

August 6, 2024

**PRELIMINARY GEOTECHNICAL INVESTIGATION
HOMESTEAD TOWNHOMES
SANTA CLARA, CALIFORNIA
SFB PROJECT NO. 1073-1**

Prepared For:

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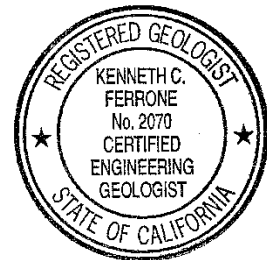
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1.0 INTRODUCTION

This report presents the results of our preliminary geotechnical investigation for the proposed Homestead Townhomes residential development to be located at 3521 & 3591 Homestead Road in Santa Clara, California, as shown on the **Vicinity Map, Figure 1**, and **Site Plan, Figure 2**. The purpose of our preliminary investigation was to evaluate the geological and geotechnical conditions at the site and provide preliminary recommendations regarding the geological and geotechnical engineering aspects of the project for planning and cost estimating purposes.

Based on the project information provided by Waymark Development and the conceptual site plan prepared by CBG and dated May 30, 2024, it is our understanding that the project will consist of re-developing approximately 5.6 acres of urban parcels (APNs 290-23-049 and -053) for about 14 residential townhome buildings that include about 153 dwelling units. The new at-grade buildings will be 3- or 4-story high. Open space areas, paved access roads and parking, and underground utilities are also planned. Nominal grading is anticipated. The actual project development details are to be determined. The existing commercial developments at the site will be demolished prior to the new construction.

The preliminary conclusions and recommendations provided in this report are based upon the information presented above; Stevens, Ferrone & Bailey Engineering Company, Inc. (SFB) should be consulted if any changes to the project occur to assess if the changes affect the validity of this report.

2.0 SCOPE OF WORK

Our preliminary investigation of the site included the following scope of work:

- Reviewing published and unpublished geotechnical and geological literature relevant to the site;
- Reviewing historical topographic maps and aerial images of the site and surrounding area;
- Reviewing available previous environmental excavation records and exploration logs of a former gas station located at the northwestern corner of the site (841 Lawrence Expressway);
- Reviewing the site Phase II environmental site assessment report prepared by Engeo (2024)¹,
- Performing a field reconnaissance of the site and surrounding area;
- Performing a subsurface exploration program to conduct two Cone Penetration Tests (CPTs) to depths of about 70 feet;
- Performing two hand auger borings to a depth of about 5 feet at the CPT locations;
- Performing preliminary engineering analysis of the field and laboratory data; and
- Preparing this report.

The data obtained and the analyses performed were for the purpose of providing feasibility level geological and geotechnical recommendations for preliminary planning and cost estimating purposes. In the future, a design-level geotechnical investigation will be necessary in order to provide detailed geotechnical design and construction criteria as well as to confirm the preliminary recommendations provided in this report.

Evaluating the potential for corrosion and toxicity of onsite soils or groundwater (including mold) were beyond our scope of work. We recommend appropriate experts be consulted regarding environmental aspects of the site and their impacts on the proposed development. In addition, it was also beyond our scope to evaluate the site's flooding potential.

¹Engeo, 2024, Phase II Environmental Site Assessment, 3521-3591 Homestead Road, Santa Clara, California, Project No. 21272.000.001, February 2.

3.0 SITE INVESTIGATION

3.1 Field Exploration

Our geotechnical field exploration program for the project included two Cone Penetration Tests (CPTs) to a maximum depth of about 70 feet on August 2, 2024. The approximate locations of the CPTs are shown on the **Site Plan, Figure 2**. The CPTs were performed by Gregg Drilling/ Pitcher Services of East Palo Alto, California, using a truck mounted CPT rig with an electronic piezocone penetrometer that has a tip area of 15 cm², a friction sleeve area of 225 cm², and a tip end area ratio of 0.75. The CPT results and an interpretation report prepared by Gregg Drilling/ Pitcher Services are included in **Appendix A**.

In addition, our field engineer logged the upper 5 feet of soils encountered during hand augering at the CPT locations. The soils are classified in general accordance with the Unified Soil Classification System (ASTM D2487 and D2488). Logs of the hand augers as well as keys for the classification of the soil (**Figure A-1**) are also included in **Appendix A**. Upon completion of our field explorations, the CPTs and hand augers were backfilled with cement grout in accordance with Santa Clara Valley Water District (Valley Water) requirements.

The approximate locations of our CPTs and hand augers were determined by pacing, tape measurements, and/or alignment from landmark references. Latitude and longitude of exploration locations shown on the exploration logs are estimated from online map data from Microsoft; actual locations were not surveyed.

Previously, an environmental boring/monitoring well to about 96-1/2 feet deep was performed by Gen-Tech Environmental² in 1993 near the northwestern corner of the site (as shown on **Figure 2**). Logs of the previous boring/monitoring well are included in **Appendix B** for reference.

It should be noted that changes in the surface and subsurface conditions can occur over time as a result of either natural processes or human activity and may affect the validity of the conclusions and recommendations in this report. In addition, our attached exploration logs and related information show our interpretation of the subsurface conditions at the dates and locations indicated, and it is not warranted that they are representative of subsurface conditions at other locations and times.

²Gen-Tech Environmental, 1993, Phase One and Phase Two Assessment, Site at 841 Lawrence Expressway, Santa Clara, CA, File No. 9313, February 24.

3.2 Surface Conditions and Site Development History

As shown on **Figure 2** and at the time of our investigation, the site was bounded by Homestead Road on the south, Lawrence Expressway on the west, Lochinvar Avenue on the north, and existing commercial and residential developments on the east.

The site was approximately rectangular in shape, generally level, and had a plan area of about 5.6 acres with maximum dimensions of about 605 feet by 410 feet. The site, except for the northwestern corner, was occupied by commercial developments that included several one-story buildings, asphalt concrete paved parking lots and drive aisles, and other associated facilities. The existing asphalt concrete pavements were generally in a fair to poor condition. According to our hand auger results, we estimate the pavement sections at our CPT/hand auger locations consist of about 3 inches asphalt concrete over about 5 inches of aggregate base. Some large and small diameter trees were also located within the site. The northwestern corner of the site was vacant and the surface was covered with a heavy growth of grasses and weeds.

Based on our review of historical topographic maps and aerial photographs of the site and vicinity, the existing commercial developments were built in the 1960s and 1970s. A gas station (that included a canopy and fueling islands, an auto repair shop building, and underground storage tanks and yard piping) was previously located at the currently vacant, northwestern corner of the site and was removed in 1992. According to Gen-Tech Environmental (1993), a large excavation was performed within the former gas station site to depths of about 11 to 26 feet to remove contaminated soils below the previous underground storage tanks, and the excavation was later backfilled and compacted. In addition, a 500-gallon waste oil tank that extended to about 8-1/2 feet deep was removed in 1988 near the northeastern corner of the previous auto repair shop building. However, there is no excavation backfill and compaction test records available regarding these previous excavations. Based on the available reports and documents we reviewed, the approximate lateral extent of the previous excavations are shown on **Figure 2** for reference.

3.3 Subsurface Conditions

The results of our CPTs and hand augers generally indicate the site, below pavement sections, is generally underlain by stiff to very stiff clays with interbeds of medium dense to very dense sand and gravel lenses that extend to the maximum depth explored of about 70 feet. Similar subsurface soil conditions were also reportedly encountered in the previous environmental boring/monitoring well by others to a maximum depth explored of about 96-1/2 feet.

The previous environmental excavation backfills of about 8-1/2 to 26 feet at the northwestern corner of the site are potentially heterogeneous, weak, and compressible if they were not placed and compacted in accordance with acceptable engineering standards.

The onsite surficial clayey soils have a medium to high plasticity and a moderate to high volumetric expansion and shrinkage potential. Detailed information and interpretations of soils encountered by our hand auguring and CPTs are presented in **Appendix A**. The previous environmental excavation records and logs of the previous boring/monitoring well are included in **Appendix B** for reference.

3.4 Groundwater

Based on the results of our CPT pore pressure dissipation tests, the groundwater level was estimated to be at depths of about 30-1/2 to 37-1/2 feet below existing ground surface. According to Engeo (2024), groundwater was encountered at depths of about 45-1/2 to 48 feet during their groundwater sampling in June 2023 for the Phase II environmental site assessment.

According to CGS (2002)³, the historically high groundwater levels in the site region are at depths of about 30 to 35 feet. Fluctuations in the groundwater level could occur due to change in seasons, variations in rainfall, pumping of the nearby wells, and other factors. The depths to groundwater below existing ground surface within the site may vary greatly.

3.5 Hydrologic Soil Group

The surface soils of the site have been mapped by the USDA Natural Resource Conservation Services (NRCS) Web Soil Survey (WSS)⁴ and categorized as Urbank land – Stevens Creek complex, 0 to 2 percent slopes (Unit 135). This unit has been assigned to Hydrologic Soil Group C and is estimated to have moderately high transmission rates (approximately 0.2 to 0.57 inches per hour). Group C soils are defined as having a slow infiltration rate when thoroughly wet and consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture.

Actual field infiltration rates will depend on the in-situ soil type, moisture, relative density, gradation, and fines content of soils, whether any water impeding clay layers exist at shallow depth, and proper and regular maintenance of the infiltration facilities. If needed, we recommend field Double Ring Infiltrometer Tests (ASTM D3385) be performed at the potential infiltration depths to evaluate the field infiltration rates.

³California Geological Survey, 2002, Seismic Hazard Zone Report for the San Jose West 7.5-Minute Quadrangle, Santa Clara County, California, Seismic Hazard Zone Report 058.

⁴USDA NRCS, <https://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx>, accessed 8/5/2024.

Sand and gravel layers with higher infiltration rates may exist below the site at shallow depths in some areas. Compacted/engineered fills should be considered as having limited infiltration ability.

3.6 Geology and Seismicity

According to Wentworth, et al. (1999)⁵, the site (below the pavement section and surficial fills if any) is underlain by Holocene older alluvial fan deposits that compose of brown gravelly sand and sandy and clayey gravel, grading upward to sandy and silty clay. The subsurface soil conditions encountered at the site are consistent with the regional geologic mapping.

The project site is in the San Francisco Bay Area, which is considered one of the most seismically active regions in the United States. Significant earthquakes have occurred in the region and are believed to be associated with crustal movements along a system of sub-parallel fault zones that generally trend in a northwesterly direction. The site is not located within an Alquist-Priolo Earthquake Fault Zone as designated by the State of California⁶. In addition, according to Santa Clara County Geologic Hazard Zones Map No. 19 (2012)⁷, the site is also not located in a fault rupture hazard zone as designated by the County. Therefore, it is our opinion that the potential for ground surface rupture due to a fault crossing the site is low.

Earthquake intensities will vary throughout the region, depending upon numerous factors including the magnitude of earthquake, the distance of the site from the causative fault, and the type of materials underlying the site. The U.S. Geological Survey (2016)⁸ indicated that there is a 72 percent chance of at least one magnitude 6.7 or greater earthquake striking the San Francisco Bay region between 2014 and 2043. Therefore, the site will be subjected to earthquakes that cause strong ground shaking.

According to 2022 CBC/ASCE 7-16, the site modified geometric mean peak ground acceleration (PGA_M) from a Maximum Considered Earthquake (MCE) event is estimated to be about 0.69g based on a stiff soil condition (Site Class D). The MCE peak ground acceleration generally has a 2% probability of being exceeded in 50 years (a mean return period of 2,475 years) except where deterministically capped along highly active faults.

⁵Wentworth, Blake, McLaughlin, and Graymer, 1999, Preliminary Geologic Map of the San Jose 30 x 60-Minute Quadrangle, California, U.S. Geological Survey Open-File Report 98-795.

⁶California Department of Conservation, Earthquake Fault Zones, CGS Special Publication 42, Revised 2018.

⁷County of Santa Clara, Geologic Hazard Zones, Version: October 26, 2012.

⁸Aagaard, Blair, Boatwright, Garcia, Harris, Michael, Schwartz, and DiLeo, Earthquake Outlook for the San Francisco Bay Region 2014–2043, USGS Fact Sheet 2016–3020, Revised August 2016 (ver. 1.1).

According to the U.S. Geological Survey's Unified Hazard Tool and applying the Dynamic: Conterminous U.S. 2014 model (v4.2.0)⁹, the resulting deaggregation calculations indicate that the site has a 10% probability of exceeding a peak ground acceleration of about 0.55g in 50 years (a design earthquake ground motion based on a Site Class D with a mean return period of 475 years). The actual ground surface acceleration might vary depending upon the local seismic characteristics of the underlying bedrock and the overlying soils.

3.7 Liquefaction

Soil liquefaction is a phenomenon primarily associated with saturated cohesionless soil layers. These soils can dramatically lose strength due to increased pore water pressure during cyclic loading, such as imposed by earthquakes. During the loss of strength, the soils acquire mobility sufficient to permit both horizontal and vertical movements. Soils that are most susceptible to liquefaction are clean, loose, uniformly graded, saturated sands that lie close to the ground surface; although, liquefaction can also occur in fine-grained soils, such as low-plasticity silts.

According to Seismic Hazard Zones Map of the San Jose West Quadrangle (2002)¹⁰, the site is not located in a liquefaction seismic hazard zone as designated by the State of California. In addition, according to Santa Clara County Geologic Hazard Zones Map No. 19 (2012), the site is also not located in a liquefaction hazard zone as designated by the County. The site and surrounding areas are mapped by Witter et al. (2006)¹¹ as being within an area having a moderate susceptibility to liquefaction hazard.

Based on our review of available geologic literature, and the results of subsurface explorations, it is our opinion that the potential for ground surface damage at the site resulting from liquefaction is low since the sandy and gravelly soils below the site are generally dense to very dense in consistency (with CPT tip resistances values of more than 200 ton per square foot) and are not susceptible to soil liquefaction.

⁹USGS Unified Hazard Tool, <https://earthquake.usgs.gov/hazards/interactive/>, accessed 8/5/2024.

¹⁰State of California, 2002, Seismic Hazard Zones, San Jose West Quadrangle, Official Map, released: February 7, 2002.

¹¹Witter, Knudsen, Sowers, Wentworth, Koehler, and Randolph, 2006, Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region, California, U.S. Geological Survey Open File Report 2006-1037.

4.0 PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

Based on the results of the preliminary geotechnical investigation, it is our opinion that the site is suitable for the proposed project from a preliminary geotechnical engineering standpoint. The following are the primary geological and geotechnical considerations for use in the preliminary planning, cost estimating, and design of the development.

WEAK SOIL AND FILL MATERIALS: We estimate that demolition of existing improvements and structures (and their associated foundations) will disturb and weaken the upper 2 to 3 feet of surface soils at the site. In addition, as described previously, the previous environmental excavation backfills of about 8-1/2 to 26 feet at the northwestern corner of the site are potentially heterogeneous, weak, and compressible if they were not placed and compacted in accordance with acceptable engineering standards. Based on the available reports and documents we reviewed, the approximate lateral extents of the previous excavations are shown on **Figure 2** for reference.

In order to reduce the potential for damaging differential settlement of overlying improvements (such as new fills, building foundations, roadways, exterior flatwork, and pavements), we recommend that these weak soils and fills/backfills be completely over-excavated and re-compacted. The over-excavation should extend to depths where competent soils are encountered. Over-excavation and re-compaction should extend at least 5 feet beyond future building footprints and at least 3 feet beyond exterior flatwork (including driveways) and pavement wherever possible. There would be no need to over-excavate and re-compact the soils or fills/backfills within areas that do not support improvements, such as within planned open space areas where ground instability or settlement is less a concern. Considerations can be given to place open spaces at the northwestern corner of the site where the former gas station and previous environmental excavations were located. We recommend new buildings and improvements be setback from an imaginary 1:1 (horizontal to vertical) plane extending upward from the bottom edges of the previous undocumented excavation backfills if they will be left in place.

Where the over-excavation limits abut adjacent property, SFB should be consulted to determine the actual vertical and lateral extent of over-excavation so that adjacent property is not adversely impacted. Over-excavations should be performed so that no more than 5 feet of differential fill thickness exists below proposed building foundations.

The extent of the removal and re-compaction may vary across the site and should be determined in the field by SFB at the time of the earthwork operation. The removed soils and fills can be used as new fill provided they are placed and compacted in accordance with geotechnical engineering recommendations presented by SFB.

EXPANSION POTENTIAL: The onsite more clayey, moderately to highly expansive soil materials will be subjected to volume changes during seasonal fluctuations in moisture content. To reduce the potential for post-construction distress to the proposed structures resulting from swelling and shrinkage of these materials, we recommend that the proposed buildings be supported on post-tensioned slab foundations that are designed to reduce the impact of the onsite expansive soils. We recommend at least 3-feet of well-blended, moisture conditioned, engineered fills be provided below the post-tensioned slab foundations. This 3-foot engineered fill layer should extend at least 5 feet beyond building footprints and at least 3 feet beyond exterior flatwork (including driveways). It should be noted that special design considerations will be required for exterior slabs.

TEMPORARY EXCAVATION SLOPE AND SHORING: If needed during the weak soil and fill over-excavation and backfill, temporary slope and shoring should be designed and implemented by specialty contractors who are familiar with working in the site subsurface conditions. The shoring system, if used, should be sufficiently rigid to reduce deflection of the sides of the excavation to prevent damage to the adjacent grounds and improvements. All excavations must comply with applicable local, state, and federal safety regulations, including OSHA requirements.

FUTURE DESIGN-LEVEL GEOTECHNICAL ENGINEERING REPORT: In order to provide detailed geotechnical engineering design and construction criteria for the project, we recommend additional borings be performed at the site. Soil samples should be retrieved for additional laboratory testing and analyses. The results of all subsurface explorations and laboratory testing will require thorough geotechnical engineering analyses, and a design-level geotechnical investigation report could then be prepared. This future report would provide all the necessary geotechnical criteria needed to design and construct the project.

4.1 Earthwork

The site should be cleared of all obstructions, including existing structures and their entire foundation systems, existing utilities and pipelines and their associated backfill, designated trees and their associated entire root systems, and debris. Holes resulting from the removal of underground obstructions extending below the proposed finished grade should be cleared and backfilled with compacted fill materials. Tree roots may extend to depths of about 3 to 4 feet. Wells and septic systems (if any) should be removed/abandoned in accordance the Santa Clara County requirements.

From a geotechnical engineering standpoint, any existing fill materials, trench backfill materials, clay or concrete pipes, and concrete that are removed can be used as new fill onsite provided debris is removed and it is broken up it is broken up to meet the size requirement for fill material. Crushed

concrete materials from building demolition can be re-used onsite as aggregate base or subbase if they meet current Caltrans specifications for aggregate base or subbase based on laboratory testing results. After clearing, areas containing heavy surface vegetation should be stripped to an appropriate depth to remove these materials.

Onsite soils and fills having an organic content of less than 3 percent by volume can be used as fill. Fill should not contain rocks or lumps larger than 6 inches in greatest dimension with not more than 15 percent larger than 2.5 inches. If needed, imported fill for general use should have a plasticity index of 20 or less and have a significant fines content.

Within the upper 5 feet of the finished ground surface, we recommend structural fill be compacted to between 88 to 92 percent relative compaction, and structural fill below a depth of 5 feet be compacted to at least 95 percent relative compaction, as determined by ASTM D1557 (latest edition). We recommend the new fill be moisture conditioned approximately 3 to 5 percent over optimum water content. The upper 6 inches of subgrade soils beneath pavements should be compacted to at least 95 percent relative compaction. Fill material should be spread and compacted in lifts not exceeding approximately 8 to 12 inches in uncompacted thickness.

Pipeline trenches should be backfilled with fill placed in lifts of approximately 8 to 12 inches in uncompacted thickness. Thicker lifts can be used provided the method of compaction is approved by SFB and the required minimum degree of compaction is achieved. Backfill should be placed by mechanical means only. Jetting is not permitted. Onsite trench backfill should be compacted to at least 90 percent relative compaction. Imported sand trench backfill should be compacted to at least 95 percent relative compaction. The upper 3 feet of trench backfill in foundation, slab, and pavement areas should be entirely compacted to at least 95 percent relative compaction.

We recommend that exterior slabs (including patios, sidewalks, walkways, and driveways) be placed directly on the properly compacted fills. We do not recommend using aggregate base, gravel, or crushed rock below these improvements. If imported granular materials are placed below these elements, subsurface water can seep through the granular materials and cause the underlying soils to saturate, heave, or pipe. Prior to placing concrete, subgrade soils should be moisture conditioned to increase their moisture content to approximately 3 to 5 percent above laboratory optimum moisture (ASTM D-1557). We recommend reinforcing exterior slabs with steel bars in lieu of wire mesh.

4.2 Foundation Support

Proposed residential buildings can be supported on post-tensioned slab foundations designed based on the expansion potential of the onsite soils. Post-tensioned slab foundations should be at least 10 inches in thickness; the actual slab thickness should be designed by the Structural Engineer.

Based on our experience, we estimate the post-tensioned slab foundations will be no greater than 12 inches thick. The project's Structural Engineer should determine the actual slab thickness. Post-tensioned slabs can be constructed directly on-grade without the need for imported fill or gravel below the post-tensioned slabs.

At least 10 feet of cover should be provided between the outer face of slabs and un-retained slope faces, as measured laterally between slope faces and the slabs. Where less than 10 feet of cover exists, deepening of the edge of slabs may be necessary in order to achieve 10 feet of cover for buildings located near tops of slopes. Where slabs are located adjacent to utility trenches, the slab bearing surface should bear below an imaginary 1:1 (horizontal to vertical) plane extending upward from the bottom edge of the adjacent utility trench. Alternatively, the slab reinforcing could be increased to span the area defined above assuming no soil support is provided.

A vapor retarder must be placed between the subgrade soils and the bottom of post-tensioned slabs. We recommend the vapor retarder consist of a single layer of Stego Wrap Vapor Barrier 15 mil Class A or equivalent. There is no need to place imported gravel or sand below the vapor retarder.

In order to reduce the potential for vapor transmission through interior, concrete slabs-on-grade, we recommend the concrete mix design for the slabs have a maximum water/cement ratio of 0.45. If a higher water/cement ratio is being considered, we recommend higher vapor transmission be taken into account in the design and construction of the structures. The actual water/cement ratio may need to be reduced if the concentration of soluble sulfates or chlorides in the supporting subgrade is detrimental to the concrete and/or reinforcing steel.

4.3 Retaining Walls

Where walls retain soil, they must be designed to resist both lateral earth pressures and any additional lateral loads caused by surcharging such as building and roadway loads. For retaining walls that need to resist earthquake induced lateral loads from nearby foundations, walls that are to be designed to resist earthquake loads, and any retaining walls that are higher than 6 feet (as required by the 2022 California Building Code), we recommend the walls also be designed to resist seismic pressures developed from a design basis earthquake. Some movement of the walls may occur during moderate to strong earthquake shaking and may result in distress as is typical for all structures subjected to earthquake shaking. Walls should be fully back-drained to prevent the build-up of hydrostatic pressures.

Global stability evaluations should be performed for retaining walls at critical locations, such as at multi-tiered wall locations, where walls are located on slopes, and where walls are subjected to significant surcharge loads.

Retaining walls and soundwalls can be supported on drilled, cast-in-place, straight shaft friction piers that develop their load carrying capacity in the soils underlying the site. Alternatively, walls can be supported by footing foundations.

4.4 Pavement

The onsite surficial clayey soils and fills are estimated to have an R-values of 5 due to their expansion potential. We anticipate that roadway sections (Traffic Index of 5.0) will consist of approximately 3 inches of asphalt concrete over 10 inches of Caltrans Class 2 baserock. Collector roadways will have thicker pavement sections; the actual pavement thickness may depend upon the traffic indices and/or minimum pavement section thicknesses required by governing agencies.

5.0 CONDITIONS AND LIMITATIONS

Future subsurface exploration, laboratory testing, and geotechnical engineering analyses will need to be performed in order to provide detailed geotechnical design and construction criteria for the project and to confirm the preliminary recommendations provided above. The future report should include detailed drainage, earthwork, building foundation, retaining wall and soundwall, flatwork, and pavement recommendations for use in the design and construction of the project. Once the future, detailed investigation is complete, we recommend SFB review the project's designs and specifications to verify that the recommendations presented in the future report have been properly interpreted and implemented in the design, plans, and specifications. We also recommend SFB be retained to provide consulting services and to perform construction observation and testing services during the construction phase of the project to observe and test the implementation of our recommendations, and to provide supplemental or revised recommendations in the event subsurface conditions are different than those described in our reports are encountered. We assume no responsibility for misinterpretation of our recommendations if we do not review the plans and specifications and are not retained during construction.

SFB is not responsible for the validity or accuracy of information, analyses, test results, or designs provided to SFB by others or prepared by others. The analysis, designs, opinions, and recommendations submitted in this report are based in part upon the data obtained from our field work and upon information provided by others. Site exploration and testing characterizes subsurface conditions only at the locations where the explorations or tests are performed; actual subsurface conditions between explorations or tests may be different than those described in this report. Variations of subsurface conditions from those analyzed or characterized in this report are not uncommon and may become evident during construction. In addition, changes in the condition of the site can occur over time as a result of either natural processes (such as earthquakes, flooding, or changes in ground water levels) or human activity (such as construction adjacent to the site, dumping of fill, or excavating). If changes to the site's surface or subsurface conditions occur since the performance of the field work described in this report, or if differing subsurface conditions are encountered, we should be contacted immediately to evaluate the differing conditions to assess if the opinions, conclusions, and recommendations provided in this report are still applicable or should be amended.

This report is a document that has been prepared in accordance with generally accepted geological and geotechnical engineering practices for the exclusive use of Waymark Development and their partners and consultants for specific application to the proposed Homestead Townhomes residential development project located at 3521 & 3591 Homestead Road in Santa Clara,

California. This report is intended to only represent our preliminary recommendations to Waymark Development for the project preliminary planning and cost estimating purposes.

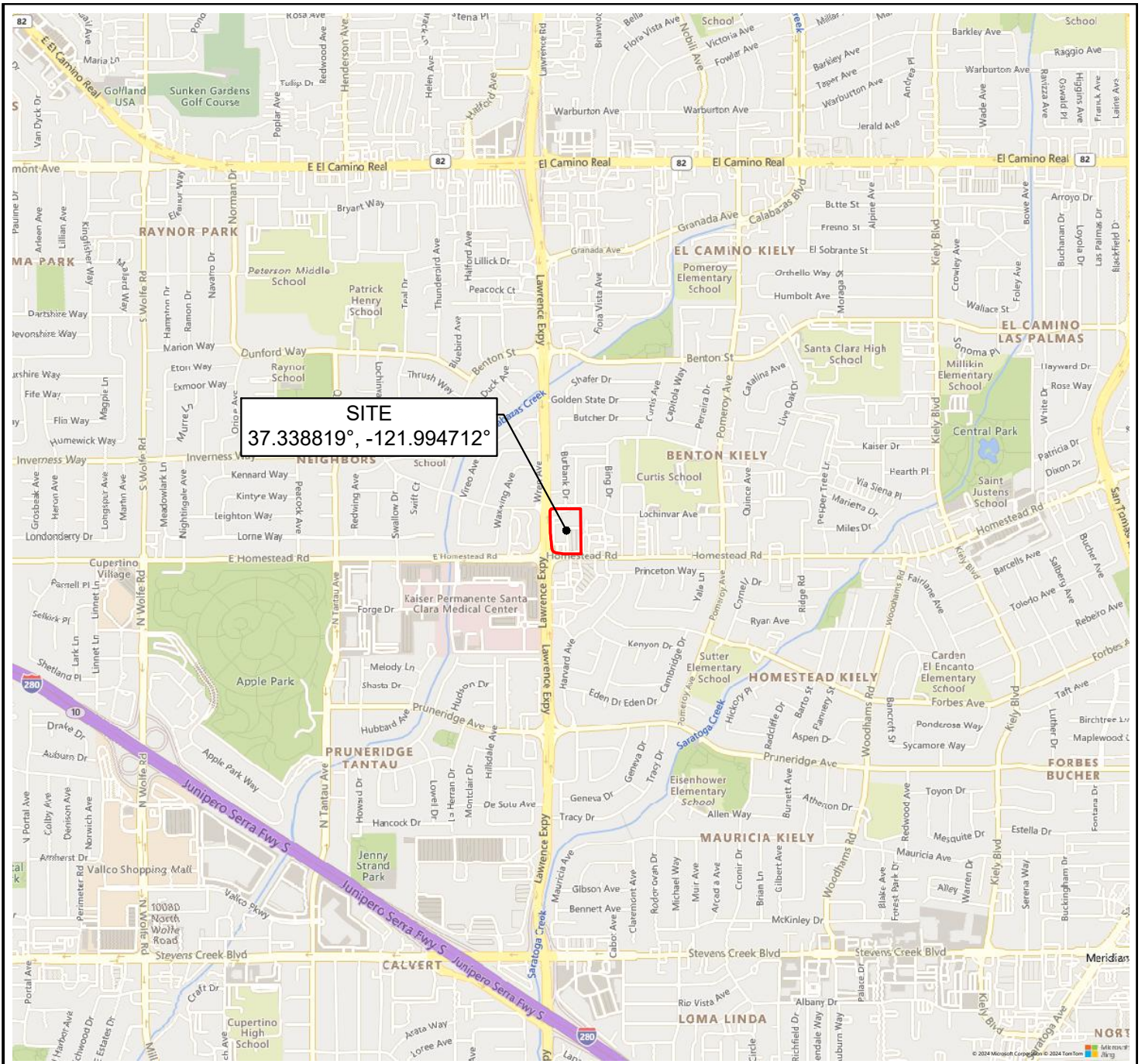
It should be understood that advancements in the practice of geotechnical engineering and engineering geology, or discovery of differing surface or subsurface conditions, may affect the validity of this report and are not uncommon. SFB strives to perform its services in a proper and professional manner with reasonable care and competence but we are not infallible. Geological engineering and geotechnical engineering are disciplines that are far less exact than other engineering disciplines; therefore, we should be consulted if it is not completely understood what the limitations to using this report are.

In the event that there are any changes in the nature or location of the project, as described in this report, the preliminary conclusions and recommendations contained in this report shall not be considered valid unless we are contacted in writing, the project changes are reviewed by us, and the preliminary conclusions and recommendations presented in this report are modified or verified in writing. The opinions, conclusions, and recommendations contained in this report are based upon the description of the project as presented in the introduction section of this report.

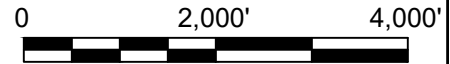
This report does not necessarily represent all of the information that has been communicated by us to Waymark Development and their consultants during the course of this engagement and our rendering of professional services to Waymark Development. Reliance on this report by parties other than those described above must be at their own risk unless we are first consulted as to the parties' intended use of this report and only after we obtain the written consent of Waymark Development to divulge information that may have been communicated to Waymark Development. We cannot accept consequences for use of segregated portions of this report.

Please refer to **Appendix C** for Geoprofessional Business Association (GBA) guidelines regarding use of this report.

FIGURES




SCALE: 1" = 2,000'








NOTE: Basemap from Microsoft/TomTom 2024.

1073-1, Homestead Townhomes, Santa Clara - August 2024.dwg
Printed: 8/1/2024

<table border="1"> <tr> <td>DATE</td> <td>August 2024</td> </tr> <tr> <td>PROJECT NO.</td> <td>1073-1</td> </tr> </table>	DATE	August 2024	PROJECT NO.	1073-1	 <p>1600 Willow Pass Court Concord, CA 94520 Tel 925.688.1001 Fax 925.688.1005 www.SFandB.com</p>	<p>VICINITY MAP</p> <p>HOMESTEAD TOWNHOMES Santa Clara, California</p>	<p>FIGURE</p> <p>1</p>
DATE	August 2024						
PROJECT NO.	1073-1						



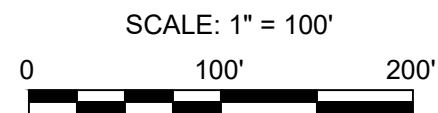
KEY

-  CPT by SFB (8/2/2024)
-  Environmental Boring & Monitoring Well by Gen-Tech Environmental (1993)
-  Estimated Extent of 1992 Environmental Excavation and Backfill
-  Estimated Location of 1988 Waste Oil Tank Removal and Backfill
-  Approximate Project Limit

NOTE: All locations shown are approximate.



NOTE: Basemap from the project conceptual site plan prepared by CBG and dated 5/30/2024. Aerial photo imagery from Microsoft 2024.



DATE	August 2024
PROJECT NO.	1073-1

Stevens
Serrone &
Bailey
Engineering Company, Inc

1600 Willow Pass Court
Concord, CA 94520
Tel 925.688.1001
Fax 925.688.1005
www.SFandB.com

SITE PLAN

HOMESTEAD TOWNHOMES
Santa Clara, California

FIGURE

2

APPENDIX A
Field Exploration

KEY TO FIELD EXPLORATION LOGS

PROJECT:

HOMESTEAD TOWNHOMES

Santa Clara, California

PROJECT NO: **1073-1**

FIGURE NO: **A-1**

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D2487 & D2488)

MAJOR DIVISIONS		GRAPHIC LOG	GROUP SYMBOL	TYPICAL DESCRIPTION	MAJOR DIVISIONS	GRAPHIC LOG	GROUP SYMBOL	TYPICAL DESCRIPTION	
COARSE-GRAINED SOILS (More than 50% retained on #200 sieve)	CLEAN GRAVELS (Less than 5% fines)		GW	Well-graded gravels, gravel-sand mixtures, trace or no fines	FINE-GRAINED SOILS (More than 50% passes #200 sieve)		ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands, clayey silts of low to medium plasticity	
			GP	Poorly-graded gravels, gravel-sand mixtures, trace or no fines			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	
	GRAVELS WITH FINES (More than 12% fines)		GM	Silty gravels, gravel-sand-silt mixtures			OL	Organic silts and clays of low plasticity	
			GC	Clayey gravels, gravel-sand-clay mixtures			MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts of high plasticity	
		CLEAN SANDS (Less than 5% fines)		SW			Well-graded sands, gravelly sands, trace or no fines	CH	Inorganic clays of high plasticity, fat clays
			SP	Poorly-graded sands, gravelly sands, trace or no fines			OH	Organic silts and clays of medium to high plasticity	
	SANDS WITH FINES (More than 12% fines)		SM	Silty sands, sand-silt mixtures		HIGHLY ORGANIC SOILS		PT	Peat and other highly organic soils
			SC	Clayey sands, sand-clay mixtures					

GRAIN SIZES

U.S. STANDARD SIEVE SIZE

BOULDERS		COBBLES		GRAVELS		SANDS			SILTS AND CLAYS	
12"		3"		3/4"		#4		#10	#40	#200
				Coarse	Fine	Coarse	Medium	Fine		
304.8 mm		76.2 mm		19.0 mm	4.75 mm	2.00 mm	0.425 mm	0.075 mm		

RELATIVE DENSITY

SANDS AND GRAVELS	BLOWS/FOOT*
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	Over 50

CONSISTENCY

SILTS AND CLAYS	BLOWS/FOOT*	UCS (KSF)**
Very Soft	0 - 2	0 - 1/2
Soft	2 - 4	1/2 - 1
Firm	4 - 8	1 - 2
Stiff	8 - 16	2 - 4
Very Stiff	16 - 32	4 - 8
Hard	Over 32	Over 8

*Number of blows for a 140-pound hammer falling 30 inches to drive a 2" O.D. (1-3/8" I.D.) split spoon sampler.

**UCS: Unconfined Compressive Strength.

SYMBOLS AND NOTES

Standard Penetration Test (SPT) Sampler (2" O.D. Split Barrel)

Shelby Tube

Groundwater Level During Drilling

Modified California Sampler (3" O.D. Split Barrel)

Bulk Sample

Groundwater Level at End of Drilling

California Sampler (2.5" O.D. Split Barrel)

Core Barrel (See Log Notes)

INCREASING VISUAL MOISTURE CONTENT

Wet
Moist
Dry

CONSTITUENT PERCENTAGE

trace < 5%
few 5 - 15%
with 16 - 30%
-y 31 - 49%



1600 Willow Pass Court
 Concord, CA 94520
 Tel: (925) 688-1001

EXPLORATORY BORING CPT-1

PROJECT NO: 1073-1	SURFACE ELEVATION: --
LOGGED / CHECKED BY: R. Ceraolo/T. Chen	DATE STARTED: 8/2/2024
DRILLER: Pitcher Services	DATE FINISHED: 8/2/2024
DRILL RIG: --	DEPTH TO INITIAL WATER: Not Encountered
DRILLING METHOD: 3-inch Hand Auger	DEPTH TO FINAL WATER: Not Encountered
HAMMER TYPE / WEIGHT / DROP: --	
BORING LOCATION: See Site Plan, Figure 2 (37.339555°;-121.994850°)	

PROJECT:
HOMESTEAD TOWNHOMES
 Santa Clara, California

SUBSURFACE MATERIAL CLASSIFICATION			DEPTH (FEET)	SAMPLER	FIELD BLOW COUNT	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS NOTES AND REMARKS
DESCRIPTION	CONSIST	GRAPHIC LOG								
ASPHALT CONCRETE (AC) about 3" thick.			0							
AGGREGATE BASE (AB) about 5" thick.										
CLAY (CL), mottled gray dark brown, silty, few sand (fine- to coarse-grained), few gravel (fine, subangular to subrounded), dry to moist.	firm to stiff									
CLAY (CL), brown, silty, with to sandy (fine- to coarse-grained), dry to moist.	stiff									
Bottom of Boring = 5 feet Notes: Stratification is approximate; variations must be expected. See report for additional details.			5							
			10							
			15							



1600 Willow Pass Court
 Concord, CA 94520
 Tel: (925) 688-1001

EXPLORATORY BORING CPT-2

PROJECT NO: 1073-1	SURFACE ELEVATION: --
LOGGED / CHECKED BY: R. Ceraolo/T. Chen	DATE STARTED: 8/2/2024
DRILLER: Pitcher Services	DATE FINISHED: 8/2/2024
DRILL RIG: --	DEPTH TO INITIAL WATER: Not Encountered
DRILLING METHOD: 3-inch Hand Auger	DEPTH TO FINAL WATER: Not Encountered
HAMMER TYPE / WEIGHT / DROP: --	
BORING LOCATION: See Site Plan, Figure 2 (37.338100°; -121.994100°)	

PROJECT:
HOMESTEAD TOWNHOMES
 Santa Clara, California

SUBSURFACE MATERIAL CLASSIFICATION			DEPTH (FEET)	SAMPLER	FIELD BLOW COUNT	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS NOTES AND REMARKS
DESCRIPTION	CONSIST	GRAPHIC LOG								
ASPHALT CONCRETE (AC) about 3" thick.			0							
AGGREGATE BASE (AB) about 5" thick.										
CLAY (CL), mottled gray dark brown, silty, few sand (fine- to coarse-grained), trace to few gravel (fine, subangular to subrounded), dry to moist.	firm to stiff									
CLAY (CL), brown, silty, with to sandy (fine- to coarse-grained), few to with gravel (fine, subangular to subrounded), dry to moist.	stiff									
SAND (SM), mottled gray brown, fine- to coarse-grained, gravelly (fine, subangular to subrounded), silty, dry to moist.	medium dense		5							
Bottom of Boring = 5 feet Notes: Stratification is approximate; variations must be expected. See report for additional details.										
			10							
			15							



CONE PENETRATION TESTING
(CPT) REPORT

Gregg Drilling LLC

Prepared for: Stevens, Ferrone & Bailey Engineering
Project P7242015
August 6, 2024

Prepared by: Chris McAleer
cmcaleer@pitcherservicesllc.com

Table of Contents

Cone Penetration Testing (CPT) Procedure	3
15cm ² Standard Cone Specifications	4
Cone Penetration Test Data & Interpretation	5
Pore Pressure Dissipation Tests (PPDTs).....	6
Seismic Cone Penetration Tests (SCPT).....	7
Soil Sampling	8
Ground Water Sampling	9
References	10
Table 1: Cone Penetration Testing Summary	11

APPENDIX A: Cone Penetration Test Plots

APPENDIX B: Pore Pressure Dissipation Test Plots

Gregg Drilling CPT Report

August 6, 2024

Stevens, Ferrone & Bailey Engineering
Attn: Taiming Chen

Subject: CPT Site Investigation
3521 & 3591 Homestead Road
Santa Clara, CA
Pitcher Project Number: P7242015

Dear Mr. Chen:

The following report presents the results of Pitcher Services’s Cone Penetration Test investigation for the above referenced site. The following testing services were performed:

1	Cone Penetration Tests	(CPTU)	<input checked="" type="checkbox"/>
2	Pore Pressure Dissipation Tests	(PPD)	<input checked="" type="checkbox"/>
3	Seismic Cone Penetration Tests	(SCPTU)	<input type="checkbox"/>
4	Groundwater Samples	(GWS)	<input type="checkbox"/>
5	Soil Samples	(SS)	<input type="checkbox"/>
6	Vapor Samples	(VS)	<input type="checkbox"/>

A list of reference papers providing additional background on the specific tests conducted is provided in the bibliography following the text of the report. If you would like a copy of any of these publications or should you have any questions or comments regarding the contents of this report, please do not hesitate to contact me at 650-328-8910.

Sincerely,

CPT Reports Team
Gregg Drilling, LLC.



Cone Penetration Testing (CPT) Procedure

Gregg Drilling carries out all Cone Penetration Tests (CPT) using an integrated electronic cone system, Figure CPT.

The cone takes measurements of tip resistance (q_c), sleeve resistance (f_s), and penetration pore water pressure (u_2). Measurements are taken at either 2.5 or 5cm intervals during penetration to provide a nearly continuous profile. CPT data reduction and basic interpretation is performed in real time facilitating on-site decision making. The above-mentioned parameters are stored electronically for further analysis and reference. All CPT soundings are performed in accordance with revised ASTM standards (D 5778-12).

The 5mm thick porous plastic filter element is located directly behind the cone tip in the u_2 location. A new saturated filter element is used on each sounding to measure both penetration pore pressures as well as measurements during a dissipation test (PPDT). Prior to each test, the filter element is fully saturated with oil under vacuum pressure to improve accuracy.

When the sounding is completed, the test hole is backfilled according to client specifications. If grouting is used, the procedure generally consists of pushing a hollow tremie pipe with a "knock out" plug to the termination depth of the CPT hole. Grout is then pumped under pressure as the tremie pipe is pulled from the hole. Disruption or further contamination to the site is therefore minimized.

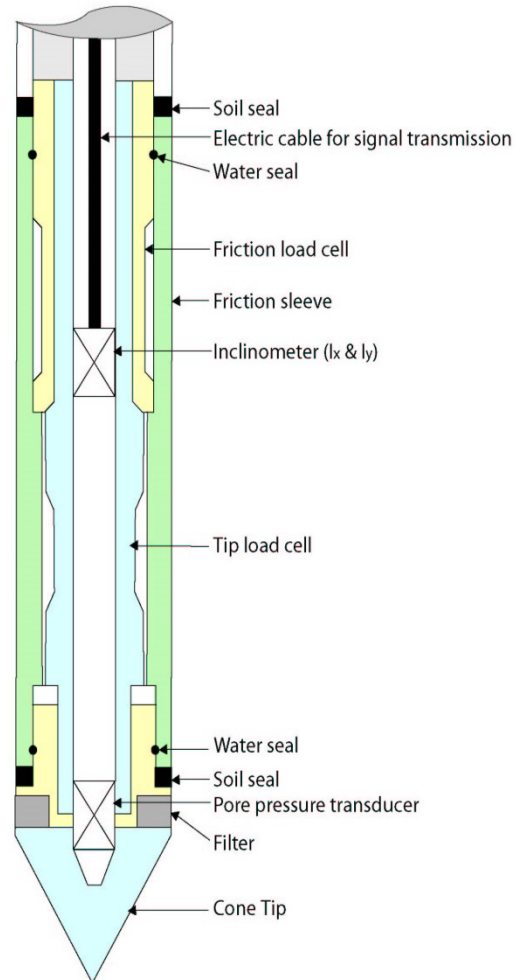


Figure CPT

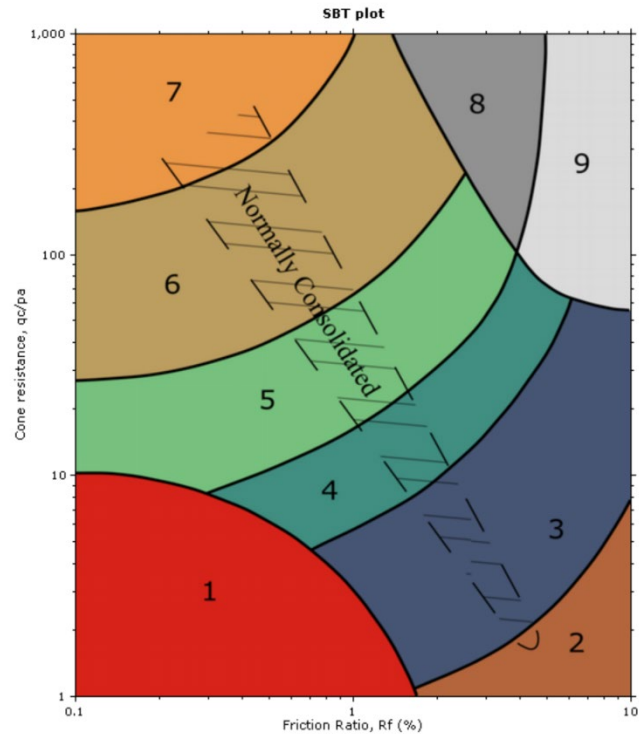
15cm² Standard Cone Specifications

Dimensions	
Cone base area	15 cm ²
Sleeve surface area	225 cm ²
Cone net area ratio	0.75
Specifications	
Cone load cell	
Full scale range	180 kN (20 tons)
Overload capacity	150%
Full scale tip stress	120 MPa (1,200 tsf)
Repeatability	120 kPa (1.2 tsf)
Sleeve load cell	
Full scale range	31 kN (3.5 tons)
Overload capacity	150%
Full scale sleeve stress	1,400 kPa (15 tsf)
Repeatability	1.4 kPa (0.015 tsf)
Pore pressure transducer	
Full scale range	7,000 kPa (1,000 psi)
Overload capacity	150%
Repeatability	7 kPa (1 psi)

Note: The repeatability during field use will depend somewhat on ground conditions, abrasion, maintenance and zero load stability.

Cone Penetration Test Data & Interpretation

The Cone Penetration Test (CPT) data collected are presented in graphical and electronic form in the report. The plots include interpreted Soil Behavior Type (SBT) based on the charts described by Robertson (2010) (Figure SBT). Typical plots display SBT based on the non-normalized charts of Robertson (2010) or normalized data (2009 and 2016). For CPT soundings deeper than 30m, we recommend the use of the normalized charts of Robertson (2009 and 2016) which can be displayed as SBTn. The report also includes spreadsheet output of computer calculations of basic interpretation in terms of SBT and SBTn and various geotechnical parameters using current published correlations based on the comprehensive review by Lunne, Robertson and Powell (1997), as well as recent updates by Robertson and Cabal (Guide to Cone Penetration Testing 7th Edition, 2022). The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg Drilling LLC does not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and does not assume any liability for use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software. Some interpretation methods require input of the groundwater level to calculate vertical effective stress. An estimate of the in-situ groundwater level has been made based on field observations and/or CPT results but should be verified by the user.



An estimate of the in-situ groundwater level has been made based on field observations and/or CPT results but should be verified by the user.

A summary of locations and depths is available in Table 1. Note that all penetration depths referenced in the data are with respect to the existing ground surface.

Note that it is not always possible to clearly identify a soil type based solely on q_t , f_s , and u_2 . In these situations, experience, judgment, and an assessment of the pore pressure dissipation data should be used to infer the correct soil behavior type.

- SBTn legend**
- 1. Sensitive fine grained
 - 4. Clayey silt to silty clay
 - 7. Gravelly sand to sand
 - 2. Organic material
 - 5. Silty sand to sandy silt
 - 8. Very stiff sand to clayey sand
 - 3. Clay to silty clay
 - 6. Clean sand to silty sand
 - 9. Very stiff fine grained

Figure SBT (After Robertson 2010) – Note: Colors may vary slightly compared to plots



Pore Pressure Dissipation Tests (PPDTs)

Pore Pressure Dissipation Tests (PPDT's) conducted at various intervals can be used to measure equilibrium water pressure (at the time of the CPT). If conditions are hydrostatic, the equilibrium water pressure can be used to determine the approximate depth of the ground water table. A PPDT is conducted when penetration is halted at specific intervals determined by the field representative. The variation of the penetration pore pressure (u) with time is measured behind the tip of the cone and recorded.

Pore pressure dissipation data can be interpreted to provide estimates of:

- Equilibrium piezometric pressure
- Phreatic Surface
- In situ horizontal coefficient of consolidation (c_h)
- In situ horizontal coefficient of permeability (k_h)

To correctly interpret the equilibrium piezometric pressure and/or the phreatic surface, the pore pressure must be monitored until it reaches equilibrium, *Figure PPDT*. This time is commonly referred to as t_{100} , the point at which 100% of the excess pore pressure has dissipated.

A complete reference on pore pressure dissipation tests is presented by Robertson et al. 1992 and Lunne et al. 1997.

A summary of the pore pressure dissipation tests completed for this project is included in Table 1.

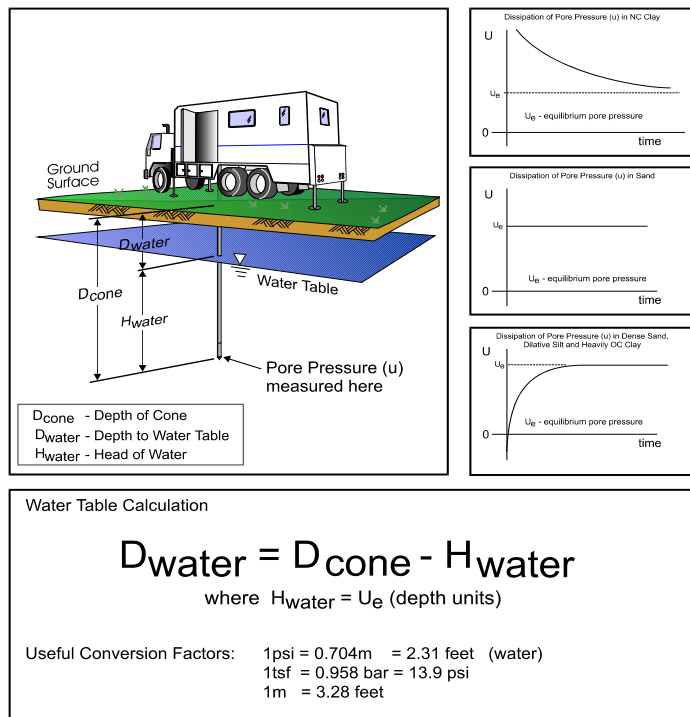


Figure PPDT

Seismic Cone Penetration Tests (SCPT)

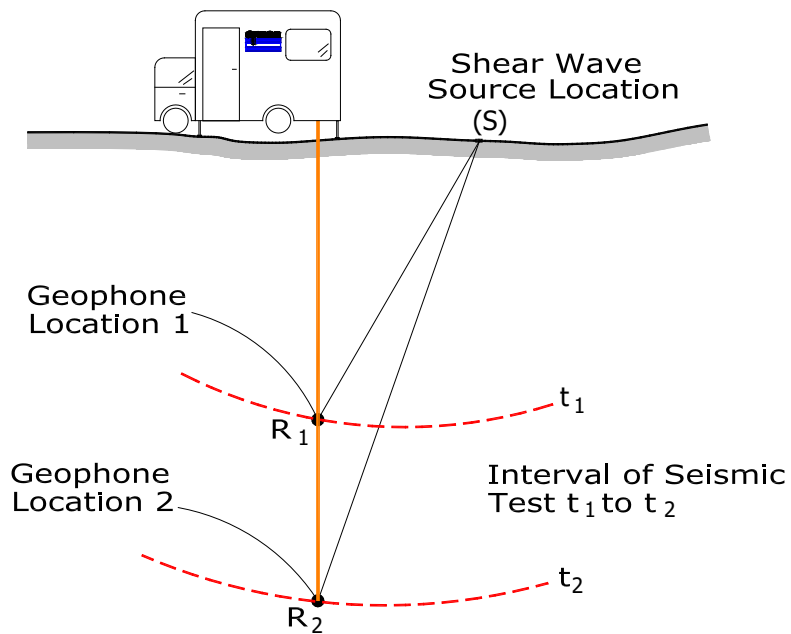
Seismic Cone Penetration Testing (SCPT) can be conducted at various intervals during the Cone Penetration Test. Shear wave velocity (V_s) can then be calculated over a specified interval with depth. A small interval for seismic testing, such as 1-1.5m (3-5ft) allows for a detailed look at the shear wave profile with depth. Conversely, a larger interval such as 3-6m (10-20ft) allows for a more average shear wave velocity to be calculated. Gregg Drilling's cones have a horizontally active geophone located 0.2m (0.66ft) behind the tip.

To conduct the seismic shear wave test, the penetration of the cone is stopped and the rods are decoupled from the rig. An automatic hammer is triggered to send a shear wave into the soil. The distance from the source to the cone is calculated knowing the total depth of the cone and the horizontal offset distance between the source and the cone. To calculate an interval velocity, a minimum of two tests must be performed at two different depths. The arrival times between the two wave traces are compared to obtain the difference in time (Δt). The difference in depth is calculated (Δd) and velocity can be determined using the simple equation: $v = \Delta d / \Delta t$

Multiple wave traces can be recorded at the same depth to improve quality of the data.

A complete reference on seismic cone penetration tests is presented by Robertson et al. 1986 and Lunne et al. 1997.

A summary of the shear wave velocities, arrival times and wave traces are provided with the report.



$$\text{Velocity } V = \frac{SR_2 - SR_1}{t_2 - t_1}$$

Figure SCPT

Soil Sampling

Gregg Drilling uses a piston-type push-in sampler to obtain small soil samples without generating any soil cuttings, *Figure SS*. Two different types of samplers (12 and 18 inch) are used depending on the soil type and density. The soil sampler is initially pushed in a "closed" position to the desired sampling interval using the CPT pushing equipment. Keeping the sampler closed minimizes the potential of cross contamination. The inner tip of the sampler is then retracted leaving a hollow soil sampler with inner 1¼" diameter sample tubes. The hollow sampler is then pushed in a locked "open" position to collect a soil sample. The filled sampler and push rods are then retrieved to the ground surface. Because the soil enters the sampler at a constant rate, the opportunity for 100% recovery is increased. For environmental analysis, the soil sample tube ends are sealed with Teflon and plastic caps. Often, a longer "split tube" can be used for geotechnical sampling.

For a detailed reference on direct push soil sampling, refer to Robertson et al, 1998.

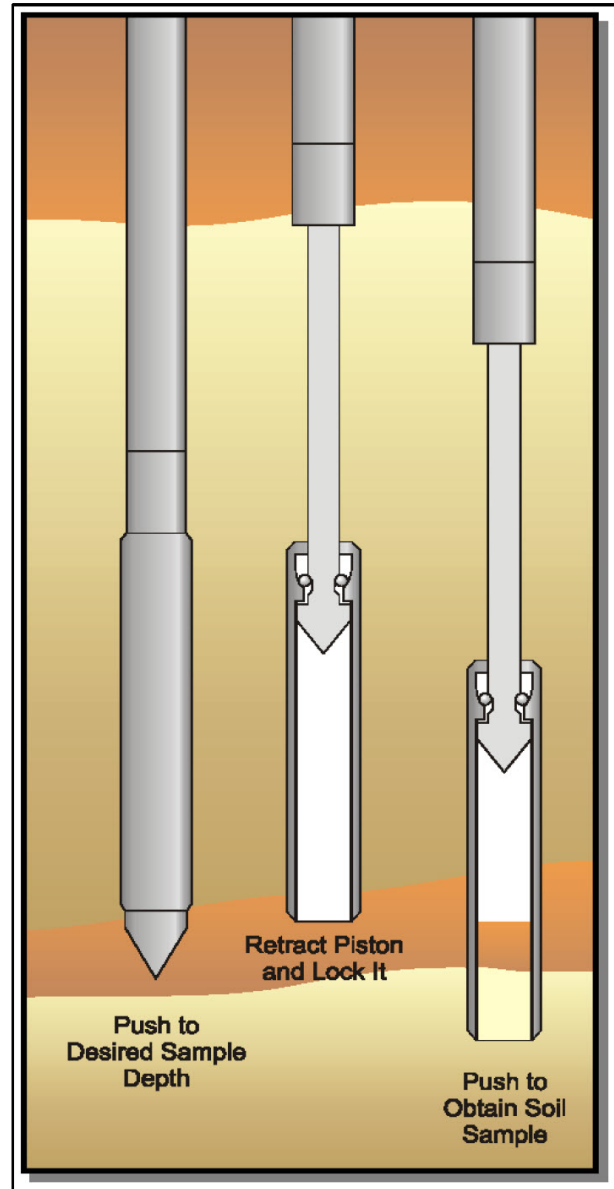


Figure SS

Ground Water Sampling

Gregg Drilling conducts groundwater sampling using a sampler as shown in *Figure GWS*. The groundwater sampler has a retrievable stainless steel or disposable PVC screen with steel drop off tip. This allows for samples to be taken at multiple depth intervals within the same sounding location. In areas of slower water recharge, provisions may be made to set temporary PVC well screens during sampling to allow the pushing equipment to advance to the next sample location while the groundwater is allowed to infiltrate.

The groundwater sampler operates by advancing 44.5mm (1¾ inch) hollow push rods with the filter tip in a closed configuration to the base of the desired sampling interval. Once at the desired sample depth, the push rods are retracted; exposing the encased filter screen and allowing groundwater to infiltrate hydrostatically from the formation into the inlet screen. A small diameter bailer (approximately ½ or ¾ inch) is lowered through the push rods into the screen section for sample collection. The number of downhole trips with the bailer and time necessary to complete the sample collection at each depth interval is a function of sampling protocols, volume requirements, and the yield characteristics and storage capacity of the formation. Upon completion of sample collection, the push rods and sampler, with the exception of the PVC screen and steel drop off tip are retrieved to the ground surface, decontaminated and prepared for the next sampling event.

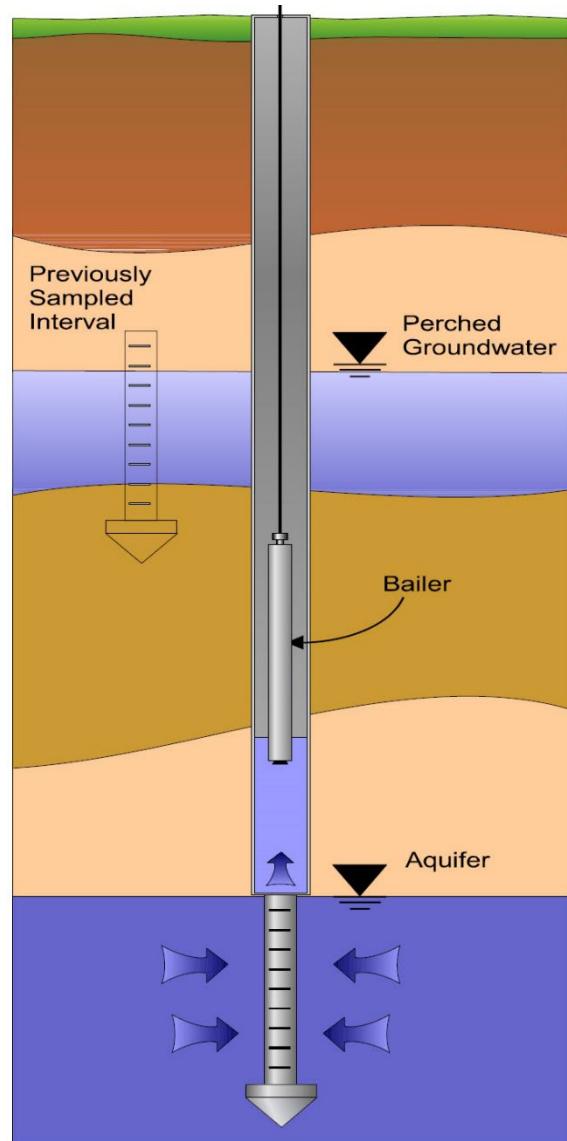


Figure GWS

For a detailed reference on direct push groundwater sampling, refer to Zemo et. al., 1992.

References

Lunne, T., Robertson, P.K. and Powell, J.J.M., “Cone Penetration Testing in Geotechnical Practice” E & FN Spon. ISBN 0 419 23750, 1997

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Mayne, P.W., “NHI (2002) Manual on Subsurface Investigations: Geotechnical Site Characterization”, available through www.ce.gatech.edu/~geosys/Faculty/Mayne/papers/index.html, Section 5.3, pp. 107-112.

Robertson, P.K., R.G. Campanella, D. Gillespie and A. Rice, “Seismic CPT to Measure In-Situ Shear Wave Velocity”, Journal of Geotechnical Engineering ASCE, Vol. 112, No. 8, 1986 pp. 791-803.

Robertson, P.K., Sully, J., Woeller, D.J., Lunne, T., Powell, J.J.M., and Gillespie, D.J., "Guidelines for Estimating Consolidation Parameters in Soils from Piezocone Tests", Canadian Geotechnical Journal, Vol. 29, No. 4, August 1992, pp. 539-550.

Robertson, P.K., T. Lunne and J.J.M. Powell, “Geo-Environmental Application of Penetration Testing”, Geotechnical Site Characterization, Robertson & Mayne (editors), 1998 Balkema, Rotterdam, ISBN 90 5410 939 4 pp 35-47.

Copies of ASTM Standards are available through www.astm.org

Table 1: Cone Penetration Testing Summary

CPT Sounding Identification	Date	Termination Depth (ft)	Depth of Soil Samples (ft)	Depth of Groundwater Samples (ft)	Depth of Pore Pressure Dissipation Tests (ft)
CPT-1	8/2/2024	69.88	-	-	39.89
CPT-2	8/2/2024	69.88	-	-	41.08, 44.42

APPENDIX A:

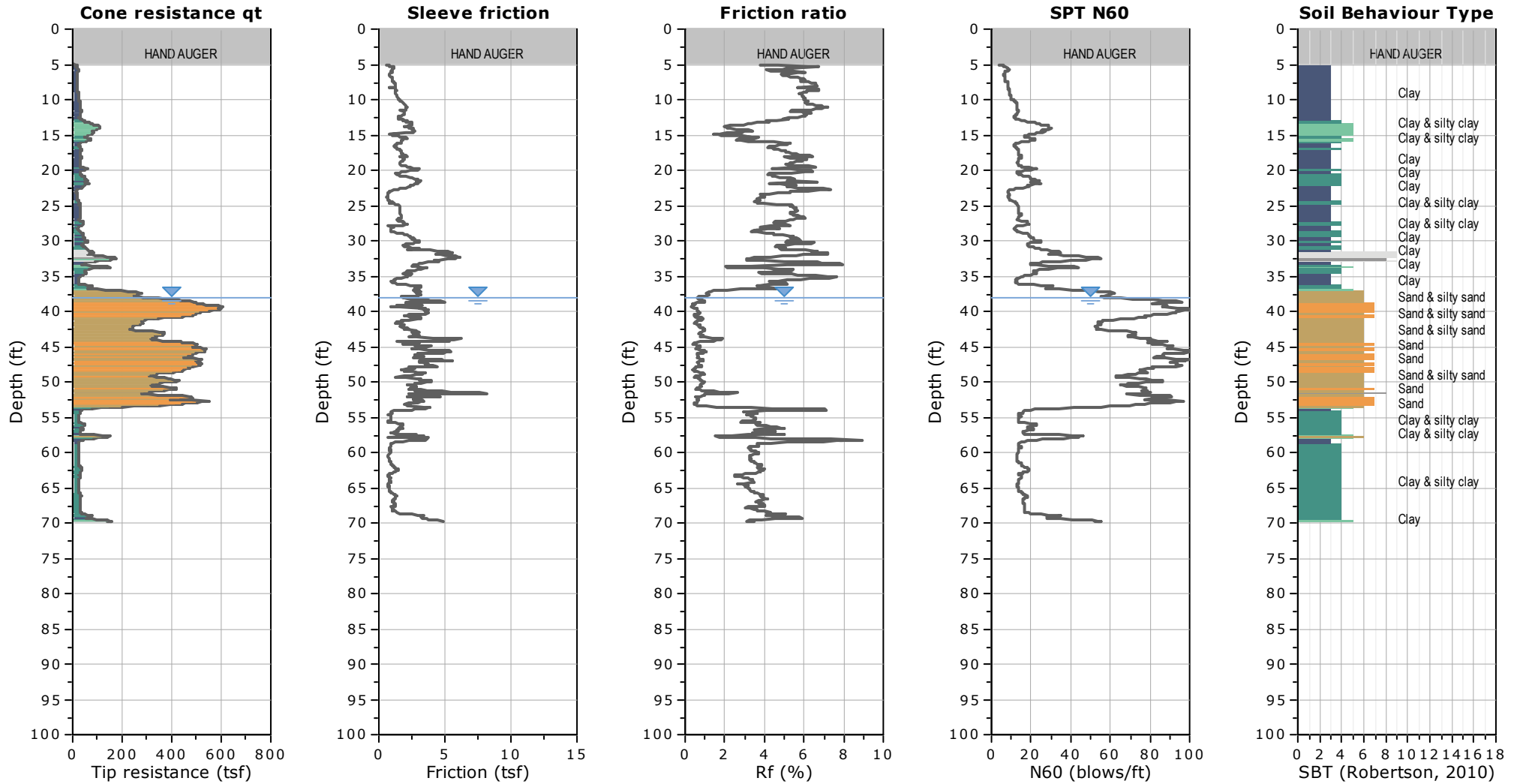
CPT PLOTS



CLIENT: Stevens, Ferrone & Bailey Engineering
SITE: 3521 & 3591 Homestead Road, Santa Clara, CA

FIELD REP: RICHARD
Cone ID: 231136

Total depth: 69.88 ft, Date: 8/02/2024

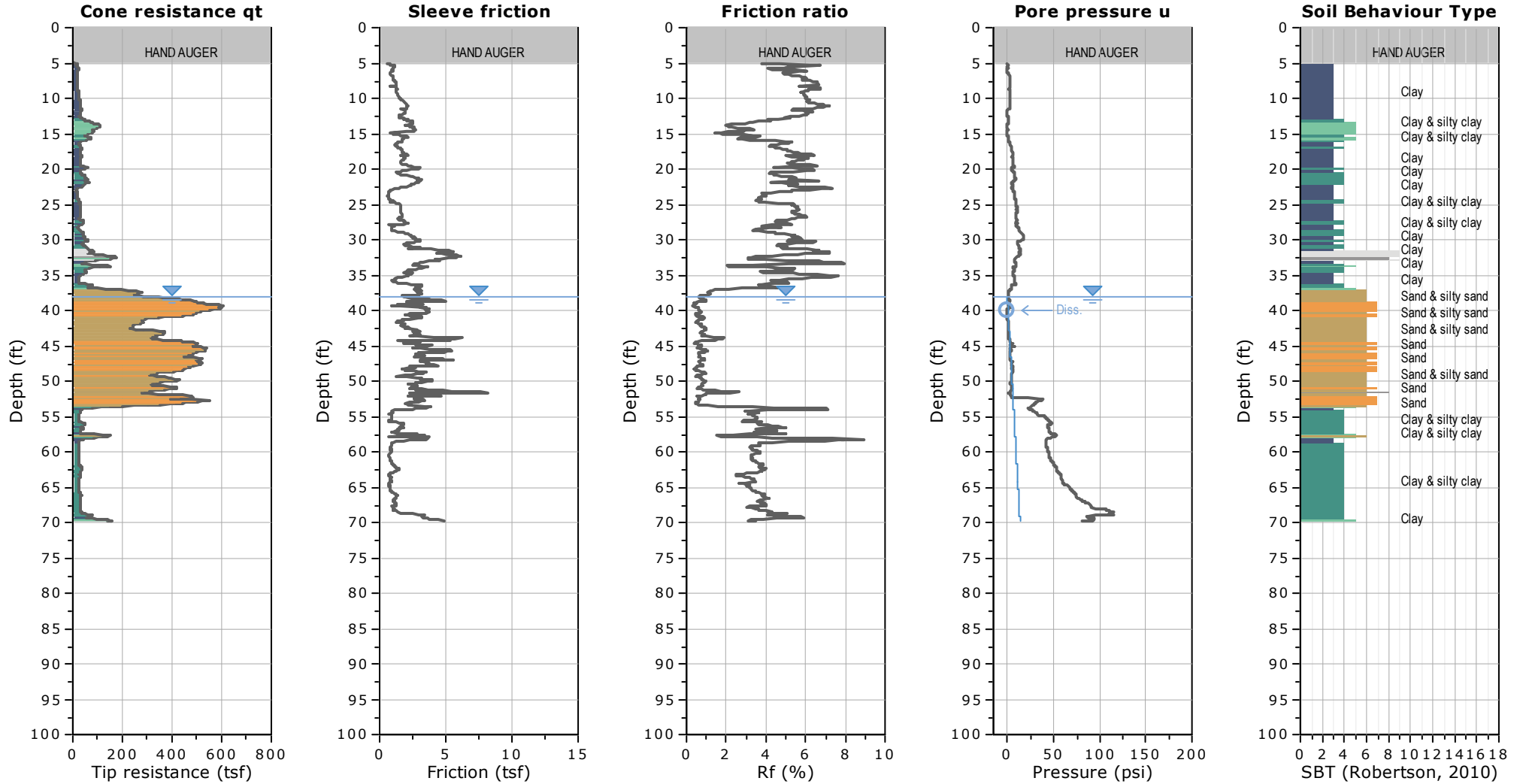




CLIENT: Stevens, Ferrone & Bailey Engineering
SITE: 3521 & 3591 Homestead Road, Santa Clara, CA

FIELD REP: RICHARD
Cone ID: 231136

Total depth: 69.88 ft, Date: 8/02/2024

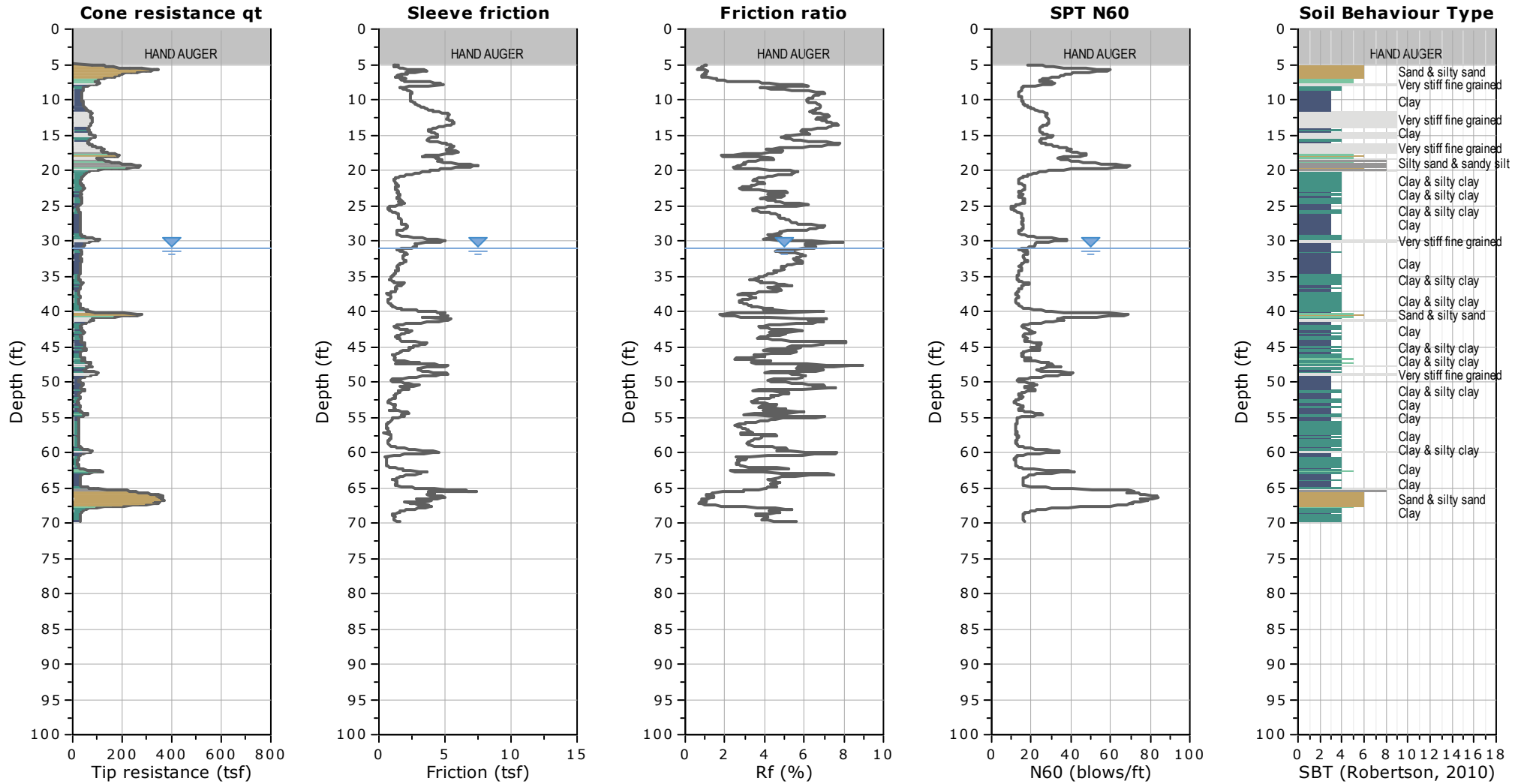




CLIENT: Stevens, Ferrone & Bailey Engineering
SITE: 3521 & 3591 Homestead Road, Santa Clara, CA

FIELD REP: RICHARD
Cone ID: 231136

Total depth: 69.88 ft, Date: 8/02/2024

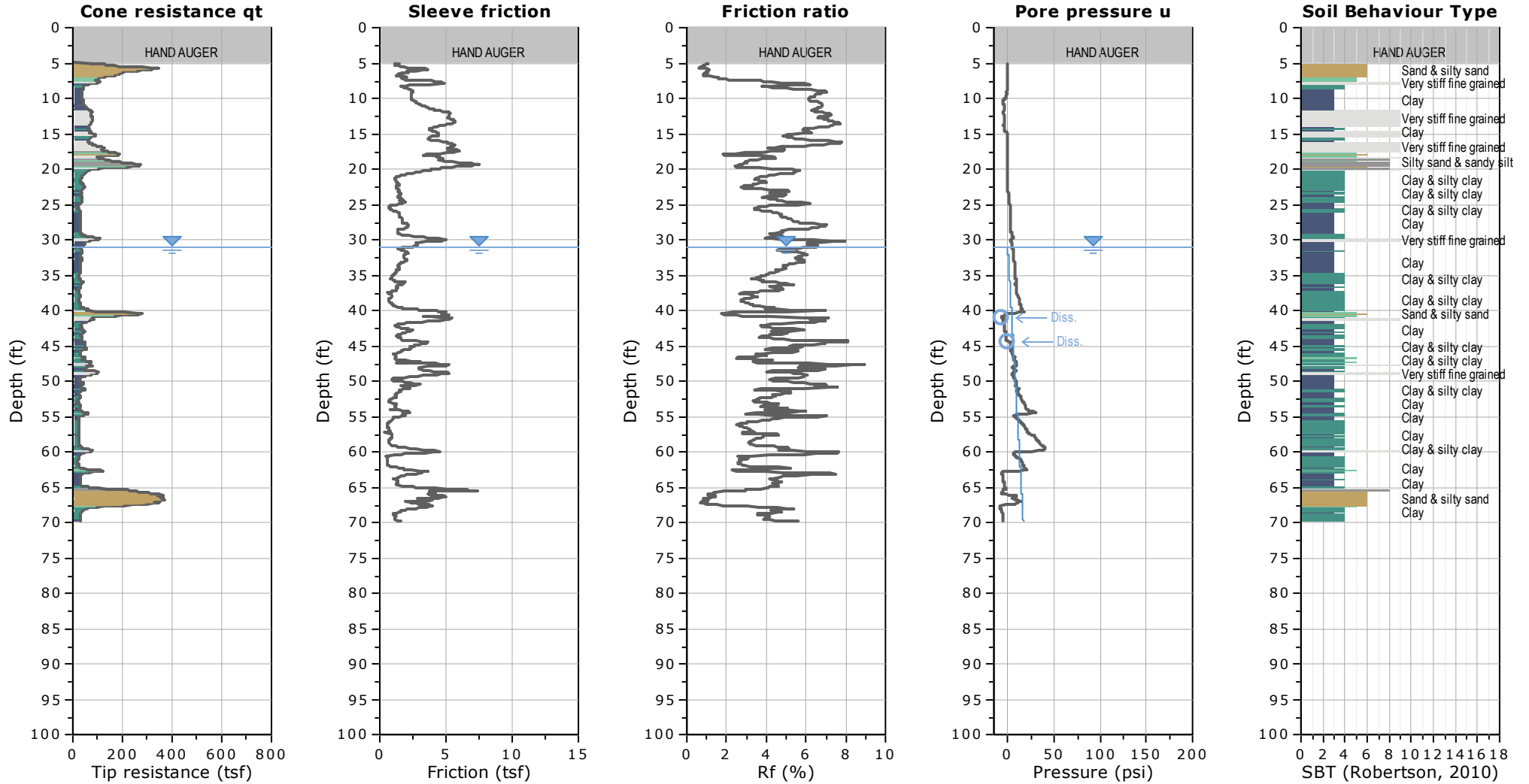




CLIENT: Stevens, Ferrone & Bailey Engineering
SITE: 3521 & 3591 Homestead Road, Santa Clara, CA

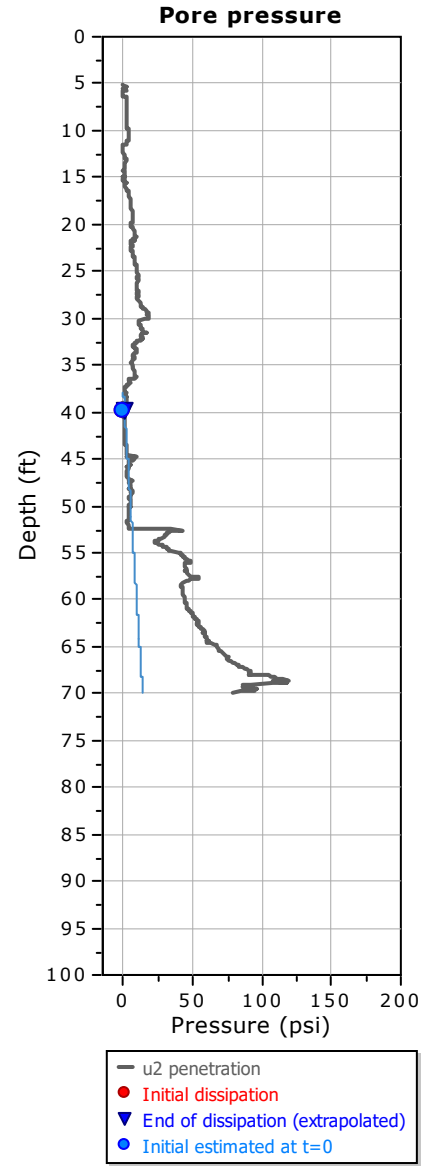
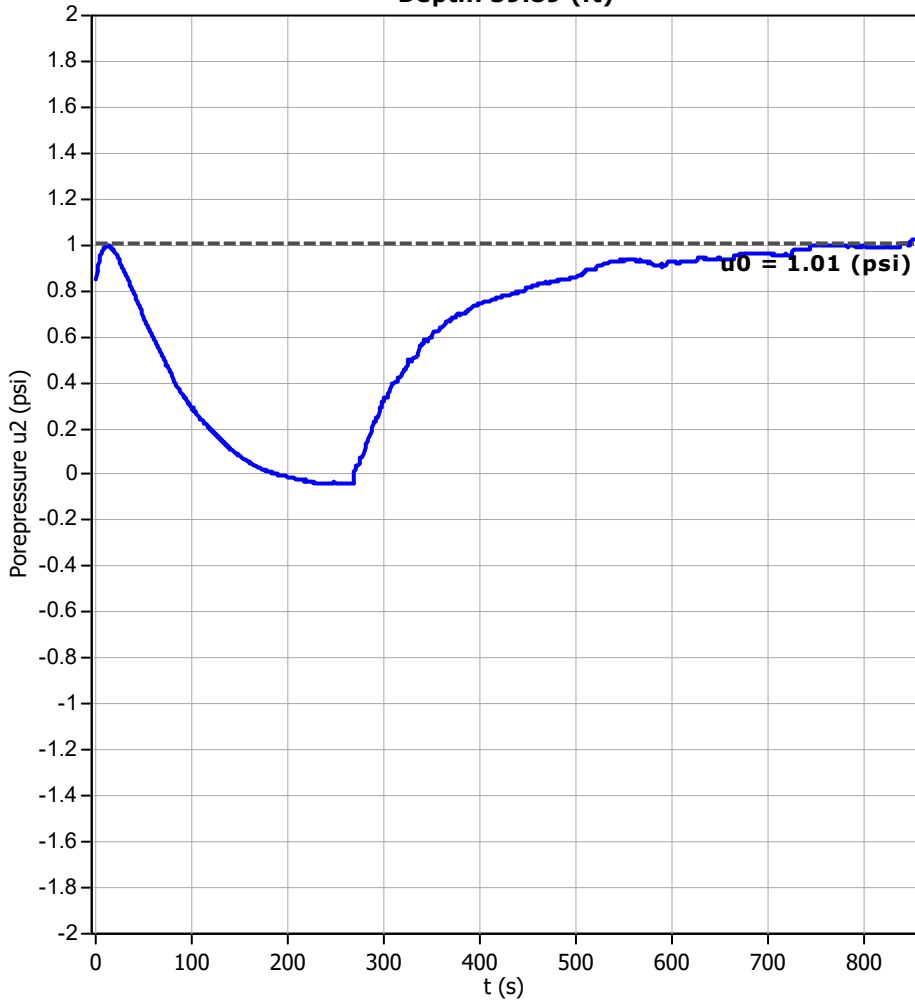
FIELD REP: RICHARD
Cone ID: 231136

Total depth: 69.88 ft, Date: 8/02/2024

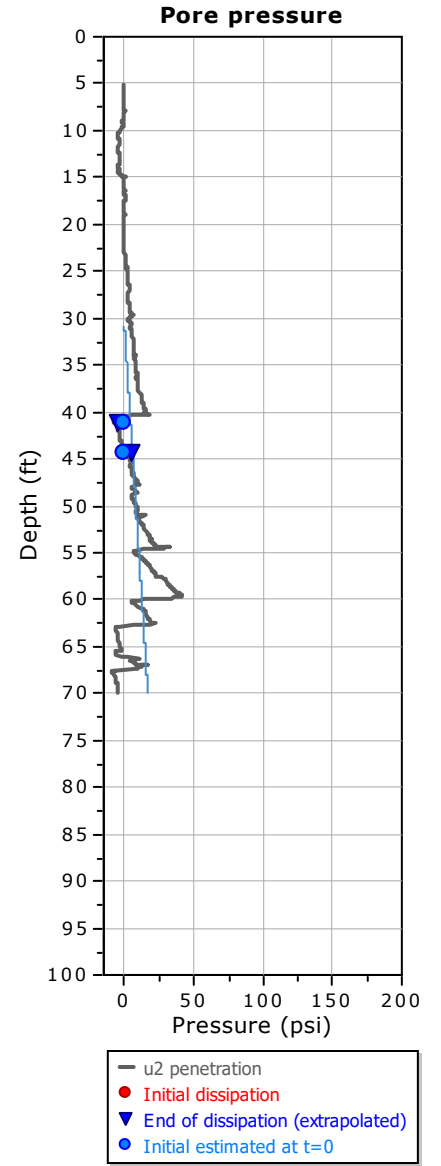
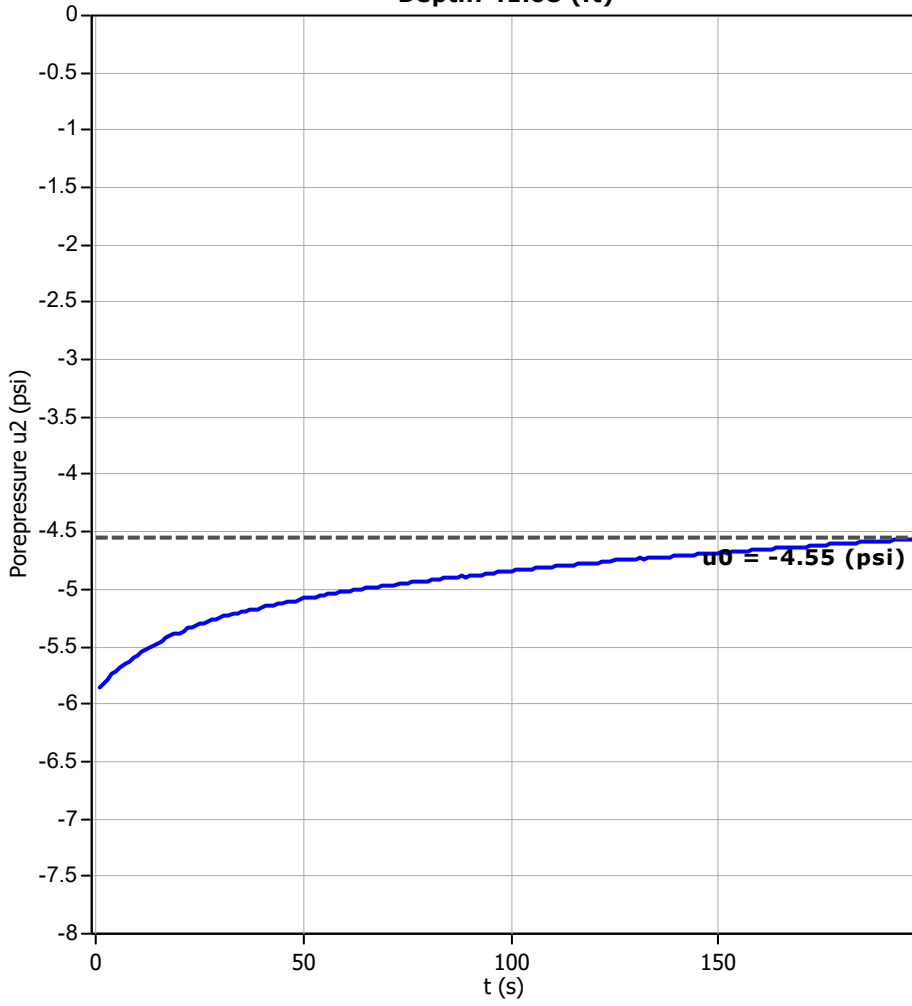


APPENDIX B:
PORE PRESSURE DISSIPATION
TEST PLOTS

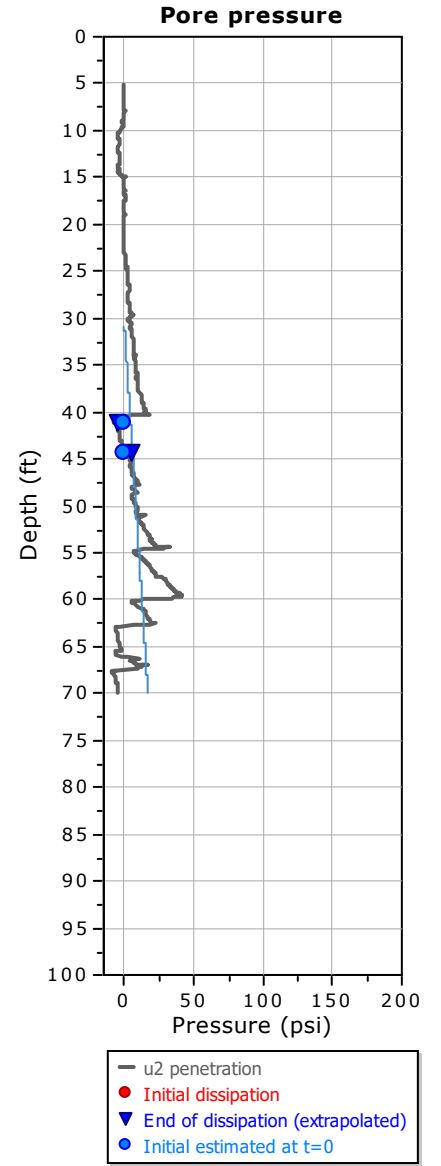
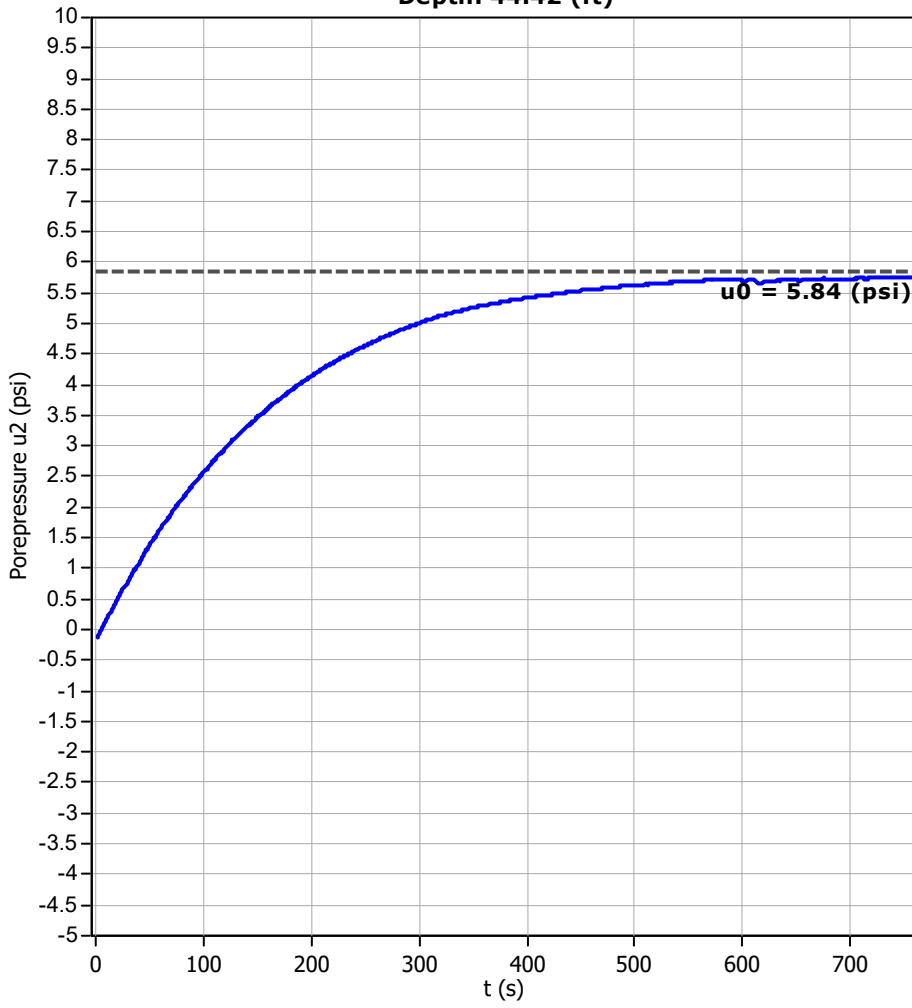
Piezocone Dissipation Test: CPT-1 Depth: 39.89 (ft)



Piezocone Dissipation Test: CPT-2 Depth: 41.08 (ft)



Piezocone Dissipation Test: CPT-2 Depth: 44.42 (ft)



APPENDIX B

Previous Environmental Excavation Records and Exploration Logs

K

CHAIN OF CUSTODY PAGE 1 OF 1

PROJECT NAME:
J.T. Petroleum

Environmental Technology

ANALYSIS REQUESTED

21782C

EPA 8240, High Boiling Point Hydrocarbons, Total oil & grease (SISE)

413.2
per 5/1/88

PROJECT NO:
SPR-0123-106

SAMPLE DESCRIPTION/LOCATION
(Be sure to indicate on map)
(Indicate: SOIL WATER DEPTH)

Soil 8 1/2' - center of tank

RECEIVED
MAY 11 1988

S.C.F.D.

ID. NO.	DATE	TIME	GRAB	COMP	NO. OF CONTAINERS
1	4-25-88		X		

REMARKS:

Soils looked & smelled clean

NAME OF GOV'T PERSON THAT WITNESSED SAMPLING:

SAMPLER(S) SIGNATURE(S):

[Signature]

Sequestered by: *[Signature]* Date: 4-27-88

Sequestered by: *[Signature]* Date: 4-25-88

Sequestered by: *[Signature]* Date: 4-25-88

Sequestered by: *[Signature]* Date: 4-28-88

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Sequestered by: *[Signature]* Date: 4-28-88

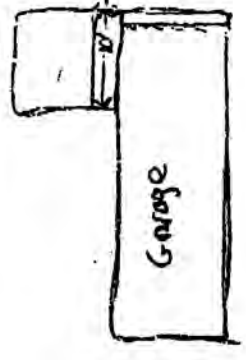
SITE MAP

Locomotive Ave

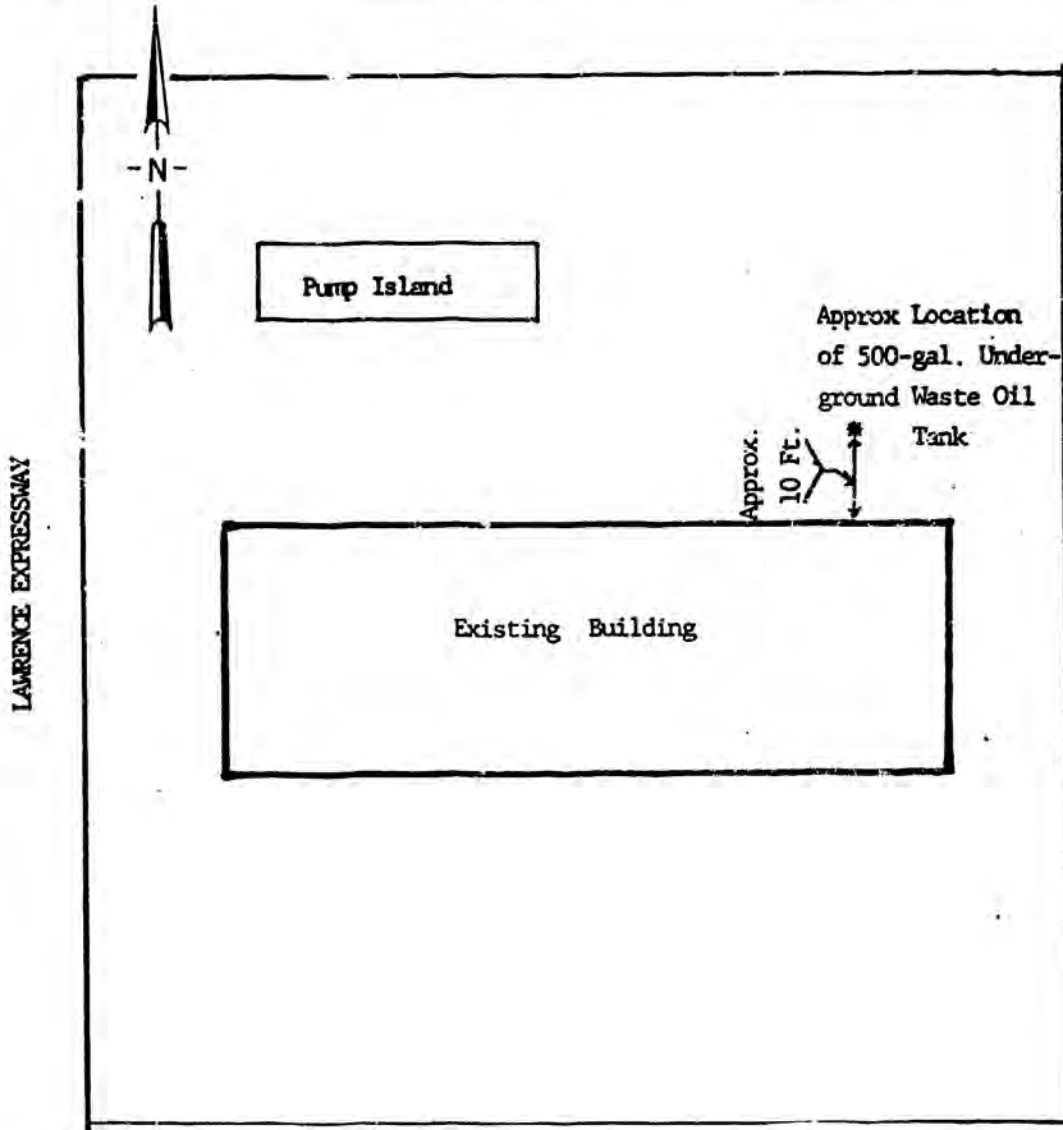
Pump Island

Laurence

Rm 134
5/3/88



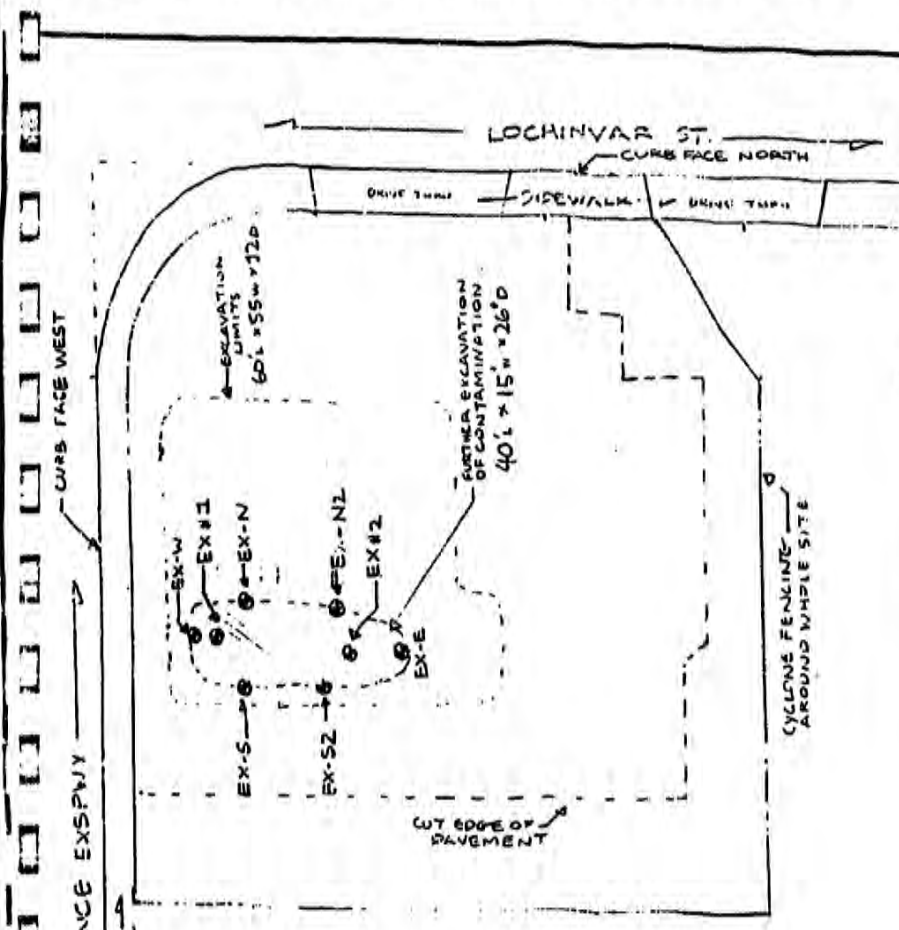
Job Name: J.T. PETROLEUM



SITE MAP
841 LAWRENCE EXPWY.
SANTA CLARA, CA

RECEIVED
MAY 11 1993

S. C. F. D.



SAMPLE DIMENSIONS	FROM CURB FACE (N)	
	C/F NORTH	C/F WEST
EX #1	90' N	21' W
EX #2	92' N	46' W
EX - N	82' N	22' W
EX - N2	83' N	43' W
EX - S	98' N	21' W
EX - S2	100' N	41' W
EX - E	91' N	56' W
EX - W	89' N	17' W

CARE-1. 841 LAWRENCE EXSPWY.
SANTA CLARA CA. 705 x 9254

SCALE: 1" = 30'

DATE: 6-26-92

APPROVED BY: _____

SOIL SAMPLE LOCATION(S) AFTER FURTHER EXCAVATION OF CONTAMINATED SOIL

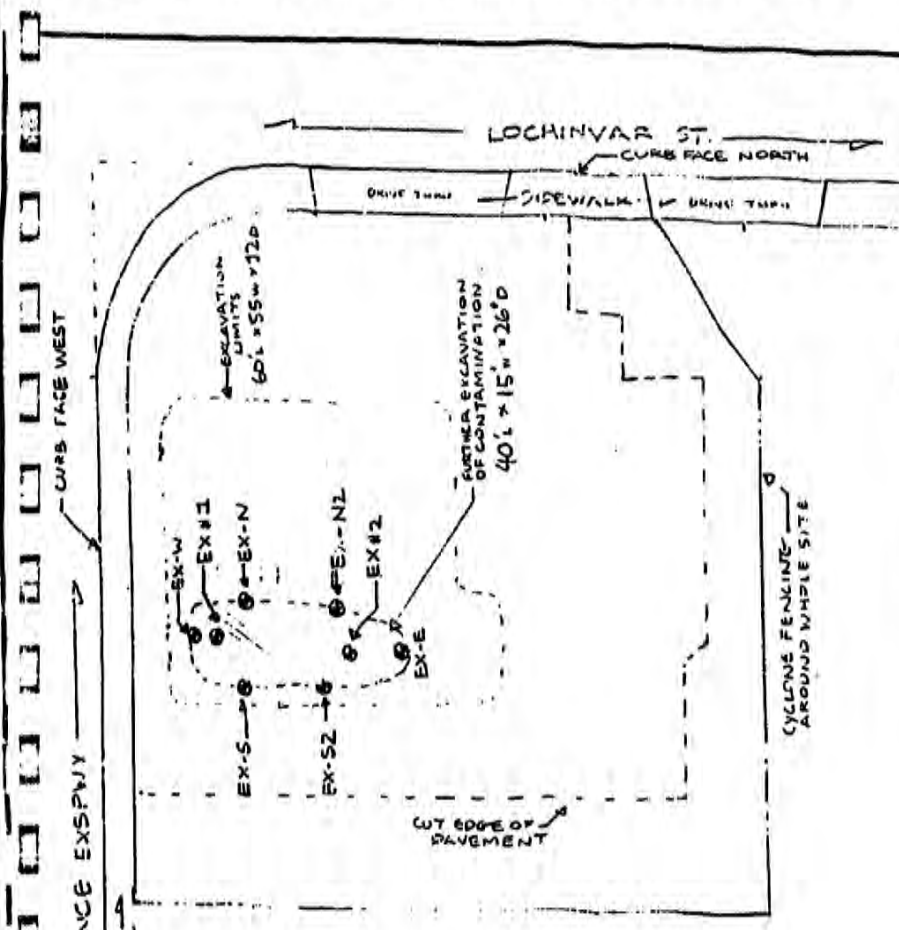
DRAWN BY: E. L.

REVISED

NOTE: SEE PROFILE SKETCH # 9254-05 FOR SAMPLE DEPTHS

DRAWING NUMBER = 9254-04





LAWRENCE EXSPWY

CURB FACE WEST

LOCHINVAR ST.
CURB FACE NORTH

DRIVE THRU SIDEWALK DRIVE THRU

EXCAVATION LIMITS
60' x 55' x 12'D

FURTHER EXCAVATION
OF CONTAMINATION
40' x 15' x 26'D

CYCLONE FENCING
AROUND WHOLE SITE

BY EDGE OF
PAVEMENT

SAMPLE DEMENSIONS FROM CURB FACE (N)

SAMPLE #	FROM CURB FACE (N)	
	C/F NORTH	C/F WEST
EX # 1	90' N	21' W
EX # 2	92' N	46' W
EX - N	82' N	22' W
EX - N2	83' N	43' W
EX - S	98' N	21' W
EX - S2	100' N	41' W
EX - E	91' N	56' W
EX - W	89' N	17' W

CARE-1. 841 LAWRENCE EXSPWY.
SANTA CLARA CA.

705 x 9254



SCALE: 1" = 30'

DATE: 6-26-92

APPROVED BY: [Signature]

DRAWN BY: E.L.

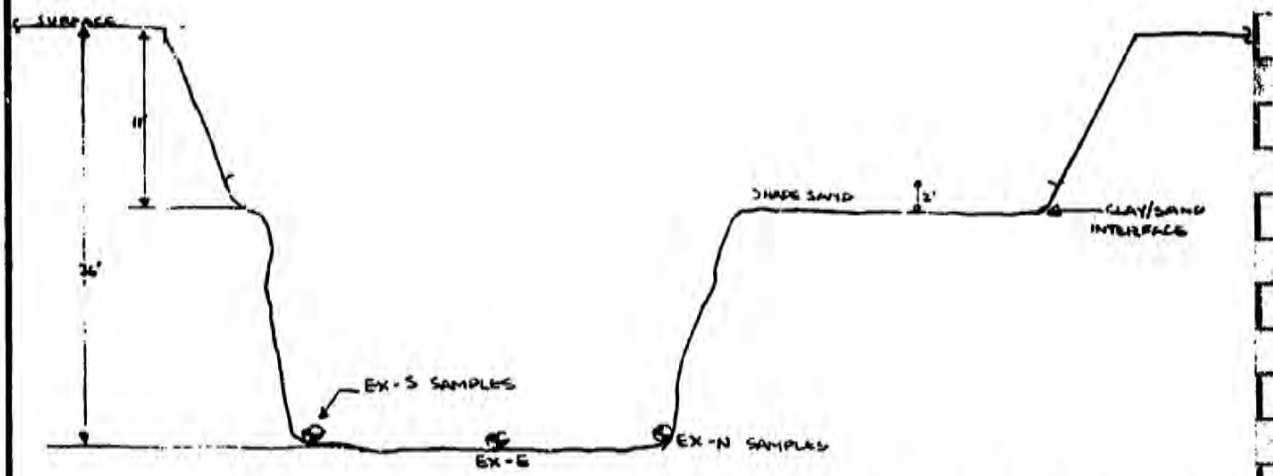
REVISED

*NOTE: SEE PROFILE SKETCH # 9254-05 FOR SAMPLE DEPTHS

DRAWING NUMBER
= 9254-04

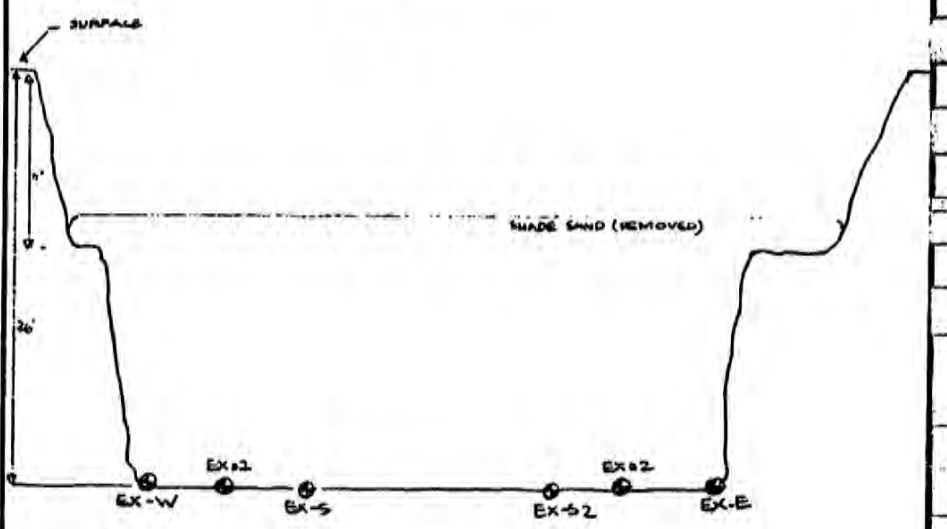
874 2 11 PRINTED ON NO. 10000 CLEARPRINT

VIEW WEST



*NOTE: ALL SAMPLES TAKEN AT 26' DEPTH FROM SURFACE

NORTH VIEW



CAREY - 841 LAWRENCE EXSPNY, SANTA CLARA, CA.

SCALE: 1" = 10'

APPROVED BY:

DATE: 6-26-72

DRAWN BY E.L.

⊙ - SOIL SAMPLE LOCATIONS

REVISED

PROFILE OF OVER EXCAVATED TANK PIT AND SAMPLE LOCATIONS w/ DEPTHS

DRAWING NUMBER: 9254-05



Lockiver Road

Expressway

Lawrence

Former Pump Islands



Former Tank Locations



Approximate Direction of Regional Groundwater Flow

Former Service Station Area



● Monitoring Well Location

**GEN TECH
ENVIRONMENTAL, INC.
SAN JOSE, CA**

**SITE PLAN
Monitoring Well Location
841 Lawrence Expressway
Santa Clara, CA**

Project No. 9313
Scale: 1' = 30'
Date: Feb. 1993

FIGURE 2

STANDARD SYMBOLS

Legend

- Soil sample location
- Soil sample collected for laboratory analysis
- No soil recovery
- First encountered groundwater level
- Potentiometric groundwater level
- Disturbed or bag soil sample

2.5 YR 6/2 Soil color according to Munsell Soil Color Charts (1975 Edition)

Penetration

Sample drive hammer weight - 140 pounds falling 30 inches.
Blows required to drive sampler 1 foot are indicated on the logs

Well Construction

- Annular seal
- Bentonite seal
- Sand pack
- Well riser section
- Well screen section

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS		GROUP SYMBOLS	TYPICAL NAMES	
COARSE-GRAINED SOILS More than half of material is larger than No. 200 sieve size	GRAVELS More than half of coarse fraction is larger than No. 4 sieve size	Clean Gravels	GW Well-graded gravels, gravel-sand mixtures, little or no fines	
		Gravels with Fines	GP Poorly graded gravels, gravel-sand mixture, little or no fines	
		SANDS More than half of coarse fraction is smaller than No. 4 sieve size	Clean Sands	GM Silty gravels, gravel-sand-silt mixtures
			Sands with Fines	GC Clayey gravels, gravel-sand-clay mixtures
	FINE-GRAINED SOILS More than half of material is smaller than No. 200 sieve size	SILTS AND CLAYS	Clean Sands	SW Well-graded sands, gravelly sand, little or no fines
			Sands with Fines	SP Poorly graded sands, gravelly sands, little or no fines
		Low Liquid Limit	SM Silty sands, sand-silt mixtures	
			S	Clayey sands, sand-clay mixtures
			ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts, with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
High Liquid Limit	OL	Organic silts and organic silty clays of low plasticity		
	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts		
	CH	Inorganic clays of high plasticity, fat clays		
		OH	Organic clays of medium to high plasticity, organic silts	
		Pt	Peat and other highly organic soils	

NOTES:

- Boundary Classification: Soils possessing characteristics of two groups are designated by combinations of group symbols. For example, GW-GC, well-graded gravel-sand mixture with clay binder.
- All sieve sizes on this chart are U.S. standard.
- The terms "silt" and "clay" are used respectively to distinguish materials exhibiting lower plasticity from those with higher plasticity.
- For a complete description of the Unified Soil Classification System, see "Technical Memorandum No. 3-357," prepared for Office, Chief of Engineers, by Waterways Equipment Station, Vicksburg, Mississippi, March 1953.

Gen Tech Environmental, Inc. San Jose, CA

Client : Carey
 Location : 841 Lawrence Expy, Santa Clara
 Drilling Method : Hollwostem Auger

Boring/Well No.: MW-1

Well Installed: Yes
 Total Depth : 96.5
 Length Screen: 17'
 Top of Sandpack: 79'

PAGE 1 OF 3
 Casing Depth: 93'
 Length Solid : 79'
 Top of Bentonite : 77'

Water Levels : 1st Enc. 85' Static : 78'

Sample No.	OV Han	Blow Count	Depth ft, bgs	Lithology Log	H2O Mark	Well Detail
			5	CL-SILTY CLAY, brown, contains 10% sand, 5% gravel, moderate plasticity, soft, damp.		
1		25	10	same as above, light brown, hard, 15% gravel		
			15			
2		17	20	same as above, color change to dark grey		
			25	same as above, <5% sand, faint laminae, stiff		
			30	SILTY SAND, greenish grey, 20% silt, damp, very dense		
			35	GRAVELY CLAY, green grey, 40% gravel disseminated, very dense, damp; field test = 50 ppm		
5		73	35	same as above, field test = ND		
			40	SILTY CLAY, greenish grey, 10% gravel, 10% sand, low plasticity, hard, damp		
6		42	40			

Gen Tech Environmental, Inc. San Jose, CA

Client : Carey Boring/Well No.: MW-1
 Location : 841 Lawrence Expy, Santa Clara
 Drilling Method : Hollwostem Auger

Well Installed: Yes
 Total Depth : 96.5
 Length Screen: 17'
 Top of Sandpack: 79'

PAGE 2 OF 3
 Casing Depth: 95'
 Length Solid : 79'
 Top of Bentonite : 77'

Water Levels : 1st Enc. 85' Static : 78'

Sample OV No.	Blow Han Count	Depth ft, bgs	Lithology Log	H2O Mark	Well Detail
8	42	45	SILTY CLAYEY SAND, 30% silt, 30% clay, 30% sand, low plasticity, dense, damp; field test = ND		
9	65	50	same as above, 6" sand interbed		
10	50	55	SANDY SILT, light brown, 10% gravel, 10% sand, 10-20% clay, 60% silt, low plasticity, hard damp; field test = nd		
11	31	60	GRAVELLY SILTY CLAY, 10-20% gravel, 10% sand, low plasticity, hard, damp to moist; field test = ND		
12	77	65	GRAVELLY SANDY CLAY, greenish grey, 10% gravel, 30% sand, mod. plasticity, damp, hard; field test= ND		
13	50	70	CLAYEY SAND, orange tan, clay 20-30%, low to mod. plasticity, 1% gravel float in clay matrix, dense, damp; field test = ND		
14	32	75	SILTY CLAY, gray green, 5% fine sand, mod. plasticity, hard, damp; field test = ND		
		80	attempted sample drive, lost tool, drill to 90' to retrieve tools.		

Gen Tech Environmental, Inc. San Jose, CA

Client : Carey Boring/Well No.: MW-1
 Location : 941 Lawrence Expy, Santa Clara
 Drilling Method : Hollowstem Auger

Well Installed: Yes
 Total Depth : 96.5
 Length Screen: 17'
 Top of Sandpack: 79'

PAGE 3 OF 3
 Casing Depth: 95'
 Length Solid : 79'
 Top of Bentonite : 77'

Water Levels : 1st Enc. 85' Static : 78'

Sample No.	QV No.	Blow Ha:	Count	Depth ft. bgs	Lithology Log	H2O Mark	Well Detail
				85	tools retrieved, rods wet	▼	
15		12		90	INTERBEDDED SAND AND CLAY, dark grey, fine sand, dissem in clay, beds about 2-3" thick, sand yield water, clay beds damp, low-mod. plasticity, med. dense to stiff.		
16		63		95	SILTY CLAY, yellow brown, 20% fine sand dissem., low plasticity, hard, damp.		
					BOTTOM OF BORING = 96.5'		

APPENDIX C
GBA Guidelines for Geotechnical Report

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer

will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept*

responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.

Most of the “Findings” Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site’s subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report’s Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are not final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals’ misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals’ plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note*

conspicuously that you’ve included the material for information purposes only. To avoid misunderstanding, you may also want to note that “informational purposes” means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled “limitations,” many of these provisions indicate where geotechnical engineers’ responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a “phase-one” or “phase-two” environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer’s services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, *proper implementation of the geotechnical engineer’s recommendations will not of itself be sufficient to prevent moisture infiltration.* **Confront the risk of moisture infiltration** by including building-envelope or mold specialists on the design team. **Geotechnical engineers are not building-envelope or mold specialists.**



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