GEOTECHNICAL INVESTIGATION

TOWNHOME DEVELOPMENT

1933-1961 PULGAS AVENUE

EAST PALO ALTO, CALIFORNIA 94303

Prepared for

Mr. Lee Xue

6044 Stevenson Boulevard

Fremont, California 94303

August 2023

Project No. 6290-1

August 15, 2023
6290-1

Mr. Lee Xue
6044 Stevenson Boulevard
Fremont, California 94538

**RE: GEOTECHNICAL INVESTIGATION
TOWNHOME DEVELOPMENT
1933-1961 PULGAS AVENUE
EAST PALO ALTO, CALIFORNIA**

Dear Mr. Xue:

In accordance with your request, we have performed a geotechnical investigation for the proposed townhome development to be constructed at 1933-1961 Pulgas Avenue in East Palo Alto, California. The accompanying report summarizes the results of our field exploration, laboratory testing, and engineering analysis, and presents geotechnical recommendations for the proposed development.

We refer you to the text of our report for specific recommendations.

Thank you for the opportunity to work with you on this project. If you have any questions or comments about the site conditions or the findings or recommendations from our site investigation, please call.

Very truly yours,

ROMIG ENGINEERS, INC.

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GEOTECHNICAL INVESTIGATION
TOWNHOME DEVELOPMENT
1933-1961 PULGAS AVENUE
EAST PALO ALTO, CALIFORNIA 94303

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AUGUST 2023

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GEOTECHNICAL INVESTIGATION
FOR
TOWNHOME DEVELOPMENT
1933-1961 PULGAS AVENUE
EAST PALO ALTO, CALIFORNIA

INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed townhome development to be constructed at 1933-1961 Pulgas Avenue in East Palo Alto, California. The location of the site is shown on the Vicinity Map, Figure 1. The purpose of this investigation was to evaluate subsurface conditions at the site and to provide geotechnical recommendations for the proposed development.

Project Description

The project consists of constructing a multi-family townhome at the referenced property in East Palo Alto. Based on the preliminary plans, the proposed development will consist of constructing one at-grade building consisting of a total of sixty (60) three-to-four story residential units with second-level decks at each unit. The three-story portion of the building will be constructed along the east (front) side (along Pulgas Avenue) with the four-story building occupying the west (rear) portion. A parking garage is planned behind the three-story portion and will extend to the ground/first level of the four-story building.

The existing single-story retail commercial building at the south portion of the property will remain in place. The existing residential, commercial, and storage structures occupying the remaining portion of the property will be demolished prior to construction. Structural loads are expected to be relatively light as is typical for this type of construction.

Scope of Work

The scope of work for this investigation was presented in detail in our agreement with you, dated May 16, 2023. In order to accomplish our investigation, we performed the following work.

- Reviewed of geologic, geotechnical, and seismic conditions in the site vicinity.

- Subsurface exploration consisting of drilling, sampling, and logging of six cone penetration tests (CPT's) and four exploratory borings throughout the site.

- Laboratory testing of a selected samples to aid in soil classification and to help evaluate their engineering properties of the soils encountered at the site.

- Engineering analysis and evaluation of the subsurface data to develop geotechnical design criteria for the project.

- Preparation of this report presenting our findings and geotechnical recommendations for the proposed project.

Limitations

This report has been prepared for the exclusive use of Mr. Lee Xue for specific application to developing geotechnical design criteria for the currently proposed townhome development to be constructed at 1933-1961 Pulgas Avenue in East Palo Alto, California. We make no warranty, expressed or implied, for the services we performed for this project. Our services were performed in accordance with geotechnical engineering principles generally accepted at this time and location. This report was prepared to provide engineering opinions and recommendations only. In the event that there are any changes in the nature, design or location of the project, or if any future improvements are planned, the conclusions and recommendations contained in this report should not be considered valid unless 1) the project changes are reviewed by us, and 2) the conclusions and recommendations presented in this report are modified or verified in writing.

The analysis, conclusions and recommendations contained in this report are based on site conditions as they existed at the time of our investigation; the currently planned improvements; review of readily available reports relevant to the site conditions; and laboratory test results. In addition, it should be recognized that certain limitations are inherent in the evaluation of subsurface conditions, and that certain conditions may not be detected during an investigation of this type. Changes in the information or data gained from any of these sources could result in changes in our conclusions or recommendations. If such changes do occur, we should be advised so that we can review our report in light of those changes.

SITE EXPLORATION AND RECONNAISSANCE

Our site reconnaissance and subsurface exploration were performed on June 21, 2023. Our subsurface exploration consisted of advancing six CPTs to depths of about 50 feet and four exploratory boring to depths ranging from 20 to 24 feet. The CPTs were advanced using an electronic cone penetration test (CPT) system, which was mounted on a truck having a downward pressure capacity of 20 tons and the exploratory borings were advanced using a portable Minuteman drilling and sampling equipment. The approximate locations of the

CPTs and borings are shown on the Site Plan, Figure 2. The CPT and boring logs are attached in Appendix A, and the results of our laboratory tests are attached in Appendix B.

Surface Conditions

The relatively flat site is located in a mixed residential and commercial area along the east side of Pulgas Avenue. The property was approximately 2.1 acres in total size extending from near the intersection of Pulgas Avenue and East Bayshore Road towards the north. At the time of our site reconnaissance, a single-story commercial building (which will remain) with an asphalt concrete parking area occupies the southern portion of the property. Multiple single-story residential, commercial, and storage structures generally occupies the rest of the property. The flatwork were generally in adequate condition. The site contained small to medium trees throughout.

Subsurface Conditions

At the locations of our borings, we generally encountered firm to very stiff sandy lean clay and fat clay of low to high plasticity to the maximum depths explored of 20 to 24 feet. We noted that the upper 2 feet of soil encountered in Borings EB-1, EB-2, and EB-4 appeared to be fill material. Some medium dense to dense clayey sands were also encountered between depths of approximately 16 to 23.5 feet in Boring EB-2.

At the locations of our CPTs, we generally encountered stiff to very stiff lean to fat clay with some interbedded strata of medium dense to dense sands in various thickness and depths to the maximum depth explored of about 50 feet. We noted that the sand layers were encountered primarily between depths of about 15 to 22 feet in CPT-01, 40 to 44 feet in CPT-02, 26 to 32 feet in CPT-04, 30 to 32 feet in CPT-05, and at depths of 20 to 21 feet and 44 to 46 feet in CPT-06.

A Liquid Limit of 57 and Plasticity Index of 33 were measured on a sample of the near-surface soils obtained from EB-1. In addition, a Liquid Limit of 35 and Plasticity Index of 14 were measured on a sample of the near-surface soils obtained from EB-3. These test results indicate that the surface and near-surface soils at the site generally have moderate to high plasticity and a moderate to very high potential for expansion.

We note that portions the medium dense sands encountered in our borings and portions of the sands and silts encountered at various depths in the CPTs appeared to be susceptible to liquefaction during strong seismic shaking. Details of our liquefaction evaluation are included in the section below titled "LIQUEFACTION EVALUATION".

Ground Water

Based on the dynamic pore pressure response, ground water was estimated to be present at depths ranging from about 7 to 9.2 feet in our CPTs during field exploration. Ground water was also encountered at depths of about 7 to 8 feet in our exploratory borings during field exploration. The CPTs and borings were backfilled with grout immediately after drilling and sampling was completed; therefore, a stabilized ground water level measurement was not have been obtained.

Information presented in the Seismic Hazard Zones Report 111 for the Palo Alto Quadrangle (California Geological Survey, 2006) indicates that the historical high ground water level near the area of the site is less than 10 feet below the existing ground surface. Based on the site location near the bay, and the findings from our investigation and our local experience, it is our opinion that the ground water level at the site may rise to within 5 feet or shallower below the existing ground surface. Please be cautioned that fluctuations in the level of ground water can occur due to variations in rainfall, local surface and subsurface drainage patterns, landscaping, and other factors.

GEOLOGIC SETTING

As part of our investigation, we have briefly reviewed our local experience and geologic literature pertinent to the area of the site. The information that we reviewed for this study indicates that the site is underlain by Holocene-age floodplain deposits, Qhfp, (Brabb, Graymer, and Jones, 2000). The floodplain deposits are generally expected to consist of dense sandy to silty clay. Lenses of coarser material may be locally present and usually occur between levee deposits and basin deposits. The geology of the site vicinity is shown on the Vicinity Geologic Map, Figure 3.

The Seismic Hazard Zones Map of the Palo Alto Quadrangle prepared by the California Division of Mines and Geology in 2006, indicates the site is located in an area that may be underlain by soils potentially susceptible to liquefaction during a major earthquake. A discussion regarding the potential for liquefaction at the site is presented in the later section of the report.

The property and the immediate site vicinity are located in an area that generally slopes very gently to the east toward the San Francisco bay. The site is located at an elevation of approximately 12 feet above sea level (see Figure 1).

Faulting and Seismicity

There are no mapped through-going faults within or adjacent to the site and the site is not located within a State of California Earthquake Fault Zone (formerly known as a Special Studies Zone), an area where the potential for fault rupture is considered probable. The closest active fault is the San Andreas fault, which is located approximately 7.2 miles southwest of the property. Thus, the likelihood of surface rupture occurring from active faulting at the site is relatively low.

The San Francisco Bay Area is, however, an active seismic region. Earthquakes in the region result from strain energy constantly accumulating because of the northwestward movement of the Pacific Plate relative to the North American Plate. On average about 1.6-inches of movement occur per year. Historically, the Bay Area has experienced large, destructive earthquakes in 1838, 1868, 1906, and 1989. In addition to the San Andreas Fault, the faults considered most likely to produce large earthquakes in the area include the San Gregorio, Hayward, and Calaveras faults. The San Gregorio fault is located approximately 17 miles southwest of the site. The Hayward and Calavera faults are located approximately 12 and 16 miles northeast of the site, respectively. These faults and significant earthquakes that have been documented in the Bay Area are listed in Table 1 below, and are shown on the Regional Fault and Seismicity Map, Figure 4.

Table 1. Earthquake Magnitudes and Historical Earthquakes
Townhome Development
East Palo Alto, California

Fault	Maximum Magnitude (Mw)	Historical Earthquakes	Estimated Magnitude
San Andreas	7.9	1989 Loma Prieta	6.9
		1906 San Francisco	7.9
		1865 N. of 1989 Loma Prieta Earthquake	6.5
		1838 San Francisco-Peninsula Segment	6.8
		1836 East of Monterey	6.5
Hayward	7.1	1868 Hayward	6.8
		1858 Hayward	6.8
Calaveras	6.8	1984 Morgan Hill	6.2
		1911 Morgan Hill	6.2
		1897 Gilroy	6.3
San Gregorio	7.3	1926 Monterey Bay	6.1

In the future, the subject property will undoubtedly experience severe ground shaking during moderate and large magnitude earthquakes produced along the San Andreas fault or other active Bay Area fault zones. Using information from recent earthquakes, improved mapping of active faults, ground motion prediction modeling, and a new model for estimating earthquake probabilities, a panel of experts convened by the U.S.G.S. have concluded there is a 72 percent chance for at least one earthquake of Magnitude 6.7 or larger in the Bay Area before 2043. The Hayward fault has the highest likelihood of an earthquake greater than or equal to magnitude 6.7 in the Bay Area, estimated at 33 percent, while the likelihood on the San Andreas and Calaveras faults is estimated at approximately 22 and 26 percent, respectively (Aagaard et al, 2016).

Earthquake Design Parameters

The State of California currently requires that buildings and structures be designed in accordance with the seismic design provisions presented in the 2022 California Building Code and in ASCE 7-16, "Minimum Design Loads for Buildings and Other Structures". Based on site geologic conditions and on information from our subsurface exploration at the site, the site may be classified as Site Class D, stiff soil, in accordance with Chapter 20 of ASCE 7-16. Spectral Response Acceleration parameters and site coefficients may be taken directly from the SEAOC/OSHPD website based on the longitude and latitude of the site. For site latitude (37.4552), longitude (-122.1303) and Site Class D, design parameters are presented on Table 2 below.

Table 2. 2022 CBC Seismic Design Criteria
Townhome Development
East Palo Alto, California

Spectral Response Acceleration Parameters			Design Value
Mapped Value for Short Period	-	S_S	1.500
Mapped Value for 1-sec Period	-	S_1	0.600
Site Coefficient	-	F_a	1.0
Site Coefficient	-	F_v	1.7
Adjusted for Site Class	-	S_{MS}	1.500
Value for Design Earthquake	-	S_{DS}	1.000

LIQUEFACTION EVALUATION

Severe ground shaking during an earthquake can cause loose to medium dense granular soils to densify. If the granular soils are below ground water, the densification can cause increases in pore water pressure, which can lead to soil softening, liquefaction, and ground deformation. Soils most prone to liquefaction are saturated, loose to medium dense, silty sands and sandy silts with limited drainage, and in some cases, sands and gravels that are interbedded with or that contain seams or layers of impermeable soil.

Liquefaction Evaluation of the CPTs

To evaluate the potential for earthquake-induced liquefaction of the soils at the site, we performed a liquefaction analysis of the CPT data using the program CLiq, developed by GeoLogismiki by applying several published methodologies, including Robertson (NCEER, 2009) and Idriss and Boulanger (2014). The results listed in Table 3 on the following page reflect the Robertson 2009 method, which included limiting vertical strains to 1 percent on clay-like soils, and the Idriss and Boulanger 2014 method, which included a weighting factor on vertical strains with depth, per Cetin et al 2009.

The sand, silty sand, sandy silt, and clayey silt to silty clay strata that we encountered at the site, below the assumed high ground water level of approximately 5 feet below the ground surface were considered in our liquefaction analysis. The results of our analyses indicate that several of the interbedded strata of sand, silty sand, sandy silt, and clayey silt to silty clay encountered in the CPTs at various depths could liquefy when subjected to a peak ground acceleration (PGA) of 0.644g, the PGA_M for maximum considered earthquake based on ASCE 7-16. The results of our liquefaction evaluation are presented in Table 3 below, and are presented in Figures C-1 and C-2 in Appendix C.

Table 3: Results of Liquefaction Evaluation
Townhome Development
East Palo Alto, California

CPT No.	Robertson 2009 Settlement (Inches)	Idriss and Boulanger 2014 Settlement (Inches)	Average Settlement (Inches)
CPT-01	0.7	1.2	1.0
CPT-02	0.4	0.5	0.5
CPT-03	0.5	0.6	0.6
CPT-04	0.5	0.9	0.7
CPT-05	0.5	0.8	0.7
CPT-06	0.4	0.5	0.5

Based on our analyses of the CPT data, total settlement that could occur at the ground surface as a result of liquefaction from the design-level earthquake is estimated to range from about 0.5 to 1 inch.

Liquefaction Evaluation of the Boring

To evaluate the potential for earthquake-induced liquefaction of the medium dense sands encountered in Boring EB-2 at the site, we performed a liquefaction analysis of the data generally following the methods described in the 2008 publication by Idriss and Boulanger titled "Soil Liquefaction During Earthquakes". The poorly-graded and clayey sands strata that we encountered at the site below the projected high ground water level of 5 feet below the ground surface was considered in our liquefaction analysis.

The results of our analysis indicate that the medium dense sands encountered between depths of about 12 to 22 feet in our Boring EB-2 could liquefy when subjected to a peak ground acceleration of 0.644g (the PGA_M for maximum considered earthquake based on ASCE 7-16). The total settlement that could occur within these sand layers as a result of liquefaction from the design-level earthquake is estimated to be approximately 2-¼ inches.

Total Liquefaction Settlement

Based on the results of our analyses of the boring and CPT data, total settlement as a result of liquefaction from the design-level earthquake at the project site is estimated to be up to about 2-¼ inches. In our opinion, differential settlement on the order of up to about 1 inch over a horizontal distance of 25 feet is possible from liquefaction at the ground surface during seismic shaking. The estimated dynamic differential settlement mentioned above should be considered during structural design of the proposed structures. This differential settlement could also affect exterior flatwork and driveway areas supported at existing surface grades during a major seismic event.

Geologic Hazards

In addition to liquefaction potential, we also reviewed the potential for other geologic hazards to impact the site and the proposed development considering the geologic setting and the soils encountered during our investigation. The results of our review are presented below.

- Fault Rupture - The site is not located in a State of California Earthquake Fault Zone or area where fault rupture is considered likely. Therefore, active faults are not believed to exist beneath the site and the potential for fault rupture to occur at the site is considered low.

- Ground Shaking - The site is located in an active seismic area. Moderate to large earthquakes are probable along several active faults in the greater Bay Area over a 30- to-50-year design life. Strong ground shaking should therefore be expected several times during the design life of the development, as is typical for sites throughout the Bay Area. The proposed townhomes and other improvements should be designed in accordance with current earthquake resistance standards.

- Dynamic Densification - Dynamic densification can occur during moderate and large earthquakes when soft or loose, natural or fill soils above the ground water table are densified and settle, often unevenly across a site. The soils encountered in our borings and CPTs above the historical high ground water table were generally stiff to very stiff clays, which is generally not prone to significant dynamic densification. In our opinion, the likelihood of significant dynamic densification affecting the proposed development is low provided the recommendations presented in our report are followed during design and construction.

- Expansive Soil - Based upon the results of our laboratory testing and visual classification, we have concluded that the surface and near-surface soils at the site have a moderate to very high potential for expansion and will be subject to expansion and contraction during wetting and/or drying cycles. However, in our opinion, the potential for building distress/damage can be greatly reduced if the proposed building is designed and constructed as recommended. We note however, that pavement/flatwork supported on expansive soils will likely be prone to differential settlement/movement and have a shorter service life due to expansive soil differential movement than a site with less expansive soil conditions.

CONCLUSIONS

From a geotechnical viewpoint, the site is suitable for the proposed townhome development provided the recommendations presented in this report are followed during design and construction.

The primary geotechnical concerns for the proposed development are: 1) the presence of the medium dense sands and silts which may be prone to liquefaction during a strong seismic shaking; 2) the presence of moderately to highly expansive surface and near-surface soils across the site; and 3) the potential for severe ground shaking at the site during a major earthquake.

As discussed above, on the order of about 1-inch of liquefaction-induced differential settlemet across a horizontal distance of 25 feet is estimated across the ground surface. The estimated dynamic settlement should be considered during the structural design of the foundation system.

In our opinion, structures supported on expansive soils will likely be subject to differential movement due to significant volume changes caused by seasonal fluctuations in the soil moisture content. To help reduce the potential for distress due to expansive soil movement and liquefaction-induced settlement, in our opinion, the proposed building may be supported on a series of reinforced concrete continuous spread footings arranged in a grid pattern with added reinforcing to provide a stiffer foundation more capable of tolerating differential soil movement. We also recommend that a layer of non-expansive fill placed below concrete slabs-on-grade. As an alternative to a grid foundation, the building may be supported on a relatively rigid structural mat foundation. Specific geotechnical recommendations for the project are presented in the following sections of this report.

Because subsurface conditions may vary from those encountered at the locations of our exploratory borings and CPTs, and to observe that our recommendations are properly implemented, we recommend that we be retained to 1) review the project plans for conformance with our recommendations and 2) observe and test during the earthwork and foundation installation phases of construction.

FOUNDATIONS

Shallow Foundations

In our opinion, the proposed building may be supported on a series of continuous spread footings bearing on undisturbed stiff native soil. Isolated footings should be avoided where differential movements would be problematic. All footings should have a width of at least 15 inches and should extend at least 32 inches below exterior grade and at least 28 inches below the bottom of concrete slabs-on-grade, and at least 20 inches below interior crawl-space grade, whichever is deeper. Continuous footings may be designed for an allowable bearing pressure of 2,500 pounds per square foot from dead plus live loads, with a one-third increase allowed when considering additional short-term wind or seismic loading.

Due to the expansive soil conditions and the potential for liquefaction, we recommend that continuous footings be arranged in a grid pattern, and we suggest that the grids be spaced at intervals no greater than approximately 20 feet or as determined by the structural engineer. In addition, we recommend that individual continuous footings be capable of spanning a distance of at least 15 feet and cantilevering a minimum distance of at least 5 feet under full dead load. From a geotechnical viewpoint, continuous footings should be reinforced with at least two no. 5 bars, top and bottom, or more as determined by the structural engineer to accommodate the spanning/cantilever criteria discussed above.

All footings located adjacent to utility lines should be embedded below a 1:1 plane extending up from the bottom edge of the utility trench. All continuous footings should be reinforced with top and bottom steel to provide structural continuity and to permit spanning of local irregularities.

The bottom of all footing excavations should be cleaned of loose and soft soil and debris. A member of our staff should observe all footing excavations prior to placement of reinforcing steel to confirm that they expose suitable material, have at least the recommended minimum dimensions, and have been properly cleaned. If firm/soft/wet or loose soils are encountered in the foundation excavations, our field representative will require these materials to be removed and may require a deeper footing embedment depth before the reinforcing steel and concrete is placed.

Structural Mat Foundation

As an alternative to a grid foundation, the proposed building may be supported on a relatively rigid structural mat foundation bearing on a 12-inch-thick section of non-expansive fill placed on a properly prepared native soil subgrade. The mat may be designed for an average allowable bearing pressure of 1,500 pounds per square foot for combined dead plus live loads, with maximum localized bearing pressures of 2,500 pounds per square foot at column or wall loads. The allowable bearing pressure may be increased by one-third for total loads including wind or seismic forces. These pressures are net values; the weight of the mat may be neglected in design.

Due to the presence of the expansive near-surface soils, the mat should also have a thickened perimeter edge. The thickened perimeter edge should have a width of at least 12 inches, should extend at least 30 inches below exterior finished grade, and at least 16 inches below the bottom of the mat (at least 4 inches below the non-expansive fill), whichever is deeper.

The mat should be reinforced to provide structural continuity and to permit spanning of local irregularities. A modulus of subgrade reaction (Kv1) of 100 pounds per cubic inch (pci) may be assumed for a 1-foot-square bearing area, which should be scaled to account for mat foundation size effects in accordance with NAVFAC Design Manual 7.02. Alternatively, the modulus of subgrade reaction (Kv) may be estimated based on the anticipated building load and differential static settlement (typically on the order of 10 to 20 pci). Once building loads are available, we should be contacted to update the modulus of subgrade reaction (Kv) for the mat subgrade based on the building loads and estimated post-construction differential settlement. In addition, due to the potential for expansive soil movement and liquefaction-induced settlement, we recommend that the mat foundation be capable of simply spanning a distance of at least 15 feet and cantilevering a minimum distance of at least 5 feet under full dead loads.

In our opinion, the mat may be constructed directly on a mat damp-proofing or capillary barrier system consisting of at least 4 inches of free-draining gravel, such as ½- to ¾-inch clean, crushed rock, over 8 inches of Class 2 Aggregate base (a total of 12 inches thick non-expansive fill) as described above. The crushed rock layer should be leveled and densified by a vibrating plate.

Prior to mat construction, the mat subgrade and non-expansive fill section should be scarified, prepared, and compacted as recommended in the section titled "Compaction". Prior to mat construction, the non-expansive fill section should be proof-rolled to provide a smooth firm surface for mat support. Our representative should observe and test during the preparation and compaction of the mat subgrade and non-expansive fill section.

Lateral Loads

Lateral loads will be resisted by friction between the bottom of the footings or mat foundation and the supporting subgrade. A coefficient of friction of 0.25 may be assumed for design. In addition to friction, lateral resistance may be provided by passive soil pressure acting against the sides of foundations cast neat in footing excavations or backfilled with properly compacted structural fill. We recommend assuming an equivalent fluid pressure of 300 pounds per cubic foot for passive soil resistance, where appropriate. The upper foot of passive soil resistance should be neglected where soil adjacent to the footing is not covered with a slab or pavement.

Settlement

Thirty-year, post-construction differential settlement due to static loads is not expected to exceed 1-inch over a horizontal distance of 25 feet across the proposed building provided foundations are designed and constructed as recommended. In addition, as stated in the above sections, we estimate that differential dynamic settlement of about 1-inch across a horizontal distance of 25 feet could occur across the site, as a result of the analyzed seismic event. The above estimated static and dynamic differential settlement should be considered during the design of the building and its foundation system.

Elevator Pit or Below-Grade Structure Damp-Proofing

We note that ground water was encountered at the site at depths ranging from 7 to 9 feet below ground surface during our subsurface exploration, and the ground water level may rise to a depth shallower than 5 feet below ground surface. We have not provided recommendations regarding the method or details for water/damp-proofing of elevator pits or below-grade structure since design of damp-proofing systems is outside of our scope of services and expertise. Installing adequate water/damp-proofing below and along the sides of the below-grade slab and the elevator pit is essential for the success of the structure. Placing concrete with a low water:cement ratio should be considered as one step of good damp-proofing as discussed in the section of this report titled “Slabs-On-Grade”. The damp-proofing system below the elevator pit may be placed directly on the compacted and approved soil subgrade, on a thin layer of crushed rock, or on a thin working slab, as determined by the water-proofing consultant.

SLABS-ON-GRADE

General Slab Considerations

At least portions of the near-surface soils at the site have a very high expansion potential. Expansive soils have a tendency to expand due to increase in moisture and shrink as they dry. This can result in slab cracking, differential settlement, and heave regardless of the geotechnical measures implementd. Our recommendations below will help reduce the impacts of the expansive soils beneath slabs-on-grade, but will not eliminate the risk entirely.

To reduce the potential for movement of the soil subgrades below at-grade concrete slabs-on-grade, at least the upper 6 inches of the expansive surface soil should be scarified, moisture conditioned, and compacted in accordance with Table 3 “Compaction Recommendations” in the later section of this report. The native soil subgrade should be kept moist up until the time the non-expansive fill, crushed rock and vapor barrier, and/or aggregate base section is placed. Slab subgrades and non-expansive fill should be prepared and compacted as recommended in the section of this report titled “Earthwork”.

Overly soft or moist soils should be removed from slab-on-grade areas. Exterior flatwork and interior slabs-on-grade should be underlain by a layer of non-expansive fill as described below. The non-expansive fill should consist of Class 2 aggregate base or clayey soil with a Plasticity Index of 15 or less.

Considering the potential for expansive soil movements of the surface soils, we expect that reinforced slabs will perform better than unreinforced slabs. Consideration should also be given to using a control joint spacing on the order of 2 feet in each direction for each inch of slab thickness.

Exterior Flatwork

Concrete walkways and exterior flatwork should be at least 4 inches thick and should be constructed on at least 12 inches of Class 2 aggregate base. The potential for distress to exterior slabs due to expansive soil movements could be reduced by placing and compacting at least 6 to 12 inches of non-expansive fill, or aggregate base, below the minimum 12-inch thick layer of aggregate base recommended above (i.e., a total of 18 to 24 inches of non-expansive fill).

To improve performance, exterior slabs-on-grade, such as for patios, may be constructed with a thickened edge to improve edge stiffness and to reduce the potential for water seepage under the edge of the slabs and into the underlying base and subgrade. In our opinion, the thickened edges should be at least 8 inches wide and ideally should extend at least 4 inches below the bottom of the underlying aggregate base layer.

Due to the presence of near-surface expansive soil, pervious flatwork/pavement is generally not recommended/desirable since the pavement will likely be prone to more significant heaving and shrinkage (uplift and downward) movement due to seasonal moisture fluctuation and introduction of surface water onto the pavement subgrade. More differential settlement under wheel loads could also occur due to soil softening/saturation. In addition, soil saturation at pervious pavement near a structure will likely cause more prominent differential settlement/movement across the building foundations. However, if pervious pavement will be required, the pavement preferably should be located at least 8 feet away from any structures. In addition, the owner must also be willing to accept a higher level of risk of differential movement damage and extra maintenance, if it occurs.

Interior Slabs

Interior concrete slab-on-grade floors should be constructed on a layer of non-expansive fill at least 24 inches thick that is placed and compacted on a properly prepared soil subgrade. If a structural mat foundation is planned, the non-expansive fill section may be reduced to 12 inches thick as recommended above. If a structural mat is not planned, it would also be preferable in non-living areas, such as the garage, for the slabs to float relative to the perimeter foundation. Due to the potential for expansive soil movement and liquefaction-induced settlement, we recommend that the interior floor slab be at least 5 inches (and preferably 6 inches) in thickness and be reinforced with more than typical steel reinforcement. Recycled aggregate base should not be used for non-expansive fill below interior slabs-on-grade, since adverse vapor could occur from crushed asphalt components.

Moisture Considerations

In areas where dampness of at-grade concrete floor slabs would be undesirable, such as within the building interior, concrete slabs should be underlain by at least 4 inches of free-draining gravel, such as ½- to ¾-inch clean crushed rock with no more than 5 percent passing the ASTM No. 200 sieve. Pea gravel should not be used for this capillary break material. The crushed rock layer should be densified and leveled with a vibratory equipment, and may be considered as the upper 4-inches of the non-expansive fill recommended above.

To reduce vapor transmission up through the at-grade concrete floor slabs/mat (to be constructed near the ground surface), the crushed rock section should be covered with a high quality, UV-resistant vapor barrier conforming to the requirements of ASTM E 1745 Class A, with a water vapor transmission rate less than or equal to 0.01 perms (such as 15-mil thick "Stego Wrap Class A"). The vapor barrier should be placed directly below the concrete slab. Sand above the vapor barrier is not recommended. Sand above the vapor barrier is not recommended. The vapor barrier should be installed in accordance with ASTM E 1643. All seams and penetrations of the vapor barrier should be sealed in accordance with manufacturer's recommendations.

The permeability of concrete is effected significantly by the water:cement ratio of the concrete mix, with lower water:cement ratios producing more damp-resistant slabs and higher strength concrete. Where moisture protection is important and/or where the concrete will be placed directly on the vapor barrier, the water:cement ratio should be 0.45 or less. To increase the workability of the concrete, mid-range plasticizers can be added to the mix. Water should not be added to the concrete mix unless the slump is less than specified and the water:cement ratio will not exceed 0.45. Other steps that may be taken to reduce moisture transmission through the concrete slabs-on-grade include moist curing for 5 to 7 days and allowing the slab to dry for a period of two months or longer prior to placing floor coverings. Also, prior to installation of the floor covering, it may be appropriate to test the slab moisture content for adherence to the manufacturer's requirements and to determine whether a longer drying time is necessary.

RETAINING WALLS

Retaining walls (if any) should be designed to resist lateral pressures from the adjacent native soil and backfill. Drained building retaining walls, or walls as part of the building, with level backfill that are not free to deflect or rotate should be designed to resist an equivalent fluid pressure of 50 pounds per cubic foot plus an additional uniform lateral pressure of 8H pounds per square foot (where H is the height of the wall in feet). If the building retaining walls are designed for undrained condition, they should be designed to resist an equivalent fluid pressure of 85 pounds per cubic foot plus an additional uniform lateral pressure of 8H pounds per square foot.

For site retaining walls (structurally separated from the proposed building) with level backfill that are free to deflect or rotate, the additional uniform lateral pressure of 8H pounds per square foot need not to be applied to the wall design.

Where the walls will be subjected to surcharge loads, such as from foundations, vehicular traffic, or construction loading, the walls should be designed for an additional uniform lateral pressure equal to one-half of the surcharge pressure.

Based on the site peak ground acceleration (PGA), on Seed and Whitman (1970); Al Atik and Sitar (2010); and Lew et al. (2010); seismic loads on building retaining walls that cannot yield may be simulated by a line load of $11H^2$ (in pounds per foot, where H is the wall height in feet). This seismic surcharge line load should be assumed to act at 1/3H above the base of the wall (in addition to the active wall design pressures of 50 and 85 pounds per cubic foot for drained and undrained conditions, respectively; the additional uniform lateral pressure of 8H psf need not to be applied for seismic condition).

For drained walls conditions, to prevent buildup of water pressure from surface water infiltration or ground water, a subsurface drainage system should be installed behind the basement walls. The drainage system may consist of a conventional gravel backdrain or an approved drainage fabric. If a gravel backdrain is used, a 4-inch diameter perforated pipe (perforations placed down) should be embedded in a section of 1/2- to 3/4-inch, clean, crushed rock at least 12 inches wide. Backfill above the perforated drain line should also consist of 1/2- to 3/4-inch, clean, crushed rock to within about 1½ to 2 feet below exterior finished grade. A filter fabric should be wrapped around the crushed rock to protect it from infiltration of native soil. The upper 1½ to 2 feet of backfill should consist of compacted native soil. The perforated pipe should discharge into a free-draining outlet or sump that pumps to a suitable location. Damp-proofing of the basement walls should be included in areas where wall dampness and efflorescence would be undesirable.

Miradrain, Enkadrain or other drainage fabrics approved by our office may be used for wall drainage as an alternative to the gravel drainage system described above. If used, the drainage fabric should extend from a depth of about 1 foot below the top of the wall down to the drain pipe at the base of the wall. A minimum 12-inch wide section of ½-inch to ¾-inch clean crushed rock and filter fabric should be placed around the drainpipe, as recommended previously.

Backfill placed behind the walls should be compacted in accordance to Table 6 "Compaction Recommendations" section of this report using light compaction equipment. If heavy equipment is used for compaction of wall backfill, the walls should be temporarily braced.

Building retaining walls may be supported on a spread footing or structural mat foundation designed in accordance with the recommendations presented previously.

VEHICLE PAVEMENTS

Asphalt Concrete Pavements

Based on the anticipated composition of the surface soils and an estimated traffic index for the proposed pavement loading conditions. We developed the minimum pavement sections presented in Table 4 below based on the Procedure 630 of Caltrans Highway Design Manual.

The Traffic Indices used in our pavement thickness calculations are considered reasonable values for this project and are based on engineering judgment rather than on a detailed traffic study. Asphalt concrete and aggregate base should conform to and be placed in accordance with the requirements of the California Department of Transportation, Standard Specifications, latest edition, except that compaction should be based on ASTM Test D1557.

Table 4. Minimum Asphalt Concrete Pavement Sections
Townhome Development
East Palo Alto, California

Traffic Loading Condition	Design Traffic Index	Asphalt Concrete (inches)	Aggregate Base* (inches)	Total Thickness (inches)
Automobile Access	4.5	3.0	8.0	11.0
Light Truck Traffic	5.0	3.0	11.0	14.0
Moderate Truck Traffic	6.0	4.0	12.0	16.0
Heavy Truck Traffic	7.0	4.0	16.0	20.0

*Caltrans Class 2 Aggregate Base (minimum R-value = 78).

We recommend that measures be taken to limit the amount of surface water that seeps into the aggregate base and subgrade below vehicle pavements, particularly where the pavements are adjacent to landscape areas. Collection of water in and below the pavement section has been shown to soften the subgrade, increasing the amount of pavement maintenance that is required, and shorten the pavement service life. Deepened curbs extending 4-inches below the bottom of the aggregate base layer are generally effective in limiting excessive water seepage. Other types of water cutoff devices or edge drains may also be considered to maintain pavement service life.

As discussed above, due to the expansive nature of the on-site soil, the pavement will likely be prone to differential settlement/movement and have a shorter service life than on a site with less expansive condition. Performance of the pavement could be improved by placing and compacting about 6 to 12 inches of additional non-expansive fill, or aggregate base below the minimum aggregate base thickness recommended above (i.e., to a total of 18 to 24 inches thick non-expansive fill). As discussed, pervious pavement is not desirable due to the presence of the near-surface expansive soil.

Rigid Concrete Pavements

The minimum thickness of the concrete pavements at the site should be based on the anticipated traffic loading, the modulus of rupture of the concrete used for pavement construction, and the composition and supporting characteristics of the subgrade below the pavement section. If rigid concrete pavement is planned for the proposed parking lot and driveway, the pavement section may be designed and constructed in accordance with ican Concrete Institute (ACI) 330R-08 – Guide for Design and Construction of Concrete Parking Lots.

Based on the variable clayey soils we encountered at the project site, a low subgrade-subbase support strength value of 100 pci was assumed in our analysis. In addition, our design assumes that pavements are restrained laterally by a concrete shoulder or curb, and the concrete should have a compressive strength, f 'c, of at least 4,000 psi and a flexural strength, M_R, of at least 500 psi. Reinforcing steel may be used for shrinkage crack control. In addition, maximum spacing should be provided between contraction joints in both directions. Our recommendations for minimum rigid pavement sections and maximum spacing between joints are presented in Table 5 on the following page. As discussed above, to help reduce the potential for differential movement/settlement, you should consider increasing the aggregate base thickness to about 18 to 24 inches.

Table 5. Rigid Concrete Pavement Design
Townhome Development
East Palo Alto, California

Traffic Categories	Maximum ADTT*	Concrete Thickness (inches)	Aggregate Base (inches)	Total Section (inches)	Maximum Spacing between Joints (feet)
Car Parking and Access Lanes	1	5.0	10.0	15.0	12.5
Truck Parking and Access Lanes	25	6.5	10.0	16.5	15.0
	300	7.0	10.0	17.0	15.0

*ADTT = Average daily truck traffic in both directions (excludes panel trucks, pickup trucks, and other four-wheel vehicles)

EARTHWORK

Clearing and Subgrade Preparation

All deleterious materials, such as existing foundations, slabs, pavements, utilities to be abandoned, vegetation, loose or soft soils, surface fills, root systems, topsoil, etc. should be cleared from areas to be built on or paved. The actual stripping depth to remove vegetation and organic topsoil should be determined by a member of our staff in the field at the time of construction. Excavations that extend below finish grade should be backfilled with structural fill that is water-conditioned, placed, and compacted as recommended in the section of this titled “Compaction.”

After the site has been properly cleared, stripped, and excavated to the required grades, exposed soil surfaces in areas to receive structural fill or slabs-on-grade, and pavements should be scarified to a depth of 6 inches, moisture conditioned, and compacted as recommended for structural fill in the section titled "Compaction."

On-site soils, foundation and utility trench excavations, exterior flatwork, pavement and slab subgrades should be kept in a moist condition throughout the construction period to help reduce the potential effects of of the expansive on-site soils on the proposed improvements.

Temporary Slopes and Excavations

The contractor should be responsible for the design and construction of all temporary slopes and any required shoring. Shoring and bracing should be provided in accordance with all applicable local, state, and federal safety regulations, including the current OSHA excavation and trench safety standards.

Due to the potential for variation of the on-site soils, field modification of temporary cut slopes may be required. Unstable materials encountered on excavations and slopes during and after excavation should be trimmed off even if this requires cutting the slopes back to a flatter inclination.

Please note that our scope or site visits do not (and will not) include reviewing the adequacy of the contractor's safety measures or stability of temporary cuts, and the contractor should be solely and completely responsible for the safety of the persons and properties at and near the excavations. In our experience, a preconstruction survey is generally performed to document existing conditions prior to construction, with intermittent monitoring of the structures during construction.

Material For Fill

All on-site soil containing less than 3 percent organic material by weight (ASTM D2974) may be suitable for use as structural fill (but not for non-expansive fill). Structural fill should not contain rocks or pieces larger than 6 inches in greatest dimension and no more than 15 percent larger than 2.5 inches. Imported, non-expansive fill should have a Plasticity Index no greater than 15, should be predominately granular, and should have sufficient binder so as not to slough or cave into foundation excavations or utility trenches. Recylced aggregate base should not be used for non-expansive fill at building interior. A member of our staff should approve proposed import materials prior to their delivery to the site.

Compaction

Scarified soil surfaces and all structural fill should be placed and compacted in uniform lifts no thicker than 8 inches in pre-compacted thickness, conditioned to the appropriate moisture content, and compacted as recommended for structural fill in Table 6 on the following page. The relative compaction and moisture content recommended in Table 6 is relative to ASTM Test D1557, latest edition.

Table 6. Compaction Recommendations
Townhome Development
East Palo Alto, California

General	Relative Compaction*	Moisture Content*
• Scarified subgrade in areas to receive structural fill.	87 to 92 percent	At least 3 percent above optimum
• Structural fill composed of on-site expansive soil.	87 to 92 percent	At least 3 percent above optimum
• Structural fill composed of non-expansive fill or low plasticity soil.	90 percent	Above optimum
• Structural fill below a depth of 5 feet.	92 percent	2 to 3 percent above optimum
Pavement Areas		
• On-site expansive soil	88 to 93 percent	At least 3 percent above optimum
• Aggregate baserock.	95 percent	Above optimum
Utility Trench Backfill		
• On-site expansive soil.	87 to 92 percent	At least 3 percent above optimum
• Imported sand	93 percent	Near optimum

* Relative to ASTM Test D1557, latest edition.

At the start of site grading and earthwork construction, and prior to subgrade preparation and placement of non-expansive fill, representative samples of on-site soil and import material will need to be collected in order for a laboratory compaction test to be performed for use during on-site density testing. Sampling of on-site soil and proposed import material should be requested by the contractor at least 5 days prior to when our staff will be needed for density testing to allow time for soil sampling and laboratory testing to be performed prior to our on-site compaction testing.

Finished Slopes

We recommend that finished slopes be cut or filled to an inclination preferably no steeper than 2:1 (horizontal:vertical). Exposed slopes may be subject to minor sloughing and erosion that may require periodic maintenance. We recommend that all slopes and soil surfaces disturbed during construction be planted with erosion-resistant vegetation.

Utility Trench Backfill

Utility trench excavations should follow in accordance with all applicable local, state and federal safety regulations, including the current OSHA excavation and trench safety standards. All trench backfill material should be moisture conditioned and compacted as recommended in the section of this report titled "Compaction". Utility penetrations through walls or footings should be properly sealed. Proper compaction of utility trenches below pavement areas is essential to help reduce future settlement and the resulting damage and maintenance costs of the pavement.

Surface Drainage

Finished grades should be designed to prevent ponding and to direct surface water runoff away from foundations, edges of slabs and pavements, and toward suitable collection and discharge facilities. Slopes of at least 2 percent are recommended for flatwork and pavement areas with 5 percent preferred in landscape areas within 8 feet of the structures, where possible. At a minimum, splash blocks should be provided at the ends of downspouts to carry surface water away from perimeter foundations. Preferably, roof downspout and concentrated drainage should be collected in a closed pipe drainage system that is routed to a storm drain system or other suitable discharge outlet.

Any drainage facilities/improvements should be observed to verify that they are adequate and that no adjustments need to be made, especially during first two years following construction. We recommend that an as-built plan showing the locations of all surface and subsurface drain lines and clean-outs be developed. The drainage facilities should be periodically checked to verify that they are continuing to function properly and will probably need to be periodically cleaned of silt and/or debris that may build up in the lines.

FUTURE SERVICES

Plan Review

Romig Engineers should review the completed grading and foundation plans for conformance with the recommendations presented in this report. We should be provided with these plans as soon as possible upon completion in order to limit the potential for delays in the permitting process that might otherwise be attributed to our review process. In addition, it should be noted that many of the local building and planning departments now require a “clean” geotechnical plan review letter prior to acceptance of plans for their final review. Since our plan reviews typically result in recommendations for modification of the plans, our generation of a “clean” review letter often requires two iterations.

At a minimum, we recommend that the following note be added to the plans:

"Earthwork, mat and/or slab subgrade and non-expansive fill preparation, foundation and slab construction, utility trench backfilling, pavement construction, retaining wall drainage installation and backfilling, and site drainage should be performed in accordance with the geotechnical report prepared by Romig Engineers, Inc., dated August 15, 2023. Romig Engineers should be notified at least 48 hours in advance of any earthwork or foundation construction and should observe and test during earthwork and foundation construction as recommended in the geotechnical report. Romig Engineers should be notified at least 5 days prior to earthwork, trench backfill and subgrade preparation work to allow time for sampling of on-site soil and laboratory compaction curve testing to be performed prior to on-site compaction density testing".

Construction Observation and Testing

The earthwork and foundation phases of construction should be observed and tested by us to 1) establish that subsurface conditions are compatible with those used in the analysis and design; 2) observe compliance with the design concepts, specifications and recommendations; and 3) allow design changes in the event that subsurface conditions differ from those anticipated. The recommendations in this report are based on a limited amount of subsurface exploration. The nature and extent of variation across the site may not become evident until construction. If variations are exposed during construction, it will be necessary to reevaluate our recommendations.

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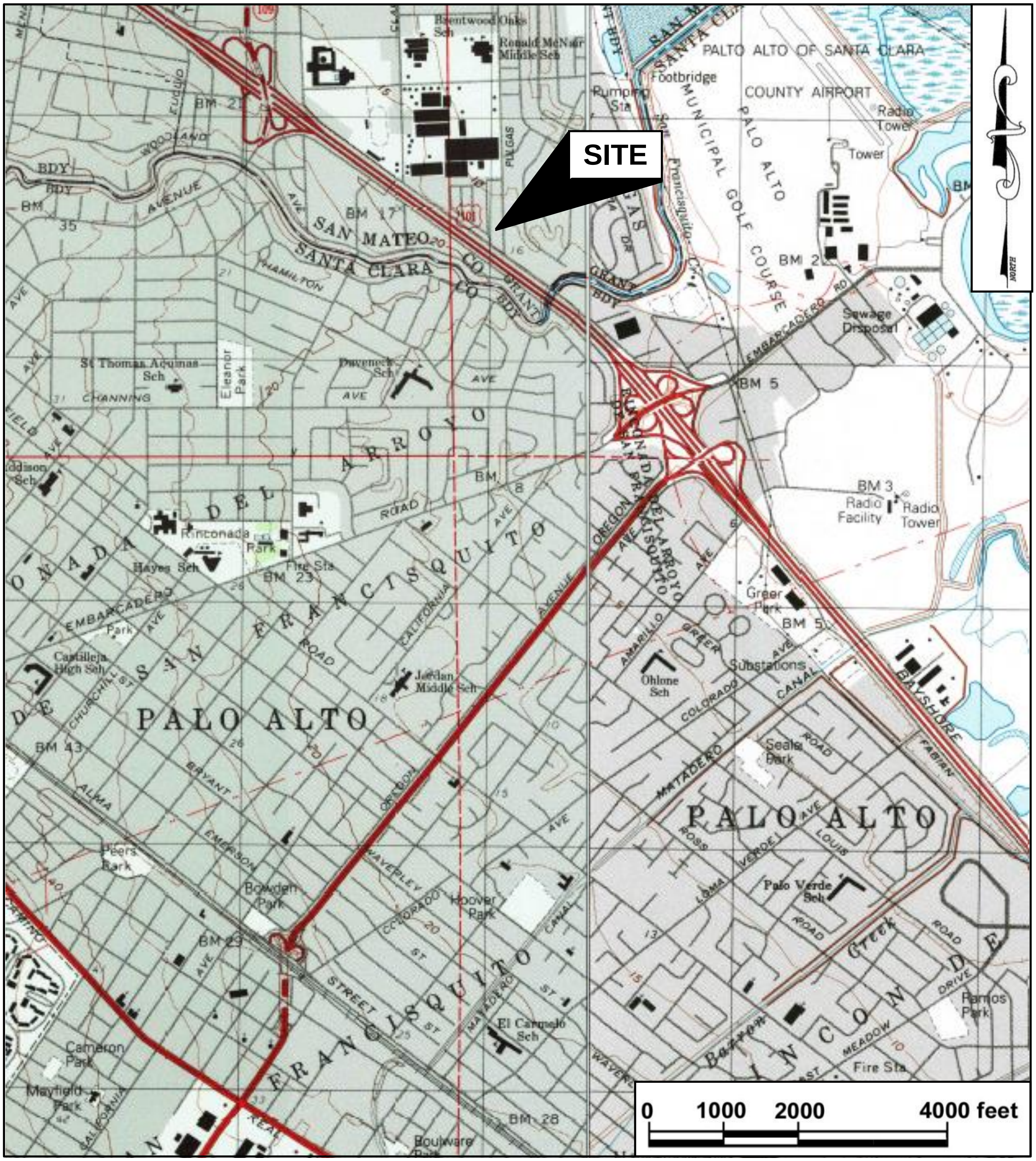


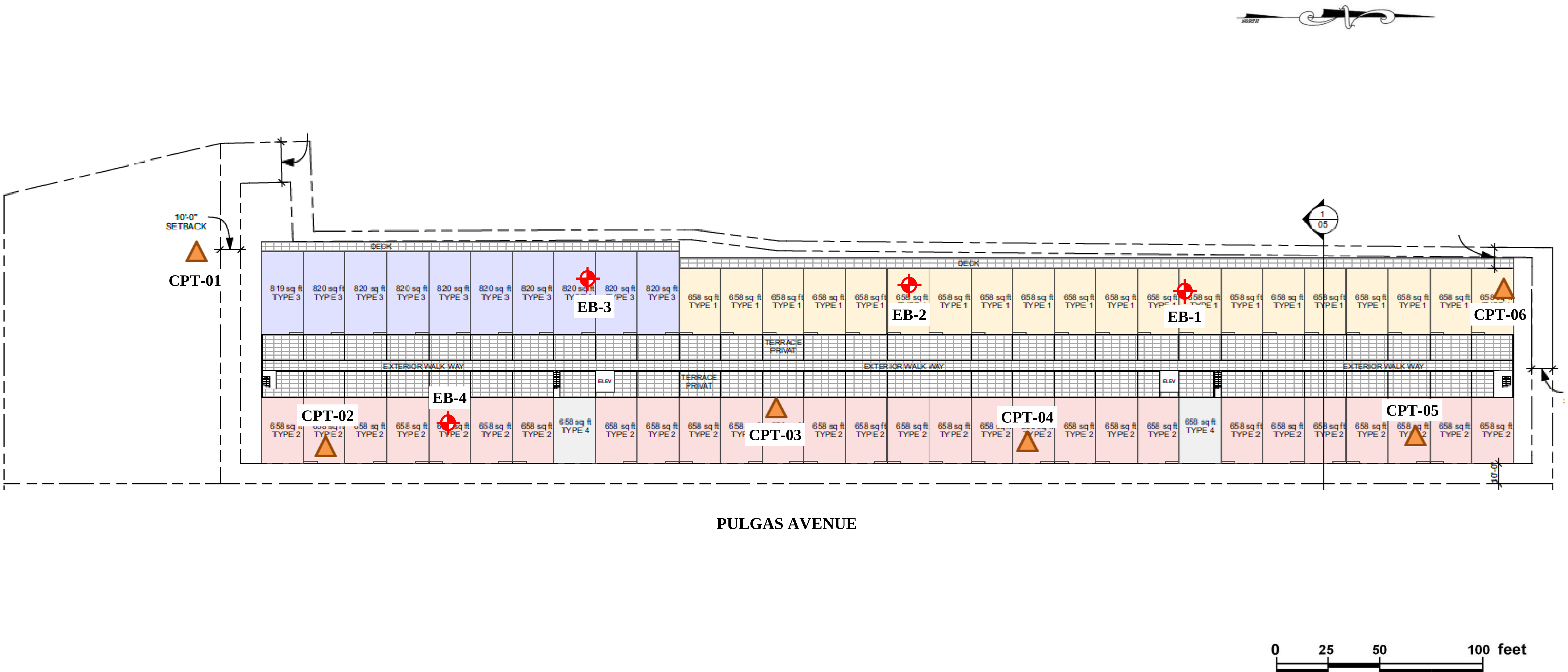
Scale: 1 inch = 2000 feet

Base is United States Geological Survey Palo Alto and Mountain View 7.5 Minute Quadrangle, dated 1997.

VICINITY MAP
XUE TOWNHOME DEVELOPMENT
EAST PALO ALTO, CALIFORNIA

FIGURE 1
AUGUST 2023
PROJECT NO. 6290-1

ROMIG ENGINEERS



LEGEND

EB-4 Approximate Location of Exploratory Borings.

CPT-06 Approximate Location of CPT Borings.

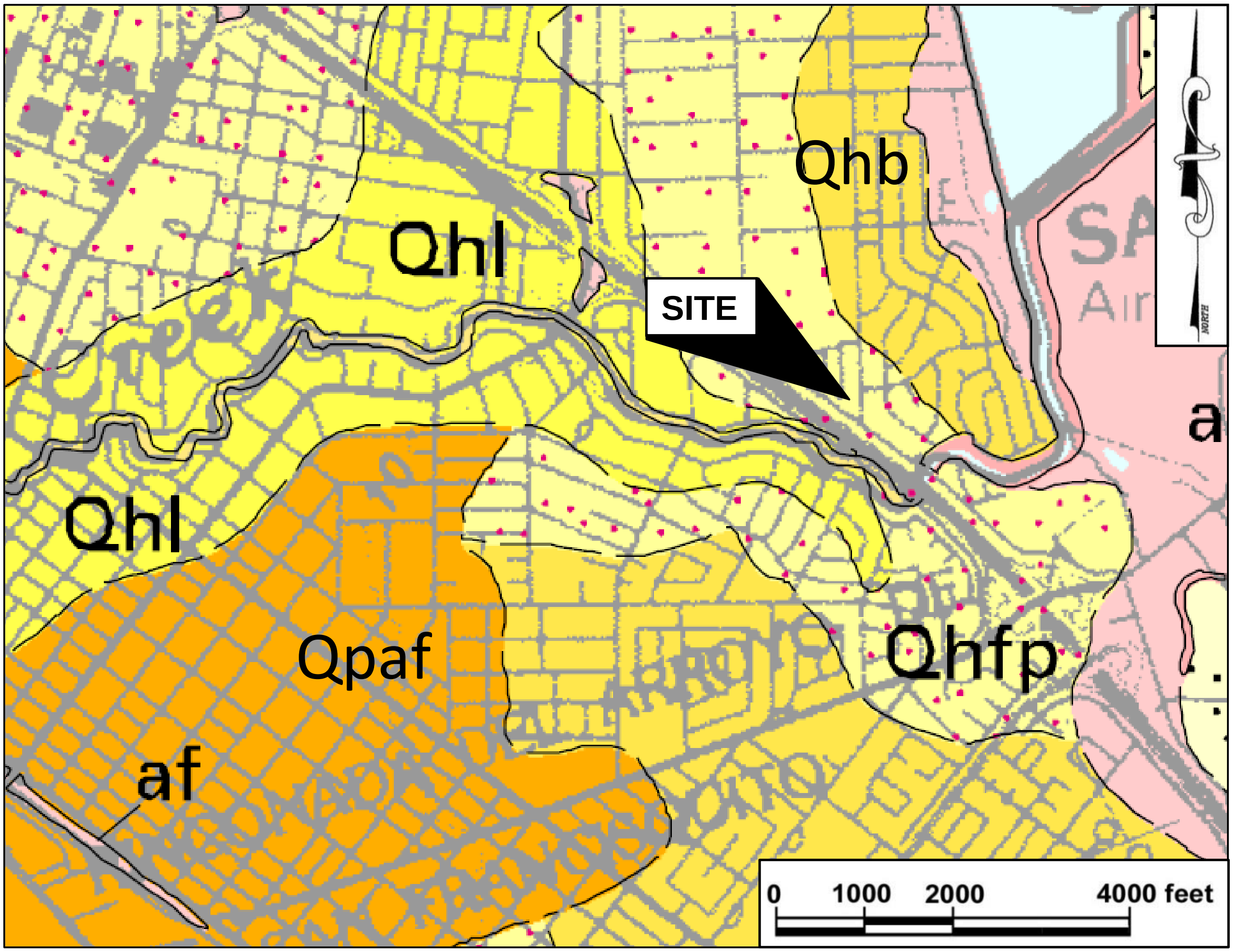
Approximate Scale: 1 inch = 50 feet.

Base is site plan prepared by RG-Architecture, dated July 14, 2023.

SITE PLAN
XUE TOWNHOME DEVELOPMENT
EAST PALO ALTO, CALIFORNIA

ROMIG ENGINEERS

FIGURE 2
AUGUST 2023
PROJECT NO. 6290-1



MAP LEGEND

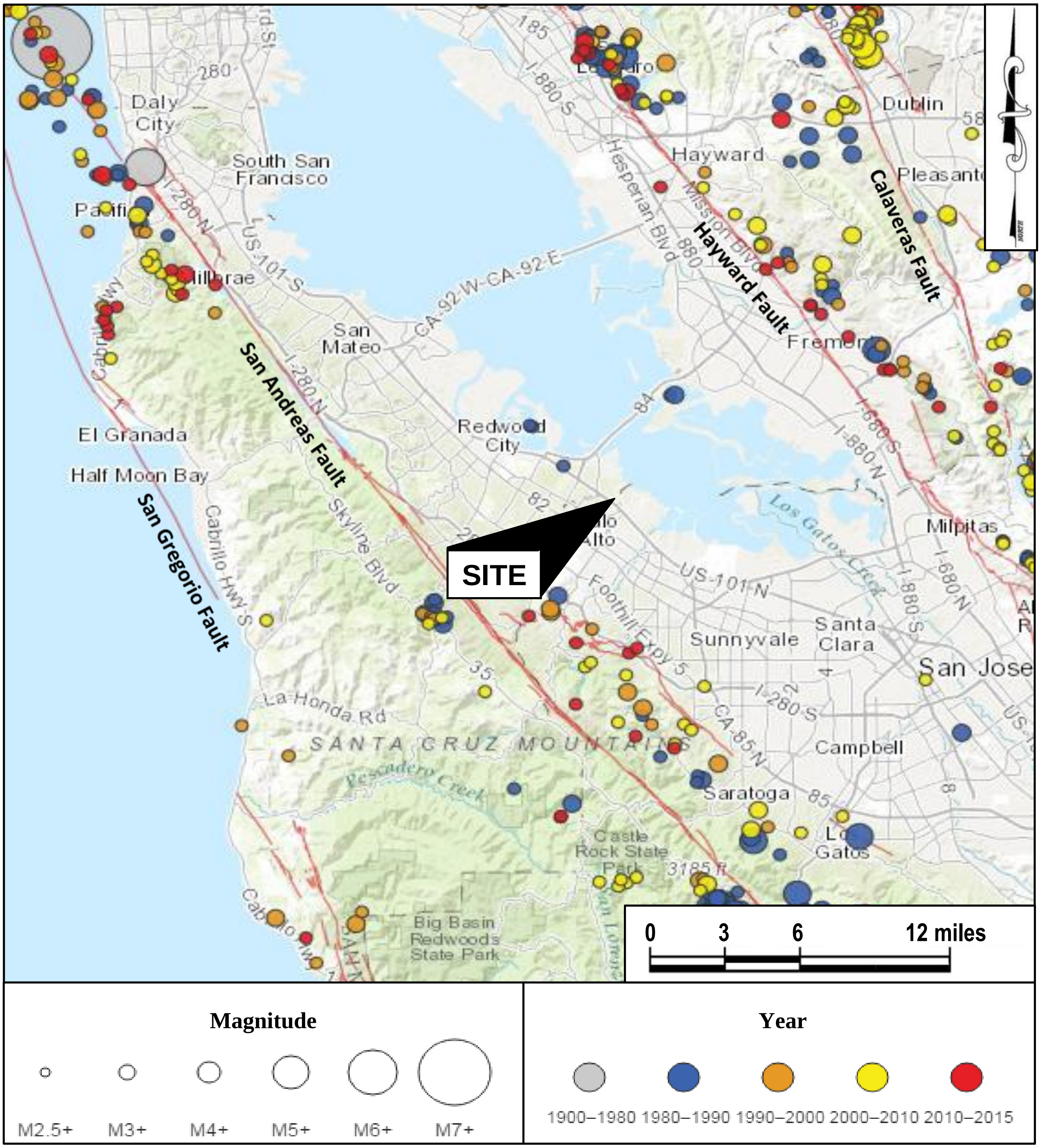
af	Artificial Fill	Qhfp	Floodplain Deposits
alf	Artificial Levee Fill	Qhl	Natural Levee Deposits
Qhb	Basin Deposits	Qpaf	Alluvial Fan and Fluvial Deposits
			Contact - dashed where approximate, dotted where inferred.

Scale: 1 inch = 2000 feet
Base is Geologic Map of Palo Alto 30 x 60 Minute Quadrangle (Brabb, Graymer, and Jones, 2000).

VICINITY GEOLOGIC MAP
XUE TOWNHOME DEVELOPMENT
EAST PALO ALTO, CALIFORNIA

FIGURE 3
AUGUST 2023
PROJECT NO. 6290-1

ROMIG ENGINEERS



Earthquakes with M5+ from 1900 to 1980, M2.5+ from 1980 to January 2015. Faults with activity in last 15,000 years. Based on data sources from Northern California Earthquake Data Center and USGS Quaternary Fault and Fold Database, accessed May 2015.

REGIONAL FAULT AND SEISMICITY MAP
XUE TOWNHOME DEVELOPMENT
EAST PALO ALTO, CALIFORNIA

FIGURE 4
AUGUST 2023
PROJECT NO. 6290-1

ROMIG ENGINEERS

APPENDIX A

FIELD INVESTIGATION

The soils encountered during drilling were logged by our representative and samples were obtained at depths appropriate to the investigation. The samples were taken to our laboratory where they were examined and classified in accordance with the Unified Soil Classification System. The logs of our borings, as well as a summary of the soil classification system (Figure A-1) used on the boring logs, are attached.

Several tests were performed in the field during drilling. The standard penetration test resistance was determined by dropping a 140-pound hammer through a 30-inch free fall, and recording the blows required to drive the 2-inch (outside diameter) sampler 18 inches. The standard penetration test (SPT) resistance is the number of blows required to drive the sampler the last 12 inches, and is recorded on the boring logs at the appropriate depths. The results of these field tests are also presented on the boring logs. Soil samples were also collected using a 2.5-inch and 3.0-inch O.D. drive samplers. The blow counts shown on the log for these samplers do not represent SPT values and have not been corrected in any way.

The Cone Penetration Tests (CPT) probes for this project were carried out by Middle Earth Geo Testing, Inc. using an integrated electronic cone system. The CPT soundings were performed in accordance with ASTM standards (D 5778-95). A 20-ton capacity cone was used for all of the soundings. The cone had a tip area of 15 cm^2 and friction sleeve area of 225 cm^2. The logs of our CPTs are attached in this Appendix.

The locations of the borings and CPTs were established by pacing using the site plan prepared by RG-Architecture, dated May 5, 2023. The locations of the borings and CPTs should be considered accurate only to the degree implied by the method used.

The boring logs, CPT logs, and related information depict our interpretation of subsurface conditions only at the specific location and time indicated. Subsurface conditions and ground water levels at other locations may differ from conditions at the location where sampling and testing were conducted. The passage of time may also result in changes in the subsurface conditions.

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USCS SOIL CLASSIFICATION

PRIMARY DIVISIONS			SOIL TYPE	SECONDARY DIVISIONS
COARSE GRAINED SOILS (< 50 % Fines)	GRAVEL	CLEAN GRAVEL (< 5% Fines)	GW	Well graded gravel, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravel or gravel-sand mixtures, little or no fines.
		GRAVEL with FINES	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines.
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines.
	SAND	CLEAN SAND (< 5% Fines)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SAND WITH FINES	SM	Silty sands, sand-silt mixtures, non-plastic fines.
			SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS (> 50 % Fines)	SILT AND CLAY Liquid limit < 50%		ML	Inorganic silts and very fine sands, with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, lean clays.
			OL	Organic silts and organic clays of low plasticity.
	SILT AND CLAY Liquid limit > 50%		MH	Inorganic silt, micaceous or diatomaceous fine sandy or silty soil.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.
BEDROCK			BR	Weathered bedrock.

RELATIVE DENSITY

SAND & GRAVEL	BLOWS/FOOT*
VERY LOOSE	0 to 4
LOOSE	4 to 10
MEDIUM DENSE	10 to 30
DENSE	30 to 50
VERY DENSE	OVER 50

CONSISTENCY

SILT & CLAY	STRENGTH^	BLOWS/FOOT*
VERY SOFT	0 to 0.25	0 to 2
SOFT	0.25 to 0.5	2 to 4
FIRM	0.5 to 1	4 to 8
STIFF	1 to 2	8 to 16
VERY STIFF	2 to 4	16 to 32
HARD	OVER 4	OVER 32

GRAIN SIZES

BOULDERS	COBBLES	GRAVEL		SAND			SILT & CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE	

12 " 3" 0.75" 4 10 40 200

SIEVE OPENINGS U.S. STANDARD SERIES SIEVE

Classification is based on the Unified Soil Classification System; fines refer to soil passing a No. 200 sieve.

* Standard Penetration Test (SPT) resistance, using a 140 pound hammer falling 30 inches on a 2 inch O.D. split spoon sampler; blow counts not corrected for larger diameter samplers.

^ Unconfined Compressive strength in tons/sq. ft. as estimated by SPT resistance, field and laboratory tests, and/or visual observation.

KEY TO SAMPLERS

Modified California Sampler (3-inch O.D.)

Mid-size Sampler (2.5-inch O.D.)

Standard Penetration Test Sampler (2-inch O.D.)

KEY TO EXPLORATORY BORING LOGS
XUE TOWNHOME DEVELOPMENT
EAST PALO ALTO, CALIFORNIA

FIGURE A-1
AUGUST 2023
PROJECT NO. 6290-1

ROMIG ENGINEERS

DRILL TYPE: Minuteman with 3-1/4" Continuous Flight Auger **LOGGED BY:** KR

DEPTH TO GROUND WATER: 7 ft. **SURFACE ELEVATION:** NA **DATE DRILLED:** 6/21/23

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Fill: Light brown, Sandy Lean Clay, moist, fine to medium grained sand, low plasticity, some roots.	Stiff	CL		0		12	19		
Dark brown to brownish gray, Fat Clay, very moist to wet, fine to medium grained sand, high plasticity, gray and orange mottling. ■ Liquid Limit = 57, Plasticity Index = 33.	Very Stiff	CH				20	28		3.0
				5	■	27	28		2.5
▼ Ground water encountered during drilling at 7 feet.					▼	32	28		2.5
Olive to tan, Sandy Lean Clay, very moist to wet, fine to medium grained sand, low to high plasticity, gray, orange, and olive mottling, some caliche.	Firm to Very Stiff	CL				5	28		0.5
				10		16	20		4.0
Increase in moisture content at 14 feet.						17	29		3.5
				15		20	33		1.5
Note: The stratification lines represent the approximate boundary between soil and rock types, the actual transition may be gradual.						12	26		1.8
*Measured using Torvane and Pocket Penetrometer devices.				20		18	30		
Bottom of Boring at 20 feet.									

EXPLORATORY BORING LOG EB-1
XUE TOWNHOME DEVELOPMENT
EAST PALO ALTO, CALIFORNIA

BORING EB-1
AUGUST 2023
PROJECT NO. 6290-1

ROMIG ENGINEERS

DRILL TYPE: Minuteman with 3-1/4" Continuous Flight Auger **LOGGED BY:** KR

DEPTH TO GROUND WATER: 8 ft. **SURFACE ELEVATION:** NA **DATE DRILLED:** 6/21/23

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	DEPTH (FEET)	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Fill: Brown, Sandy Lean Clay, moist, fine to coarse grained sand, fine to coarse sub-angular to sub-rounded gravel, low plasticity, concrete fragments.	Hard	CL	0	41	12		
Brown to dark brown, Sandy Fat Clay, moist to very moist, fine to medium grained sand, high plasticity. Note: The stratification lines represent the approximate boundary between soil and rock types, the actual transition may be gradual. *Measured using Torvane and Pocket Penetrometer devices. ▼ Ground water measured at 8 feet after drilling.	Very Stiff	CH	5 ▼ 10	18 19 16 28	22 31 27 23		2.0 2.3 1.8 2.3
Tan to gray, Sandy Lean Clay, very moist to wet, fine to coarse grained sand, trace fine sub-rounded gravel, moderate plasticity, gray, olive, and orange mottling.	Very Stiff	CL		21	22		1.5
Tan to gray, Clayey Sand with gravel, wet, fine to coarse grained sand, fine to coarse sub-angular to sub-rounded gravel, low plasticity fines. ● 31% Passing No. 200 Sieve.	Medium Dense	SC	15	12 12	20 19		
Brown, Poorly-Graded Sand to Clayey Sand, moist, fne to coarse grained sand, fine to coarse sub-angular to sub-rounded gravel. ● 11% Passing No. 200 Sieve.	Medium Dense to Dense	SP/ SC	20	23 27	12 13		

Continued on Next Page.

DRILL TYPE: Minuteman with 3-1/4" Continuous Flight Auger **LOGGED BY:** KR

DEPTH TO GROUND WATER: 8 ft. **SURFACE ELEVATION:** NA **DATE DRILLED:** 6/21/23

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS∗ (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
	Medium Dense to Dense	SP/ SC		20					
● 17% Passing No. 200 Sieve.					●	18	12		
							13		
						31	17		
Transition to sandy lean clay at 23.5 feet.									
Bottom of Boring at 24 feet.				25					
				30					
				35					
				40					

EXPLORATORY BORING LOG EB-2
XUE TOWNHOME DEVELOPMENT
EAST PALO ALTO, CALIFORNIA

BORING EB-2
AUGUST 2023
PROJECT NO. 6290-1

ROMIG ENGINEERS

DRILL TYPE: Minuteman with 3-1/4" Continuous Flight Auger **LOGGED BY:** KR

DEPTH TO GROUND WATER: 8 ft. **SURFACE ELEVATION:** NA **DATE DRILLED:** 6/21/23

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Brown, Sandy Lean Clay, moist to very moist, fine to medium grained sand, low to moderate plasticity, roots at upper 2 feet.	Very Stiff	CL		0		22	16		4.0
■ Liquid Limit = 35, Plasticity Index = 14.					■	17	15		3.0
Dark brown to grayish brown, Fat Clay, very moist to wet, fine to medium grained sand, high plasticity, orange mottling, trace fine gravel.	Stiff to Very Stiff	CH		5		19	33		3.0
▼ Ground water encountered during drilling at 8 feet.					▼	18	37		0.5
				10		13	26		2.3
						26	25		2.3
Tan to gray, Sandy Lean Clay, very moist to wet, fine to coarse grained sand, fine to coarse sub-angular to sub-rounded gravel, low to moderate plasticity.	Firm to Very Stiff	CL				34	20		3.3
Interbedded clayey sand at 14 to 16 feet.				15					
● 33% Passing No. 200 Sieve.					●	7	20		0.5
Note: The stratification lines represent the approximate boundary between soil and rock types, the actual transition may be gradual.						15	22		3.0
*Measured using Torvane and Pocket Penetrometer devices.				20		18	22		3.3
Bottom of Boring at 20 feet.									

EXPLORATORY BORING LOG EB-3
XUE TOWNHOME DEVELOPMENT
EAST PALO ALTO, CALIFORNIA

BORING EB-3
AUGUST 2023
PROJECT NO. 6290-1

ROMIG ENGINEERS

DRILL TYPE: Minuteman with 3-1/4" Continuous Flight Auger **LOGGED BY: KR**

DEPTH TO GROUND WATER: 8 ft. **SURFACE ELEVATION:** NA **DATE DRILLED:** 6/21/23

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Fill: Brown, Sandy Lean Clay, moist, fine to coarse grained sand, trace fine to coarse gravel, low plasticity, brick fragments.	Very Stiff	CL		0		19	14		3.5
Dark brown to grayish brown, Fat Clay, very moist to wet, fine grained sand, high plasticity, brown and orange mottling.	Stiff to Very Stiff	CH				9	29		2.8
				5		14	26		1.5
						21	36		1.3
▼ Ground water encountered during drilling at 8 feet.									
				10		10	28		2.0
							23		
Olive to gray, Sandy Lean Clay, very moist to wet, fine to medium grained sand, low to moderate plasticity, orange and white mottling, caliche.	Stiff to Hard	CL				30	21		2.5
Note: The stratification lines represent the approximate boundary between soil and rock types, the actual transition may be gradual.						40	20		3.8
*Measured using Torvane and Pocket Penetrometer devices.				15		15	25		2.3
Increase in moisture content at 16 feet.						16	30		1.3
Interbedded clayey sand and gravel.				20		15	14		1.5
Bottom of Boring at 20 feet.									

EXPLORATORY BORING LOG EB-4
XUE TOWNHOME DEVELOPMENT
EAST PALO ALTO, CALIFORNIA

BORING EB-4
AUGUST 2023
PROJECT NO. 6290-1

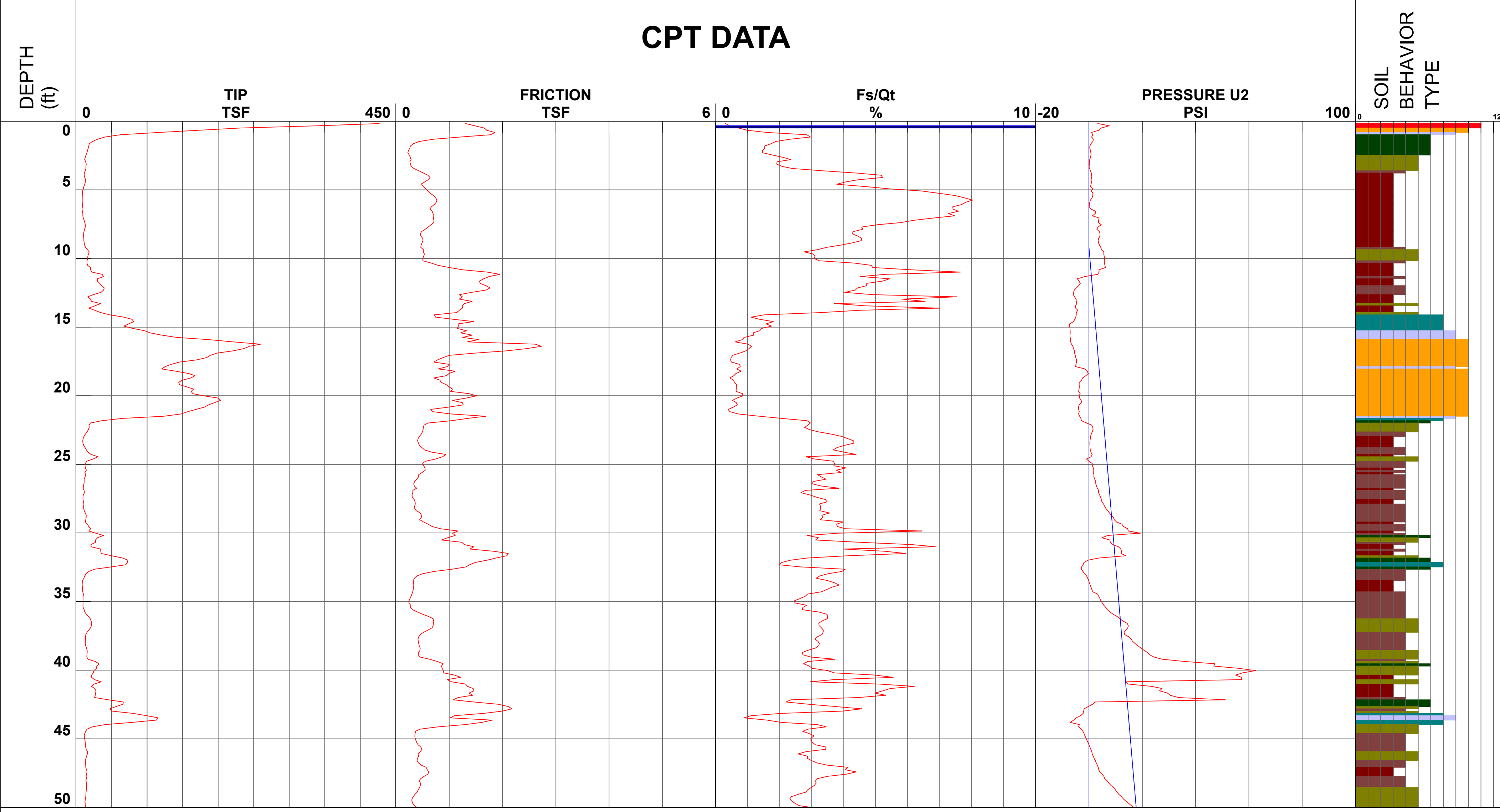
ROMIG ENGINEERS

Romig Engineers

Project	Xue Townhomes (1933 Pulgas Avenue)	Operator	JM-GM	Filename	SDF(119).cpt
Job Number	6290-1	Cone Number	DDG1596	GPS	
Hole Number	CPT-01	Date and Time	6290-1	Maximum Depth	50.69 ft
EST GW Depth During Test		9.20 ft			

Net Area Ratio .8

CPT DATA



- 1 - sensitive fine grained
- 2 - organic material
- 3 - clay
- 4 - silty clay to clay
- 5 - clayey silt to silty clay
- 6 - sandy silt to clayey silt
- 7 - silty sand to sandy silt
- 8 - sand to silty sand
- 9 - sand
- 10 - gravelly sand to sand
- 11 - very stiff fine grained (*)
- 12 - sand to clayey sand (*)

Cone Size 15cm squared

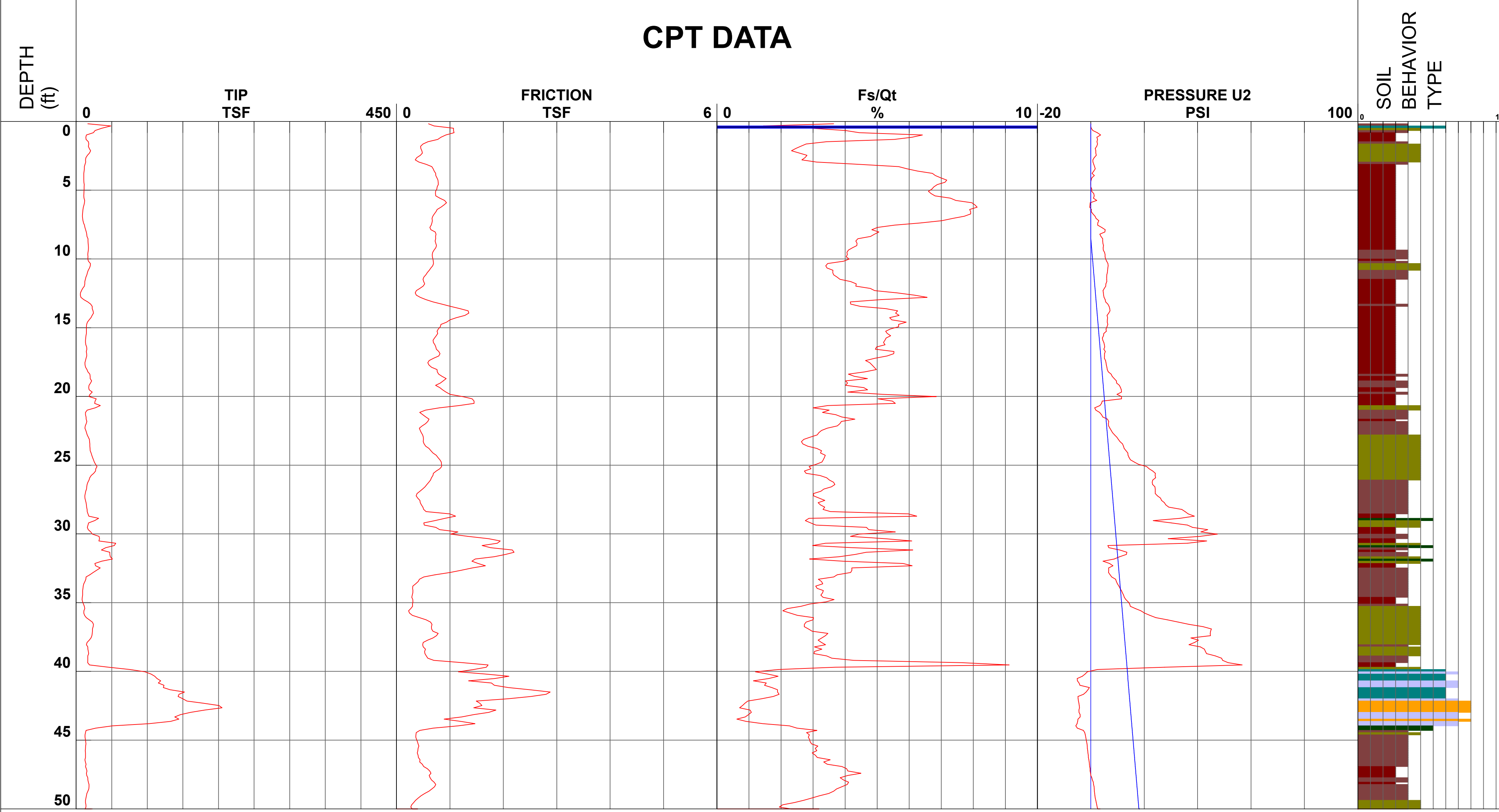
S*Soil behavior type and SPT based on data from UBC-1983

Romig Engineers

Middle Earth Geo Testing Inc.

Project	Xue Townhomes (1933 Pulgas Avenue)	Operator	JM-GM	Filename	SDF(120).cpt
Job Number	6290-1	Cone Number	DDG1596	GPS	
Hole Number	CPT-02	Date and Time	6290-1	Maximum Depth	50.69 ft
EST GW Depth During Test		8.50 ft			

Net Area Ratio .8



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 15cm squared

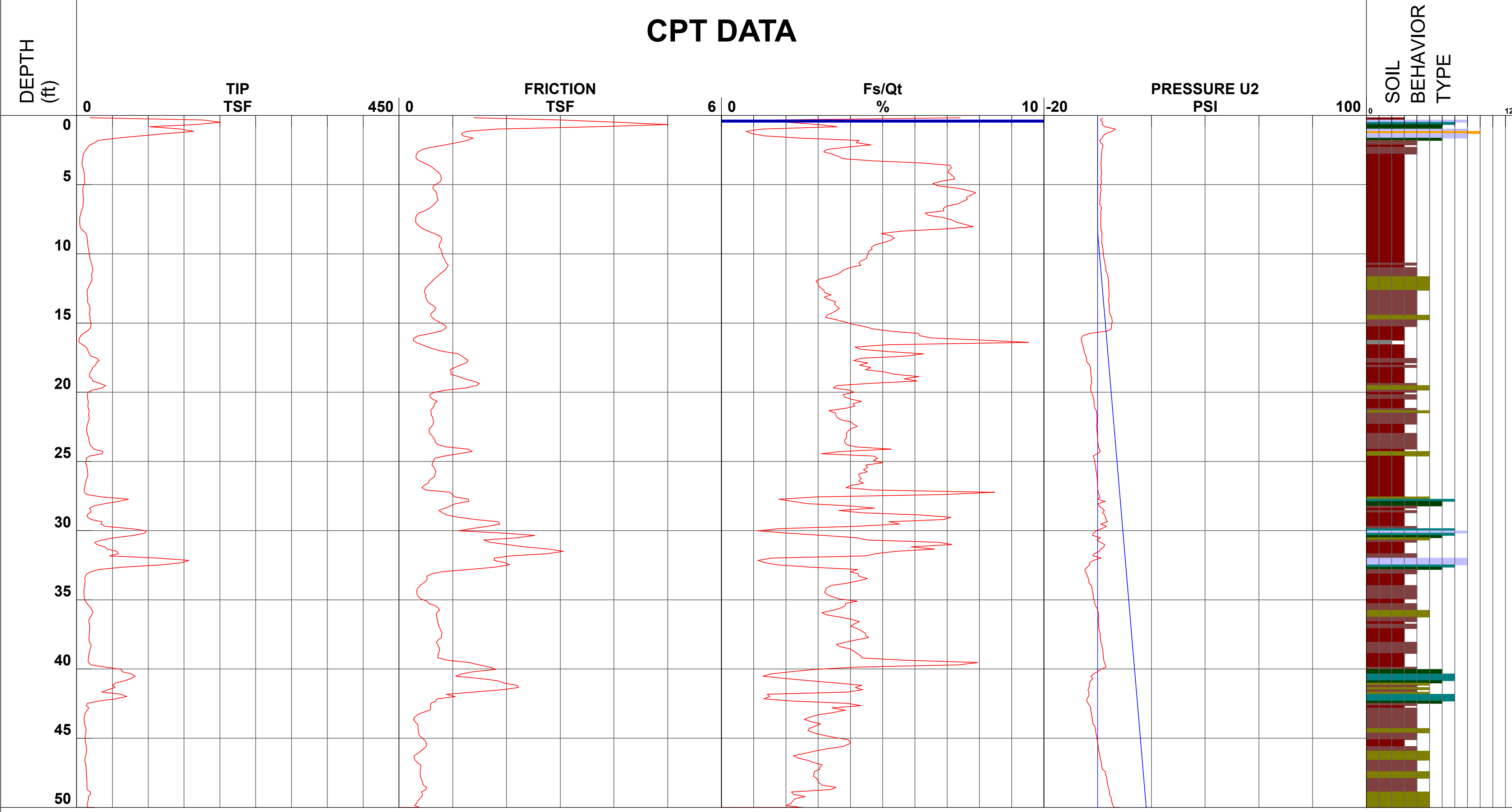
S*Soil behavior type and SPT based on data from UBC-1983

Romig Engineers

Project	Xue Townhomes (1933 Pulgas Avenue)	Operator	JM-GM	Filename	SDF(121).cpt
Job Number	6290-1	Cone Number	DDG1596	GPS	
Hole Number	CPT-03	Date and Time	6290-1	Maximum Depth	50.52 ft
EST GW Depth During Test		8.40 ft			

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

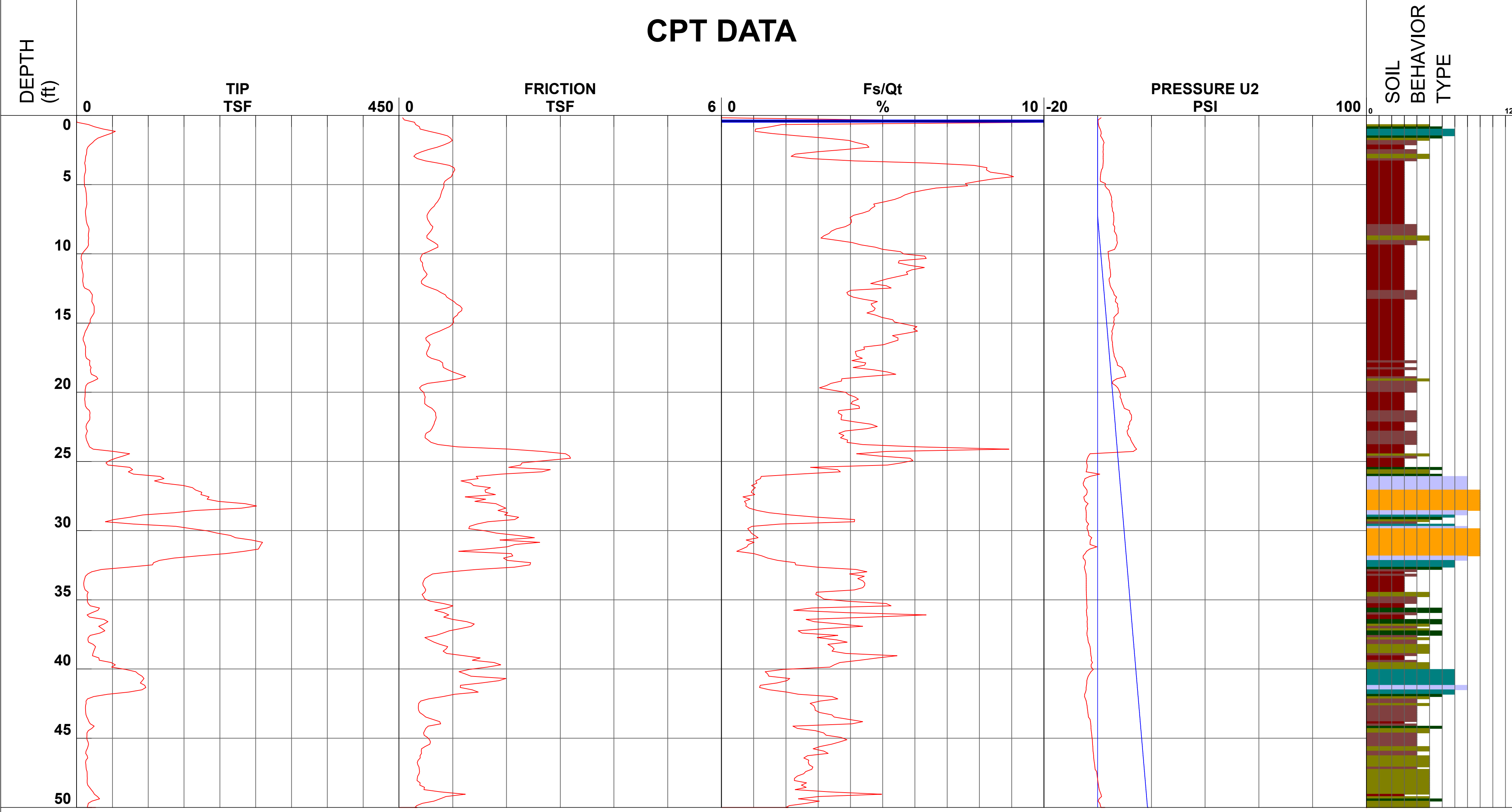
Cone Size 15cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Romig Engineers

Project	Xue Townhomes (1933 Pulgas Avenue)	Operator	JM-GM	Filename	SDF(122).cpt
Job Number	6290-1	Cone Number	DDG1596	GPS	
Hole Number	CPT-04	Date and Time	6290-1	Maximum Depth	50.69 ft
EST GW Depth During Test		7.30 ft			

Net Area Ratio .8



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 15cm squared

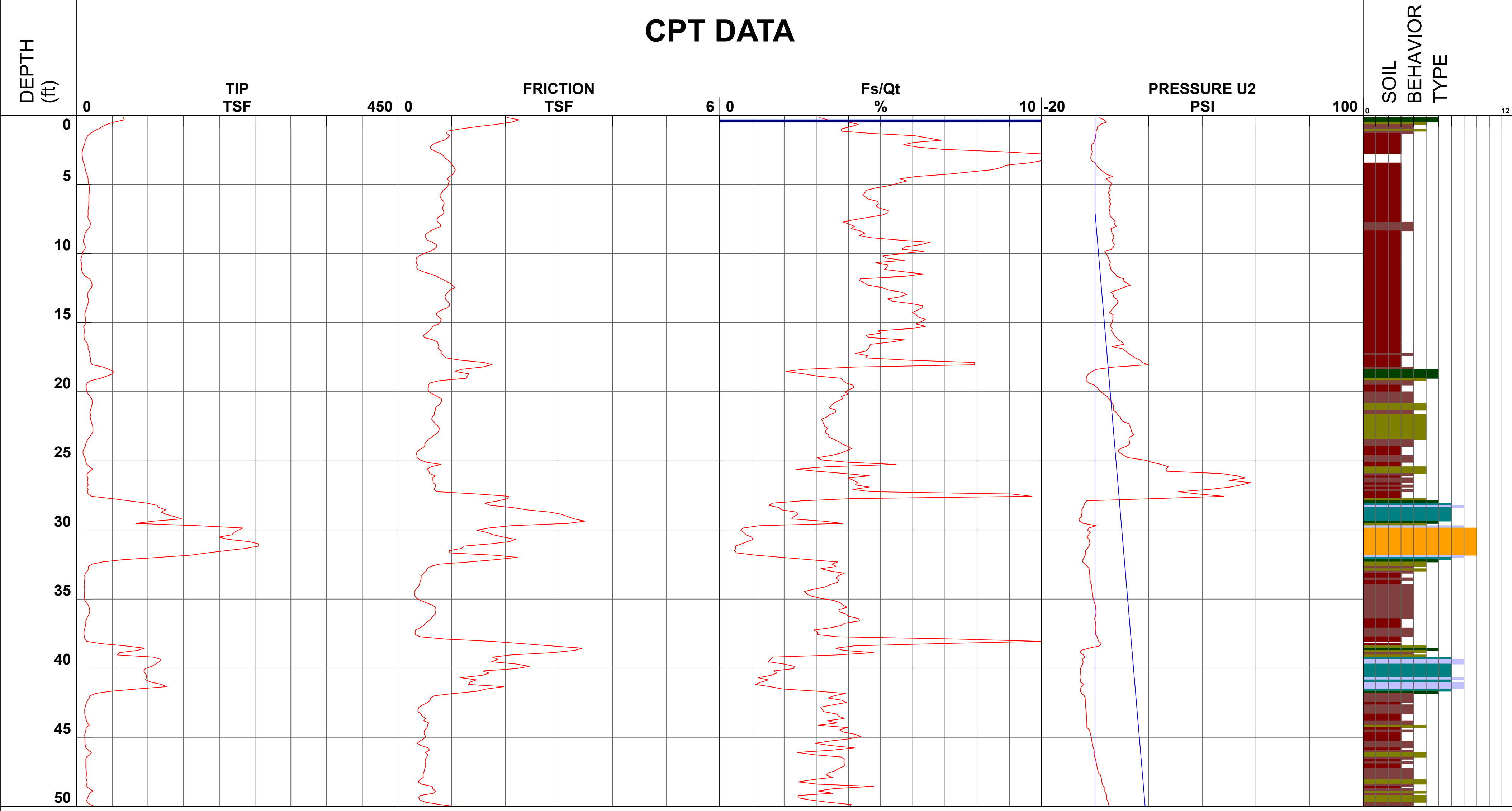
S*Soil behavior type and SPT based on data from UBC-1983

Romig Engineers

Project	Xue Townhomes (1933 Pulgas Avenue)	Operator	JM-GM	Filename	SDF(118).cpt
Job Number	6290-1	Cone Number	DDG1596	GPS	
Hole Number	CPT-05	Date and Time	6290-1	Maximum Depth	50.69 ft
EST GW Depth During Test		7.00 ft			

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 15cm squared

S*Soil behavior type and SPT based on data from UBC-1983

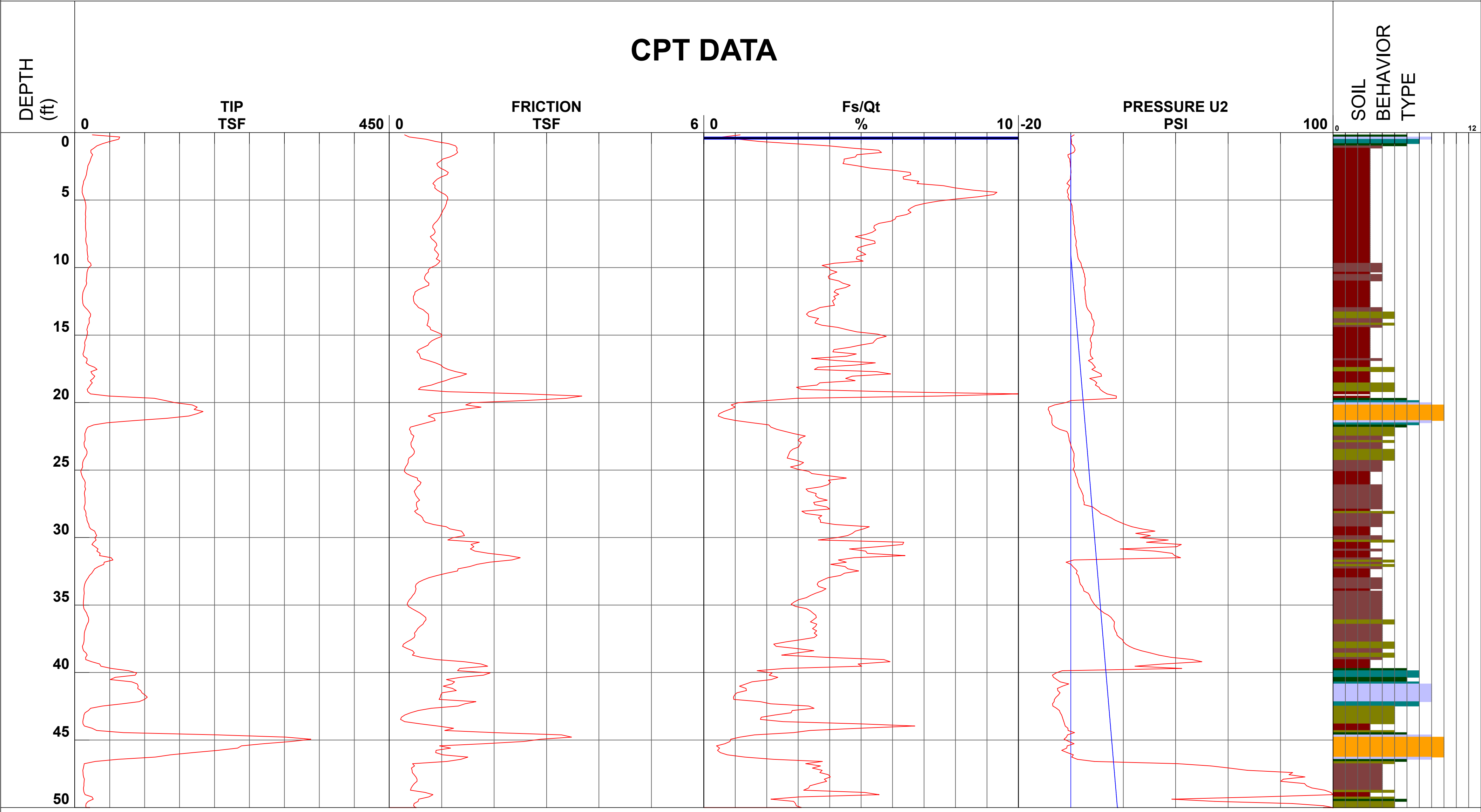
Romig Engineers

Project	Xue Townhomes (1933 Pulgas Avenue)	Operator	JM-GM
Job Number	6290-1	Cone Number	DDG1596
Hole Number	CPT-06	Date and Time	6290-1
EST GW Depth During Test		9.10 ft	

Filename	SDF(117).cpt
GPS	
Maximum Depth	50.69 ft

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 15cm squared

S*Soil behavior type and SPT based on data from UBC-1983

APPENDIX B

LABORATORY TESTS

Samples from the subsurface exploration were selected for tests to help evaluate the physical and engineering properties of the soils. The tests performed are briefly described below.

The natural moisture content was determined in accordance with ASTM D 2216 on nearly all of the samples recovered from the borings. This test determines the moisture content, representative of field conditions, at the time the samples were collected. The results are presented on the boring logs at the appropriate sample depths.

The Atterberg Limits were determined on two samples of soil in accordance with ASTM D4318. The Atterberg Limits are the moisture content within which the soil is workable or plastic. The results of these tests are presented in Figure B-1 and on the log of Borings EB-1 and EB-3 at the appropriate sample depths.

The amount of silt and clay-sized material present was determined on four samples of soil in accordance with ASTM D422. The results are presented on the boring logs at the appropriate sample depths.

❖ ❖ ❖ ❖ ❖

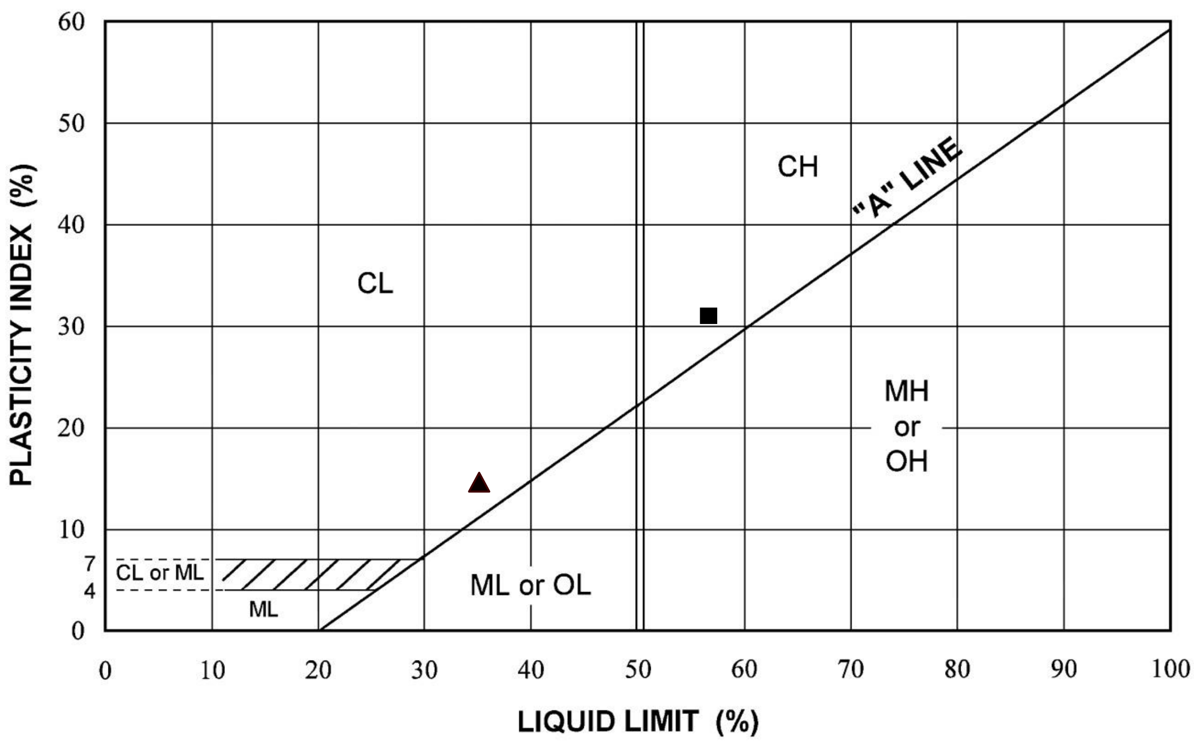


Chart Symbol	Boring Number	Sample Depth (feet)	Water Content (percent)	Liquid Limit (percent)	Plasticity Index (percent)	Liquidity Index (percent)	Passing No. 200 Sieve (percent)	USCS Soil Classification
■	EB-1	4-6	28	57	31	6		CH
▲	EB-3	2-4	15	35	14	-43		CL

PLASTICITY CHART
XUE TOWNHOME DEVELOPMENT
EAST PALO ALTO, CALIFORNIA

FIGURE B-1
AUGUST 2023
PROJECT NO. 6290-1

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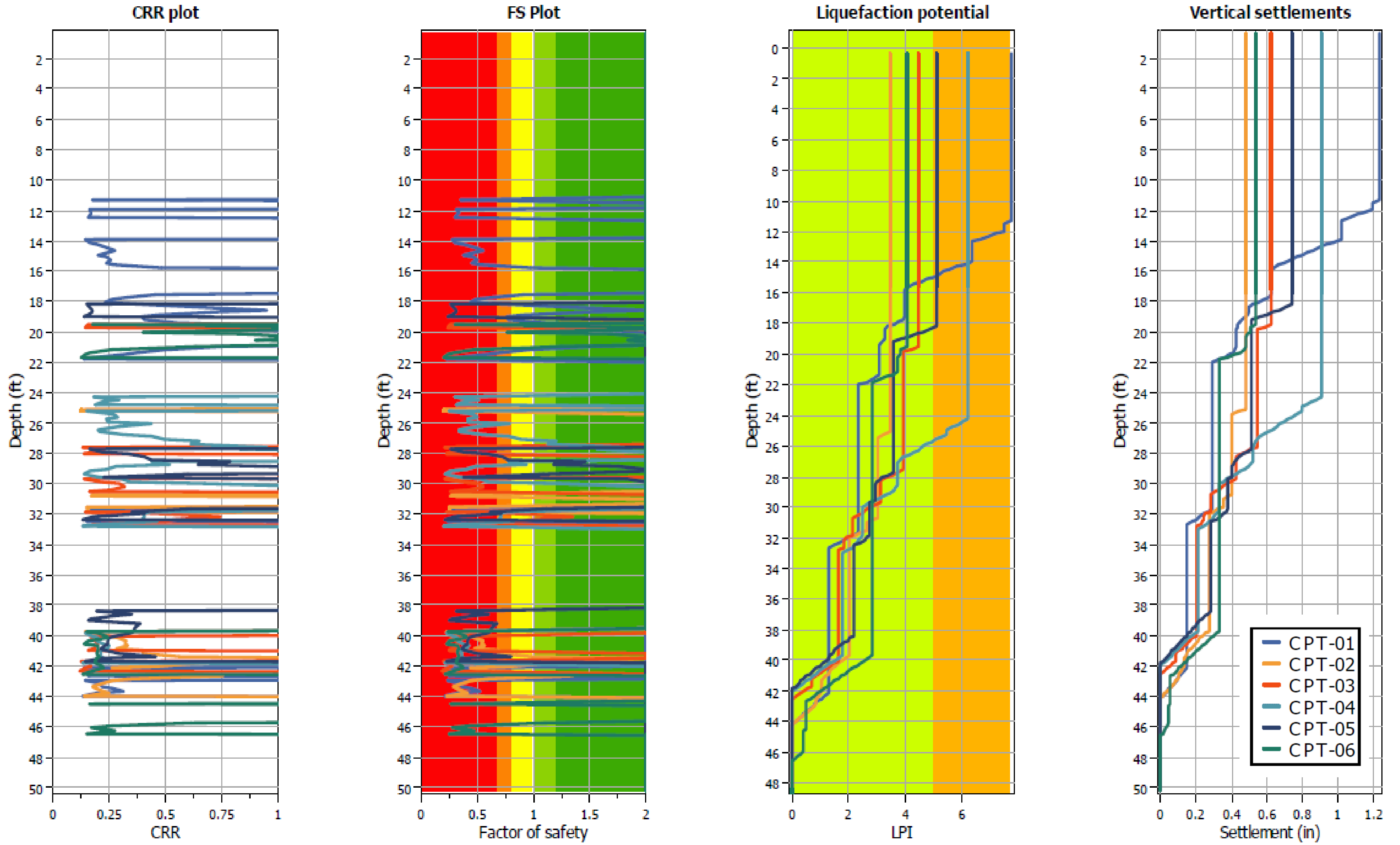
APPENDIX C

LIQUEFACTION ANALYSES

To evaluate the potential for earthquake-induced liquefaction of the soils at the site, we performed a liquefaction analysis of the CPT data using the program CLiq, developed by GeoLogismiki. The program applied several published methodologies, including Idriss and Boulanger (2014) and Robertson (2009). The results of our liquefaction evaluation and the details regarding the potentially liquefiable layers are presented on the attached Figures C-1 and C-2.

❖ ❖ ❖ ❖ ❖

LIQUEFACTION ANALYSIS (IDRISS AND BOULANGER, 2014)



LIQUEFACTION ANALYSIS (IDRISS AND BOULANGER, 2014)
XUE TOWNHOME DEVELOPMENT
EAST PALO ALTO, CALIFORNIA

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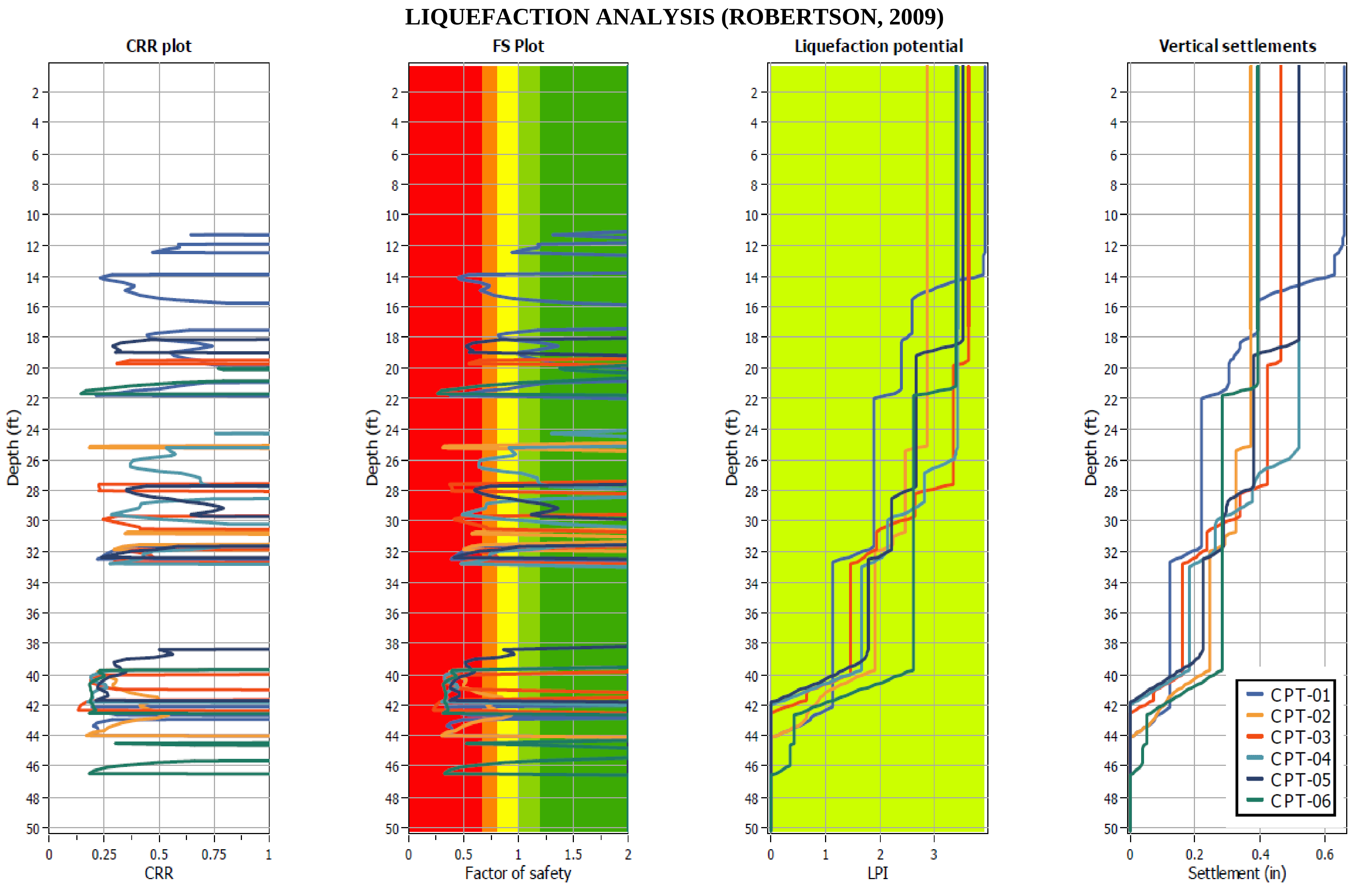
FIGURE C-1
AUGUST 2023
PROJECT NO. 6290-1



LIQUEFACTION ANALYSIS (ROBERTSON, 2009)
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FIGURE C-2
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