

TYPE OF SERVICES	Geotechnical Investigation
PROJECT NAME	Halsey House
LOCATION	University Avenue and University Terrace Los Altos, California
CLIENT	David J. Powers & Associates
PROJECT NUMBER	118-161-1
DATE	February 16, 2024



GEOTECHNICAL



CORNERSTONE EARTH GROUP

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Client Address	1871 The Alameda, Suite 200 San Jose, California
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FIGURE 1: VICINITY MAP

FIGURE 2: SITE PLAN

FIGURE 3: REGIONAL FAULT MAP

FIGURE 4A TO 4C: LIQUEFACTION ANALYSIS SUMMARY – CPT-01 TO CPT-03

APPENDIX A: FIELD INVESTIGATION

APPENDIX B: LABORATORY TEST PROGRAM

Type of Services	Geotechnical Investigation
Project Name	Halsey House
Location	University Avenue and University Terrace Los Altos, California

SECTION 1: INTRODUCTION

This geotechnical report was prepared for the sole use of David J. Powers & Associates for the Halsey House project in Los Altos, California. The location of the site is shown on the Vicinity Map, Figure 1.

1.1 PROJECT DESCRIPTION

The project will consist of evaluating the existing Halsey House structure for rehabilitation or reconstruction. The original structure was built in 1923 and was primarily used as an education and recreational nature center until 2008, when it was then closed due to its deteriorating conditions. As a result of these conditions, the Halsey House structure has remained closed since 2008. The City of Los Altos has planned for the structure to be renovated for re-use, or to be demolished and reconstructed to replace the Halsey House with a purpose-designed facility. The structure is located within the approximately 6-acre Redwood Grove Nature Preserve. The Halsey House structure is reportedly built on a concrete foundation, is of wood-framed construction, has a clay tile roof, and has an approximate footprint of 3,400 square feet.

Structural loads are not currently known for the proposed structure; however, structural loads for rehabilitation or a new structure are expected to be relatively light and typical of similar type structures. Grading plans are not available at this time; however, as the current project scope focuses on the existing structure, grading is anticipated to consist of minor cuts and fills on the order of 1 to 3 feet to potentially facilitate the rehabilitation of the existing structure, which could include underpinning and the installation of new improvements such as flatwork and utilities. Therefore, the overall topography of the Preserve is expected to remain the same following this project.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated July 10, 2023, and consisted of field and laboratory programs to evaluate physical and engineering properties of the subsurface soils, engineering analysis to prepare recommendations for site work and grading, building

foundations, flatwork, and pavements, and preparation of this report. Brief descriptions of our exploration and laboratory programs are presented below.

1.3 EXPLORATION PROGRAM

Field exploration consisted of two borings drilled on January 23, 2024 with track-mounted, limited-access, hollow-stem auger drilling equipment, as well as three Cone Penetration Tests (CPTs) advanced on January 16, 2024. The borings were drilled to depths of 25 and 30 feet; the CPTs were each advanced to depths of 44½ feet. Seismic shear wave velocity measurements were collected from CPT-1. Borings EB-1 and EB-2 were advanced adjacent to CPT-1 and CPT-2, respectively, for direct evaluation of physical samples to correlated soil behavior. The borings and CPTs were backfilled with cement grout in accordance with local requirements.

The approximate locations of our exploratory borings are shown on the Site Plan, Figure 2. Details regarding our field program are included in Appendix A.

1.4 LABORATORY TESTING PROGRAM

In addition to visual classification of samples, the laboratory program focused on obtaining data for foundation design and seismic ground deformation estimates. Testing included moisture contents, dry densities, and washed sieve analyses. Details regarding our laboratory program are included in Appendix B.

1.5 ENVIRONMENTAL SERVICES

Environmental services were not requested for this project. If environmental concerns are determined to be present during future evaluations, the project environmental consultant should review our geotechnical recommendations for compatibility with the environmental concerns.

SECTION 2: REGIONAL SETTING

2.1 GEOLOGICAL SETTING

The site is located within the Santa Clara Valley, which is a broad alluvial plane between the Santa Cruz Mountains to the southwest and west, and the Diablo Range to the northeast. The San Andreas Fault system, including the Monte Vista-Shannon Fault, exists within the Santa Cruz Mountains and the Hayward and Calaveras Fault systems exist within the Diablo Range. Alluvial soil thicknesses in the project area have been mapped at less than 100 feet (Rogers & Williams, 1974).

2.2 REGIONAL SEISMICITY

While seismologists cannot predict earthquake events, geologists from the U.S. Geological Survey have recently updated (in 2015) earlier estimates from their 2014 Uniform California Earthquake Rupture Forecast (Version 3; UCERF3) publication. The estimated probability of

one or more magnitude 6.7 earthquakes (the size of the destructive 1994 Northridge earthquake) expected to occur somewhere in the San Francisco Bay Area has been revised (increased) to 72 percent for the period 2014 to 2043 (Aagaard et al., 2016). The faults in the region with the highest estimated probability of generating damaging earthquakes between 2014 and 2043 are the Hayward (33%), Calaveras (26%), and San Andreas Faults (22%). In this 30-year period, the probability of an earthquake of magnitude 6.7 or larger occurring is 22 percent along the San Andreas Fault and 33 percent for the Hayward Fault.

The faults considered capable of generating significant earthquakes are generally associated with the well-defined areas of crustal movement, which trend northwesterly. The table below presents the State-considered active faults within 25 kilometers of the site.

Table 1: Approximate Fault Distances

Fault Name	Distance	
	(miles)	(kilometers)
Monte Vista-Shannon	2.7	4.3
San Andreas (1906)	4.3	7.0
Hayward (Southeast Extension)	13.7	22.0
Hayward (Total Length)	15.4	24.8

A regional fault map is presented as Figure 3, illustrating the relative distances of the site to significant fault zones.

SECTION 3: SITE CONDITIONS

3.1 SITE BACKGROUND

As mentioned above, the Halsey House was originally constructed in 1923. We reviewed historic aerial imagery provided online by Historic Aerials (historicaerials.com). A summary of pertinent surface changes at and near the site is as follows:

- 1948: The existing Halsey House structure is observed and the residential development to the north and east of the project site appears to have been constructed by this time. The Halsey House structure is bounded by agricultural fields to the west and south.
- 1980: Additional residential development is constructed to the west of the project site.
- 1997: Residential development to the south of the project site appears completed. No pertinent changes were observed since this image.

3.2 SURFACE DESCRIPTION

As noted above, the existing historic Halsey House is located within the Redwood Grove Nature Preserve. The Preserve is generally characterized by gentle slopes and plentiful mature trees and vegetation, with dirt and gravel walking trails present throughout. The Preserve appears to

generally slope downwards to the north, and the Halsey House structure is situated within the southern section of the Preserve. The current Adobe Creek runs approximately 50 feet north of the Halsey House. Grades around the property range from Elevation 212 feet to the south to Elevation 206 feet to the north. Numerous mature trees are present within the Preserve and the immediate vicinity of the Halsey House.

3.3 SUBSURFACE CONDITIONS

Below the surficial topsoil, our explorations generally encountered undocumented fill consisting of sandy lean clay to a depth of up to 1 foot. Beneath the undocumented fill, Boring EB-1 generally encountered native stiff to hard sandy lean clay down to a depth of 14½ feet, underlain by medium dense to dense clayey sand with variable amounts of clay and gravel to 21½ feet and very stiff lean clay to 23½ feet before terminating in dense clayey sand at a depth of 25 feet. Beneath the undocumented fill Boring EB-2 generally encountered native stiff sandy lean clay to a depth of 3 feet, underlain by medium dense to dense clayey sands with variable amounts of clay and gravel down to 30 feet, the maximum boring depth.

Beneath the terminal boring depths, our CPTs generally encountered interbedded dense sands and hard clays to the maximum depth explored of 44½ feet.

3.3.1 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents within the upper 10 feet range from 3 percent under to 5 percent over the estimated laboratory optimum moisture.

3.4 GROUNDWATER

Groundwater was encountered in our explorations EB-1 and EB-2 at depths ranging from 10 to 15 feet below current grades. All measurements were taken at the time of drilling and may not represent the stabilized levels that can vary from the initial levels encountered.

Based on the above, we recommend a design groundwater depth of 8 feet. Fluctuations in groundwater levels occur due to many factors including seasonal fluctuation, underground drainage patterns, regional fluctuations, and other factors.

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT SURFACE RUPTURE

As discussed above several significant faults are located within 25 kilometers of the site. The site is not located within a State-designated Alquist Priolo Earthquake Fault Zone, or a Santa Clara County Fault Hazard Zone. As shown in Figure 3, no known surface expression of fault traces is thought to cross the site; therefore, fault surface rupture hazard is not a significant geologic hazard at the site.

4.2 ESTIMATED GROUND SHAKING

Moderate to severe (design-level) earthquakes can cause strong ground shaking, which is the case for most sites within the Bay Area. A peak ground acceleration (PGA) was estimated for analysis using a value equal to $F_{PGA} \times PGA$, as allowed in the 2022 edition of the California Building Code when an exception has been taken per ASCE 7-16, Section 11.4.8. For our liquefaction analysis we used a PGA_M of 1.112g.

4.3 LIQUEFACTION POTENTIAL

The site is within a State-designated Liquefaction Hazard Zone (CGS, Cupertino 7.5-Minute Quadrangle, 2002) as well as a Santa Clara County Liquefaction Hazard Zone (Santa Clara County, 2003). Our field and laboratory programs addressed this issue by testing and sampling potentially liquefiable layers to maximum depths explored, performing visual classification on sampled materials, evaluating CPT data, and performing various tests to further classify soil properties.

4.3.1 Background

During strong seismic shaking, cyclically induced stresses can cause increased pore pressures within the soil matrix that can result in liquefaction triggering, soil softening due to shear stress loss, potentially significant ground deformation due to settlement within sandy liquefiable layers as pore pressures dissipate, and/or flow failures in sloping ground or where open faces are present (lateral spreading) (NCEER 1998). Limited field and laboratory data is available regarding ground deformation due to settlement; however, in clean sand layers settlement on the order of 2 to 4 percent of the liquefied layer thickness can occur. Soils most susceptible to liquefaction are loose, non-cohesive soils that are saturated and are bedded with poor drainage, such as sand and silt layers bedded with a cohesive cap.

4.3.2 Analysis

As discussed in the “Subsurface” section above, several sand layers were encountered below the design groundwater depth of 8 feet. Following the liquefaction analysis framework in the 2008 monograph, *Soil Liquefaction During Earthquakes* (Idriss and Boulanger, 2008), incorporating updates in *CPT and SPT Based Liquefaction Triggering Procedures* (Boulanger and Idriss, 2014), and in accordance with CDMG Special Publication 117A guidelines (CDMG, 2008) for quantitative analysis, these layers were analyzed for liquefaction triggering and potential post-liquefaction settlement. These methods compare the ratio of the estimated cyclic shaking (Cyclic Stress Ratio - CSR) to the soil’s estimated resistance to cyclic shaking (Cyclic Resistance Ratio - CRR), providing a factor of safety against liquefaction triggering. Factors of safety less than or equal to 1.3 are considered to be potentially liquefiable and capable of post-liquefaction re-consolidation (i.e. settlement).

The CSR for each layer quantifies the stresses anticipated to be generated due to a design-level seismic event, is based on the peak horizontal acceleration generated at the ground surface discussed in the “Estimated Ground Shaking” section above, and is corrected for

overburden and stress reduction factors as discussed in the procedure developed by Seed and Idriss (1971) and updated in the 2008 Idriss and Boulanger monograph.

In estimating post-liquefaction settlement at the site, we have implemented a depth weighting factor proposed by Cetin (2009). Following evaluation of 49 high-quality, cyclically induced, ground settlement case histories from seven different earthquakes, Cetin proposed the use of a weighting factor based on the depth of layers. The weighting procedure was used to tune the surface observations at liquefaction sites to produce a better model fit with measured data. Aside from the better model fit it produced, the rationale behind the use of a depth weighting factor is based on the following: 1) upward seepage, triggering void ratio redistribution, and resulting in unfavorably higher void ratios for the shallower sublayers of soil layers; 2) reduced induced shear stresses and number of shear stress cycles transmitted to deeper soil layers due to initial liquefaction of surficial layers; and 3) possible arching effects due to nonliquefied soil layers. All these may significantly reduce the contribution of volumetric settlement of deeper soil layers to the overall ground surface settlement (Cetin, 2009).

The soil's CRR is estimated from the in-situ measurements from CPTs and laboratory testing on samples retrieved from our borings. SPT "N" values obtained from hollow-stem auger borings were not used in our analyses, as the "N" values obtained are less reliable in sands below groundwater. The tip pressures are corrected for effective overburden stresses, taking into consideration both the groundwater level at the time of exploration and the design groundwater level, and stress reduction versus depth factors. The CPT method utilizes the soil behavior type index (I_c) to estimate the plasticity of the layers.

The results of our CPT analyses (CPT-1 through CPT-3) are presented in Figures 4A through 4C of this report.

4.3.3 Summary

Our analyses indicate that several layers could potentially experience liquefaction triggering that could result in post-liquefaction total settlement at the ground surface ranging up to $\frac{2}{3}$ -inch based on the Yoshimine (2006) method. As discussed in SP 117A, differential movement for level ground sites over deep soil sites will be up to about two-thirds of the total settlement between independent foundation elements. In our opinion, differential settlements are anticipated to be on the order of up to $\frac{1}{2}$ -inch over a horizontal distance of 30 feet.

4.3.4 Ground Deformation and Surficial Cracking Potential

The methods used to estimate liquefaction settlements assume that there is a sufficient cap of non-liquefiable material to prevent ground deformation or sand boils. For ground deformation to occur, the pore water pressure within the liquefiable soil layer will need to be great enough to break through the overlying non-liquefiable layer, which could cause significant ground deformation and settlement. The work of Youd and Garris (1995) indicates that the 25-foot thick layer of non-liquefiable cap is sufficient to prevent ground deformation and significant surficial cracking; therefore, the above total settlement estimates are reasonable.

4.4 LATERAL SPREADING

Lateral spreading is horizontal/lateral ground movement of relatively flat-lying soil deposits towards a free face such as an excavation, channel, or open body of water; typically, lateral spreading is associated with liquefaction of one or more subsurface layers near the bottom of the exposed slope. As failure tends to propagate as block failures, it is difficult to analyze and estimate where the first tension crack will form.

The current Adobe Creek runs approximately 50 feet north of the Halsey House. The channel bottom was observed to be roughly 6 to 7 feet deep relative to the existing adjacent site grades. As part of our liquefaction analyses, we calculated the Lateral Displacement Index (LDI) for potentially liquefiable layers based on methods presented in the 2008 monograph, *Soil Liquefaction During Earthquakes* (Idriss and Boulanger, 2008). LDI is a summation of the maximum shear strains versus depth, which is a measurement of the potential maximum displacement at that exploration location. Summations of the LDI values to a depth equal to twice the open face height were included. Estimated displacements in the vicinity of CPT-1 through CPT-3 based on the LDI calculations are considered negligible.

We also reviewed the historical ground failure map presented in the Seismic Hazard Zone Report for the vicinity (CGS, Cupertino 7.5-Minute Quadrangle, 2002), which indicated no occurrences of lateral spread within the project site vicinity. Therefore, in our opinion, the potential for lateral spreading to impact the project is relatively low.

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose unsaturated sandy soils can settle during strong seismic shaking. We evaluated the potential for seismic compaction of the loose to medium dense sands based on the work by Robertson and Shao (2010). Based on our analyses, the potential for significant seismic settlement affecting the proposed improvements is low.

4.6 TSUNAMI/SEICHE

The terms tsunami or seiche are described as ocean waves or similar waves usually created by undersea fault movement or by a coastal or submerged landslide. Tsunamis may be generated at great distance from shore (far field events) or nearby (near field events). Waves are formed, as the displaced water moves to regain equilibrium, and radiates across the open water, similar to ripples from a rock being thrown into a pond. When the waveform reaches the coastline, it quickly raises the water level, with water velocities as high as 15 to 20 knots. The water mass, as well as vessels, vehicles, or other objects in its path create tremendous forces as they impact coastal structures.

Tsunamis have affected the coastline along the Pacific Northwest during historic times. The Fort Point tide gauge in San Francisco recorded approximately 21 tsunamis between 1854 and 1964. The 1964 Alaska earthquake generated a recorded wave height of 7.4 feet and drowned eleven people in Crescent City, California. For the case of a far-field event, the Bay area would

have hours of warning; for a near field event, there may be only a few minutes of warning, if any.

A tsunami or seiche originating in the Pacific Ocean would lose much of its energy passing through San Francisco Bay. Based on the mapping of tsunami inundation potential for the San Francisco Bay Area by CGS (conservation.ca.gov/cgs/tsunami/maps), areas most likely to be inundated are marshlands, tidal flats, and former bay margin lands that are now artificially filled, but are still at or below sea level, and are generally within 1½ miles of the shoreline. The site is approximately 5¾ miles inland from the San Francisco Bay shoreline and is approximately 206 to 212 feet above mean sea level (WGS84). Therefore, the potential for inundation due to tsunami or seiche is considered low.

4.7 FLOODING

Based on our internet search of the Federal Emergency Management Agency (FEMA) flood map public database, the site is located within Zone AE, defined as a “Regulatory Floodway” and Zone A, with base flood elevations between 204 and 208. We recommend the project civil engineer be retained to confirm this information and verify the base flood elevation, if appropriate.

The Department of Water Resources (DWR), Division of Safety of Dams (DSOD) compiled a database of Dam Failure Inundation Hazard Maps (DSOD, 2015). The generalized hazard maps were prepared by dam owners as required by the State Office of Emergency Services; they are intended for planning purposes only. Based on our review of these maps, the site is not located within a dam failure inundation area.

SECTION 5: CONCLUSIONS

5.1 SUMMARY

From a geotechnical viewpoint, retrofitting the existing structure or building new facilities should both be feasible provided the concerns listed below are addressed in the project design. Descriptions of each concern with brief outlines of our recommendations follow the listed concerns.

- Potential for liquefaction-induced settlements
- Presence of undocumented fill and redevelopment considerations
- Presence of undocumented fill
- Shallow groundwater

5.1.1 Potential for Liquefaction-Induced Settlements

As discussed, our liquefaction analysis indicates that there is a potential for liquefaction of localized sand layers during a significant seismic event. Although the potential for liquefied sands to vent to the ground surface through cracks in the surficial soils is low, our analysis

indicates that liquefaction-induced settlement on the order of up to $\frac{2}{3}$ -inch could occur, resulting in differential settlement up to $\frac{1}{2}$ -inch. New foundations should be designed to tolerate the anticipated total and differential settlements. Liquefaction does not appear to pose a severe risk to the structure; therefore, existing foundations should not be impacted to a degree that would necessitate replacement. Detailed foundation recommendations are presented in the “Foundations” section.

5.1.2 Presence of Undocumented Fill and Redevelopment Considerations

As discussed, the site is currently occupied by the existing Halsey House and appurtenant flatwork, site fixtures, and landscaping. We understand that the City is considering renovating the existing structure, which may include new foundation elements, or demolishing the existing structure and reconstructing a purpose-designed facility as a replacement. Potential issues that are often associated with new foundations or redeveloping sites include demolition of existing improvements, abandonment of existing utilities, and undocumented fills. As discussed, our borings generally encountered up to one foot of undocumented fill. Any undocumented fill encountered during site grading should be completely removed from within improvement areas and to a lateral distance equal to fill depth below the perimeter footing. Provided the fills meet the “Material for Fill” requirements below, the fills may be reused when backfilling the excavations. The existing fill should not affect the performance of the existing structure, and should not impact new improvements provided the recommendations in this report to remove and replace the undocumented fill as engineered fill are followed. Please refer to the “Earthwork” section below for further recommendations.

5.1.3 Shallow Groundwater

Shallow groundwater was measured at depths ranging from approximately 10 to 15 feet below the existing ground surface. As discussed above, we recommend a design groundwater depth of 8 feet. Our experience with similar sites in the vicinity indicates that shallow groundwater could significantly impact grading and underground construction, depending on the depth of excavations. These impacts typically consist of potentially wet and unstable subgrade, difficulty achieving compaction, and difficult underground utility installation. Dewatering and shoring of utility trenches may be required in some isolated areas of the site. Shallow groundwater should not impact the performance of the existing structure. Detailed recommendations addressing this concern are presented in the “Earthwork” section of this report.

5.2 PLANS AND SPECIFICATIONS REVIEW

We recommend that we be retained to review the geotechnical aspects of the project structural, civil, and landscape plans and specifications, allowing sufficient time to provide the design team with any comments prior to issuing the plans for construction.

5.3 CONSTRUCTION OBSERVATION AND TESTING

As site conditions may vary significantly between the small-diameter borings performed during this investigation, we also recommend that a Cornerstone representative be present to provide

geotechnical observation and testing during earthwork and foundation construction. This will allow us to form an opinion and prepare a letter at the end of construction regarding contractor compliance with project plans and specifications, and with the recommendations in our report. We will also be allowed to evaluate any conditions differing from those encountered during our investigation and provide supplemental recommendations as necessary. For these reasons, the recommendations in this report are contingent of Cornerstone providing observation and testing during construction. Contractors should provide at least a 48-hour notice when scheduling our field personnel.

SECTION 6: EARTHWORK

6.1 SITE DEMOLITION

If renovation and seismic retrofit of the existing structure is planned, the areas designated for improvements should be cleared of all surface and subsurface deleterious materials including any pre-existing foundations, flatwork, pavements, utilities, and other improvements, if encountered. If the existing structure will be removed and reconstructed as a purpose-designed facility, all existing improvements not to be reused for the development should be demolished and removed from the site. Excavations extending below the planned finished improvement grades should be cleaned and backfilled with suitable material compacted to the recommendations in the “Compaction” section of this report.

Cornerstone should be notified prior to the start of demolition and should be present on at least a part-time basis during all backfill and mass grading as a result of demolition. Occasionally, other types of buried structures (wells, cisterns, debris pits, etc.) can be found on sites with prior development. If encountered, Cornerstone should be contacted to address these types of structures on a case-by-case basis.

6.1.1 Demolition of Existing Slabs, Foundations and Pavements

All slabs and foundations not designated for reuse should be completely removed from within planned building areas.

Special care should be taken during the demolition and removal of existing floor slabs, foundations, utilities, and pavements to minimize disturbance of the subgrade. Excessive disturbance of the subgrade, which includes either native or previously placed engineered fill, resulting from demolition activities can have serious detrimental effects on planned foundation and paving elements.

Existing foundations are typically mat-slabs, shallow footings, or piers/piles. If slab or shallow footings are encountered, they should be completely removed. If drilled piers are encountered, they should be cut off at an elevation at least 60-inches below proposed footings or the final subgrade elevation, whichever is deeper. The remainder of the drilled pier could remain in place. Foundation elements to remain in place should be surveyed and superimposed on the proposed development plans to determine the potential for conflicts or detrimental impacts to

the planned construction. Following review, additional mitigation or planned foundation elements may need to be modified.

6.1.2 Abandonment of Existing Utilities

All utilities not designated for reuse should be completely removed from within planned building areas. For any utility line to be considered acceptable to remain within building areas, the utility line must be completely backfilled with grout or sand-cement slurry (sand slurry is not acceptable), the ends outside the building area capped with concrete, and the trench fills either removed and replaced as engineered fill with the trench side slopes flattened to at least 1:1, or the trench fills are determined not to be a risk to the structure. The assessment of the level of risk posed by the particular utility line will determine whether the utility may be abandoned in place or needs to be completely removed. The contractor should assume that all utilities will be removed from within building areas unless provided written confirmation from both the owner and the geotechnical engineer.

Utilities extending beyond the building area may be abandoned in place provided the ends are plugged with concrete, they do not conflict with planned improvements, and that the trench fills do not pose significant risk to the planned surface improvements.

The risk for owners associated with abandoning utilities in place include the potential for future differential settlement of existing trench fills, and/or partial collapse and potential ground loss into utility lines that are not completely filled with grout.

6.2 SITE CLEARING AND PREPARATION

6.2.1 Site Stripping

The site should be stripped of all surface vegetation, and surface and subsurface improvements to be removed within the proposed development area. Demolition of existing improvements is discussed in the prior paragraphs. Surface vegetation and topsoil should be stripped to a sufficient depth to remove all material greater than 3 percent organic content by weight. Based on our site observations, surficial stripping should extend about 3 to 6 inches below existing grade in vegetated areas.

6.2.2 Tree and Shrub Removal

Trees and shrubs designated for removal should have the root balls and any roots greater than ½-inch diameter removed completely. Mature trees are estimated to have root balls extending to depths of 2 to 4 feet, depending on the tree size. Significant root zones are anticipated to extend to the diameter of the tree canopy. Grade depressions resulting from root ball removal should be cleaned of loose material and backfilled in accordance with the recommendations in the "Compaction" section of this report.

6.3 MITIGATION OF UNDOCUMENTED FILLS

Undocumented fill was encountered in our Boring EB-1 and EB-2 ranging from approximately ½ to 1 foot. All undocumented fills should be completely removed from within proposed improvement areas and to a lateral distance of at least equal to fill depth below the perimeter footing, whichever is greater. Provided the fills meet the “Material for Fill” requirements below, the fills may be reused when backfilling the excavations. Based on review of the samples collected from our borings, it appears that the fill may be reused. If materials are encountered that do not meet the requirements, such as debris, wood, trash, those materials should be screened out of the remaining material and be removed from the site. Backfill of excavations should be placed in lifts and compacted in accordance with the “Compaction” section below.

Fills extending into planned pavement and flatwork areas may be left in place provided they are determined to be a low risk for future differential settlement and that the upper 12 to 18 inches of fill below pavement subgrade is re-worked and compacted as discussed in the “Compaction” section below. In our opinion, the fills encountered at this site are acceptable to be left in place beneath planned new pavement and flatwork.

6.4 TEMPORARY CUT AND FILL SLOPES

The contractor is responsible for maintaining all temporary slopes and providing temporary shoring where required. Temporary shoring, bracing, and cuts/fills should be performed in accordance with the strictest government safety standards. On a preliminary basis, the upper 10 feet at the site may be classified as OSHA Soil Type C materials.

Excavations performed during site demolition and fill removal should be sloped at 2:1 (horizontal:vertical) within the upper 5 feet below building subgrade. Actual excavation inclinations should be reviewed in the field during construction, as needed. Excavations below building subgrade and excavations in pavement and flatwork areas should be sloped in accordance with OSHA soil classification requirements.

6.5 SUBGRADE PREPARATION

After the areas designated for improvements have been properly cleared, stripped and necessary excavations have been made, the exposed surface soils in those areas to receive fills, slabs-on-grade, and/or pavements should be scarified to a depth of 6 inches, moisture conditioned, and compacted in accordance with the “Compaction” section below.

6.6 WET SOIL STABILIZATION GUIDELINES

Native soil and fill materials, especially soils with high fines contents such as clays and silty soils, can become unstable due to high moisture content, whether from high in-situ moisture contents or from winter rains. As the moisture content increases over the laboratory optimum, it becomes more likely the materials will be subject to softening and yielding (pumping) from construction loading or become unworkable during placement and compaction.

As discussed in the “Subsurface” section in this report, the in-situ moisture contents can range up to 15 percent over the estimated laboratory optimum in the upper 10 of the soil profile. The contractor should anticipate drying any overly wet soils prior to reusing them as fill. In addition, repetitive rubber-tire loading will likely de-stabilize the overly wet soils.

There are several methods to address potential unstable soil conditions and facilitate fill placement and trench backfill. Some of the methods are briefly discussed below. Implementation of the appropriate stabilization measures should be evaluated on a case-by-case basis according to the project construction goals and the site conditions.

6.6.1 Scarification and Drying

The subgrade may be scarified to a depth of 6 to 12 inches and allowed to dry to near optimum conditions, if sufficient dry weather is anticipated to allow sufficient drying. More than one round of scarification may be needed to break up the soil clods.

6.6.2 Removal and Replacement

As an alternative to scarification, the contractor may choose to over-excavate the unstable soils and replace them with dry on-site or import materials. A Cornerstone representative should be present to provide recommendations regarding the appropriate depth of over-excavation, whether a geosynthetic (stabilization fabric or geogrid) is recommended, and what materials are recommended for backfill.

6.6.3 Chemical Treatment

Where the unstable area exceeds about 5,000 to 10,000 square feet and/or site winterization is desired, chemical treatment with quicklime (CaO), kiln-dust, or cement may be more cost-effective than removal and replacement. Recommended chemical treatment depths will typically range from 12 to 18 inches depending on the magnitude of the instability.

6.7 MATERIAL FOR FILL

6.7.1 Re-Use of On-site Soils

On-site soils with an organic content less than 3 percent by weight may be reused as general fill. General fill should not have lumps, clods, or cobble pieces larger than 6 inches in diameter; 85 percent of the fill should be smaller than 2½ inches in diameter. Minor amounts of oversized material (smaller than 12 inches in diameter) may be allowed provided the oversized pieces are not allowed to nest together and the compaction method will allow for loosely placed lifts not exceeding 12 inches.

6.7.2 Potential Import Sources

Imported and non-expansive material should be inorganic with a Plasticity Index (PI) of 15 or less, and not contain recycled asphalt concrete where it will be used within the habitable

building areas. To prevent significant caving during trenching or foundation construction, imported material should have sufficient fines. Samples of potential import sources should be delivered to our office at least 10 days prior to the desired import start date. Information regarding the import source should be provided, such as any site geotechnical reports. If the material will be derived from an excavation rather than a stockpile, potholes will likely be required to collect samples from throughout the depth of the planned cut that will be imported. At a minimum, laboratory testing will include PI tests. Material data sheets for select fill materials (Class 2 aggregate base, ¾-inch crushed rock, quarry fines, etc.) listing current laboratory testing data (not older than 6 months from the import date) may be provided for our review without providing a sample. If current data is not available, specification testing will need to be completed prior to approval.

Environmental and soil corrosion characterization should also be considered by the project team prior to acceptance. Suitable environmental laboratory data to the planned import quantity should be provided to the project environmental consultant; additional laboratory testing may be required based on the project environmental consultant's review. The potential import source should also not be more corrosive than the on-site soils, based on pH, saturated resistivity, and soluble sulfate and chloride testing.

6.8 COMPACTION REQUIREMENTS

All fills, and subgrade areas where fill, slabs-on-grade, and pavements are planned, should be placed in loose lifts 8 inches thick or less and compacted in accordance with ASTM D1557 (latest version) requirements as shown in the table below. In general, clayey soils should be compacted with sheepsfoot equipment and sandy/gravelly soils with vibratory equipment; open-graded materials such as crushed rock should be placed in lifts no thicker than 18 inches and consolidated in place with vibratory equipment. Each lift of fill and all subgrade should be firm and unyielding under construction equipment loading in addition to meeting the compaction requirements to be approved. The contractor (with input from a Cornerstone representative) should evaluate the in-situ moisture conditions, as the use of vibratory equipment on soils with high moistures can cause unstable conditions. General recommendations for soil stabilization are provided in the "Wet Soil Stabilization Guidelines" section of this report.

Table 2: Compaction Requirements

Description	Material Description	Minimum Relative ¹ Compaction (percent)	Moisture ² Content (percent)
General Fill (within upper 5 feet)	On-Site Soils	90	>1
General Fill (below a depth of 5 feet)	On-Site Soils	95	>1
Trench Backfill	On-Site Soils	90	>1
Trench Backfill (upper 6 inches of subgrade)	On-Site Soils	95	>1
Crushed Rock Fill	¾-inch Clean Crushed Rock	Consolidate In-Place	NA
Non-Expansive Fill	Imported Non-Expansive Fill	90	Optimum
Flatwork Subgrade	On-Site Soils	90	>1
Flatwork Aggregate Base	Class 2 Aggregate Base ³	90	Optimum
Pavement Subgrade	On-Site Soils	95	>1
Pavement Aggregate Base	Class 2 Aggregate Base ³	95	Optimum
Asphalt Concrete	Asphalt Concrete	95 (Marshall)	NA

1 – Relative compaction based on maximum density determined by ASTM D1557 (latest version)

2 – Moisture content based on optimum moisture content determined by ASTM D1557 (latest version)

3 – Class 2 aggregate base shall conform to Caltrans Standard Specifications, latest edition, except that the relative compaction should be determined by ASTM D1557 (latest version)

6.9 TRENCH BACKFILL

Utility lines constructed within public right-of-way should be trenched, bedded and shaded, and backfilled in accordance with the local or governing jurisdictional requirements. Utility lines in private improvement areas should be constructed in accordance with the following requirements unless superseded by other governing requirements.

All utility lines should be bedded and shaded to at least 6 inches over the top of the lines with crushed rock (¾-inch-diameter or greater) or well-graded sand and gravel materials conforming to the pipe manufacturer's requirements. Open-graded shading materials should be consolidated in place with vibratory equipment and well-graded materials should be compacted to at least 90 percent relative compaction with vibratory equipment prior to placing subsequent backfill materials.

General backfill over shading materials may consist of on-site native materials provided they meet the requirements in the "Material for Fill" section, and are moisture conditioned and compacted in accordance with the requirements in the "Compaction" section.

Where utility lines will cross perpendicular to strip footings, the footing should be deepened to encase the utility line, providing sleeves or flexible cushions to protect the pipes from anticipated foundation settlement, or the utility lines should be backfilled to the bottom of footing with sand-

cement slurry or lean concrete. Where utility lines will parallel footings and will extend below the “foundation plane of influence,” an imaginary 1:1 plane projected down from the bottom edge of the footing, either the footing will need to be deepened so that the pipe is above the foundation plane of influence, or the utility trench will need to be backfilled with sand-cement slurry or lean concrete within the influence zone. Sand-cement slurry used within foundation influence zones should have a minimum compressive strength of 75 psi.

6.10 SITE DRAINAGE

Ponding should not be allowed adjacent to building foundations, slabs-on-grade, or pavements. Hardscape surfaces should slope at least 2 percent towards suitable discharge facilities; landscape areas should slope at least 3 percent to at least 10 feet from the structure. Roof runoff should be directed away from building areas in closed conduits, to approved infiltration facilities, or on to hardscaped surfaces that drain to suitable facilities. Retention, detention, or infiltration facilities should be spaced at least 10 feet from buildings, and preferably at least 5 feet from slabs-on-grade or pavements. However, if retention, detention, or infiltration facilities are located within these zones, we recommend that these treatment facilities meet the requirements in the Storm Water Treatment Design Considerations section of this report.

Where minimal side yards are planned (10 feet or less), we recommend that area drains collect surface runoff and transmit the runoff to other suitable landscape drainage facilities to prevent ponding adjacent to building foundations. Landscape drainage such as drain inlets and storm water filtration and/or infiltration trenches should be provided to collect and transmit storm water runoff to project storm drains, and/or detention or retention facilities.

SECTION 7: 2022 CBC SEISMIC DESIGN CRITERIA

7.1 SEISMIC DESIGN CRITERIA

We understand that the project structural design for a seismic retrofit or structure reconstruction will be based on the 2022 California Building Code (CBC), which provides criteria for the seismic design of buildings in Chapter 16. The “Seismic Coefficients” used to design buildings are established based on a series of tables and figures addressing different site factors, including the time-weighted average shear wave velocity of the top 100 feet (30 meters) of the soil profile (V_{S30}), and soil profile in the upper 100 feet below grade and mapped spectral acceleration parameters based on distance to the controlling seismic source/fault system.

Our explorations generally encountered stiff clay and medium dense to dense sand deposits to a depth of 44½ feet, the maximum depth explored. Shear wave velocity (V_s) measurements were performed while advancing CPT-1, resulting in a time-averaged shear wave velocity for the top 30 meters (V_{S30}) of 367 meters per second. Therefore, we have classified the site as Soil Classification C. The mapped spectral acceleration parameters S_s and S_1 were calculated using the web-based program ATC Hazards by Locations, located at <https://hazards.atcouncil.org/>, based on the site coordinates presented below and the site classification. Recommended values for design are presented in Table 3. The table below lists the various factors used to determine the seismic coefficients and other parameters.

Table 3: CBC Site Categorization and Site Coefficients

Classification/Coefficient	Design Value
Site Class	C
Site Latitude	37.372911°
Site Longitude	-122.119684°
0.2-second Period Mapped Spectral Acceleration ¹ , S_s	2.242g
1-second Period Mapped Spectral Acceleration ¹ , S_1	0.806g
Short-Period Site Coefficient – F_a	1.2
Long-Period Site Coefficient – F_v	1.4
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{MS}	2.69g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	1.128g
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.794g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	0.752g
Site Amplification Factor at PGA – F_{PGA}	1.2
Site Modified Peak Ground Acceleration – PGA_M	1.112g

SECTION 8: FOUNDATIONS

8.1 SUMMARY OF RECOMMENDATIONS

In our opinion, both a proposed remodel and seismic retrofit or a reconstructed structure may be supported on shallow foundations provided the recommendations in the “Earthwork” section and the sections below are followed. If ground anchors or other foundation types are desired, we should be retained to provide further recommendations as needed.

8.2 SHALLOW FOUNDATIONS

8.2.1 Conventional Shallow Footings

New conventional shallow footings should bear on natural, undisturbed soil or engineered fill, be at least 15 inches wide, and extend at least 12 inches below the lowest adjacent grade. Lowest adjacent grade is defined as the deeper of the following: 1) bottom of the adjacent interior slab-on-grade, or 2) finished exterior grade, excluding landscaping topsoil.

Footings constructed to the above dimensions and in accordance with the “Earthwork” recommendations of this report are capable of supporting maximum allowable bearing pressures of 2,500 psf for dead loads, 3,750 psf for combined dead plus live loads, and 5,000 psf for all loads including wind and seismic. These pressures are based on factors of safety of 3.0, 2.0, and 1.5 applied to the ultimate bearing pressure for dead, dead plus live, and all loads,

respectively. These pressures are net values; the weight of the footing may be neglected for the portion of the footing extending below grade (typically, the full footing depth). Top and bottom mats of reinforcing steel should be included in continuous footings to help span irregularities and differential settlement.

8.2.2 Footing Settlement

Structural loads were not provided to us at the time this report was prepared; therefore, we assumed the typical loading in the following table.

Table 4: Assumed Structural Loading

Foundation Area	Range of Assumed Loads
Isolated Column Footing	20 to 25 kips
Perimeter Strip Footing	1 to 3 kips per lineal foot

Based on the above loading and the allowable bearing pressures presented above, we estimate that the total static footing settlement will be on the order of $\frac{1}{3}$ -inch, with $\frac{1}{4}$ -inch or less of post-construction differential settlement between adjacent foundation elements. In addition, we estimate that differential seismic movement will be on the order of $\frac{1}{2}$ -inch over a horizontal distance of 30 feet, resulting in a total estimated differential footing movement of $\frac{3}{4}$ -inch between foundation elements, assumed to be on the order of 30 feet. As our footing loads were assumed, we recommend we be retained to review the final footing layout and loading, and verify the settlement estimates above.

8.2.3 Lateral Loading

Lateral loads may be resisted by friction between the bottom of footing and the supporting subgrade, and also by passive pressures generated against footing sidewalls. An ultimate frictional resistance of 0.40 applied to the footing dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 450 pcf may be used in design. The structural engineer should apply an appropriate factor of safety (such as 1.5) to the ultimate values above. Where footings are adjacent to landscape areas without hardscape, the upper 12 inches of soil should be neglected when determining passive pressure capacity.

8.2.4 Conventional Shallow Footing Construction Considerations

Where utility lines will cross perpendicular to strip footings, the footing should be deepened to encase the utility line, providing sleeves or flexible cushions to protect the pipes from anticipated foundation settlement, or the utility lines should be backfilled to the bottom of footing with sand-cement slurry or lean concrete. Where utility lines will parallel footings and will extend below the "foundation plane of influence," an imaginary 1:1 plane projected down from the bottom edge of the footing, either the footing will need to be deepened so that the pipe is above the foundation plane of influence, or the utility trench will need to be backfilled with sand-cement slurry or lean

concrete within the influence zone. Sand-cement slurry used within foundation influence zones should have a minimum compressive strength of 75 psi.

Footing excavations should be filled as soon as possible or be kept moist until concrete placement by regular sprinkling to prevent desiccation. A Cornerstone representative should observe all footing excavations prior to placing reinforcing steel and concrete. If there is a significant schedule delay between our initial observation and concrete placement, we may need to re-observe the excavations.

8.3 GROUND ANCHORS

As discussed, we understand the City is considering a renovation and seismic retrofit for the existing Halsey House, which may include new foundation elements, or potentially new ground anchors to support new shear elements within the structure. Cuts adjacent to the existing slab or foundations should be properly supported to prevent soil from sloughing out from underneath the existing slab or footings. Excavations extending into the influence zone of existing foundations may need to be performed in sequence and may need to be underpinned depending on the specific situation. There are several methods that could be used to underpin existing footings, if needed. In our opinion, helical anchors or drilled piers could be used as ground anchors to support the existing foundations or new foundation elements. Helical anchors should be designed and installed by a helical anchor design build contractor.

Drilled piers may be used to provide additional uplift and vertical capacity. The piers should have a minimum diameter of 18 inches and extend to a depth of at least 10 feet below the surface. The vertical capacity may be designed based on an allowable skin friction of 200 pounds per square foot (psf) for the upper 5 feet and 500 psf for depths deeper than 5 feet. These allowable values are for combined dead plus live loads based on a factor of safety of 2.0; dead loads should not exceed two-thirds of the allowable capacities. The allowable skin friction may be increased by one-third for wind and seismic loads. Frictional resistance to uplift loads may be developed along the pier shafts based on 80 percent of the ultimate frictional resistance; the structural engineer should apply an appropriate factor of safety (such as 1.5) to the ultimate uplift capacity.

Cornerstone should review any proposed underpinning design prior to the start of construction and should be onsite while operations for underpinning or excavation in sequence beneath foundations are being performed.

SECTION 9: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

9.1 INTERIOR SLABS-ON-GRADE

As the Plasticity Index (PI) of the surficial soils is 15 or less, the proposed new areas of slabs-on-grade may be supported directly on subgrade prepared in accordance with the recommendations in the "Earthwork" section of this report. If moisture-sensitive floor coverings are planned, the recommendations in the "Interior Slabs Moisture Protection Considerations" section below may be incorporated in the project design if desired. If significant time elapses

between initial subgrade preparation and slab-on-grade construction, the subgrade should be proof-rolled to confirm subgrade stability, and if the soil has been allowed to dry out, the subgrade should be re-moisture conditioned to near optimum moisture content.

The structural engineer should determine the appropriate slab reinforcement for the loading requirements and consider the expansion potential of the underlying soils. For unreinforced concrete slabs, ACI 302.1R recommends limiting control joint spacing to 24 to 36 times the slab thickness in each direction, or a maximum of 18 feet.

9.2 INTERIOR SLABS MOISTURE PROTECTION CONSIDERATIONS

The following general guidelines for concrete slab-on-grade construction where floor coverings are planned are presented for consideration by the developer, design team, and contractor. These guidelines are based on information obtained from a variety of sources, including the American Concrete Institute (ACI) and are intended to reduce the potential for moisture-related problems causing floor covering failures, and may be supplemented as necessary based on project-specific requirements. The application of these guidelines or not will not affect the geotechnical aspects of the slab-on-grade performance.

- Place a minimum 15-mil vapor retarder conforming to ASTM E 1745, Class C requirements or better directly below the concrete slab; the vapor retarder should extend to the slab edges and be sealed at all seams and penetrations in accordance with manufacturer's recommendations and ASTM E 1643 requirements. A 4-inch-thick capillary break, consisting of crushed rock should be placed below the vapor retarder and consolidated in place with vibratory equipment. The mineral aggregate shall be of such size that the percentage composition by dry weight as determined by laboratory sieves will conform to the following gradation:

Sieve Size	Percentage Passing Sieve
1"	100
$\frac{3}{4}$ "	90 – 100
No. 4	0 – 10
No. 200	0 – 5

- The concrete water:cement ratio should be 0.45 or less. Mid-range plasticizers may be used to increase concrete workability and facilitate pumping and placement.
- Water should not be added after initial batching unless the slump is less than specified and/or the resulting water:cement ratio will not exceed 0.45.
- Polishing the concrete surface with metal trowels is not recommended.
- Where floor coverings are planned, all concrete surfaces should be properly cured.

- Water vapor emission levels and concrete pH should be determined in accordance with ASTM F1869-98 and F710-98 requirements and evaluated against the floor covering manufacturer's requirements prior to installation.

9.3 EXTERIOR FLATWORK

New exterior concrete flatwork subject to pedestrian and/or occasional light pick up loading should be at least 4 inches thick and supported on at least 4 inches of Class 2 aggregate base overlying subgrade prepared in accordance with the "Earthwork" recommendations of this report. Flatwork that will be subject to heavier or frequent vehicular loading should be designed in accordance with the recommendations in the "Vehicular Pavements" section below. To help reduce the potential for uncontrolled shrinkage cracking, adequate expansion and control joints should be included. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness. Flatwork should be isolated from adjacent foundations or retaining walls except where limited sections of structural slabs are included to help span irregularities in retaining wall backfill at the transitions between at-grade and on-structure flatwork.

SECTION 10: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of David J. Powers & Associates specifically to support the design of the Halsey House project in Los Altos, California. The opinions, conclusions, and recommendations presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in Northern California at the time this report was prepared. No warranty, expressed or implied, is made or should be inferred.

Recommendations in this report are based upon the soil and groundwater conditions encountered during our subsurface exploration. If variations or unsuitable conditions are encountered during construction, Cornerstone must be contacted to provide supplemental recommendations, as needed.

David J. Powers & Associates may have provided Cornerstone with plans, reports and other documents prepared by others. David J. Powers & Associates understands that Cornerstone reviewed and relied on the information presented in these documents and cannot be responsible for their accuracy.

Cornerstone prepared this report with the understanding that it is the responsibility of the owner or their representatives to see that the recommendations contained in this report are presented to other members of the design team and incorporated into the project plans and specifications, and that appropriate actions are taken to implement the geotechnical recommendations during construction.

Conclusions and recommendations presented in this report are valid as of the present time for the development as currently planned. Changes in the condition of the property or adjacent properties may occur with the passage of time, whether by natural processes or the acts of

other persons. In addition, changes in applicable or appropriate standards may occur through legislation or the broadening of knowledge. Therefore, the conclusions and recommendations presented in this report may be invalidated, wholly or in part, by changes beyond Cornerstone's control. This report should be reviewed by Cornerstone after a period of three (3) years has elapsed from the date of this report. In addition, if the current project design is changed, then Cornerstone must review the proposed changes and provide supplemental recommendations, as needed.

An electronic transmission of this report may also have been issued. While Cornerstone has taken precautions to produce a complete and secure electronic transmission, please check the electronic transmission against the hard copy version for conformity.

Recommendations provided in this report are based on the assumption that Cornerstone will be retained to provide observation and testing services during construction to confirm that conditions are similar to that assumed for design, and to form an opinion as to whether the work has been performed in accordance with the project plans and specifications. If we are not retained for these services, Cornerstone cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of Cornerstone's report by others. Furthermore, Cornerstone will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services.

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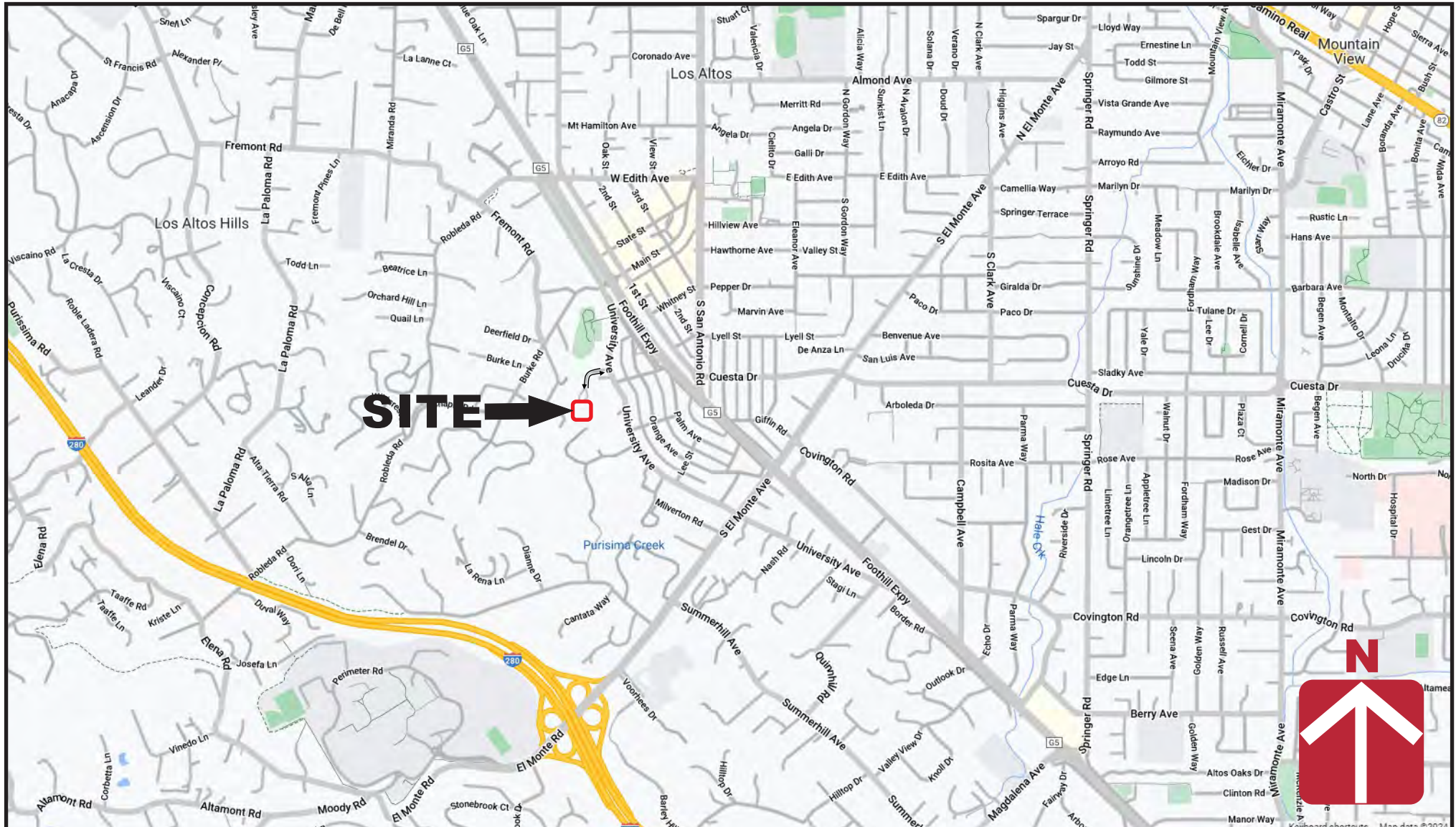
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Vicinity Map

Halsey House
Los Altos, CA

Project Number

118-161-1

Figure Number

Figure 1

Date

January 2024

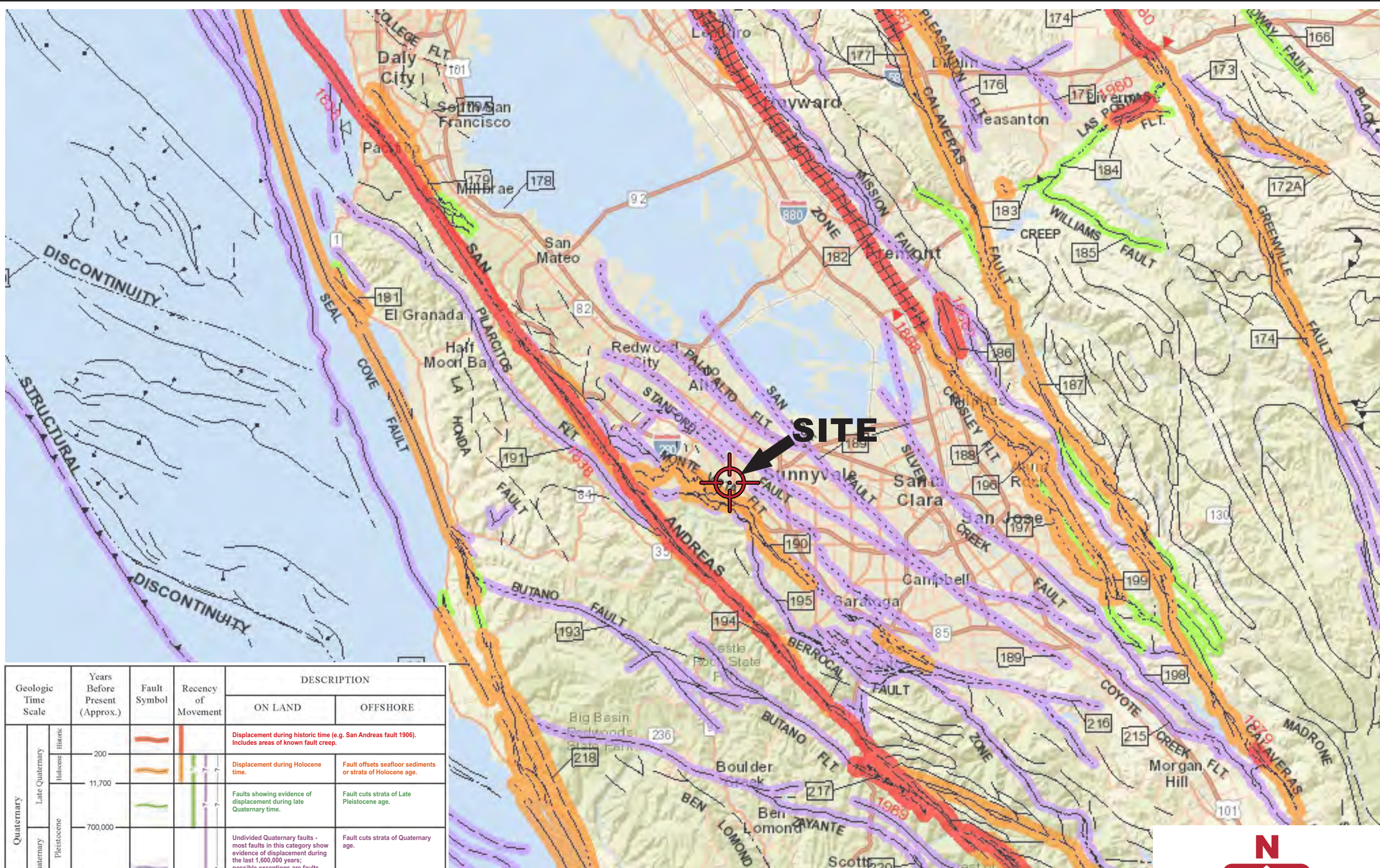
Drawn By

RRN



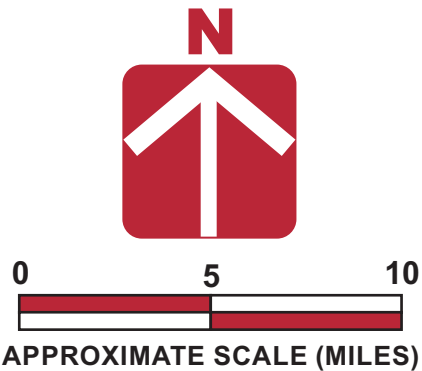
Base by Google Earth, dated 04/05/2022

Project Number		118-161-1
Figure Number		Figure 2
Date	January 2024	Drawn By RRN
Site Plan		
Halsey House Los Altos, CA		
CORNERSTONE EARTH GROUP		



Geologic Time Scale		Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION	
					ON LAND	OFFSHORE
Quaternary	Late Quaternary	Holocene			Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		200			Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
	Pleistocene	11,700			Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
Early Quaternary		700,000			Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
		1,600,000				
Pre-Quaternary		4.5 billion (Age of Earth)			Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

Base by California Geological Survey - 2010 Fault Activity Map of California (Jennings and Bryant, 2010)



Project Number118-161-1

Figure NumberFigure 3

DateJanuary 2024

Drawn ByRRN

Regional Fault Map

Halsey House
Los Altos, CA

CORNERSTONE
EARTH GROUP

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PROJECT/CPT DATA

Project Title **Halsey House**

Project No. **118-161-1**

Project Manager **SCO**

SEISMIC PARAMETERS

Controlling Fault **San Andreas**

Earthquake Magnitude (Mw) **7.6**

PGA (Amax) **1.02** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet) **20**

Design Water Depth (feet) **8**

Ave. Unit Weight Above GW (pcf) **120**

Ave. Unit Weight Below GW (pcf) **125**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM **8** FEET

0.00 (Inches)

LIQUEFACTION SETTLEMENT FROM **50** FEET

0.33 (Inches)

TOTAL SEISMIC SETTLEMENT 0.3 INCHES

POTENTIAL LATERAL DISPLACEMENT

LDI² **0.00** L/H **5.0**

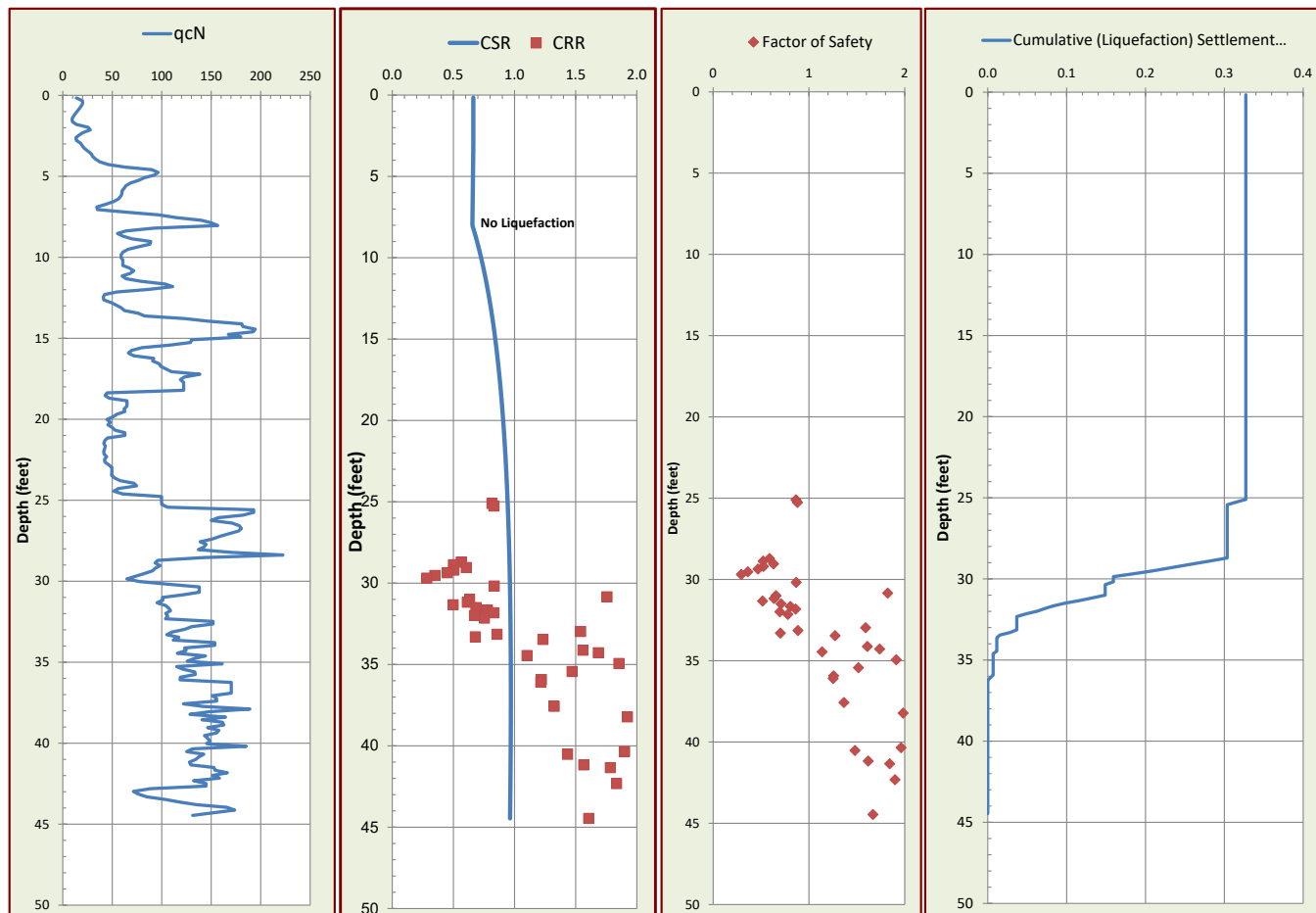
LDI¹ Corrected for Distance **0.00** (4 < L/H < 40)

EXPECTED RANGE OF DISPLACEMENT

0.0 to **0.0** feet

¹Not Valid for L/H Values < 4 and > 40.

²LDI Values Only Summed to 2H Below Grade.



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PROJECT/CPT DATA

Project Title **Halsey House**

Project No. **118-161-1**

Project Manager **SCO**

SEISMIC PARAMETERS

Controlling Fault **San Andreas**

Earthquake Magnitude (Mw) **7.6**

PGA (Amax) **1.02** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet) **20**

Design Water Depth (feet) **8**

Ave. Unit Weight Above GW (pcf) **120**

Ave. Unit Weight Below GW (pcf) **125**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM **8** FEET

0.00 (Inches)

LIQUEFACTION SETTLEMENT FROM **50** FEET

0.62 (Inches)

TOTAL SEISMIC SETTLEMENT 0.6 INCHES

POTENTIAL LATERAL DISPLACEMENT

LDI² **0.00** L/H **18.6**

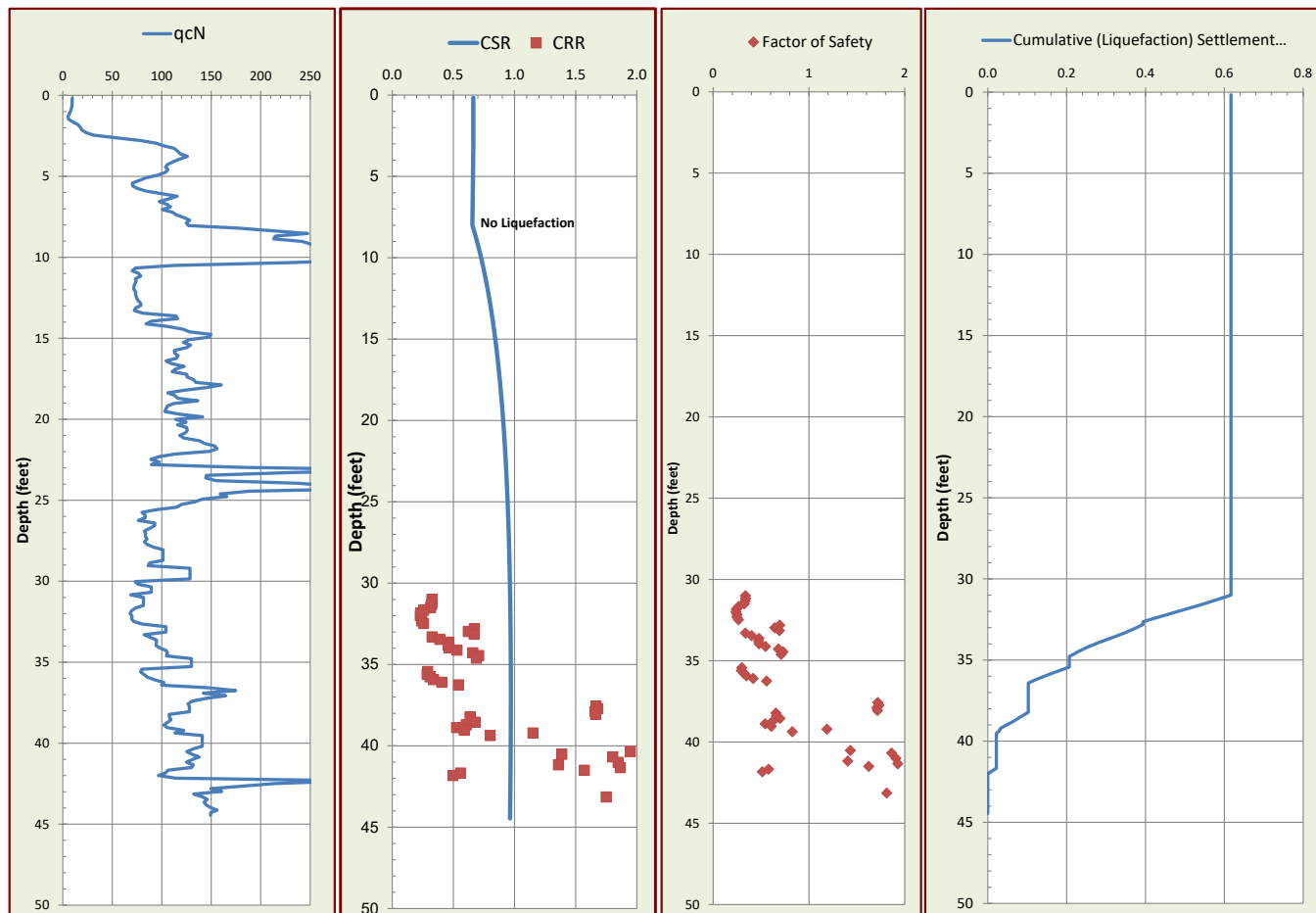
LDI¹ Corrected for Distance **0.00** (4 < L/H < 40)

EXPECTED RANGE OF DISPLACEMENT

0.0 to **0.0** feet

¹Not Valid for L/H Values < 4 and > 40.

²LDI Values Only Summed to 2H Below Grade.



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PROJECT/CPT DATA

Project Title **Halsey House**

Project No. **118-161-1**

Project Manager **SCO**

SEISMIC PARAMETERS

Controlling Fault **San Andreas**

Earthquake Magnitude (Mw) **7.6**

PGA (Amax) **1.02** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet) **20**

Design Water Depth (feet) **8**

Ave. Unit Weight Above GW (pcf) **120**

Ave. Unit Weight Below GW (pcf) **125**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM **8** FEET

0.06 (Inches)

LIQUEFACTION SETTLEMENT FROM **50** FEET

0.35 (Inches)

TOTAL SEISMIC SETTLEMENT **0.4** INCHES

POTENTIAL LATERAL DISPLACEMENT

LDI² **0.00** L/H **12.1**

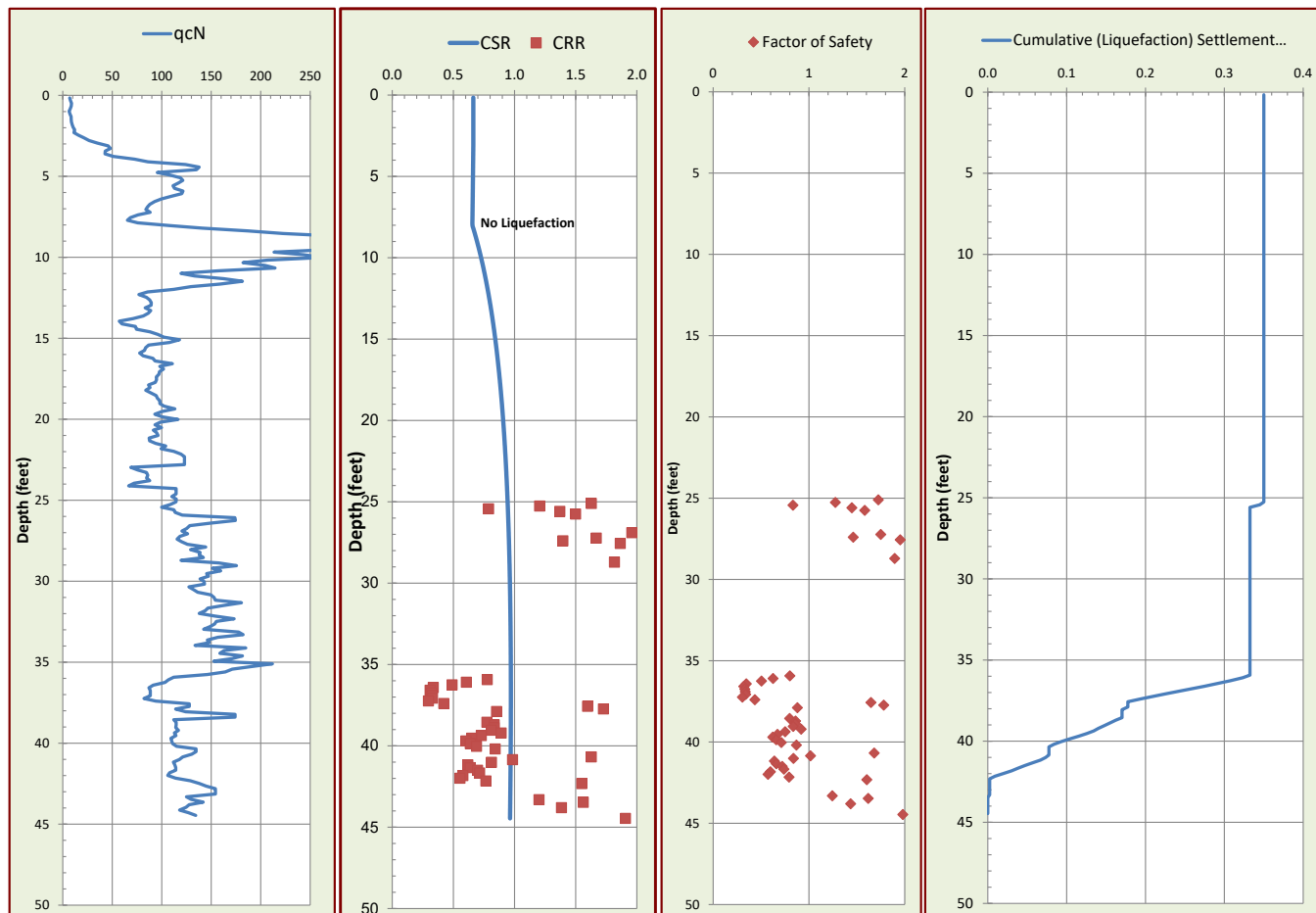
LDI¹ Corrected for Distance **0.00** (4 < L/H < 40)

EXPECTED RANGE OF DISPLACEMENT

0.0 to **0.0** feet

¹Not Valid for L/H Values < 4 and > 40.

²LDI Values Only Summed to 2H Below Grade.



APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using track-mounted, limited-access, hollow-stem auger drilling equipment and track-mounted Cone Penetration Test equipment. Two 6½-inch-diameter exploratory borings were drilled on January 23, 2024 to depths of 25 and 30 feet. Three CPT soundings were also performed in accordance with ASTM D 5778-95 (revised, 2002) on January 16, 2024, to depths of approximately 44½ feet each. The approximate locations of exploratory borings and CPTs are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Boring logs, as well as a key to the classification of the soil and bedrock, are included as part of this appendix.

Boring and CPT locations were approximated using existing site boundaries, a hand-held GPS unit, and other site features as references. Boring and CPT elevations were not determined. The locations of the borings and CPTs should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. The standard penetration resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The 2-inch O.D. split-spoon sampler was driven 18 inches, and the number of blows was recorded for each 6 inches of penetration (ASTM D1586). 2.5-inch I.D. samples were obtained using a Modified California Sampler driven into the soil with the 140-pound hammer previously described. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows required to drive the last 12 inches. The various samplers are denoted at the appropriate depth on the boring logs.

The CPT involved advancing an instrumented cone-tipped probe into the ground while simultaneously recording the resistance at the cone tip (q_c) and along the friction sleeve (f_s) at approximately 5-centimeter intervals. Based on the tip resistance and tip to sleeve ratio (R_f), the CPT classified the soil behavior type and estimated engineering properties of the soil, such as equivalent Standard Penetration Test (SPT) blow count, internal friction angle within sand layers, and undrained shear strength in silts and clays. A pressure transducer behind the tip of the CPT cone measured pore water pressure (u_2). Graphical logs of the CPT data are included as part of this appendix.

Field tests included an evaluation of the unconfined compressive strength of the soil samples using a pocket penetrometer device. The results of these tests are presented on the individual boring logs at the appropriate sample depths.

Attached boring and CPT logs and related information depict subsurface conditions at the locations indicated and on the date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these boring and CPT locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition,

any stratification lines on the logs represent the approximate boundary between soil types and the transition may be gradual.

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-10)

MATERIAL TYPES	CRITERIA FOR ASSIGNING SOIL GROUP NAMES			GROUP SYMBOL	SOIL GROUP NAMES & LEGEND	
COARSE-GRAINED SOILS >50% RETAINED ON NO. 200 SIEVE	GRAVELS >50% OF COARSE FRACTION RETAINED ON NO 4. SIEVE	CLEAN GRAVELS <5% FINES	$Cu>4$ AND $1<Cc<3$	GW	WELL-GRADED GRAVEL	
			$Cu>4$ AND $1>Cc>3$	GP	POORLY-GRADED GRAVEL	
		GRAVELS WITH FINES >12% FINES	FINES CLASSIFY AS ML OR CL	GM	SILTY GRAVEL	
			FINES CLASSIFY AS CL OR CH	GC	CLAYEY GRAVEL	
	SANDS >50% OF COARSE FRACTION PASSES ON NO 4. SIEVE	CLEAN SANDS <5% FINES	$Cu>6$ AND $1<Cc<3$	SW	WELL-GRADED SAND	
			$Cu>6$ AND $1>Cc>3$	SP	POORLY-GRADED SAND	
		SANDS AND FINES >12% FINES	FINES CLASSIFY AS ML OR CL	SM	SILTY SAND	
			FINES CLASSIFY AS CL OR CH	SC	CLAYEY SAND	
FINE-GRAINED SOILS >50% PASSES NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT<50	INORGANIC	$PI>7$ AND PLOTS>"A" LINE	CL	LEAN CLAY	
			$PI>4$ AND PLOTS<"A" LINE	ML	SILT	
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OL	ORGANIC CLAY OR SILT	
	SILTS AND CLAYS LIQUID LIMIT>50	INORGANIC	PI PLOTS >"A" LINE	CH	FAT CLAY	
			PI PLOTS <"A" LINE	MH	ELASTIC SILT	
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OH	ORGANIC CLAY OR SILT	
HIGHLY ORGANIC SOILS		PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC ODOR		PT	PEAT	

OTHER MATERIAL SYMBOLS	
	Poorly-Graded Sand with Clay
	Clayey Sand
	Sandy Silt
	Artificial/Undocumented Fill
	Poorly-Graded Gravelly Sand
	Topsoil
	Well-Graded Gravel with Clay
	Well-Graded Gravel with Silt
	Sand
	Silt
	Well Graded Gravelly Sand
	Gravelly Silt
	Asphalt
	Boulders and Cobble

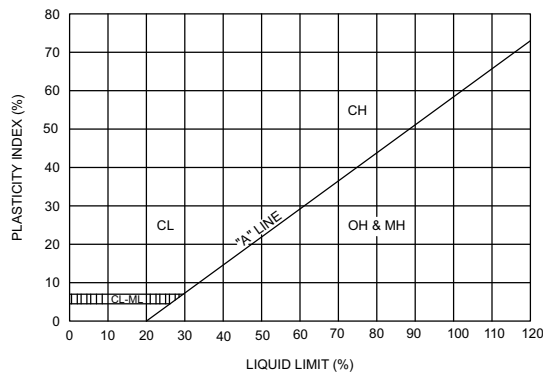
SAMPLER TYPES

	SPT		Shelby Tube
	Modified California (2.5" I.D.)		No Recovery
	Rock Core		Grab Sample

ADDITIONAL TESTS

CA - CHEMICAL ANALYSIS (CORROSIVITY)	PI - PLASTICITY INDEX
CD - CONSOLIDATED DRAINED TRIAXIAL	SW - SWELL TEST
CN - CONSOLIDATION	TC - CYCLIC TRIAXIAL
CU - CONSOLIDATED UNDRAINED TRIAXIAL	TV - TORVANE SHEAR
DS - DIRECT SHEAR	UC - UNCONFINED COMPRESSION
PP - POCKET PENETROMETER (TSF)	(1.5) - (WITH SHEAR STRENGTH IN KSF)
(3.0) - (WITH SHEAR STRENGTH IN KSF)	
RV - R-VALUE	UU - UNCONSOLIDATED UNDRAINED TRIAXIAL
SA - SIEVE ANALYSIS: % PASSING #200 SIEVE	
- WATER LEVEL	

PLASTICITY CHART



PENETRATION RESISTANCE (RECORDED AS BLOWS / FOOT)

SAND & GRAVEL		SILT & CLAY		
RELATIVE DENSITY	BLOWS/FOOT*	CONSISTENCY	BLOWS/FOOT*	STRENGTH** (KSF)
VERY LOOSE	0 - 4	VERY SOFT	0 - 2	0 - 0.25
LOOSE	4 - 10	SOFT	2 - 4	0.25 - 0.5
MEDIUM DENSE	10 - 30	MEDIUM STIFF	4 - 8	0.5 - 1.0
DENSE	30 - 50	STIFF	8 - 15	1.0 - 2.0
VERY DENSE	OVER 50	VERY STIFF	15 - 30	2.0 - 4.0
		HARD	OVER 30	OVER 4.0

* NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1-3/8 INCH I.D.) SPLIT-BARREL SAMPLER THE LAST 12 INCHES OF AN 18-INCH DRIVE (ASTM-1586 STANDARD PENETRATION TEST).

** UNDRAINED SHEAR STRENGTH IN KIPS/SQ. FT. AS DETERMINED BY LABORATORY TESTING OR APPROXIMATED BY THE STANDARD PENETRATION TEST, POCKET PENETROMETER, TORVANE, OR VISUAL OBSERVATION.



CORNERSTONE EARTH GROUP

BORING NUMBER EB-1

PAGE 1 OF 1

PROJECT NAME Halsey HousePROJECT NUMBER 118-161-1PROJECT LOCATION Los Altos, CADATE STARTED 1/23/24 DATE COMPLETED 1/23/24DRILLING CONTRACTOR Cuesta GeoDRILLING METHOD MPP LAD Track Rig, 6½ inch Hollow-Stem AugerLOGGED BY JDS

NOTES _____

GROUND ELEVATION _____ BORING DEPTH 25 ft.LATITUDE 37.3730383° LONGITUDE -122.1196333°**GROUND WATER LEVELS:**▼ AT TIME OF DRILLING 15 ft.▼ AT END OF DRILLING 10 ft.

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)

DEPTH (ft)

SYMBOL

DESCRIPTIONN-Value (uncorrected)
blows per footSAMPLES
TYPE AND NUMBERDRY UNIT WEIGHT
pcfNATURAL
MOISTURE CONTENT

PLASTICITY INDEX, %

PERCENT PASSING
No. 200 SIEVEUNDRAINED SHEAR STRENGTH,
ksf

○ HAND PENETROMETER

△ TORVANE

● UNCONFINED COMPRESSION

▲ UNCONSOLIDATED-UNDRAINED
TRIAXIAL

1.0 2.0 3.0 4.0

>4.5

>4.5



CORNERSTONE EARTH GROUP

BORING NUMBER EB-2

PAGE 1 OF 2

PROJECT NAME Halsey HousePROJECT NUMBER 118-161-1PROJECT LOCATION Los Altos, CADATE STARTED 1/23/24 DATE COMPLETED 1/23/24DRILLING CONTRACTOR Cuesta GeoDRILLING METHOD MPP LAD Track Rig, 6½ inch Hollow-Stem AugerLOGGED BY JDS

NOTES _____

GROUND ELEVATION _____ BORING DEPTH 30 ft.LATITUDE 37.3727601° LONGITUDE -122.1195807°**GROUND WATER LEVELS:**▼ AT TIME OF DRILLING 12 ft.▼ AT END OF DRILLING 12 ft.

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)

DEPTH (ft)

SYMBOL

DESCRIPTION**Sandy Lean Clay (CL) [Fill]**

stiff, moist, brown, fine to coarse sand, low plasticity

Sandy Lean Clay (CL)

stiff, moist, dark brown to brown, fine to coarse sand, low plasticity

Clayey Sand (SC)

medium dense, moist, reddish brown, fine to coarse sand, some fine subangular gravel

Clayey Sand with Gravel (SC)

dense, moist, brown, fine to coarse sand, fine to coarse subangular to subrounded gravel, some cobbles

Clayey Sand (SC)

medium dense, moist, reddish brown, fine to coarse sand, some fine subangular gravel

becomes dense

*Continued Next Page*N-Value (uncorrected)
blows per footSAMPLES
TYPE AND NUMBERDRY UNIT WEIGHT
pcfNATURAL
MOISTURE CONTENT

PLASTICITY INDEX, %

PERCENT PASSING
No. 200 SIEVE

UNDRAINED SHEAR STRENGTH,
ksf

○ HAND PENETROMETER

△ TORVANE

● UNCONFINED COMPRESSION

▲ UNCONSOLIDATED-UNDRAINED
TRIAXIAL

1.0 2.0 3.0 4.0

11

MC-1B

101

17

27

MC-2B

105

10

41

MC-3B

111

13

76

MC-4B

132

7

21

SPT-5

30

41

SPT-6

24

17

40

SPT



PROJECT NAME Halsey House

PROJECT NUMBER 118-161-1

PROJECT LOCATION Los Altos, CA

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf ○ HAND PENETROMETER △ TORVANE ● UNCONFINED COMPRESSION ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL 1.0 2.0 3.0 4.0				
			Clayey Sand (SC) dense, moist, reddish brown, fine to coarse sand, some fine subangular gravel	37	SPT-8		23							
			some thin lenses of Sandy Lean Clay (CL)	35	SPT-9		29							
	30		Bottom of Boring at 30.0 feet.											
	35													
	40													
	45													
	50													
	55													



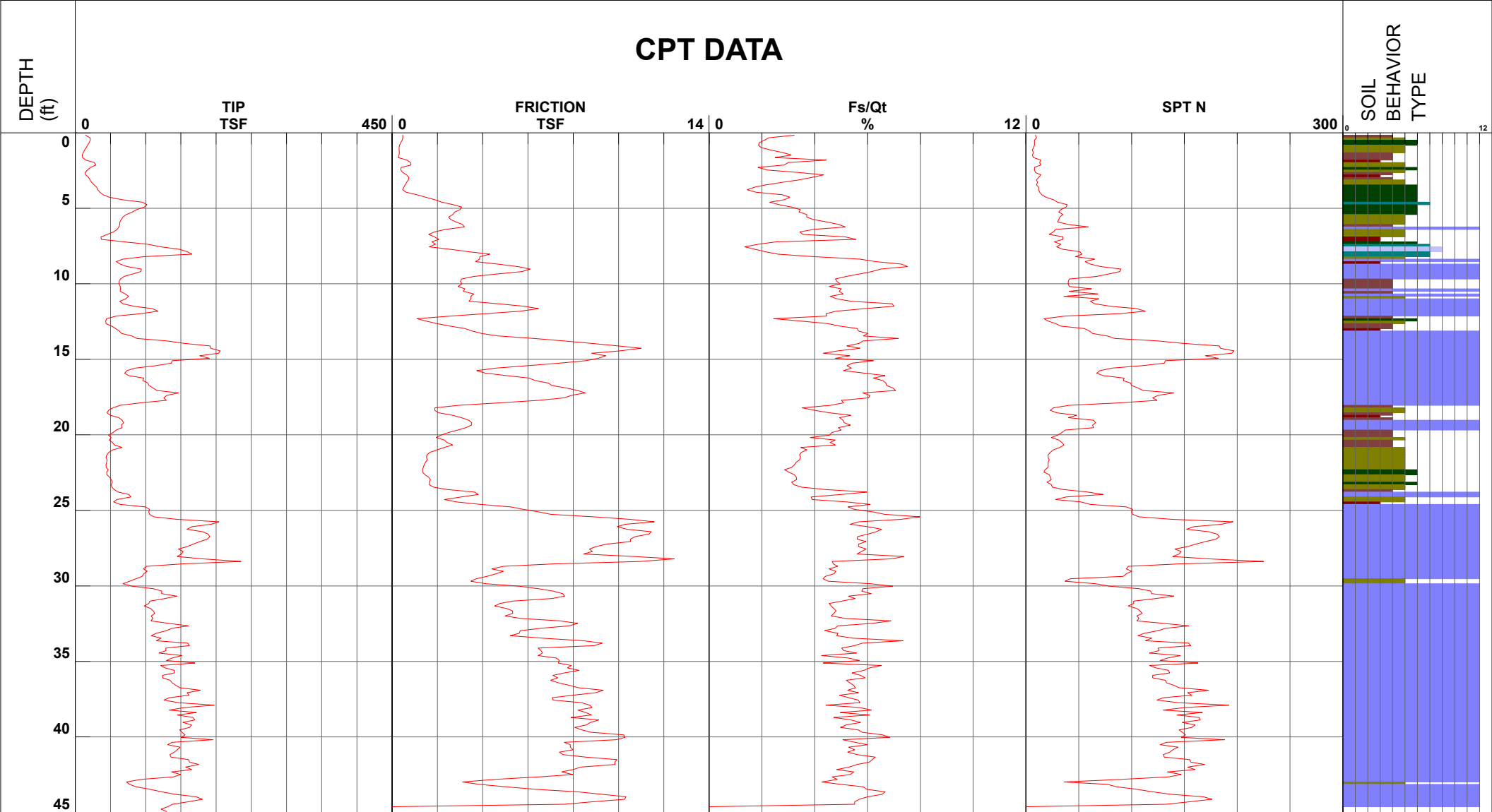
Cornerstone Earth Group

Project Halsey House
Job Number 118-161-1
Hole Number CPT-01
EST GW Depth During Test

Operator JM-FA
Cone Number DDG1596
Date and Time 1/16/2024 11:09:52 AM
NO GW

Filename SDF(279).cpt
GPS
Maximum Depth 44.95 ft

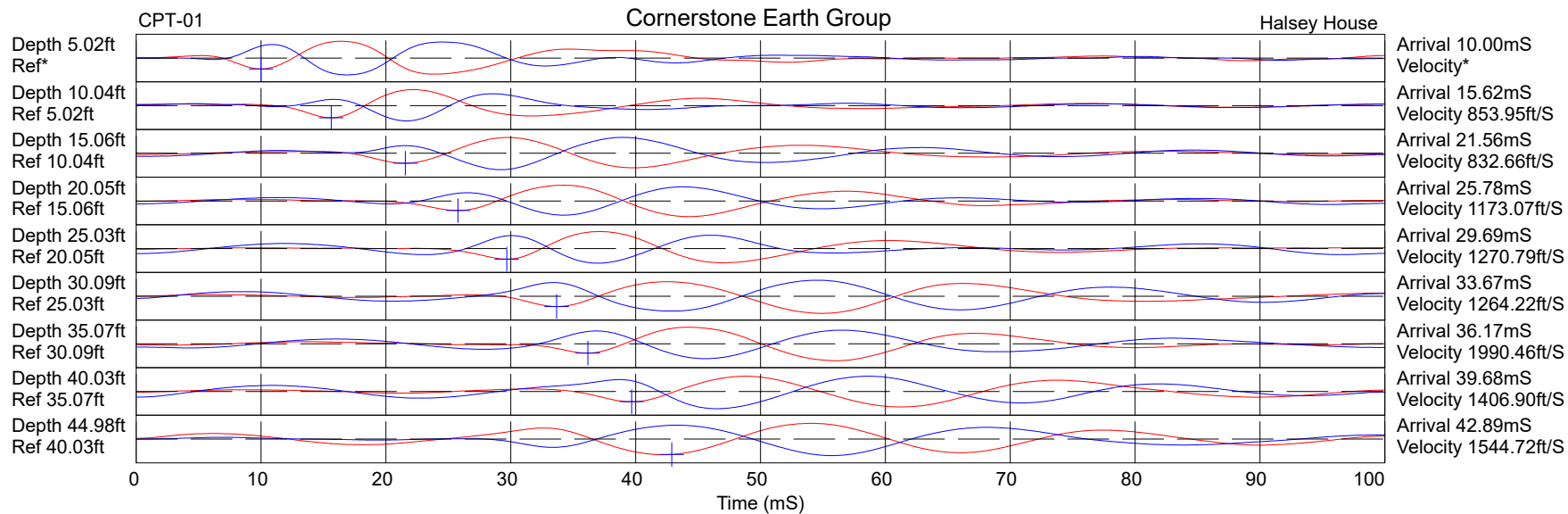
Net Area Ratio .8



- | | | | |
|----------------------------|-------------------------------|------------------------------|----------------------------------|
| 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 15cm²

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



Hammer to Rod String Distance (ft): 2.17
* = Not Determined

COMMENT:



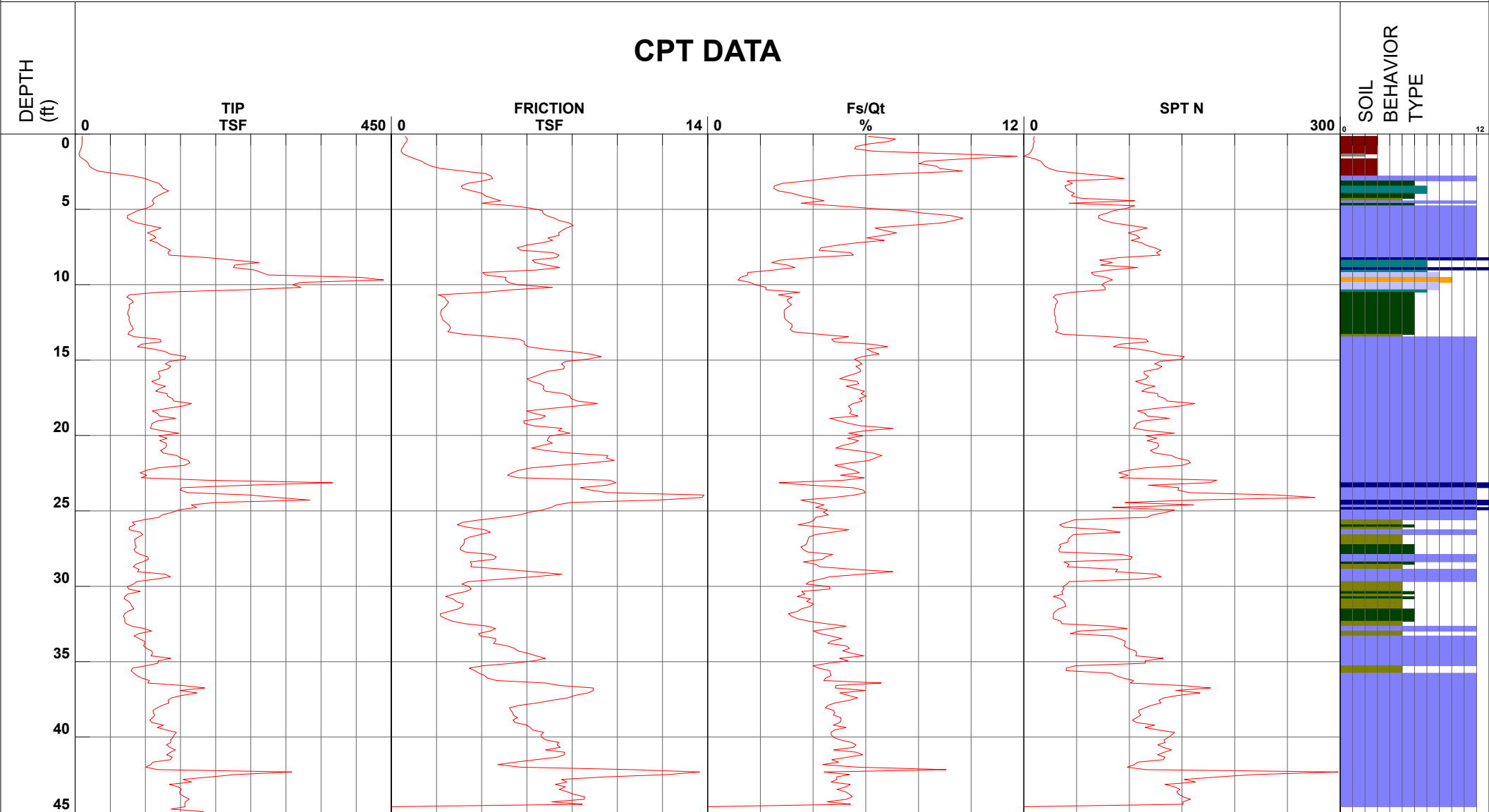
Cornerstone Earth Group

Project Halsey House
Job Number 118-161-1
Hole Number CPT-02
EST GW Depth During Test

Operator JM-FA
Cone Number DDG1596
Date and Time 1/16/2024 12:01:25 PM
NO GW

Filename SDF(280).cpt
GPS
Maximum Depth 44.95 ft

Net Area Ratio .8



- | | | | |
|----------------------------|-------------------------------|------------------------------|----------------------------------|
| 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 15cm²

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



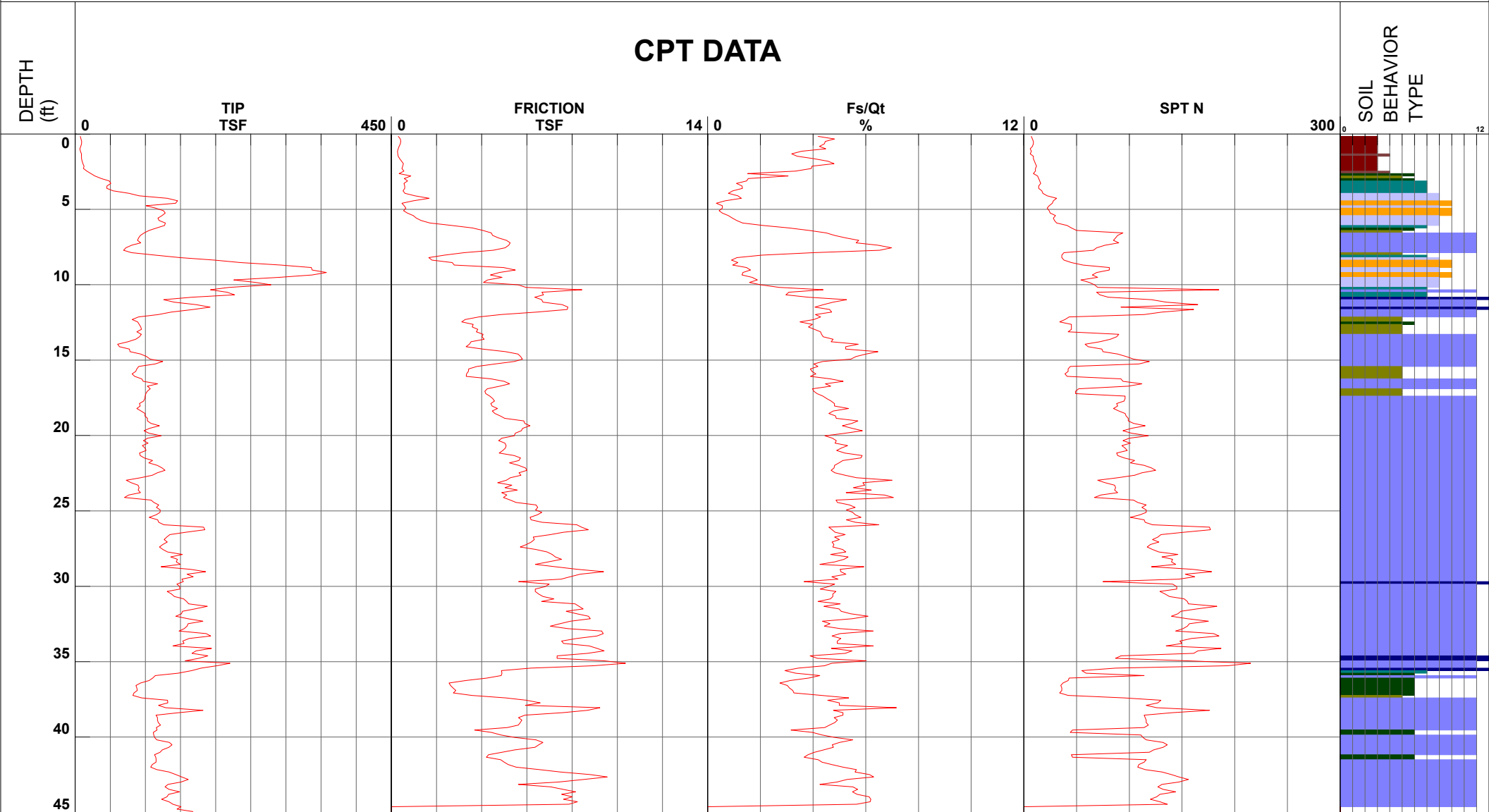
Cornerstone Earth Group

Project Halsey House
Job Number 118-161-1
Hole Number CPT-03
EST GW Depth During Test

Operator JM-FA
Cone Number DDG1596
Date and Time 1/16/2024 10:04:52 AM
NO GW

Filename SDF(278).cpt
GPS
Maximum Depth 44.95 ft

Net Area Ratio .8



- | | | | |
|----------------------------|-------------------------------|------------------------------|----------------------------------|
| 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 15cm²

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

APPENDIX B: LABORATORY TEST PROGRAM

The laboratory testing program was performed to evaluate the physical and mechanical properties of the soils retrieved from the site to aid in verifying soil classification.

Moisture Content: The natural water content was determined (ASTM D2216) on 16 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry Densities: In place dry density determinations (ASTM D2937) were performed on 10 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on two samples of the subsurface soils to aid in the classification of these soils. Results of these tests are shown on the boring logs at the appropriate sample depths.