grouting. Groundwater was not encountered in the other CPTs. Considering the relatively high clay content of the subsurface soils, we judge the groundwater levels in the borings and CPTs may not have been fully stabilized at the time of these measurements.

The report prepared by the California Geological Survey titled *Seismic Hazard Zone Report for the Mountain View 7.5-minute Quadrangle, Santa Clara County, California* (2006) indicates the historic high groundwater level in the site vicinity is approximately 35 feet bgs.

The groundwater level at the site is expected to fluctuate several feet seasonally with potentially larger fluctuations annually, depending on the amount of rainfall. Based on available

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

groundwater information, we conclude a design high groundwater depth of 35 feet bgs be used for this project.

5.0 SEISMIC CONSIDERATION

5.1 Regional Seismicity and Faulting

The site is located in the Coast Ranges Geomorphic Province of California that 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 North American plates and subsequent strike-slip faulting along the San Andreas Fault system. The San Andreas Fault is more than 600 miles long 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 San Andreas, Hayward, and Monte Vista faults. These and other faults in the region are shown on Figure 4. Numerous damaging earthquakes have occurred along these faults in recorded time. 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).

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[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.

TABLE 1
Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction	Characteristic Moment Magnitude
Monte Vista - Shannon	3.9	Southwest	7.14
Total North San Andreas (SAO+SAN+SAP+SAS)	10	Southwest	8.04
North San Andreas (Peninsula, SAP)	10	Southwest	7.38
Total Hayward + Rodgers Creek (RC+HN+HS+HE)	19	Northeast	7.58
Hayward (South, HS)	19	Northeast	7.00
Butano	20	Southwest	6.93
North San Andreas (Santa Cruz Mts, SAS)	24	Southeast	7.15
Hayward (Extension, HE)	25	East	6.18
Total Calaveras (CN+CC+CS+CE)	25	East	7.43
Calaveras (Central, CC)	25	East	6.85
Calaveras (North, CN)	25	East	6.86
Zayante-Vergeles (2011 CFM)	27	Southwest	7.48
San Gregorio (North)	28	West	7.44
Sargent	30	Southeast	6.71
Las Positas	34	Northeast	6.50
Zayante-Vergeles	34	Southeast	7.00
Mount Diablo Thrust	45	Northeast	6.67
Hayward (North, HN)	45	North	6.90
Mount Diablo Thrust South	45	Northeast	6.50
Mount Diablo Thrust North CFM	46	Northeast	6.72
Reliz	46	Southwest	7.44
Greenville (North)	49	East	6.86
Greenville (South)	50	East	6.64

Since 1800, four major earthquakes have been recorded on the North San Andreas Fault. In 1836, an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale occurred east of Monterey Bay on the San Andreas Fault (Toppozada and Borchardt 1998). The estimated moment magnitude (M_w) for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to an M_w of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas Fault from Shelter Cove to San Juan Bautista approximately 470 kilometers in length. It had a maximum intensity of XI (MM), an M_w of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The Loma Prieta Earthquake of October 17, 1989 had an M_w of 6.9 and occurred about 43 kilometers south of the site.

In 1868, an earthquake with an estimated maximum intensity of X on the MM scale occurred on the southern segment (between San Leandro and Fremont) of the Hayward Fault. The estimated M_w for the earthquake is 7.0. In 1861, an earthquake of unknown magnitude (estimated M_w of about 6.5) was reported on the Calaveras Fault. The most recent significant earthquake on this fault was the 1984 Morgan Hill earthquake, which corresponds to an M_w of 6.2.

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 the North 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 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 field investigation to evaluate the potential of these phenomena occurring at the project site.

5.2.1 Ground Shaking

The ground shaking intensity felt at the project site will depend on: 1) the size of the earthquake (magnitude), 2) the distance from the site to the fault source, 3) the directivity (focusing of earthquake energy along the fault in the direction of the rupture), and 4) subsurface conditions. The site is less than 25 kilometers from three major faults (San Andreas, Hayward, and Calaveras faults). Therefore, the potential exists for a large earthquake to induce strong to very strong ground shaking at the site during the life of the project.

5.2.2 Fault 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. We, therefore, conclude the risk of fault offset at the site from a known active fault is 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 risk of surface faulting and consequent secondary ground failure from previously unknown faults is also very low.

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[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 Associated Hazards

When a saturated, cohesionless soil liquefies, it experiences a temporary loss of shear strength created by a temporary 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.

As shown on Figure 5, the site is not located within zone of liquefaction potential on the map titled *Earthquake Zones of Required Investigation, Mountain View Quadrangle, Official Map,* prepared by the California Geological Survey (CGS), dated October 18, 2006. Considering the soil encountered below the design groundwater level (35 feet bgs) is very stiff to hard clay and dense to very dense sand, which is not susceptible to liquefaction, we conclude the potential for liquefaction and associated hazards to occur at the site is nil.

5.2.4 Cyclic Densification

Cyclic densification (also referred to as differential compaction) of non-saturated sand (sand above groundwater table) can occur during an earthquake, resulting in settlement of the ground surface and overlying improvements. The borings and CPTs indicate the soil above the groundwater at the site consists of soil that is sufficiently dense and/or sufficiently cohesive to resist cyclic densification. Consequently, we conclude the potential for building settlement resulting from cyclic densification is nil.

6.0 DISCUSSIONS 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 for this project are the following:

- the need for an adequate shoring system to support the proposed excavation for the two basement parking levels of the proposed mixed-use building;
- providing adequate foundation support for the proposed buildings.

These and other geotechnical issues, as they pertain to the proposed development, are discussed in this section.

6.1 Design Groundwater Table

Based on the historic high groundwater level in the site vicinity as discussed in Section 4.1, we conclude a design groundwater depth of 35 feet bgs should be used for this project. We understand the finished floor for the lower basement level of the proposed mixed-use building will be about 21 feet below grade, which is at about 14 feet above the design groundwater table.

6.2 Foundation and Settlement

We anticipate the foundation level of the proposed at-grade commercial building and the proposed mixed-use building with two basement levels will be underlain by firm native alluvium that can support moderate building loads. Therefore, we conclude the proposed buildings can be supported on shallow foundations, such as conventional spread footings or mats.

We estimate total and differential settlements of properly constructed spread footings or mats designed based on the recommendations presented in Section 7.2 of this report will be less than one inch and 1/2 inch across a 30-foot horizontal distance, respectively.

6.3 Excavation Support

We anticipate excavation of about 23 to 24 feet in depth will be needed to construct the basement levels of the proposed mixed-use building. Excavations that will be deeper than five feet and will be entered by workers should be shored or sloped in accordance with the Occupational Safety and Health Administration (OSHA) standards (29 CFR Part 1926). The shoring designer should be responsible for the shoring design. The contractor should be responsible for the construction and safety of temporary slopes and shoring.

We judge that a soldier pile-and-lagging shoring system is most appropriate for support of the proposed excavations for this project. A soldier pile-and-lagging 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 tiebacks, and therefore will not encroach beyond the property line. Where cuts exceed about 12 feet in height, soldier pile-and-lagging systems are typically more economical if they include tieback anchors; however, tieback anchors installed close to the site perimeter will likely extend beneath the streets and sidewalks, which will require an encroachment agreement with the City of Mountain View and/or Caltrans (W. El Camino Real). Where it is not feasible to install tiebacks and the excavation height is too great for a cantilevered soldier pile system, then internal bracing of the excavation will be required.

A structural/civil engineer knowledgeable in this type of construction should be retained to design the shoring. The shoring designer should design the shoring system for lateral deformation of less than 1/2 inch at any location on the shoring where there is a structure within a horizontal distance equal to twice the retained soil height and one inch where there are no structures within that horizontal distance. We should review the final shoring plans and calculations to check that they are consistent with the recommendations presented in this report.

6.4 Excavation, Monitoring and Construction Considerations

The soil to be excavated for the proposed basement levels and foundations is expected to consist primarily of clays and sands which can be excavated with conventional earth-moving equipment such as backhoes.

During excavation, the shoring system may deform laterally, which could cause the ground surface adjacent to the shoring to settle. The magnitudes of shoring movements and the resulting settlements are difficult to estimate because they depend on many factors, including the method of installation and the contractor's skill in the shoring installation. Ground movements due to a properly designed and constructed shoring system should be within ordinary accepted limits of about one inch where there are no improvements within a horizontal distance equal to twice the retained soil height of the shoring and 1/2 inch where there are improvements, including structures and critical buried utility pipelines) within that horizontal distance. A monitoring program should be established to evaluate the effects of the excavation on the adjacent buildings and surrounding ground.

The contractor should also survey and take photographs of existing buildings within a horizontal distance equal to twice the retained soil height prior to the start of construction. The survey points should be used to monitor the vertical and horizontal movements of the shoring and surrounding structures and streets during construction.

6.5 Soil Corrosivity

Corrosivity tests were performed by Project X Corrosion Engineering of Murrieta, California on selected soil samples obtained from Borings B-1, B-2, and B-3 at 2.5, 5.5, and 5.25 feet bgs. The corrosivity test results are presented in Appendix B of this report.

The resistivity test results (1,206 to 3,417 ohm-cm) indicate the near-surface soil is “highly corrosive[6]” to buried metallic structures. Accordingly, all buried iron, steel, cast iron, ductile 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 tests (7.2 to 7.6) indicate the near-surface is “negligibly corrosive” to buried metallic and concrete structures. The chloride ion concentrations (11.4 to 23.9 mg/kg) indicate 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

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[6] Roberge, Pierre R. (2018). Corrosion Basics, an Introduction, Third Edition. NACE International, p. 189.

sulfate ion concentrations (8.2 to 27.8 mg/kg) are sufficiently low such that sulfates do not to pose a threat to buried concrete.

7.0 RECOMMENDATIONS

Our recommendations for site preparation and grading, temporary cut slope and shoring, foundation support, design of permanent below-grade walls, and other geotechnical aspects of the project are presented in this section.

7.1 Site Preparation, Excavation, and Fill Placement

Any vegetation and organic topsoil (if present) should be stripped in areas to receive improvements (i.e., building, pavement, or flatwork). Tree roots with a diameter greater than 1/2 inch within three feet of subgrade should be removed. Site demolition should include the removal of all existing pavements, former foundation elements and underground utilities. 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 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 that extend below finished improvements should be properly backfilled with engineered fill under our observation and following the recommendations provided later in this section.

In areas that will receive fill or improvements (i.e., at-grade building pad and exterior concrete flatwork), the soil subgrade exposed following stripping and clearing should be scarified to a depth of at least eight inches, moisture-conditioned to above optimum moisture content, and compacted to at least 90 percent relative compaction[7]. Where firm native soil is exposed at

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[7] 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.

basement subgrade, scarification and recompaction is not necessary and the subgrade should be proof-rolled instead.

7.1.1 Fill Materials and Compaction Criteria

On-site soil may be used as fill or backfill, provided it is free of organic matter, contains no rocks or lumps larger than three inches in greatest dimension, and is approved by the Geotechnical Engineer. Fill consisting of imported soil (select fill) should be free of organic matter, contain no rocks or lumps larger than three inches in greatest dimension, have a liquid limit of less than 40 and a plasticity index lower 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 available, up to two weeks should be allowed to perform analytical testing on the proposed imported material.

Fill should be placed in horizontal lifts not exceeding eight inches in uncompacted thickness, moisture-conditioned to above optimum moisture content, and compacted to at least 90 percent relative compaction. Fill material consisting of clean sand or gravel (defined as poorly graded soil with less than five percent fines by weight) or greater than five feet in thickness should be compacted to at least 95 percent relative compaction. Fill placed within the upper six inches of vehicular pavement soil subgrade should also be compacted to at least 95 percent relative compaction and be non-yielding.

Where the compaction recommendations presented in this section are in conflict with the City of Mountain View standard details for pavements and sidewalks within the public right-of-way, the City Engineer or Inspector should determine which compaction requirements should take precedence.

7.1.2 Exterior Flatwork Subgrade Preparation

We recommend a minimum of four inches of Class 2 aggregate base be placed below exterior concrete flatwork, such as patios and sidewalks. The subgrade should be scarified, moisture-conditioned to above optimum moisture content and compacted to at least 90 percent relative compaction. The prepared subgrade should be kept moist until it is covered with the Class 2 aggregate base. The Class 2 aggregate base should be moisture-conditioned to above optimum moisture content and compacted to at least 90 percent relative compaction.

7.1.3 Utility Trench Backfill

Excavations for utility trenches can be readily made with a backhoe. All temporary excavations used in construction should be designed, planned, constructed, and maintained by the contractor and should conform to the current CAL-OSHA requirements.

To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of clean sand or fine gravel. After the pipes and conduits are tested, inspected (if required) and approved, they should be covered to a depth of six inches with clean sand or fine gravel, which should be compacted with a vibratory plate compactor. Backfill for utility trenches and other excavations is also considered fill, and should be placed and compacted as according to the recommendations previously presented in Section 7.1.1. If imported clean sand or gravel (defined as soil with less than five percent fines) is used as backfill, it should be compacted to at least 95 percent relative compaction. Jetting of trench backfill should not be permitted. Special care should be taken when backfilling utility trenches in pavement areas. Poor compaction may cause excessive settlements, resulting in damage to improvements.

Foundations for the proposed buildings should be bottomed below an imaginary line extending up at a 1.5:1 (horizontal to vertical) inclination from the base of utility trenches running parallel to the foundation. Alternatively, the portion of the utility trench (excluding bedding) that is below the 1.5:1 line can be backfilled with controlled low-strength material (CLSM) with a 28-day unconfined compressive strength of at least 100 pounds per square inch (psi) or Class 2 aggregate base compacted to at least 95 percent relative compaction. Excavation of utility trenches below an imaginary line extending down at a 1.5:1 (horizontal to vertical) inclination from the base of the foundation shall not be permitted after the foundation is poured, unless the utility excavation is shored and the shoring design is reviewed and approved by the Geotechnical Engineer and Structural Engineer.

7.1.4 Drainage and Landscaping

Positive surface drainage should be provided around the buildings to direct surface water away from the foundations and below-grade walls. To reduce the potential for water ponding adjacent to the buildings, we recommend the ground surface within a horizontal distance of five feet from the buildings slope down away from the buildings with a surface gradient of at least two percent in unpaved areas and one percent in paved areas. In addition, roof downspouts should be discharged into controlled drainage facilities to keep the water away from the foundations and basement walls. The use of water-intensive landscaping around the perimeters of the proposed buildings should be avoided.

We recommended that bioswales constructed at the site be provided with underdrains and/or drain inlets. The subdrain pipes should be installed eight inches above the bottom of the infiltration area for treatment areas that are at least five feet away from the structure and pavements. The intent of this recommendation is to allow infiltration into the underlying soil, but to reduce the potential for bio-retention areas to flood during periods of heavy rainfall. The sides of bioswales should be sloped at a maximum gradient of 1:1 (horizontal: vertical).

Where bioswales will be located within five feet of the buildings, the bottom of the treatment area should be lined with an impermeable liner. Where bioswales will be located within five feet of pavements, a four-inch-diameter perforated subdrain pipe should be placed four inches above the base of the treatment area or the bottom of the treatment area should be lined with an impermeable liner. Where a vertical curb or foundation is constructed near a bioswale, the curb and the edge of the foundation should be founded below an imaginary line extending up at an inclination of 1.5:1 (horizontal: vertical) from the base of the bioswale. For bio-retention features that will have vertical concrete walls, the walls should be designed to resist lateral earth pressures and, where appropriate, vehicular surcharge pressures imposed on the walls by either: 1) constructing a footing for the wall, or 2) installing horizontal struts inside the feature.

Care should be taken to minimize the potential for subsurface water to collect beneath flatwork and pavements. Where landscape beds and tree wells are immediately adjacent to pavements and flatwork that are not designed as permeable systems, we recommend vertical cutoff barriers be incorporated into the design to prevent irrigation water from saturating the subgrade and aggregate base. These barriers may consist of either flexible impermeable membranes or deepened concrete curbs.

7.2 Foundations

We conclude the proposed at-grade commercial building and the proposed mixed-use building with two basement levels may be supported on conventional spread footings or mats bearing on firm native soil. Recommendations for spread footings and mats are presented in this section.

7.2.1 Spread Footings

Spread footings should bear on firm native alluvium. Continuous footings should be at least 18 inches wide and isolated spread footings should be at least 24 inches wide. Footings for the at-grade commercial building and the below-grade mixed-use building should be founded at least 24 and 18 inches below the lowest adjacent soil subgrade (not counting the capillary moisture break, where present), respectively.

Footings for the proposed at-grade commercial building may be designed using allowable bearing pressures of 3,000 pounds per square foot (psf) for dead-plus-live loads and 4,000 psf for total design loads, which include wind or seismic forces. Footings for the proposed mixed-use building with two basement levels may be designed using allowable bearing pressures of 5,000 pounds per square foot (psf) for dead-plus-live loads and 6,650 psf for total design loads, which include wind or seismic forces. These allowable bearing pressures include factors of safety of at least 2.0 and 1.5 for dead-plus-live loads and total loads, respectively.

Lateral loads may be resisted by a combination of passive pressure on the vertical faces of the footings and friction between the bottoms of the footings and the supporting soil. To compute frictional resistance, we recommend using an allowable friction coefficient of 0.3. To calculate the passive resistance, we recommend using an equivalent fluid weight of 300 pounds per cubic foot (pcf) be used. Passive resistance for the upper foot of soil should be ignored unless it is confined by a pavement or slab. The values for the friction coefficient and passive pressure include a factor of safety of 1.5 and may be used in combination without further reduction.

Footing excavations should be free of standing water, debris, and disturbed materials prior to placing concrete. The bottoms and sides of the footing excavations should be moistened following excavation and maintained in a moist condition until concrete is placed. We should check footing excavations prior to placement of reinforcing steel.

7.2.2 Mat Foundation

For structural design of the mat foundation we recommend using a coefficient of vertical subgrade reaction of 25 pounds per cubic inch (pci) for the proposed at-grade commercial building and 50 pci for the proposed mixed-use building with two basement levels for dead-plus-live loads. These values has been reduced to account for the size of the mat/equivalent footings (therefore, this is not k_{v1} for 1-foot-square plate) and may be increased by one-third for total loads. Once the structural engineer evaluates the initial distribution of bearing stress on the bottom of the mat, we can review the distribution and revise the coefficients of subgrade reaction.

Considering the large area of the mat, we expect the average bearing stress under the mat to be low; however, concentrated stresses will occur at column locations and at the edges of the mat. The mat should be designed to impose a maximum dead-plus-live-load bearing pressure of 3,000 and 5,000 psf on the foundation subgrade soil for the proposed at-grade commercial building and mixed-use building with two basement levels, respectively. These pressures may be increased by one-third for total load conditions.

Lateral loads may be resisted by a combination of passive pressure on the vertical faces of the mat and friction between the bottoms of the mat and the supporting soil. To compute passive resistance, we recommend using an allowable equivalent fluid weight of 300 pcf. The upper foot of soil should be ignored unless confined by a slab or pavement (for any at-grade foundations). The allowable friction factor will depend on the type of vapor retarder used at the base of the mat. If no membrane is used, an allowable base friction coefficient of 0.30 may be used in design. If a vapor retarder is used, a base friction factor of 0.20 should be used. The passive pressure and frictional resistance values include 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 standing water, debris, and disturbed materials prior to placing the vapor retarder and concrete. We should check the mat subgrade prior to placement of the vapor retarder, aggregate base, or concrete.

7.3 Concrete Slab-on-Grade Floor

The subgrade for the slab-on-grade floor (for the spread footing foundation option) or mat foundation should be prepared in accordance with our recommendations in Section 7.1. Where water vapor transmission through the floor slab/mat is not desirable, we recommend installing a capillary moisture break and water vapor retarder beneath the floor slab/mat.

A capillary moisture break consists of at least four inches of clean, free-draining gravel or crushed rock. The particle size of the capillary break material should meet the gradation requirements presented in Table 2.

TABLE 2
Gradation Requirements for Capillary Moisture Break

Sieve Size	Percentage Passing Sieve
1 inch	90 – 100
3/4 inch	30 – 100
1/2 inch	5 – 25
3/8 inch	0 – 6

The vapor retarder should meet the requirements for Class B vapor retarders stated in ASTM E1745. Where the building will be supported on a mat slab, the capillary moisture break may be omitted provided the vapor retarder meets the requirements for Class A vapor retarders. The vapor retarder should be placed in accordance with the requirements of ASTM E1643. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder.

Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and results in excessive vapor transmission through the slab. Where the concrete is poured directly over the vapor retarder, we recommend the w/c ratio of the concrete not exceed 0.45. Water should not be added to the concrete mix in the field. If necessary, workability should be increased by adding plasticizers. In addition, the slab should be properly cured. Before the floor covering is placed, the contractor should check that the concrete surface and the moisture emission levels (if emission testing is required) meet the manufacturer’s requirements.

7.4 Permanent Below-Grade Walls

Below-grade walls should be designed to resist, static lateral earth pressures, lateral pressures caused by earthquakes, vehicular surcharge pressures, and surcharges from adjacent foundations, where appropriate. We recommend restrained below-grade walls at the site be designed for the more critical of the following criteria:

- At-rest equivalent fluid weight of 55 pcf
- Active pressure of 35 pcf plus a seismic increment of 33 pcf (triangular distribution)

The recommended lateral earth pressures above are based on a level backfill condition with no additional surcharge loads. Where traffic loads are expected within 10 feet of the walls, an additional design load of 50 psf should be applied to the upper 10 feet of the wall. Basement walls adjacent to adjacent structures (i.e., new commercial building) should be designed for surcharge pressures if the foundations supporting the adjacent buildings are founded above the zone-of-influence for the basement walls. This zone is defined as an imaginary line extending up from the bottom of the wall at an inclination of 1.5:1. The influence on a wall from a foundation that is founded within this zone of influence should be analyzed on an individual basis after the geometry has been determined.

To protect against moisture migration, below-grade walls should be waterproofed and water stops should be placed at all construction joints. The design pressures recommended are based on fully drained walls. Although the below-grade walls will likely be above the design groundwater level, water can accumulate behind the walls from other sources, such as rainfall, irrigation, and broken water lines, etc. One acceptable method for backdraining a basement 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 on all sides by at least four 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 basement walls prior to pouring the podium slabs, the walls should be braced, or hand compaction equipment used, to prevent unacceptable surcharges on walls (as determined by the structural engineer).

7.5 Temporary Cut Slopes and Shoring

The safety of workers and equipment in or near the excavation is the responsibility of the contractor. The selection, design, construction, and performance of the shoring system should be the responsibility of the contractor. A structural engineer/civil engineer knowledgeable in this type of construction should design the shoring. We should review the geotechnical aspects of the proposed shoring system to ensure that it meets our requirements. During construction, we should observe the installation of the shoring system and check the condition of the soil encountered during excavation.

We judge that temporary cuts in on-site soil which are less than 25 feet high, above groundwater, and inclined (1:1) in accordance to OSHA guidelines for Type B soil will be stable provided that they are not surcharged by equipment or building material. Temporary shoring will be required where temporary slopes are not possible because of space constraints. As discussed in Section 6.3, we conclude viable shoring systems for this project include cantilevered and tied-back soldier-pile-and-lagging systems, depending on the retained soil height. Recommendations regarding the design and construction of both shoring types are presented below.

7.5.1 Cantilever Soldier Pile and Lagging Shoring System

We recommend a cantilevered soldier pile-and-lagging shoring system be designed to resist active equivalent fluid weights of 35 pcf. Where traffic loads 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 pile should be computed using an equivalent fluid weight 300 pcf; and a maximum passive earth pressure of 3,000 psf. The upper foot of soil should be ignored when computing passive resistance. Passive pressure can be assumed to act over an area of three soldier pile widths assuming the toe of the soldier pile is filled with structural concrete. If lean concrete is placed in the soldier pile shaft, the passive pressure can be assumed to act over two pile diameters. These passive pressure values include a factor of safety of at least 1.5.

Soldier piles should be placed in pre-drilled holes backfilled with concrete. Where granular layers susceptible to caving are present, installing the soldier piles may require casing or use of drilling slurry to reduce caving of the holes. Installing soldier piles by driving or using vibratory methods is acceptable, but should not be permitted within 25 feet of existing structures.

7.5.2 Soldier Pile and Lagging Shoring System with Tiebacks

Recommended lateral pressures for the design of soldier pile and lagging shoring with tiebacks are presented on Figure 6. Where it is not feasible to install tiebacks, then internal bracing of the excavation will be required. Internal bracing should be preloaded to limit movement of the shoring. Recommendations for passive resistance at the toe of the soldier pile and the installation of soldier piles and laggings are presented in the previous Section 7.5.1.

The penetration of the soldier piles must be sufficient to ensure stability and resist the downward loading of tiebacks. Vertical loads can be resisted by skin friction along the portion of the soldier piles below the excavation. We recommend using an allowable skin friction value of 1,500 psf to compute the required soldier pile embedment. End bearing should be neglected.

Design criteria for tiebacks are also presented on Figure 6. As shown, tiebacks should derive their load-bearing capacity from the soil behind an imaginary line sloping upward from a point H/5 feet away from the bottom of the excavation at an angle of 60 degrees from horizontal, where H is the wall height in feet. The minimum stressing lengths for strand and bar tendons should be 15 and 10 feet, respectively. The minimum bond length for strand and bar tendons should both be 15 feet.

Allowable capacities of the tiebacks will depend upon the drilling method, hole diameter, grout pressure, and workmanship. The shoring contractor should be prepared to use smooth-cased method (such as a Klemm rig) to install the tiebacks where granular soil susceptible to caving is present. Where it is not feasible to install tiebacks, then internal bracing of the excavation will be required. Internal bracing should be preloaded to limit movement of the shoring.

The shoring designer should be responsible for determining the actual length of tiebacks required to resist the design loads. The determination should be based on the designer’s familiarity with the installation method to be used.

7.5.3 Tieback Testing

The computed bond length of tiebacks should be confirmed by a performance- and proof-testing program under the observation of our field engineer. The first two production tiebacks and two percent of the remaining tiebacks should be performance-tested to 1.5 times the design load. The remaining tiebacks should be confirmed by a proof-test to 1.25 times the design load. The movement of each tieback should be monitored with a free-standing, tripod-mounted dial gauge during performance and proof testing. The bottom of excavation should not extend more than two feet below a row of unsecured tiebacks.

The performance test is used to verify the capacity and the load-deformation behavior of the tiebacks. It is also used to separate and identify the causes of tieback movement, and to check that the designed unbonded length has been established. In the performance test, the load is applied to the tieback in several cycles of incremental loading and unloading. During the test, the tieback load and movement are measured. The maximum test load should be held for a minimum of 10 minutes, with readings taken at 0, 1, 3, 6, and 10 minutes. If the difference between the 1- and 10-minute reading is less than 0.04 inch during the loading, the test is discontinued. If the difference is more than 0.04 inch, the holding period is extended by 50 minutes to 60 minutes, and the movements should be recorded at 15, 20, 25, 30, 45, and 60 minutes.

A proof test is a simple test used to measure the total movement of the tieback during one cycle of incremental loading. The maximum test load should be held for a minimum of 10 minutes, with readings taken at 0, 1, 2, 3, 6, and 10 minutes. If the difference between the 1- and 10-minute reading is less than 0.04 inch, the test is discontinued. If the difference is more than 0.04 inch, the holding period is extended by 50 minutes to 60 minutes, and the movements should be recorded at 15, 20, 25, 30, 45, and 60 minutes.

We should evaluate the tieback test results and determine whether the tiebacks are acceptable. A performance- or proof-tested tieback with a 10-minute hold is acceptable if the tieback carries the maximum test load with less than 0.04 inch movement between 1 and 10 minutes, and total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length.

A performance- or proof-tested tieback with a 60-minute hold is acceptable if the tieback carries the maximum test load with less than 0.08 inch movement between 6 and 60 minutes, and total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length. Tiebacks that failed to meet the first criterion will be assigned a reduced capacity.

7.5.4 Construction Monitoring

The contractor should establish survey points on the shoring and on the ground surface at critical locations behind the shoring prior to the start of excavation. These survey points should be used to monitor the vertical and horizontal movements of the shoring and the ground behind the shoring during construction.

During excavation, the shoring system may deform laterally, which could cause the ground surface adjacent to the shoring wall to settle. The magnitudes of shoring movements and the resulting settlements are difficult to estimate because they depend on many factors, including the method of installation and the contractor's skill in the shoring installation. A monitoring program should be established to evaluate the effects of the construction on the adjacent properties.

7.7 Pavers

The following sections present geotechnical recommendations for the design and construction of non-permeable and permeable pavers.

7.7.1 Non-Permeable Concrete Pavers

We recommend non-permeable pedestrian pavers and sand bedding be underlain by at least four inches of Class 2 aggregate base compacted to at least 90 percent relative compaction. We recommend non-permeable pavers subject to vehicular traffic be underlain by Class 2 aggregate base compacted to at least 95 percent relative compaction. The aggregate base thickness beneath non-permeable pavers subject to vehicular traffic will depend on the traffic index (TI). For TIs of 4.5, 5, 5.5, and 6, we recommend aggregate base thicknesses of 6.5, 8.5, 10, and 12 inches, respectively. Recommendations for concrete pavers in areas with TI greater than 6 can be provided upon request.

7.7.2 Permeable Interlocking Concrete Pavers

We recommend permeable interlocking concrete pavements (ICP) be designed in accordance with the guidelines presented by the Interlocking Concrete Pavement Institute (ICPI 2005). These guidelines include specific recommendations for permeable aggregate subbase, base, and bedding courses to be placed beneath ICP pavements. We recommend permeable pavements for pedestrian traffic be designed for partial exfiltration of water into the subgrade soil. This requires installing a subdrain system at the base of the pervious aggregate materials, which are underlain by a filter fabric (see Figure 7). We recommend permeable pavements for vehicular traffic be designed for no-exfiltration of water into the subgrade soil. This requires installing a subdrain system at the base of the pervious aggregate materials, which are underlain by an impermeable liner (see Figure 8).

The soil subgrade beneath ICP pavements should be prepared and compacted in accordance with the recommendations presented in Section 7.1. In addition, the subgrade should be a firm and non-yielding surface. The subgrade should be proof-rolled under the observation of our field engineer to confirm it is non-yielding prior to placing the filter fabric and aggregate base materials. The soil subgrade at the bottom of the permeable section should slope down toward the drain pipe trench at a gradient of at least two percent. The perforated pipe should slope down to a suitable outlet at a minimum gradient of one percent. The pipe should be placed with the perforations down on a minimum of two inches of permeable subbase.

ICPI’s guidelines call for 1-1/2 to 2 inches of bedding material consisting of ASTM No. 8 aggregate directly below the pavers. This material is also recommended for fill material between the pavers. As shown in Table 4 below, this material consists of fine gravel with 10 to 30 percent sand.

TABLE 4
Gradation Requirements for ASTM No. 8 Aggregate

Sieve Size	Percentage Passing Sieve
1/2 inch	100
3/8 inch	85 – 100
No. 4	10 – 30
No. 8	0 – 10
No. 16	0 – 5

The ASTM No. 8 bedding should be underlain by a permeable base course of ASTM No. 57 crushed aggregate. As shown in Table 5, ASTM No. 57 aggregate consists of open-graded gravel with a gradation between that of the 3/4-inch drain rock and the ASTM No. 8 aggregate.

TABLE 5
Gradation Requirements for ASTM No. 57 Aggregate

Sieve Size	Percentage Passing Sieve
1-1/2 inch	100
1 inch	95 – 100
1/2 inch	25 – 60
No. 4	0 – 10
No. 8	0 – 5

The ASTM No. 57 permeable base course should be underlain by a permeable subbase course of ASTM No. 2 crushed aggregate. The gradation requirements for ASTM No. 2 crushed aggregate subbase are presented in Table 6.

TABLE 6
Gradation Requirements for ASTM No. 2 Aggregate

Sieve Size	Percentage Passing Sieve
3 inch	100
2-1/2 inch	90-100
2 inch	35-70
1-1/2 inch	0-15
3/4 inch	0 -5

The No. 2 aggregate subbase course should be placed in lifts not exceeding 6 inches in loose thickness and compacted using a smooth-drum roller, operated in static (non-vibratory) mode. The subsequent course of No. 57 aggregate may be placed in one lift and should be compacted with a smooth-drum roller in vibratory mode with sufficient passes to create an unyielding surface. Placement and compaction of the permeable aggregate base and subbase should be performed under the observation of our field engineer. Following compaction of the No. 57 aggregate, the No. 8 bedding, not exceeding 2 inches in loose thickness, should be placed and screeded to a level, undisturbed surface immediately prior to paver installation.

The required thicknesses of the permeable aggregate base and subbase courses depends on the infiltration and water storage design requirements, as well as the traffic loading demand. Our recommendations for the minimum permeable ICP pavement sections (based on traffic demand) are presented in Table 7. Also included in Table 7 is a recommended section for permeable ICPs subject to pedestrian traffic only.

TABLE 7
Recommended Pavement Sections for Permeable Interlocking Concrete Pavers

TI	ASTM No. 8 Bedding Aggregate (inches)	ASTM No. 57 Stone Base (inches)	ASTM No. 2 Stone Subbase (inches)
Pedestrian	1.5-2.0	4.0 (10)	6.0 (0)
6 to 8.5	1.5-2.0	4.0	8.0

The above recommended ICP pavement sections are based on the ICPI technical guidelines (ICPI 2005). From a geotechnical standpoint, it is acceptable to design the pedestrian ICP section to exclude the No. 2 subbase course, in which case the No. 57 base course should be increased to 10 inches. From a geotechnical standpoint, it is also acceptable to use compacted structural planting mix in lieu of the No. 57 and No. 2 base courses in locations where the pedestrian ICP section is adjacent to tree wells and is required for promoting root growth.

7.8 Seismic Design

The latitude and longitude of the site are 37.3851° and -122.0837°, respectively. For design in accordance with 2019 or 2022 CBC, we recommend the following:

- Site Class D – Stiff Soil
- $S_S = 1.836g$, $S_1 = 0.651g$

The 2019 and 2022 CBC are based on the guidelines contained within ASCE 7-16. Per ASCE 7-16, where S_1 is greater than 0.2 times gravity (g) for Site Class D, a ground motion hazard analysis is needed unless the seismic response coefficient (C_s) value will be calculated as outlined in Section 11.4.8, Exception 2 of ASCE 7-16. Assuming the C_s value will be calculated as outlined in Section 11.4.8, Exception 2 of ASCE 7-16, we recommend the following seismic design parameters:

- F_a = 1.0, F_v = 1.7
- S_{MS} = 1.836g, S_{M1} = 1.107g
- S_{DS} = 1.224g, S_{D1} = 0.738g
- Seismic Design Category D for Risk Factors I, II, and III

8.0 ADDITIONAL GEOTECHNICAL SERVICES

Prior to construction, Rockridge Geotechnical should review the project plans and specifications to check that they conform to the intent of our recommendations. During construction, our field engineer should provide on-site observation and testing during site preparation, placement and compaction of fill and aggregate base, installation of shoring system, and installation of foundations. These observations will allow us to compare actual with anticipated soil conditions and to check that 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 the borings and CPTs. If any variations or undesirable conditions are encountered during construction, we should be notified so that 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 project vicinity.

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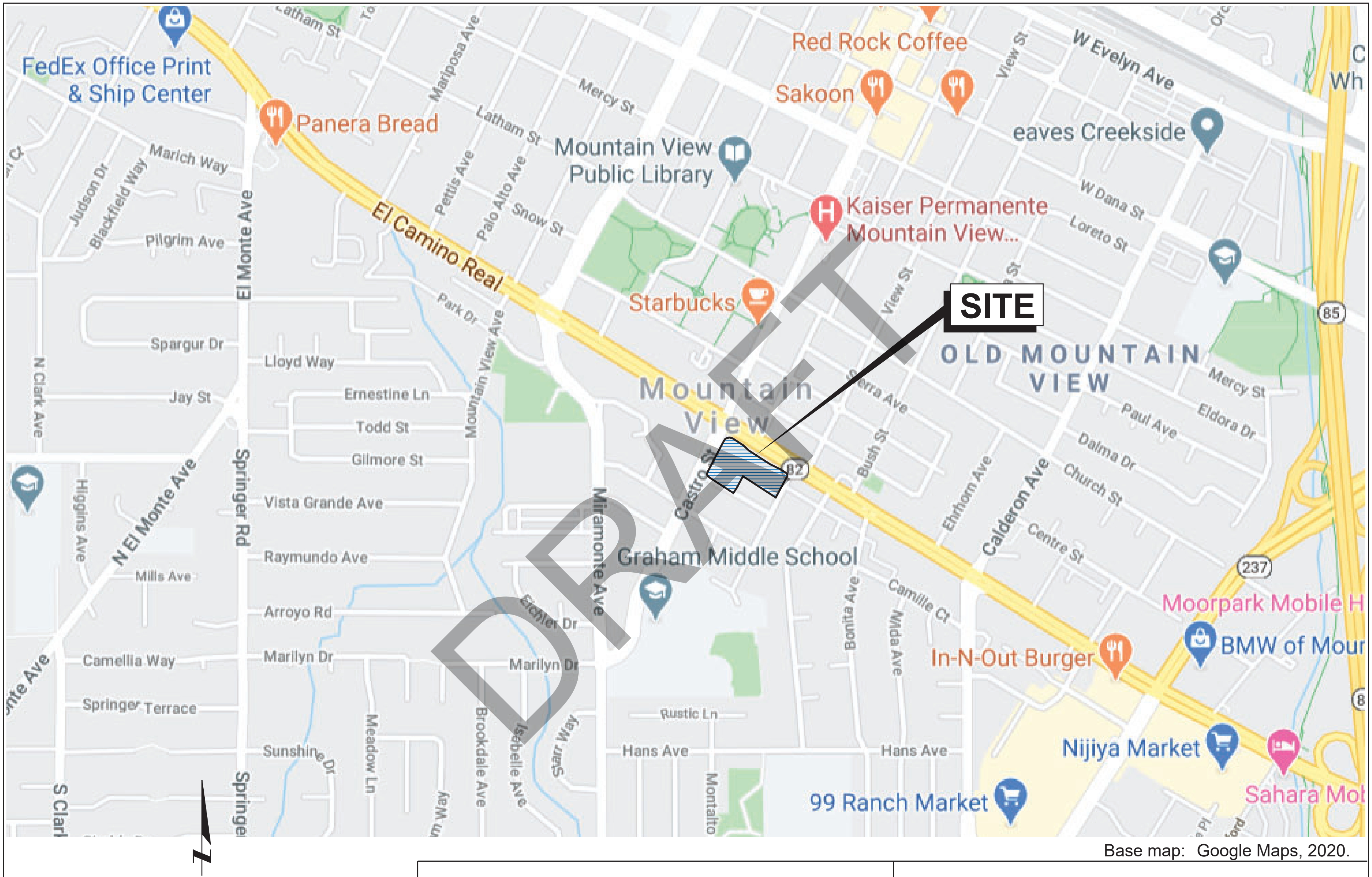
FIGURES



Base map: Google Maps, 2020.

0 1,000 2,000 Feet

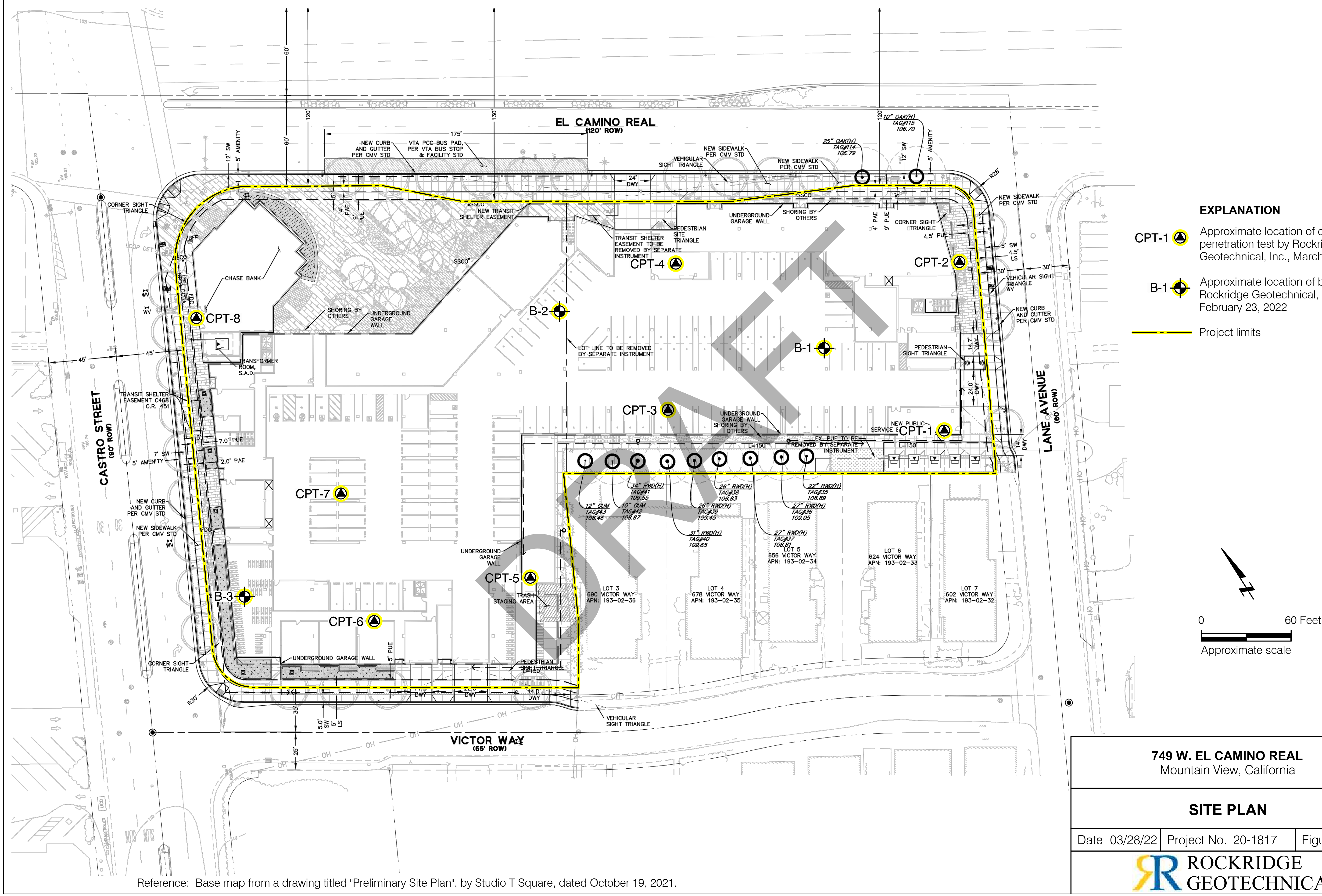
Approximate scale

749 W. EL CAMINO REAL
Mountain View, California

ROCKRIDGE
GEOTECHNICAL

SITE LOCATION MAP

Date 04/07/22	Project No. 20-1817	Figure 1



EXPLANATION

CPT-1 ▲	Approximate location of cone penetration test by Rockridge Geotechnical, Inc., March 3, 2022
B-1 ⊕	Approximate location of boring by Rockridge Geotechnical, Inc., February 23, 2022
— - —	Project limits

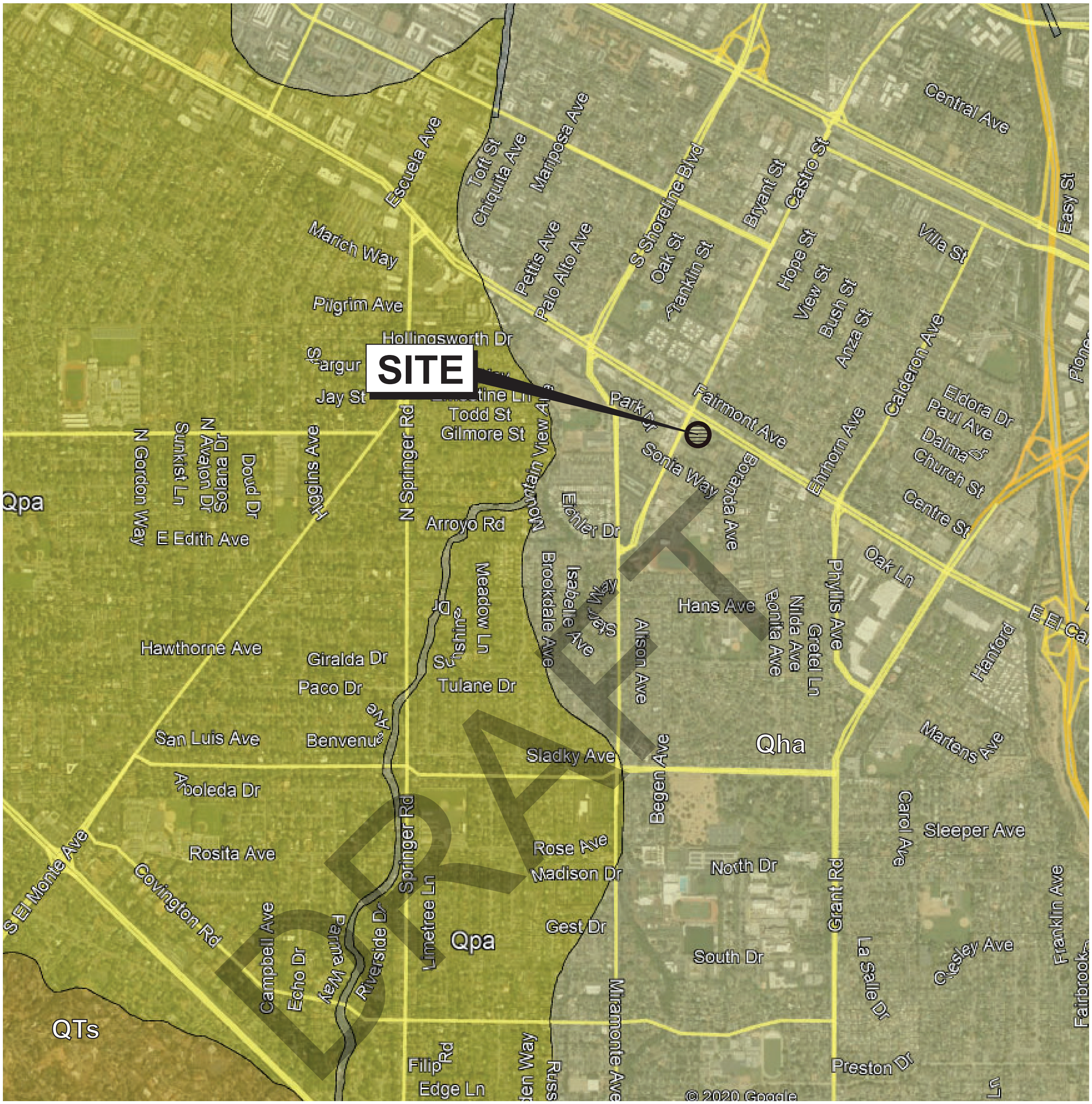
Reference: Base map from a drawing titled "Preliminary Site Plan", by Studio T Square, dated October 19, 2021.

749 W. EL CAMINO REAL
Mountain View, California

SITE PLAN

Date 03/28/22 | Project No. 20-1817 | Figure 2

ROCKRIDGE GEOTECHNICAL



Base map: Google Earth with U.S. Geological Survey (USGS), Santa Clara County, 2020.

EXPLANATION

Qha Alluvium (Holocene)

Qpa Alluvium (Pleistocene)

QTs Sediments (early Pleistocene and (or) Pliocene)

Geologic contact:
dashed where approximate and dotted
where concealed, queried where uncertain

N

0 2,000 4,000 Feet

Approximate scale

749 W. EL CAMINO REAL
Mountain View, California

ROCKRIDGE
GEOTECHNICAL

REGIONAL GEOLOGIC MAP

Date 04/07/22 | Project No. 20-1817 | Figure 3

Base Map: U.S. Geological Survey (USGS), National Seismic Hazards Maps - Fault Sources, 2014.

EXPLANATION

🟥 Strike slip

🟨 Thrust (Reverse)

🟦 Normal

0 5 10 Miles

Approximate scale

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Mountain View, California

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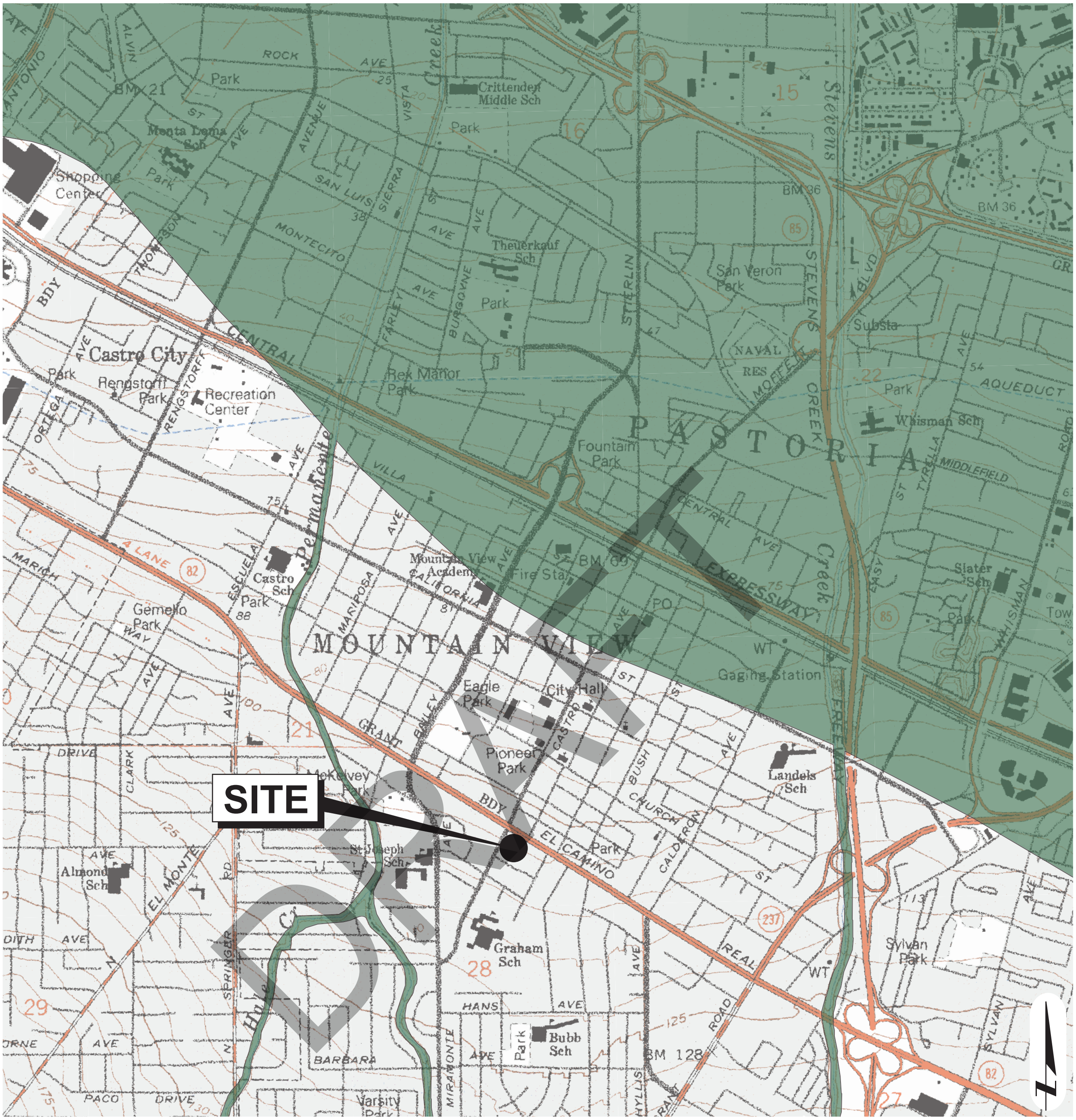
REGIONAL FAULT MAP

Date 03/28/22	Project No. 20-1817	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
Mountain View Quadrangle.
California Geological Survey
Released October 18, 2006

0 2,000 4,000 Feet
Approximate scale

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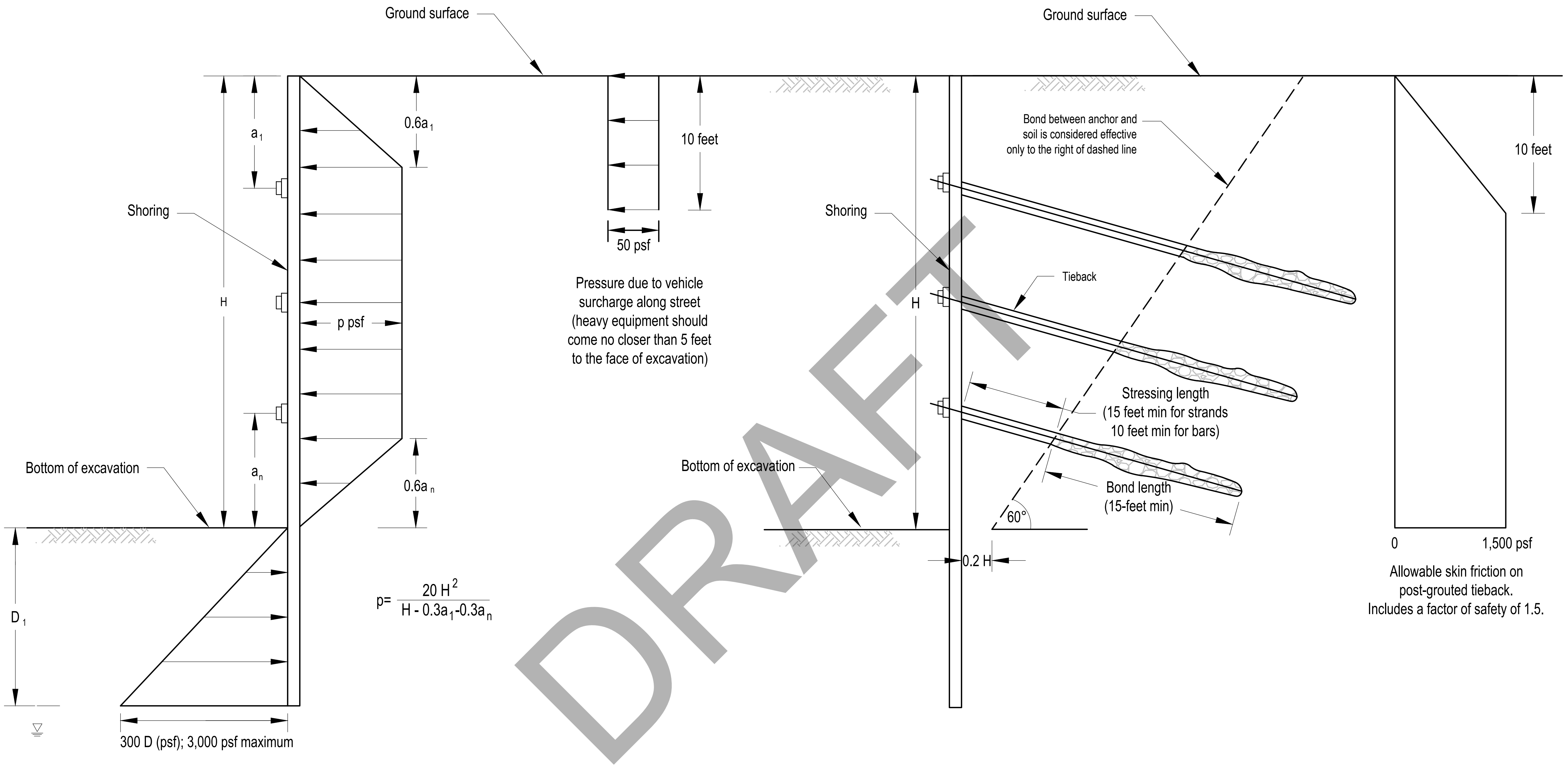
EARTHQUAKE ZONES OF REQUIRED INVESTIGATION MAP

Date 04/07/22 | Project No. 20-1817 | Figure 5



NOT TO SCALE

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, assuming the tow of the soldier pile is filled with structural concrete. If lean concrete is placed in the toe, the passive pressure can be assumed to act over two pile diameters.

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DESIGN PARAMETERS FOR ANCHORED SOLDIER-PILE-AND-LAGGING TEMPORARY SHORING SYSTEM

Date 04/07/22	Project No. 20-1817	Figure 6

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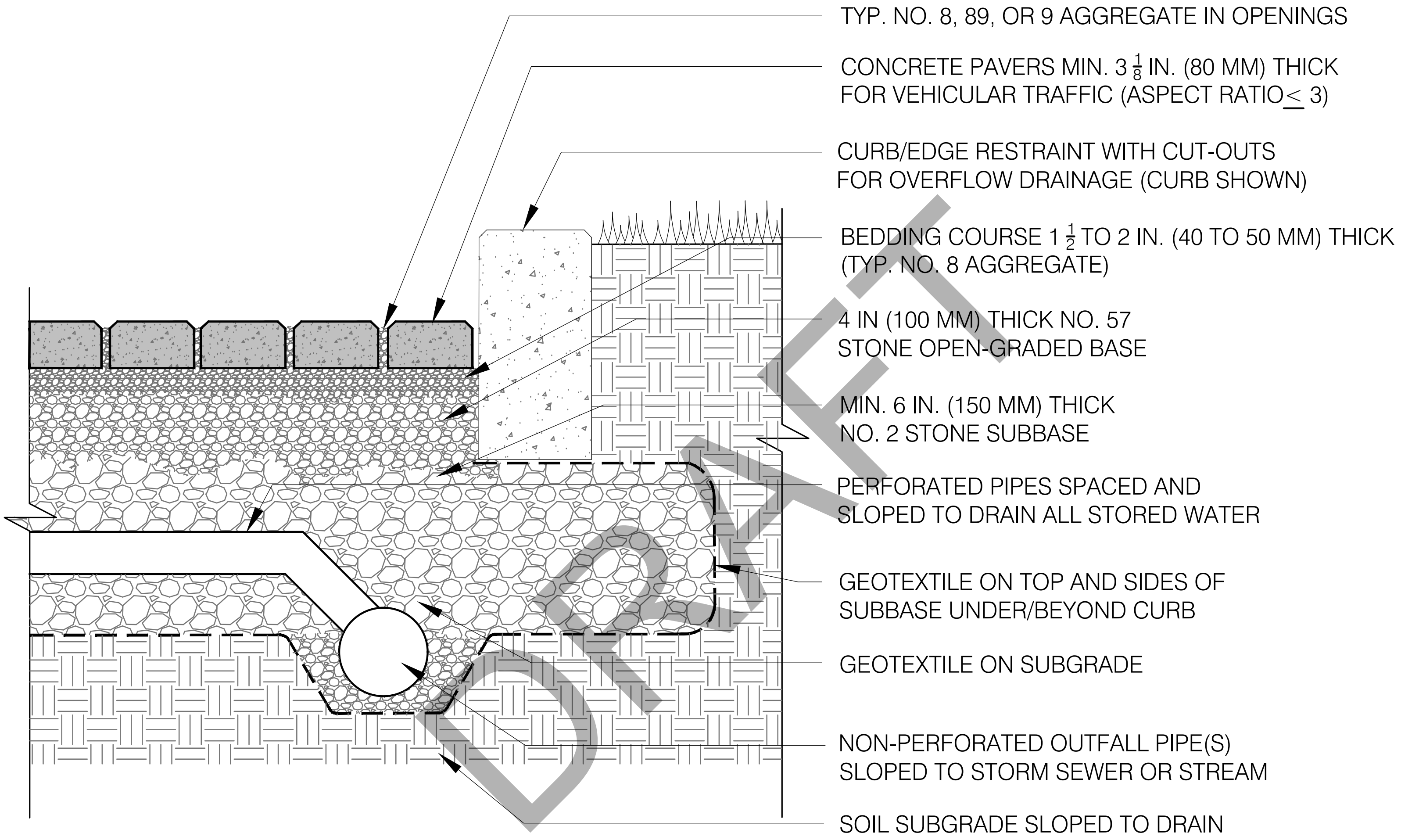

NOTES:

1. 2 3/8 IN. (60 MM) THICK PAVERS MAY BE USED IN RESIDENTIAL APPLICATIONS.
2. NO. 2 STONE SUBBASE THICKNESS VARIES WITH DESIGN. CONSULT ICPI PERMEABLE INTERLOCKING CONCRETE PAVEMENT MANUAL..
3. PERFORATED PIPES MAY BE RAISED FOR WATER STORAGE FROM LARGE RAIN EVENTS WITH OUTLET(S) AT LINER BOTTOM TO DRAIN SMALL RAIN EVENTS.

Reference: "Permeable Interlocking Concrete Pavements", Third Edition, prepared by Interlocking Concrete Pavement Institute, dated 2005.

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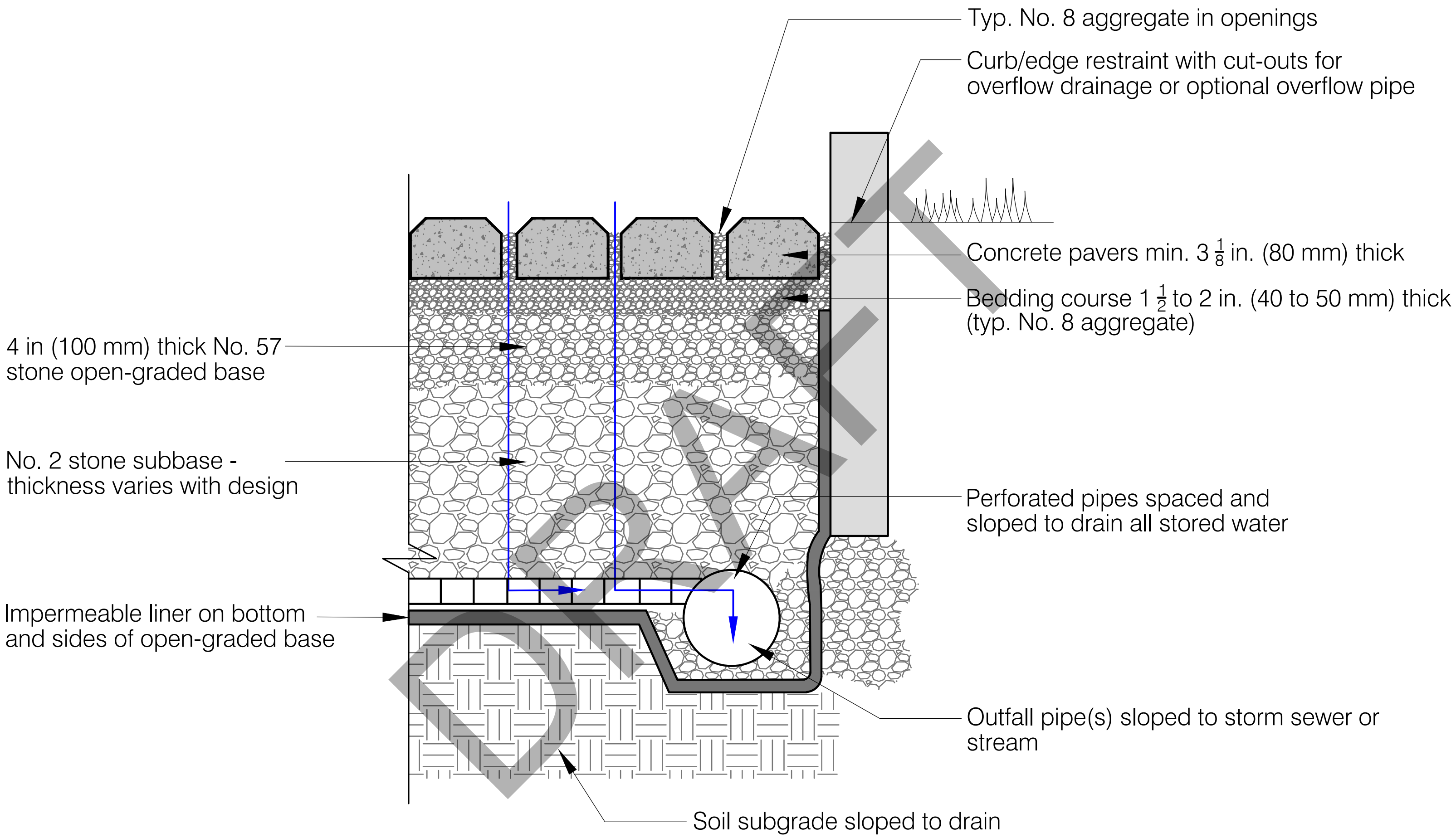
GENERALIZED ICPI PERMEABLE
PAVER DETAIL
FOR PARTIAL EXFILTRATION

Date 04/07/22	Project No. 20-1817	Figure 7



Reference: "Permeable Interlocking Concrete Pavements", Third Edition, prepared by Interlocking Concrete Pavement Institute, dated 2005.

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GENERALIZED ICPI PERMEABLE PAVER DETAIL FOR NO EXFILTRATION

Date 04/01/22	Project No. 20-1817	Figure 8

APPENDIX A

Logs of Borings and Cone Penetration Test Results

PROJECT: **749 W. EL CAMINO REAL**
Mountain View, California

Log of Boring B-1

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2
Date started: 02/23/2022
Date finished: 02/23/2022
Drilling method: 8-inch-diameter hollow-stem auger
Logged by: A. Limpert
Drilled by: Exploration Geoservices, Inc.
Rig: Mobile B53R
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Downhole Safety hammer
Sampler: Modified California (MC), 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/Cut Ft
						CLAY with SAND (CL) dark brown to gray-brown, very stiff, moist, trace fine sand, roots and organics						
1												
2												
3	MC	■	14 19 22	26		LL = 35, PI = 18; see Appendix B					15.9	112
4					CL	Soil Corrosivity Test; see Appendix B						
5						dark brown and light brown, hard, moist, fine sand						
6	MC	■	20 23 33	35							14.9	101
7												
8						SANDY CLAY (CL) brown, very stiff, moist, fine to coarse sand, trace fine rounded gravel						
9	MC	■	17 23 26	31								
10					CL							
11												
12						CLAYEY SAND (SC) brown, dense to very dense, fine to medium sand						
13												
14	SPT	◩	13 22 24	50	SC							
15												
16												
17						CLAY with SAND (CL) brown, hard, moist, fine to medium sand						
18												
19	SPT	◩	17 20 23	46	CL							
20												
21												
22						SANDY CLAY (CL) brown, hard, moist, fine to medium sand, trace fine subrounded to subangular gravel						
23												
24	SPT	◩	15 24 22	50								
25												
26					CL							
27												
28												
29	SPT	◩	18 23 46	75								
30												

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Project No.: 20-1817
Figure: A-1a

PROJECT: **749 W. EL CAMINO REAL**
Mountain View, California

Log of Boring B-1

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						olive-gray and red-brown mottled						
34	SPT	◩	17 15 28	46	CL							
35												
36												
37												
38						CLAYEY SAND (SC) olive-gray, very dense, moist						
39	SPT	◩	15 21 31	56	SC							
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 40 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] MC and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.63, and 1.08, respectively, to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 20-1817

Figure: A-1b

PROJECT: **749 W. EL CAMINO REAL**
Mountain View, California

Log of Boring B-2

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2
Date started: 02/23/2022
Date finished: 02/23/2022
Drilling method: 8-inch-diameter hollow-stem auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Downhole safety hammer
Sampler: Modified California (MC), Standard Penetration Test (SPT)
Logged by: A. Limpert
Drilled by: Exploration Geoservices, Inc.
Rig: Mobile B53R

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value¹	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
						4 inches of asphalt concrete 7 inches of aggregate base						
1						SANDY CLAY (CL)						
2						dark brown, hard, moist, fine sand, trace fine gravel						
3	MC	■	28 50/1"	32/ 11"								
4					CL							
5						very stiff						
6	MC	■	9 14 18	20		LL = 43, PI = 23; see Appendix B Soil Corrosivity Test; see Appendix B					21.3	106
7												
8												
9	MC	■	8 9 10	12		CLAY with SAND (CL) brown, stiff, moist, fine to medium sand					19.1	103
10					CL							
11												
12												
13					SC	CLAYEY SAND (SC) brown, medium dense, moist, fine to medium sand						
14	MC	■	7 8 9	11		Particle Size Distribution; see Appendix B				18	8.5	111
15						SANDY CLAY (CL) brown, stiff, moist, fine to medium sand						
16					CL							
17												
18						CLAY with SAND (CL) brown with red-brown mottling, very stiff, moist, fine sand, trace fine angular gravel						
19	MC	■	19 16 21	23								
20												
21					CL							
22												
23												
24	MC	■	19 29 50/5"	50/ 11"		hard						
25						CLAYEY SAND with GRAVEL (SC) brown with gray-brown, very dense, moist, fine to coarse sand, fine subrounded to subangular gravel						
26												
27												
28					SC							
29	SPT	◩	25 32 38	76		brown with some black sand, some angular gravel						
30												

ROCKRIDGE GEOTECHNICAL

Project No.: 20-1817
Figure: A-2a

PROJECT: **749 W. EL CAMINO REAL**
Mountain View, California

Log of Boring B-2

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	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
					SC	CLAYEY SAND with GRAVEL (SC) (continued)						
31												
32						CLAY with SAND (CL) olive-brown, hard, moist, fine to medium sand						
33												
34	SPT	◩	14 18 26	48	CL							
35												
36												
37						CLAYEY SAND (SC) dark brown, dense, moist, medium sand						
38					SC							
39	SPT	◩	14 16 22	41	CL							
40						CLAY (CL) light brown mottled with olive-gray, hard, trace sand						
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 40 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] MC and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.63, and 1.08, respectively, to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 20-1817

Figure: A-2b

PROJECT: **749 W. EL CAMINO REAL**
Mountain View, California

Log of Boring B-3

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2
Date started: 02/23/2022
Date finished: 02/23/2022
Drilling method: 8-inch-diameter hollow-stem auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Downhole safety hammer
Sampler: Modified California (MC), Standard Penetration Test (SPT)
Logged by: A. Limpert
Drilled by: Exploration Geoservices, Inc.
Rig: Mobile B53R

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/Cut Ft
						3 inches of asphalt concrete 7 inches of aggregate base						
1												
2					CL	SANDY CLAY (CL) dark brown, hard, moist, medium sand, trace gravel						
3	MC	■	20 21 33	34								
4						CLAY with SAND (CL) gray-brown to dark brown, hard, moist, fine to medium sand, trace gravel						
5					CL							
6	MC	■	23 25 33	37		brown, trace roots Soil Corrosivity Test; see Appendix B					15.3	108
7						CLAYEY SAND (SC) brown, medium dense, moist, fine to medium sand, trace gravel						
8	MC	■	13 14 17	20	SC						13.7	108
9												
10	MC	■	40 50/6"	32/6"	GC	CLAYEY GRAVEL with SAND (GC) gray and brown, very dense, moist, fine to coarse sub-rounded to subangular gravel						
11					SC							
12						CLAYEY SAND (SC) brown to red-brown, very dense, moist, fine to coarse sand, trace fine gravel						
13						CLAY with SAND (CL) brown, very stiff, moist, fine to coarse sand, trace fine subangular to rounded gravel						
14	SPT	◪	9 12 14	28								
15					CL							
16												
17												
18						SANDY CLAY (CL) brown, very stiff to hard, moist, fine sand						
19	MC	■	13 26 24	32	CL							
20						trace rounded coarse sand at 20 feet (bottom of sample)						
21												
22												
23						CLAYEY GRAVEL with SAND (GC) brown, very dense, moist, fine to coarse sand, fine to coarse subrounded gravel						
24	MC	■	50/6"	32/6"	GC							
25												
26												
27						CLAYEY SAND with GRAVEL (SC) brown, very dense, moist, fine to coarse sand, fine subangular to subrounded gravel						
28					SC							
29	SPT	◪	28 31 50/6"	88/ 12"								
30												

ROCKRIDGE GEOTECHNICAL

Project No.: 20-1817
Figure: A-3a

PROJECT: **749 W. EL CAMINO REAL**
Mountain View, California

Log of Boring B-3

PAGE 2 OF 2

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
					SC	CLAYEY SAND with GRAVEL (SC) (continued)						
31												
32						SANDY CLAY with GRAVEL (CL) olive-brown, hard, moist, fine sand, fine gravel						
33												
34	SPT	◩	13 19 33	56	CL							
35												
36												
37						CLAYEY SAND (SC) brown with light brown, dense, moist, fien to medium sand						
38					SC							
39	SPT	◩	13 16 14	32								
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 40 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] MC and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.63, and 1.08, respectively, to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 20-1817
Figure: A-3b

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

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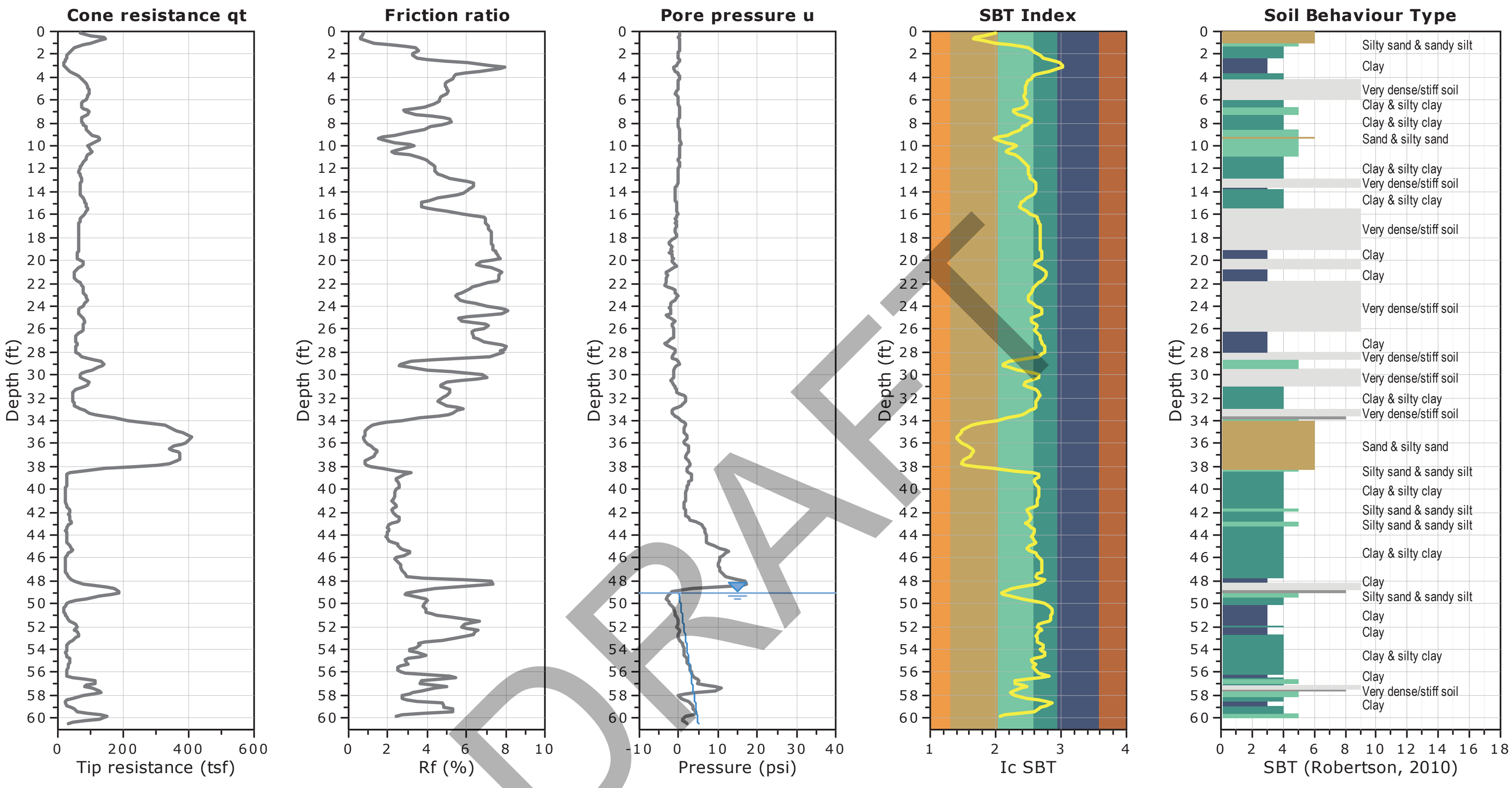
CLASSIFICATION CHART

Date 03/27/22 | Project No. 20-1817 | Figure A-4



Total depth: 60.5 ft, Date: March 3, 2022
Groundwater Depth: 49 feet (assumed based on CPT-2 and CPT-5)
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty clay
- 🟩 5. Silty sand to sandy silt
- 🟨 6. Clean sand to silty sand
- 🟧 7. Gravelly sand to sand
- ⬛ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

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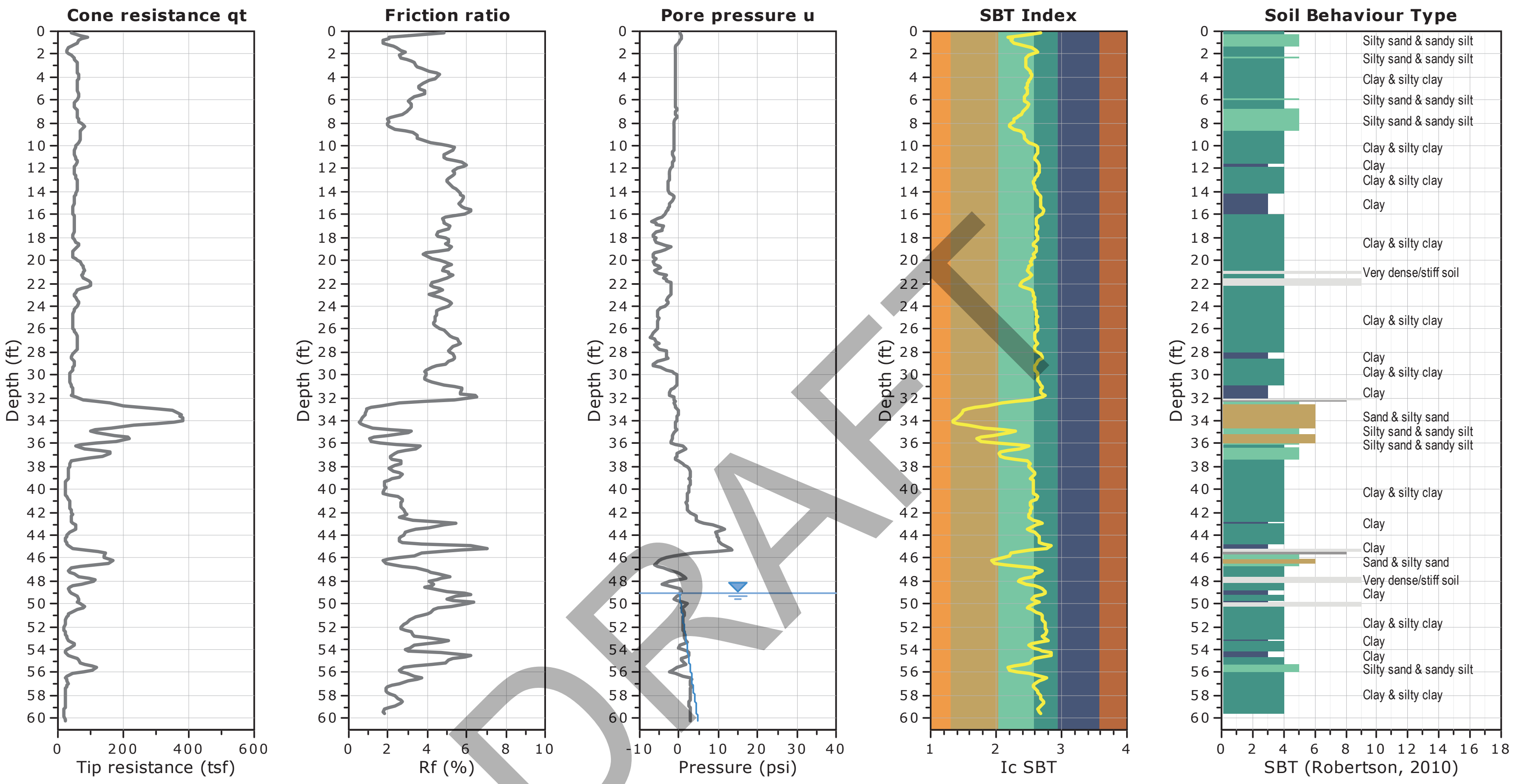
CONE PENETRATION TEST RESULTS
CPT-1

Date 04/07/22 | Project No. 20-1817 | Figure A-5



Total depth: 60.2 ft, Date: March 3, 2022
Groundwater Depth: 49 feet (based on tape-drop measurement)
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty clay
- 🟩 5. Silty sand to sandy silt
- 🟨 6. Clean sand to silty sand
- 🟧 7. Gravelly sand to sand
- ⬛ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

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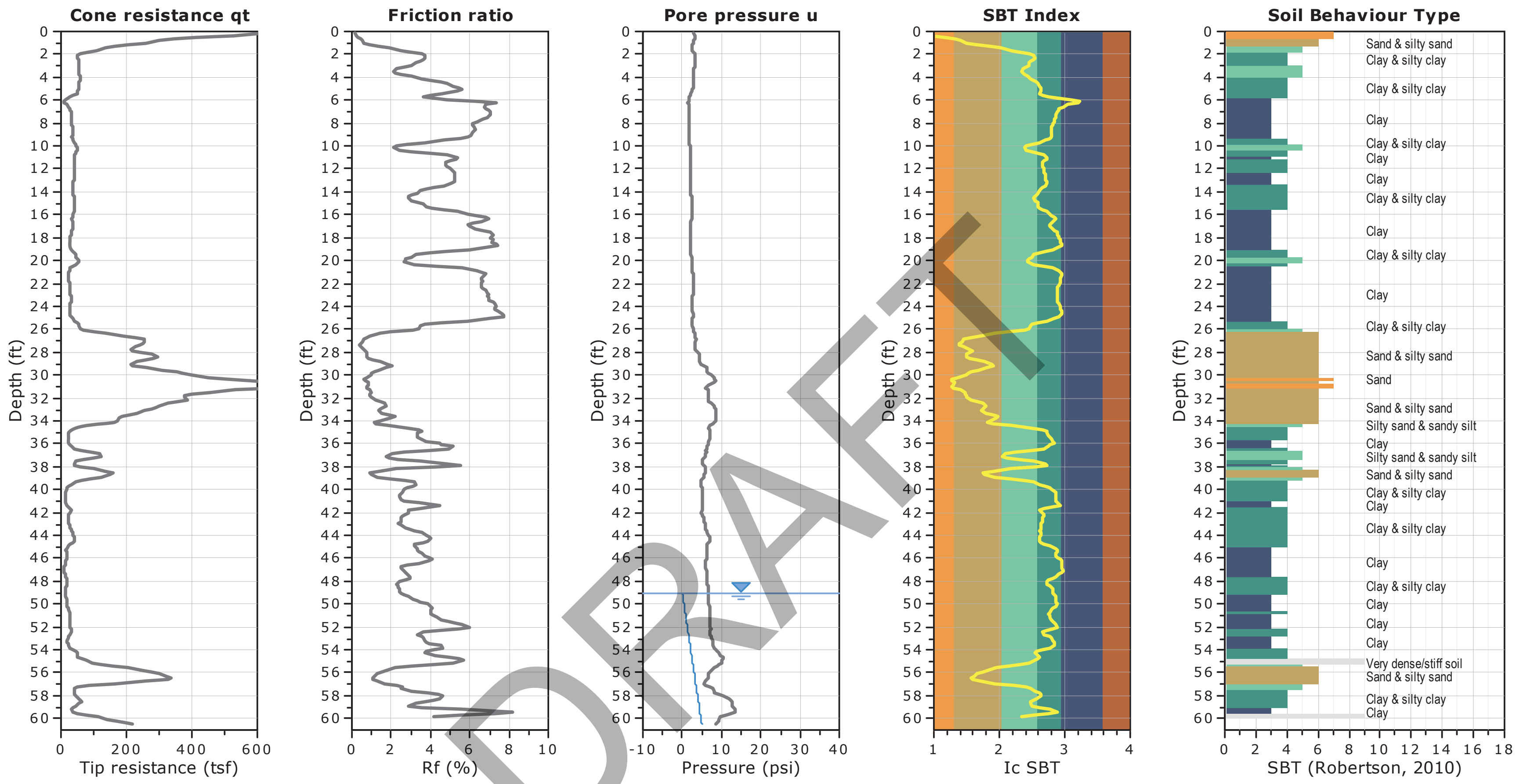
CONE PENETRATION TEST RESULTS
CPT-2

Date 04/07/22	Project No. 20-1817	Figure A-6



Total depth: 60.5 ft, Date: March 3, 2022
Groundwater Depth: 49 feet (assumed based on CPT-2 and CPT-5)
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty clay
- 🟩 5. Silty sand to sandy silt
- 🟨 6. Clean sand to silty sand
- 🟧 7. Gravelly sand to sand
- ⬛ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

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Mountain View, California

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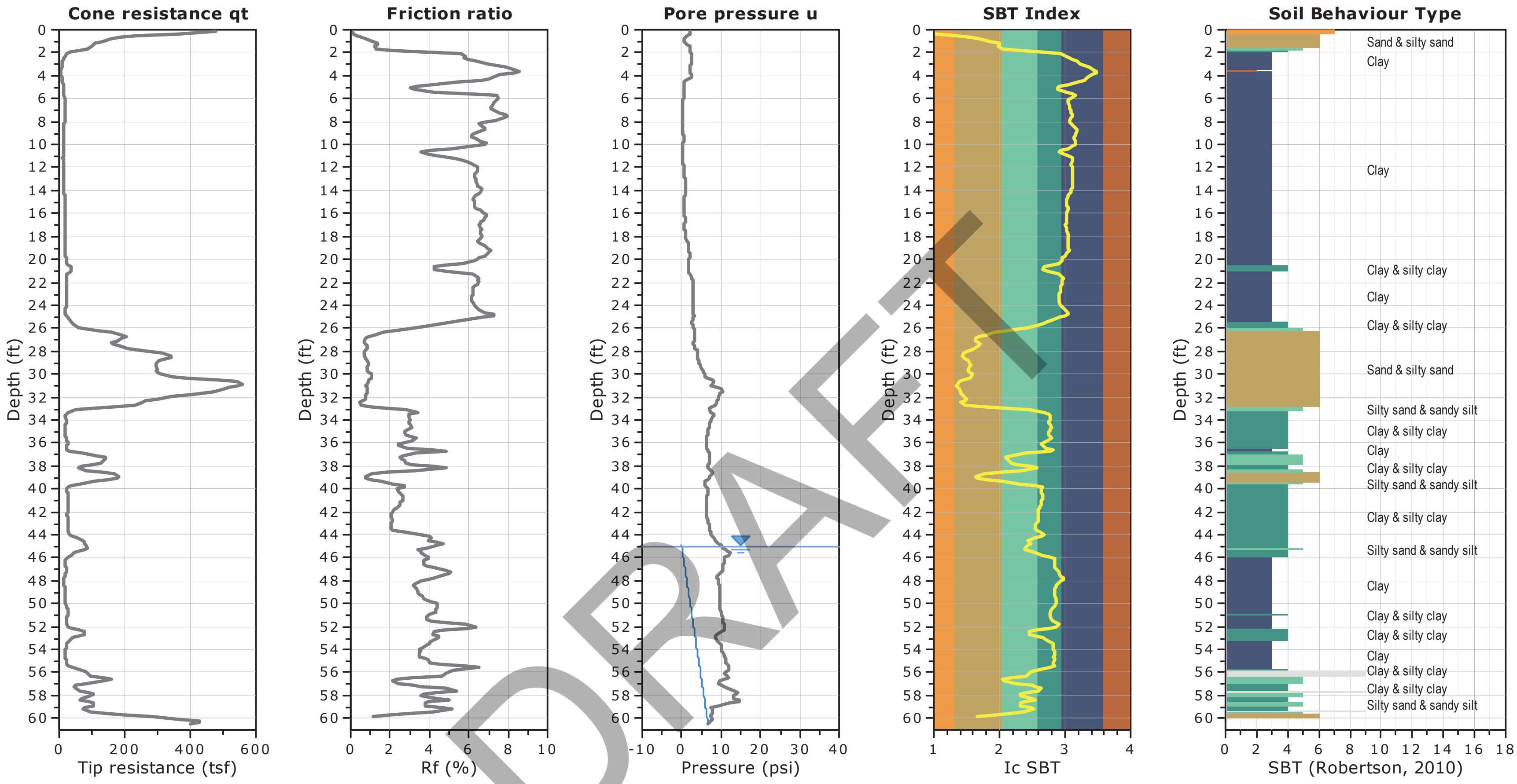
CONE PENETRATION TEST RESULTS
CPT-3

Date 04/07/22 | Project No. 20-1817 | Figure A-7



SBT legend

- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty clay
- 🟩 5. Silty sand to sandy silt
- 🟫 6. Clean sand to silty sand
- 🟧 7. Gravelly sand to sand
- ⬜ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

Total depth: 60.5 ft, Date: March 3, 2022
Groundwater Depth: 45 feet (based on tape-drop measurement)
Cone Operator: Middle Earth Geo Testing, Inc.

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Mountain View, California

ROCKRIDGE
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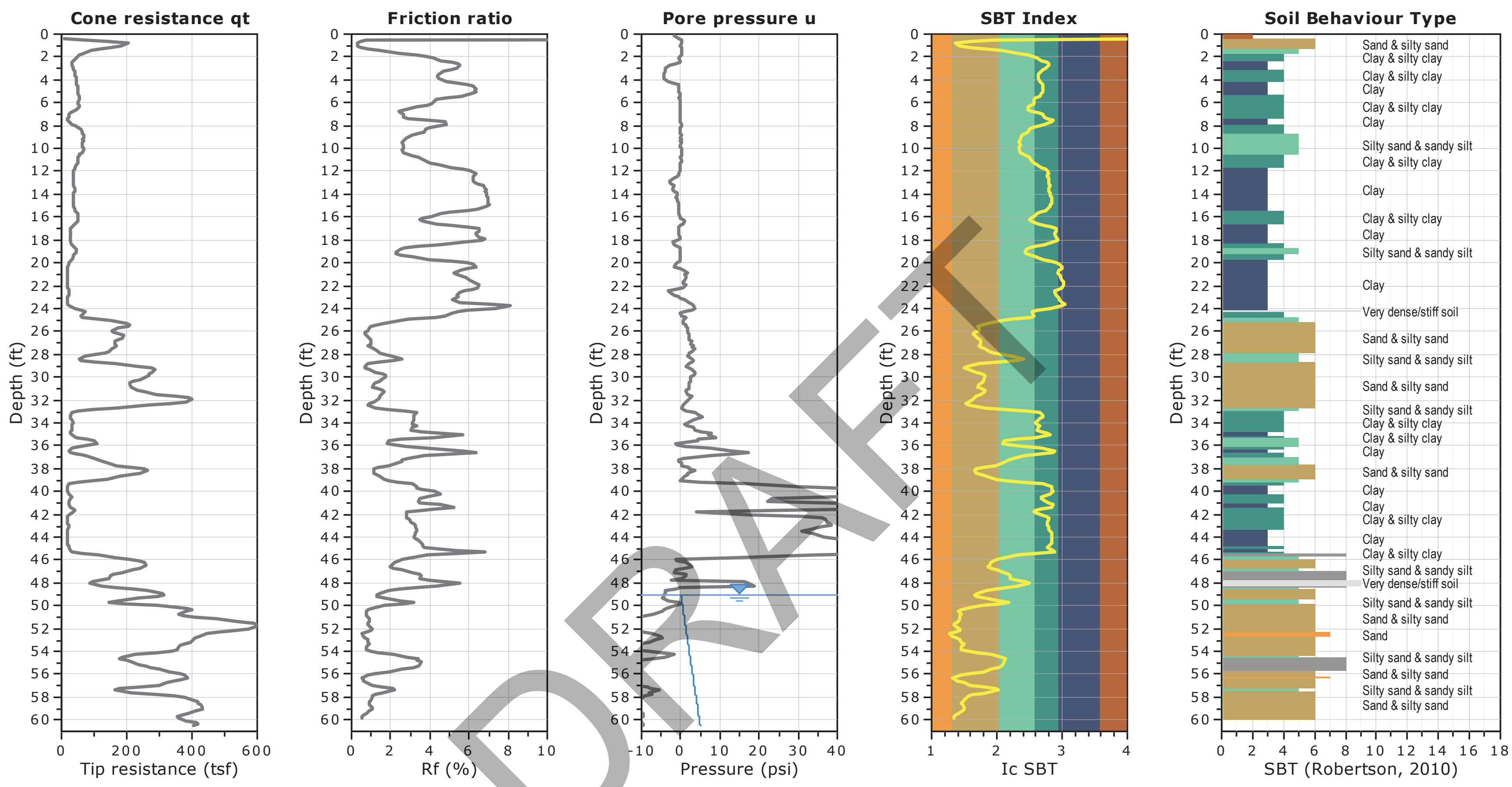
CONE PENETRATION TEST RESULTS
CPT-4

Date 04/07/22	Project No. 20-1817	Figure A-8



Total depth: 60.5 ft, Date: March 3, 2022
Groundwater Depth: 49 feet (based on tape-drop measurement)
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

🟥 1. Sensitive fine grained	🟩 4. Clayey silt to silty clay	🟧 7. Gravelly sand to sand
🟫 2. Organic material	🟩 5. Silty sand to sandy silt	⬛ 8. Very stiff sand to clayey sand
🟦 3. Clay to silty clay	🟨 6. Clean sand to silty sand	⬜ 9. Very stiff fine grained

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Mountain View, California

ROCKRIDGE
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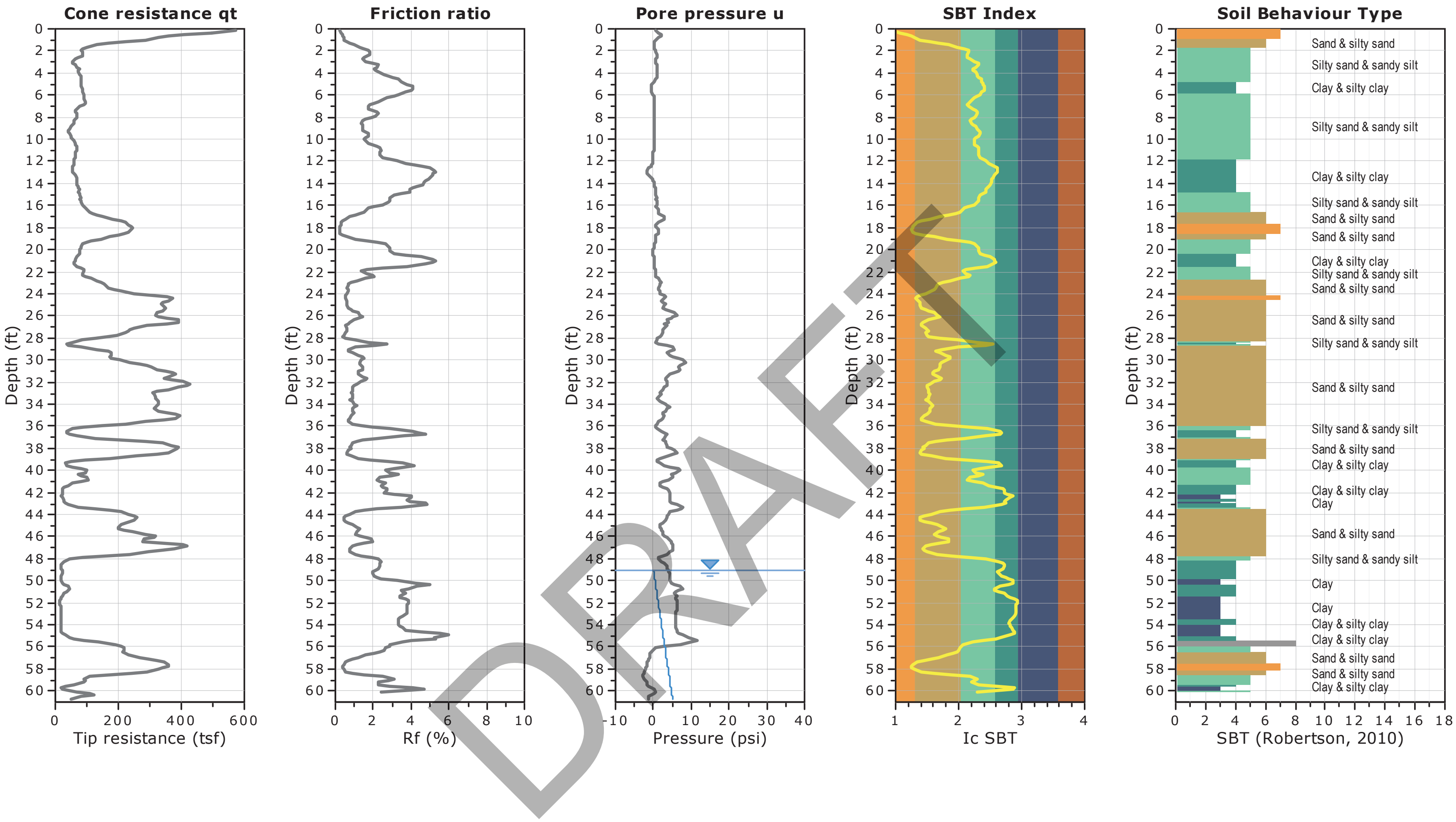
CONE PENETRATION TEST RESULTS
CPT-5

Date 04/07/22 | Project No. 20-1817 | Figure A-9



Total depth: 60.7 ft, Date: March 3, 2022
Groundwater Depth: 49 feet (assumed based on CPT-2 and CPT-5)
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty clay
- 🟩 5. Silty sand to sandy silt
- 🟨 6. Clean sand to silty sand
- 🟧 7. Gravelly sand to sand
- ⬛ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

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ROCKRIDGE
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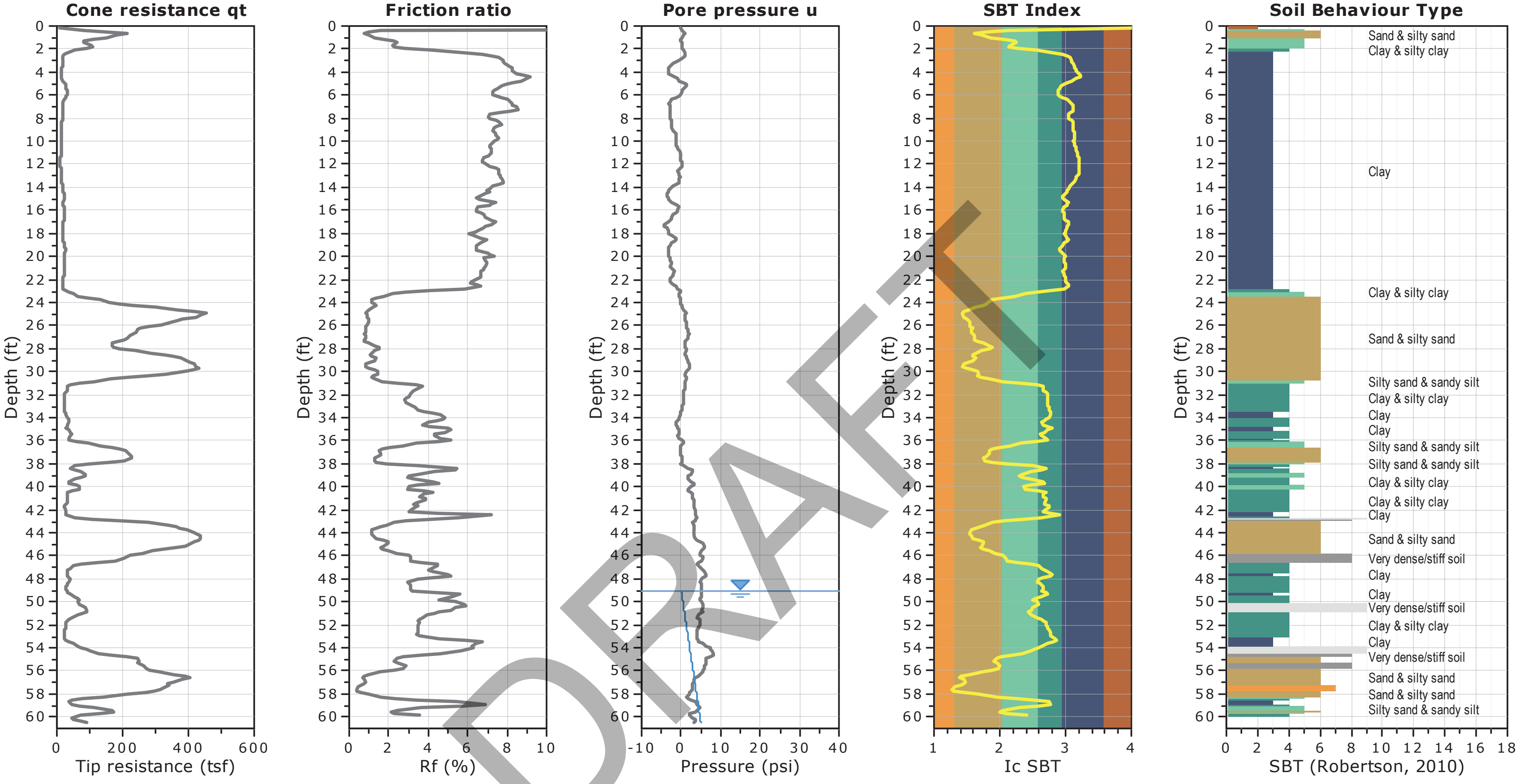
CONE PENETRATION TEST RESULTS
CPT-6

Date 04/07/22	Project No. 20-1817	Figure A-10



Total depth: 60.7 ft, Date: March 3, 2022
Groundwater Depth: 49 feet (assumed based on CPT-2 and CPT-5)
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty clay
- 🟩 5. Silty sand to sandy silt
- 🟨 6. Clean sand to silty sand
- 🟧 7. Gravelly sand to sand
- ⬛ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

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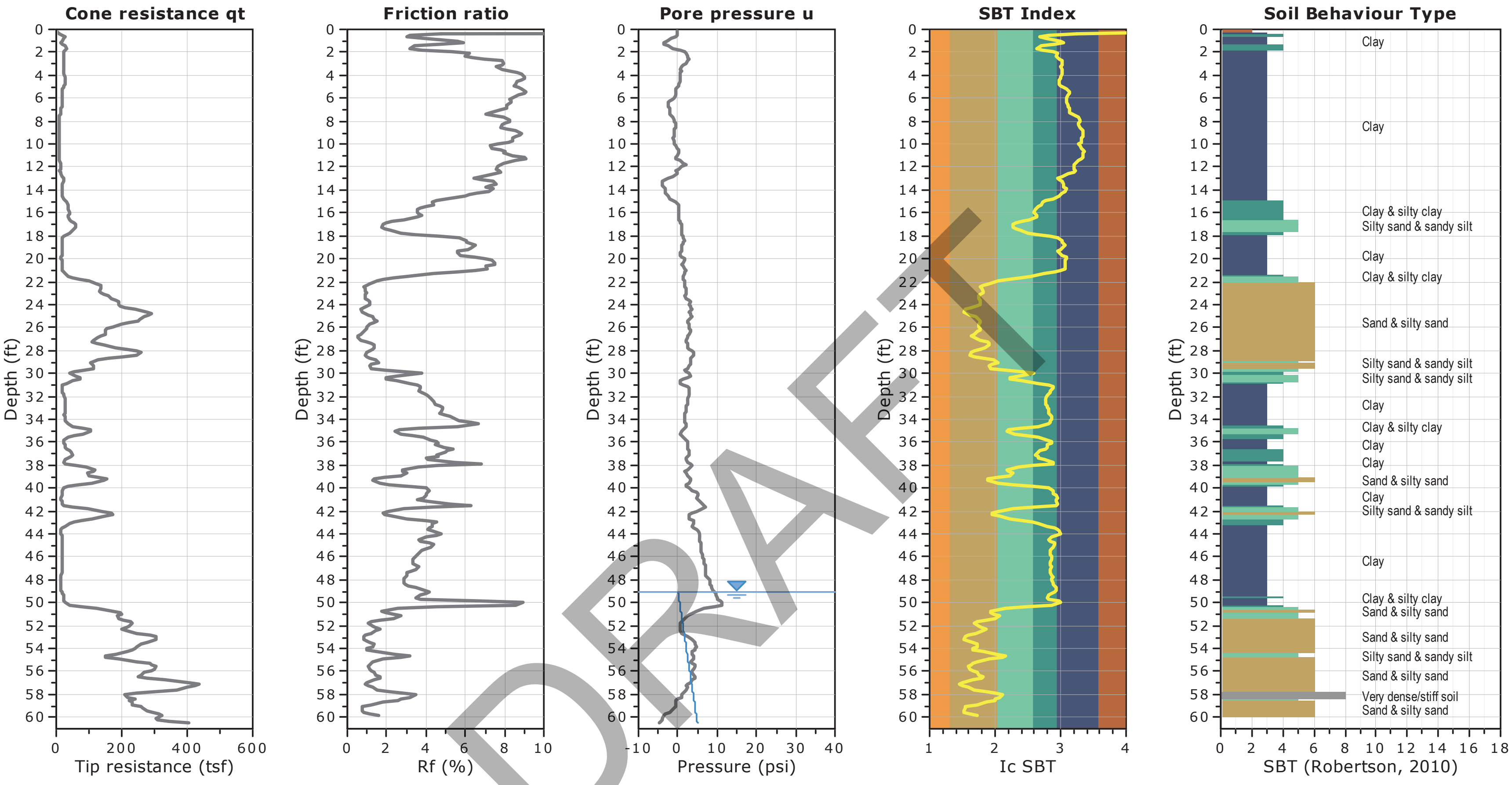
CONE PENETRATION TEST RESULTS
CPT-7

Date 04/07/22	Project No. 20-1817	Figure A-11



SBT legend

- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty clay
- 🟩 5. Silty sand to sandy silt
- 🟨 6. Clean sand to silty sand
- 🟧 7. Gravelly sand to sand
- ⬛ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

Total depth: 60.5 ft, Date: March 3, 2022
Groundwater Depth: 49 feet (assumed based on CPT-2 and CPT-5)
Cone Operator: Middle Earth Geo Testing, Inc.

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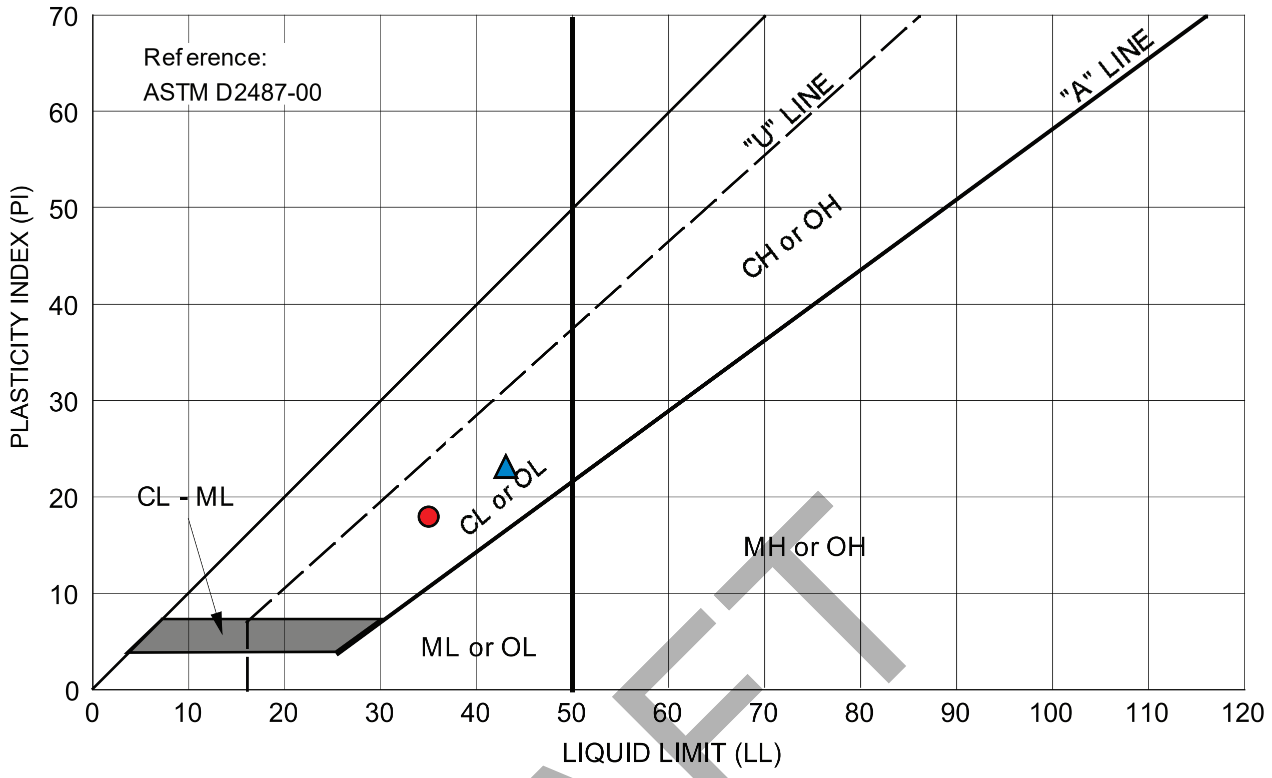
CONE PENETRATION TEST RESULTS
CPT-8

Date 04/07/22 | Project No. 20-1817 | Figure A-12

APPENDIX B
Laboratory Test Results



Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-1 at 3.0 feet	CLAY with SAND (CL), dark brown to gray-brown	15.9	35	18	--
▲	B-2 at 6.0 feet	SANDY CLAY (CL), dark brown	21.3	43	23	--

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PLASTICITY CHART

Date 04/06/22 | Project No. 20-1817 | Figure B-1

	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	15.8	16.4	10.8	19.2	19.8	18.0	

SOIL DATA

SYMBOL	SOURCE	DEPTH (ft.)	Material Description	USCS
○	B-2	14.0'	CLAYEY SAND, brown	SC

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PARTICLE SIZE DISTRIBUTION REPORT

Date 03/28/22 | Project No. 20-1817 | Figure B-2

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

REPORT S220308A

March 10, 2022

	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^{--}	**Phosphate** PO_4^{3-}
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ohm-cm)	(Ohm-cm)		(mV)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)
B-1: CLAY with SAND (CL) dark brown to gray-brown	2.5	15.9	0.0016	11.4	0.0011	32,830	3,417	7.2	177	0.36	1.8	1.8	ND	10.9	6.5	26.8	17.5	0.4	4.2
B-2: SANDY CLAY (CL) dark brown	5.5	27.8	0.0028	17.9	0.0018	1,608	1,206	7.6	266	0.42	118.4	4.1	0.06	51.6	6.2	89.6	4.8	2.7	0.7
B-3: CLAY with SAND (CL) brown	5.25	8.2	0.0008	23.9	0.0024	3,417	1,139	7.6	268	0.33	30.5	1.2	0.28	9.1	2.6	2.2	1.8	0.4	1.2

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)

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SOIL CORROSIVITY
TEST RESULTS

Date 03/29/22	Project No. 20-1817	Figure B-3