Appendix D
Geotechnical Report

ESA

MUNICIPAL WATER OFFICE REDEVELOPMENT

SAN JOSE, CALIFORNIA

GEOTECHNICAL EXPLORATION

SUBMITTED TO
Ms. Meryka Dirks
Environmental Science Associates
787 The Alameda, Suite 250
San Jose, CA 95126

PREPARED BY
ENGEO Incorporated

March 14, 2022

PROJECT NO.
19886.000.001

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GEOTECHNICAL
ENVIRONMENTAL
WATER RESOURCES
CONSTRUCTION SERVICES
COASTAL/MARINE GEOTECHNICS

Project No.
19886.000.001

March 14, 2022

Ms. Meryka Dirks
Environmental Science Associates
787 The Alameda, Suite 250
San Jose, CA 95126

Subject: Municipal Water Office Redevelopment
3025 Tuers Road
San Jose, California

GEOTECHNICAL EXPLORATION

Dear Ms. Dirks:

We prepared this geotechnical report for Environmental Science Associates as outlined in our agreement dated January 18, 2022. We characterized the subsurface conditions at the site to provide the enclosed geotechnical recommendations for design.

Our experience and that of our profession clearly indicate that the risk of costly design, construction, and maintenance problems can be significantly lowered by retaining the design geotechnical engineering firm to review the project plans and specifications and provide geotechnical observation and testing services during construction. Please let us know when working drawings are nearing completion, and we will be glad to discuss these additional services with you.

If you have any questions or comments regarding this report, please call and we will be glad to discuss them with you.

Sincerely,

ENGEO Incorporated

Eleni Korogianos

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

We prepared this geotechnical report for design of the Municipal Water Office Redevelopment in San Jose, California, as outlined in our agreement dated January 18, 2022. We developed our scope to present our geotechnical recommendations for the design and construction of the proposed project. Our scope of work included:

- Review of relevant background information, including available literature, geologic maps, and reports pertinent to the site.
- Exploration of subsurface conditions.
- Laboratory testing of select samples.
- Evaluation of geotechnical conditions and performing analyses of collected data.
- Preparation of this geotechnical report.

We reviewed the following documents during our preparation of this design report.

1. City of San Jose; 9813 San Jose Municipal Water New Offices, 30% Design Plans, San Jose, California; May 19, 2021, Project No. 2020.185.
2. HMH; Utility Coordination, San Jose Municipal Water System, San Jose, California; July 19, 2021, HMH #5889.00.
3. TRC; Geotechnical Investigation, Four Photovoltaic Solar Arrays for the City of San Jose, San Jose, California, January 5, 2012, Report Number 188828.

We prepared this report for the exclusive use of the City of San Jose (under our agreement with Environmental Sciences Associates) and their consultants for the design of this project. In the event that any changes are made in the character, design, or layout that could impact the geotechnical conclusions and recommendations provided in this report, we should be provided the opportunity to review the conclusions and recommendations contained in this report to evaluate whether modifications may be necessary.

1.2 PROJECT LOCATION AND PROPOSED DEVELOPMENT

The approximately 3.25-acre site is located at 3025 Tuers Road in San Jose and identified as Assessor's Parcel Number (APN) 449-350-010 (Figure 1). The site is located on the edge of the Windmill Springs neighborhood of San Jose and is bounded by Loupe Avenue and Los Lagos Golf Course on the northwest, Tuers Road on the northeast, and open space on the south. Coyote Creek is located approximately 1,300 feet southwest and is aligned roughly parallel with the western property boundary.

Based on the 30 percent design plans, we understand the proposed project will consist of an approximately 22,300-square-foot, two-story office building, separate storage building, outdoor area, and associated parking and utilities. Associated improvements will include retaining walls up to 5 feet in height, concrete and asphalt pavement, hardscape areas, underground utilities to support the proposed offices and associated facilities, and landscaped areas.

The new Municipal Water Office and associated facilities will replace the existing facilities on the site. Figure 2 shows site boundaries, proposed building and pavement areas, and our exploration locations.

2.0 FINDINGS

2.1 PREVIOUS GEOTECHNICAL AND GEOLOGICAL STUDY

In January 2012, TRC published a geotechnical investigation report for proposed solar photovoltaic solar arrays above the existing parking structures at the site. The previous exploration included geotechnical borings, laboratory testing of subsurface samples, geotechnical analysis, and preparation of a report. TRC opined that the proposed development was feasible provided the seismic hazards identified in the report were addressed in the design (TRC, 2012).

2.2 REGIONAL GEOLOGY

The site is located in the San Francisco Bay Region, on the west flank of the Diablo Range foothills of the Coast Range geomorphic province, prominent northwest-trending mountains defining the eastern boundary of Santa Clara Valley.

Regional geologic mapping by Dibblee (2005) indicates that the site is predominantly underlain by Holocene-age alluvial deposits (Qa), which consist of gravel, sand, and clay soil of valley areas. Holocene-age sand and gravel of the Coyote Creek channel (Qg) is mapped west of the site but not extending into the project boundaries. A geologic map of the site is shown in Figure 3.

2.3 SEISMICITY

The San Francisco Bay Region contains numerous active earthquake faults. The site is located within the Santa Clara Valley region, which lies to the east of the San Andreas Fault and to the west of the Hayward and Calaveras Faults.

The California Geologic Survey (CGS) defines an active fault as one that has experienced surface displacement within Holocene time (about the last 11,700 years) (CGS SP42, 2018). The Working Group on California Earthquake Probabilities (WGCEP, 2017) evaluated the 30-year probability of a Moment Magnitude 6.7 or greater earthquake occurring on the known active fault systems in the Bay Area in the Third Uniform California Rupture Forecast (UCERF3). UCERF3 estimated an overall probability of 72 percent for the Bay Area as a whole, 14.3 percent for the Hayward Fault, 7.4 percent for the Calaveras Fault, and 6.4 percent for the Northern San Andreas Fault.

To assess the site’s seismicity, including nearby active faults that are capable of generating strong seismic ground shaking at the site, we utilized the United States Geological Survey (USGS) Unified Hazard Tool and disaggregated the hazard at the peak ground acceleration (PGA) for a 2,475-year return period, with the resulting faults listed below in Table 2.3-1. The locations of the faults are also presented in Figure 5. The closest distance to the rupture plane (rupture distance) (RRUP) is measured from the location listed below.

**TABLE 2.3-1: Active Faults Capable of Producing Significant Ground Shaking at the Site
Latitude: 37.342467 degrees, Longitude: -121.878061 degrees**

SOURCE	RUPTURE DISTANCE (R_{RUP}) (MILES)	MAXIMUM MAGNITUDE
Hayward (So) [0]	6.25	7.07
Calaveras (Central) [8]	6.68	7.21
San Andreas (Peninsula) [1]	10.64	7.94
Hayward Extension	12.99	6.45
Hayward (So) [1]	16.76	6.82
Silver Creek [5]		6.92

Based on USGS Unified Hazard Tool: Dynamic Conterminous U.S. 2014 (update) (v4.2.0)

The faults listed above represent sources contributing at least one percent to the seismic hazard at the site, at the PGA, and for the given return period; we did not include gridded or areal sources.

The project site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone or a Santa Clara County Hazard Zone, and no known active faults cross the site.

2.4 FIELD EXPLORATION

To characterize the subsurface conditions at the site, we advanced two borings and three cone penetration tests (CPTs) at the locations shown on the Site Plan in Figure 2. We advanced the CPTs and borings on January 14 and February 7, 2022, respectively. We obtained the necessary geotechnical drilling permits from Santa Clara Valley Water District for our CPT and boring explorations. We backfilled the explorations in accordance with Valley Water requirements.

Approximate ground surface elevations at exploration locations, and the total exploration depths, are summarized in Table 2.4-1. Exploration details are provided in the following sections. All elevations referred to in this report are relative to the WGS84 Datum.

TABLE 2.4-1: Exploration Summary

EXPLORATION IDENTIFICATION	EXPLORATION METHOD	APPROXIMATE SURFACE ELEVATION (FEET)	APPROXIMATE DEPTH (FEET)
1-B1	Solid-Flight-Auger	80	31½
1-B2	Solid-Flight-Auger/ Mud-Rotary	81	51½
1-SCPT1	Seismic CPT	80	100
1-CPT2	CPT	80	50½
1-CPT3	CPT	80	100½

2.4.1 Borings

A representative from our firm observed the drilling and logged the subsurface conditions at both boring locations. We retained the services of Britton Exploration to advance the borings with a track-mounted drill rig using solid-flight-auger and mud-rotary methods to depths ranging from 31½ to 51½ feet below the existing ground surface (bgs). At 1-B1, we used the solid flight auger method for the full depth of exploration. At 1-B2, we used the solid-flight-auger method in the upper 14 to 15 feet before switching to mud-rotary drilling for the remainder of the boring depth. Our drilling subcontractor placed drilling spoils in 55-gallon steel drums. Britton Exploration transported the drums off site for disposal at an appropriate waste facility.

We collected select soil samples using a 2½-inch inside diameter (I.D.) California-type split-spoon sampler fitted with 6-inch-long steel liners, or a 2-inch outside diameter (O.D.) standard penetration test (SPT) split-spoon sampler, or a 3-inch I.D. Shelby tube piston sampler. We recorded the penetration of the sampler into the subsurface material as the number of blow counts needed to drive the sampler 18 inches in 6-inch increments with a 140-pound hammer dropped through a 30-inch free-fall employing an automatic trip system. Unless otherwise indicated, the blows per foot recorded on the boring logs represent the accumulated number of blows to drive the last 1 foot of penetration; the blow counts have not been converted using any correction factors.

The boring logs are presented in Appendix A. The logs depict subsurface conditions at the exploration locations for the date of exploration; however, subsurface conditions may vary with time.

2.4.2 Cone Penetration Tests

We retained the services of a CPT subcontractor, ConeTec, Ltd., to advance a cone penetrometer in three locations to a maximum depth of about 100 feet in general accordance with ASTM D5778. Measurements include the tip resistance to penetration of the cone (Qc), the resistance of the surface sleeve (Fs), and pore pressure (U) (Robertson and Campanella, 1988).

We measured shear-wave velocities (V_S) in 1-SCPT3, using the downhole seismic method specified in ASTM D7400. We present the CPT logs and the V_S profile in Appendix D. The time-averaged shear-wave velocity over the top 61 feet is approximately 795 feet per second (ft/s).

2.5 LABORATORY TESTING

We performed geotechnical laboratory testing on select soil samples recovered during our field exploration to evaluate their physical index properties and strength characteristics. The laboratory tests that were performed, the associated ASTM procedures, and location of the results in this report are shown in Table 2.5-1.

TABLE 2.5-1: Laboratory Testing

SOIL CHARACTERISTIC	TESTING METHOD	LABORATORY TEST RESULT LOCATION
Moisture Content	ASTM D2216	Appendices A and B
Unit Weight and Moisture Content	ASTM D7263	Appendices A and B
Atterberg Limits	ASTM D4318	Appendices A and B
Grain Size Distribution	ASTM D1140	Appendices A and B
Unconsolidated-Undrained Isotropic Triaxial Compression	ASTM D2850	Appendices A and B
Unconfined Compression	ASTM D2166	Appendices A and B

In addition to the tests listed in Table 2.5-1, corrosivity testing of a sample collected during our exploration was performed by CERCO Analytical. The results are discussed in Section 3.5 and presented in Appendix C.

2.6 SURFACE CONDITIONS

During our site reconnaissance, we observed that the site is paved with asphalt and concrete. The site is relatively level, with a ground surface ranging between approximately Elevation 153 and 158 feet.

2.7 SUBSURFACE CONDITIONS

We encountered existing fill in both borings. The fill, which comprised silt and clayey gravel layers, was approximately 5 feet and 2½ feet thick in borings 1-B1 and 1-B2, respectively. Below the fill and to a depth of approximately 10 feet in both of our borings, we generally encountered medium stiff to stiff lean clay with varying amounts of sand, silt, and gravel, which appeared to be native material. Below 10 feet, our borings generally encountered soft to medium stiff lean clay with variable sand content and interbedded layers of clayey sand and silt extending to approximately 26 feet bgs. Below 26 feet, our borings encountered approximately 3 to 6-foot-thick layers of poorly graded sand and silty sand with varying amounts of fines and gravel to approximately 36½ feet bgs. Below 36½ feet bgs, boring 1-B2 encountered fat clay to the exploration terminus depth of 51½ feet.

The subsurface conditions encountered at the time of the exploration are graphically depicted on our boring logs, Appendix A. The logs contain the soil type, color, consistency, and visual classification in general accordance with the Unified Soil Classification System.

2.8 GROUNDWATER CONDITIONS

We viewed groundwater monitoring data viewed online through Valley Water's groundwater elevation database at https://gis.valleywater.org/GroundwaterElevations/ that indicates the shallowest groundwater depth in the project vicinity has varied between 40 and 50 feet since 2021. A historic shallowest groundwater depth of approximately 25 feet bgs is mapped for the site (Seismic Hazard Zone report, 2000).

Groundwater was encountered between depths of approximately 24½ feet and 63 feet bgs in our CPTs. For purposes of the planning and design of the project, we recommend considering an estimated design groundwater depth of 25 feet bgs. Given the depth of excavation anticipated to remove and replace existing fill, we do not consider shallow groundwater to be of concern.

Fluctuations in the level of groundwater may occur due to variations in rainfall, irrigation practices, and other factors, which may result in groundwater levels that differ from the levels measured during our exploration.

3.0 DISCUSSION AND CONCLUSIONS

We evaluated the site with respect to known potential geologic and geotechnical hazards common to the greater San Francisco Bay Region. We discuss the primary hazards, their anticipated risk of occurrence, and potential impacts on the proposed project in the following sections.

From a geotechnical engineering viewpoint, in our opinion, the site is suitable for the proposed development, provided the geotechnical recommendations in this report are properly incorporated into the design plans and specifications. The primary geotechnical concerns that could affect development on the site are potential existing fill and seismic-induced settlement. We summarize our conclusions below.

3.1 EXISTING FILL

We encountered material in both borings that we identified as fill; because there is no record of fill placement, we recommend it be considered non-engineered. Non-engineered fill can experience excessive settlement, especially under new fill or building loads. Without proper documentation of existing fill placed on the site, we recommend complete removal and recompaction of the existing fill. We present fill removal recommendations in Section 5.2.

3.2 EXPANSIVE SOIL

Expansive soil changes in volume with changes in moisture. It can shrink or swell and cause heaving and cracking of slabs-on-grade, pavements, and structures founded on shallow foundations.

We performed sampling and testing of near-surface soil at the site to characterize the physical properties in relation to expansion potential. The results of Atterberg Limit tests indicate plasticity indexes (PIs) ranging between 6 and 13 for samples collected from the upper 10 feet of the site (in native clayey soil). Our geotechnical laboratory test results indicate that the soil at the project site exhibits low to moderate expansion potential. Site soil that is re-used as engineered fill should be placed in accordance with our fill placement recommendations to reduce the potential for changes in volume.

3.3 SEISMIC HAZARDS

Potential seismic hazards resulting from a nearby moderate to major earthquake can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking, soil liquefaction, and lateral spreading. We discuss these hazards in the following sections.

3.3.1 Ground Rupture

The site is not located within a State of California Earthquake Fault Hazard Zone (Alquist-Priolo Zone) or a Santa Clara County Hazard Zone, and no known active faults cross the site. Therefore, it is our opinion that ground rupture is unlikely at the site.

3.3.2 Ground Shaking

As discussed in Section 2.4, an earthquake of moderate to high magnitude generated within the San Francisco Bay Region could cause considerable ground shaking at the site. To mitigate the shaking effects, all structures should be designed using sound engineering judgment and the latest California Building Code (CBC) requirements as a minimum.

Seismic design provisions of current building codes generally prescribe minimum lateral forces, applied statically to the structure, combined with the gravity forces of dead-and-live loads. The code-prescribed lateral forces are generally substantially smaller than the expected peak forces that would be associated with a major earthquake. Therefore, structures should be able to: (1) resist minor earthquakes without damage, (2) resist moderate earthquakes without structural damage but with some nonstructural damage, and (3) resist major earthquakes without collapse but with some structural, as well as nonstructural damage (SEAOC, 1996). Conformance to the current building code recommendations does not constitute any kind of guarantee that significant structural damage would not occur in the event of a maximum magnitude earthquake; however, it is reasonable to expect that well-designed and well-constructed structures will not collapse or cause loss of life in a major earthquake.

3.4 LIQUEFACTION ANALYSES

The site is located within a mapped State of California Seismic Hazard Zone for areas that may potentially experience liquefaction (Figure 4). Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. The soil considered most susceptible to liquefaction is clean, loose, saturated, uniformly graded fine sand below the groundwater table. Empirical evidence indicates that loose silty sand is also potentially liquefiable. When seismic ground shaking occurs, the soil is subjected to cyclic shear stresses that can cause excess pore water pressures to develop, shear strength to temporarily decrease, and soil to liquefy. If the sand settles or vents to the surface during and following liquefaction, ground settlement and surface deformation may occur. In addition to liquefaction of sandy materials, clayey soil can also experience "cyclic-softening" or strength loss as a result of cyclic loading and resulting strain and pore pressure increase. We do not expect the clay layers encountered at the site to be susceptible to liquefaction due to stiffness and plasticity of clay below the design groundwater table, thus we did not assess the liquefaction potential of fine-grained soil.

We performed an analysis of liquefaction potential based on the CPT data. As previously mentioned, we evaluated liquefaction triggering and associated settlements for sand-like behavior only. For our analysis, we used a design groundwater level of 25 feet bgs and used the mapped Maximum Considered Earthquake (MCE) geometric mean peak ground acceleration (PGA_M) of 0.69g, based on the 2019 California Building Code. We assumed an earthquake Moment Magnitude of 6.9 based on our disaggregation of the 2 percent in 50 years probability Uniform Hazard Spectra. Our CPT-based liquefaction analysis indicates layers of sandy soil with variable fines content encountered, generally between 25 feet and 50 feet bgs, may be susceptible to liquefaction.

We recommend considering total potential liquefaction-induced settlement of 1 inch at the site. We recommend that the proposed structure be designed to accommodate seismically induced differential settlement of up to ½ inch over a lateral distance of 30 feet.

3.5 SOIL CORROSION POTENTIAL

As part of this study, we obtained a representative soil sample and submitted to a qualified analytical lab for determination of pH, resistivity, sulfate, and chloride. The results are included in Appendix A and summarized in the table below.

TABLE 3.6-1: Corrosivity Test Results

SAMPLE LOCATION	DEPTH (FT)	PH	RESISTIVITY (OHMS-CM)	CHLORIDE (MG/KG)	SULFATE (MG/KG)
1-B1	2.5-4	8.08	1,600	N.D.	64

* ASTM D4327

The 2019 CBC references the 2014 American Concrete Institute Manual, ACI 318-14, Section 19.3.1 for concrete durability requirements. ACI Table 19.3.1.1 provides exposure categories and classes, and Table 19.3.2.1 provides requirements for concrete in contact with soil based upon the exposure class.

In accordance with the criteria presented in ACI 318, the tested soil is categorized as Not Applicable, and is within S0 sulfate exposure class and C0 corrosion class. Cement type, water-cement ratio, and concrete strength are not specified for this range. It should be noted, however, that the structural engineering design requirements for concrete may result in more stringent concrete specifications.

Based on the resistivity measurements, the soil is considered corrosive to buried metal piping. All buried metallic piping should be properly protected against corrosion.

If desired to investigate this further, we recommend a corrosion consultant be retained to evaluate if specific corrosion recommendations are advised for the project. Note that ASTM Test Method D4327 was used in lieu of the ACI designated sulfate test methods as it provides more repeatable test results.

3.6 2019 CBC SEISMIC DESIGN PARAMETERS

The 2019 CBC was developed using design criteria in the 2016 ASCE 7-16 Standard. We used in-situ shear-wave velocity measurements from our seismic CPT (Appendix C) to estimate the average shear-wave velocity of the upper 100 feet of site soil. Based on this shear-wave velocity testing, we characterized the site as a Site Class D. We provide the 2019 CBC seismic design parameters in Table 3.6-1 below, which includes design spectral response acceleration parameters based on the mapped Risk-Targeted Maximum Considered earthquake (MCE_R) spectral response acceleration parameters.

TABLE 3.6-1: 2019 CBC Seismic Design Parameters, Latitude: 37.300920 **Longitude:** -121.826333

PARAMETER	VALUE
Site Class	D
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	1.5
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.6
Site Coefficient, F_A	1.0
Site Coefficient, F_V	Null*
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	1.5
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	Null*
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	1.0
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	Null*
MCE_G Peak Ground Acceleration adjusted for Site Class effects, PGA_M (g)	0.686
Long period transition-period, T_L	12 sec

*Required site-specific ground motion hazard analysis per ASCE 7-16 Section 11.4.8.

The CBC indicates that site-specific ground motion hazard analysis if the structural engineer determines there is a need for performing a site-specific seismic-hazard analysis, we can provide a scope for site-specific seismic-hazard analysis and ground motion study under separate cover.

4.0 CONSTRUCTION MONITORING

Our experience and that of our profession clearly indicate that the risk of costly design, construction, and maintenance problems can be significantly lowered by retaining the design geotechnical engineering firm to:

1. Review the final grading and foundation plans and specifications prior to construction to evaluate whether our recommendations have been implemented, and to provide additional or modified recommendations, as needed. This also allows us to check if any changes have occurred in the nature, design or location of the proposed improvements and provides the opportunity to prepare a written response with updated recommendations.

2. Perform construction monitoring to check the validity of the assumptions we made to prepare this report. Earthwork operations should be performed under the observation of our representative to check that the site is properly prepared, the selected fill materials are satisfactory, and that placement and compaction of the fills has been performed in accordance with our recommendations and the project specifications. Sufficient notification to us prior to earthwork is important.

If we are not retained to perform the services described above, then we are not responsible for any party's interpretation of our report (and subsequent addenda, letters, and verbal discussions).

5.0 EARTHWORK RECOMMENDATIONS

The relative compaction and optimum moisture content of soil and aggregate base referred to in this report are based on the most recent ASTM D1557 test method. Compacted soil is not acceptable if it is unstable. It should exhibit only minimal flexing or pumping, as observed by our field representative. The term "moisture condition" refers to adjusting the moisture content of the soil by either drying if too wet or adding water if too dry. We define "structural areas" as any area sensitive to settlement of compacted soil. These areas include, but are not limited to building pads, sidewalks, pavement areas, and retaining walls.

5.1 DEMOLITION AND SITE PREPARATION

Site development should commence with the removal of existing pavement, as well as excavation and removal of existing structures, including utilities and foundation remnants. All debris and soft compressible soil should be removed from any location to be graded, from areas to receive fill or structures, and from areas to serve as borrow. The depth of removal of such materials should be determined by a qualified agency representative at the time of grading.

Existing vegetation should be removed from areas to receive fill or improvements and those areas to serve for borrow. Tree roots should be removed down to a depth of at least 3 feet below existing grade.

5.2 EXISTING FILL REMOVAL

Existing fill should be completely removed beneath the proposed structure footprint. We encountered fill in both borings in thicknesses ranging from approximately 2½ to 5 feet; as such, we anticipate fill beneath the proposed building area.

The depth of removal required to expose native soil will vary across the building footprint. The existing fill should be removed and replaced by compacted engineered fill, placed in accordance with the recommendations in Section 4.6.

5.3 OVER-OPTIMUM SOIL MOISTURE CONDITIONS

The contractor should anticipate encountering excessively over-optimum (wet) soil moisture conditions during winter or spring grading, or during or following periods of rain. Wet soil can make proper compaction difficult or impossible. Wet soil conditions can be mitigated by:

1. Frequent spreading and mixing during warm dry weather.
2. Mixing with drier materials.
3. Mixing with a lime, lime flyash, or cement product.
4. Stabilizing with aggregate, geotextile stabilization fabric, or both.

We should evaluate Options 3 and 4 prior to implementation.

5.4 ACCEPTABLE FILL MATERIAL

With the exception of construction debris (wood, brick, asphalt, concrete, metal, etc.), trees, organically contaminated material (soil which contains more than 3 percent organic content by weight), and otherwise unsuitable soil, we anticipate the site soil is suitable for use as engineered fill. Unsuitable material and debris, including trees with their roots and particles larger than 6 inches, should be removed from the project site. Oversized soil or rock material (those exceeding two-thirds of the lift thickness or 6 inches in dimension, whichever is less) should be removed from the fill and broken down to meet this requirement or otherwise offhauled.

Import fill should meet the recommendations above, have a PI less than 25 and at least 20 percent by dry weight passing the No. 200 sieve.

5.5 REUSE OF ON-SITE RECYCLED MATERIALS

Asphalt or Portland Cement concrete recycled from existing pavement may be re-used as general structural fill beneath the building pad and proposed improvements.

The material will need to be broken down, but not pulverized, to have a maximum particle size less than 6 inches if used for fill. Materials planned for reuse as roadway base should be tested to confirm Caltrans Class 2 aggregate base specifications are met. The material should be moisture conditioned and compacted according to the specifications in Section 4.7.

It should be noted that materials derived from crushed concrete and asphalt are generally not suitable for supporting plant growth. The landscape designer should be consulted for additional information.

5.6 FILL PLACEMENT

5.6.1 Grading in Structural Areas

After preparing the site, as recommended in Sections 5.1 and 5.2, the contractor should mechanically compact the subgrade in accordance with the recommendations in this section. The loose lift thickness should not exceed 8 inches or the depth of penetration of the compaction equipment used, whichever is less. Engineered fill should be compacted and moisture conditioned in accordance with the requirements presented in Table 5.6.1-1.

TABLE 5.6.1-1: Engineered Fill Compaction and Moisture Content Requirements

MATERIALS	MINIMUM RELATIVE COMPACTION (%)	MINIMUM RELATIVE COMPACTION (%) - UPPER 12 INCHES OF FILL IN PAVEMENT AREAS	MINIMUM MOISTURE CONTENT (percentage points above optimum)
Engineered Fill (all acceptable fill)	90	95	3
Aggregate Base	95	--	0

5.6.2 Underground Utility Backfill

Project consultants involved in utility design should specify pipe-bedding materials. Utility trench backfill should conform to the recommendations in Section 5.6.1.

Care should be exercised where trenches are located beside foundation areas. Trenches constructed parallel to foundations should be located entirely above a plane extending down from the lower edge of the footing at an angle of 45 degrees.

The contractor is responsible for conducting trenching and shoring in accordance with CAL/OSHA requirements. Compaction of the pipe bedding or backfill by means of jetting or flooding should not be allowed.

5.7 SURFACE DRAINAGE

The project civil engineer is responsible for designing surface drainage improvements. With regard to geotechnical engineering issues, we recommend that finish grades be sloped away from the building and other surface improvements to the maximum extent practical. The latest California Building Code Section 1804.4 specifies minimum slopes of 5 percent away from foundation for pervious surfaces. As a minimum, we recommend the following.

1. Roof downspouts should discharge into closed conduits and direct away from foundations to appropriate drainage devices.

2. Water should not be allowed to pond near foundations, pavements, or exterior flatwork.

6.0 FOUNDATION RECOMMENDATIONS

We developed foundation recommendations using data obtained from our field exploration, laboratory test results, and engineering analysis. We recommend supporting the proposed building on a spread footing foundation system. Alternatively, the office building may be supported on a structural mat foundation.

The foundation system should be designed to accommodate estimated liquefaction-induced differential settlement of up to ½ inch over a horizontal distance of 30 feet.

6.1 SPREAD FOOTING DESIGN

Table 6.1-1 provides our recommendations for minimum footing dimensions for footings excavated into firm native soil and/or competent engineered fill. Based on these footing dimensions, we recommend and allowable bearing capacity of 2,500 pounds per square foot (psf) for dead-plus-live loads; we anticipate footings designed in accordance with the dimensions recommended in the following section will experience approximately ½ inch of total static settlement with a differential of approximately one-half the total. The allowable bearing capacity can be increased by one-third for the short-term effects of wind or seismic loading.

TABLE 6.1-1: Minimum Footing Dimensions

FOOTING TYPE	MINIMUM DEPTH (INCHES)*	MINIMUM WIDTH (INCHES)
Continuous	24	12
Isolated	24	18

*Below lowest adjacent finish grade.

The structural engineer should design footing reinforcement to support the intended structural loads without excessive settlement. Continuous footings should be reinforced with top and bottom steel to provide structural continuity and to permit spanning of local irregularities. At a minimum, continuous footings should be designed to structurally span a clear distance of 5 feet and include, at a minimum, at least four No. 4 steel reinforcement bars, two top and two bottom.

Lateral loads may be resisted by friction along the base and by passive pressure along the sides of footings. The passive pressure is based on an equivalent fluid pressure in pounds per cubic foot (pcf). We recommend the following ultimate values.

- Passive Lateral Pressure: 350 pcf
- Coefficient of Friction: 0.35

The ultimate values above should be factored, as appropriate, based on the design case and design method used. If the two resistance values are combined, one should be reduced by half to address strain incompatibility in developing the peak value of each resistance mechanism.

If a two-pour system is used for footings and slab, the cold joint between the exterior footing and slab-on-grade should be located at least 4 inches above adjacent finish exterior grade. If this is not done, then we recommend the addition of a waterstop between the two pours to reduce moisture penetration through the cold joint and migration under the slab. Use of a monolithic pour would eliminate the need for the waterstop.

We recommend the slab-on-grade floor have a minimum thickness of 5 inches and minimum steel reinforcing of No. 3 rebar on 18-inch centers each way within the middle third of the slab to help control the width of shrinkage cracking that inherently occurs as concrete cures.

6.2 MAT FOUNDATIONS

Alternatively, the structure may be supported on a rigid mat foundation. The thickness of the mat will be driven by the structural design. The structural mat should be designed to impose an average allowable bearing pressure of at most 1,500 pounds per square foot (psf) for dead-plus-live loads. The allowable bearing capacity may be increased to 2,500 psf in areas of loading concentration. If a spring constant is needed for design, a modulus of subgrade reaction (k_s) of 35 pounds per square inch per inch of deflection (psi/in) can be used.

Lateral loads may be resisted by friction along the base of the mat using an ultimate coefficient of friction of 0.2 (assuming a slab underlayment as described below). If the mat has thickened areas, or footings, at columns, an ultimate passive resistance of 350 pcf may be used. We recommend these values be factored, as appropriate, based on the design case and design method used. If the two resistance values are combined, one should be reduced by half to address strain incompatibility in developing the peak value of each resistance mechanism.

6.3 FOUNDATION SUBGRADE AND UNDERLAYMENT

The pad subgrade should not be allowed to dry before placing concrete. The pad subgrade should be checked by a representative of our firm prior to concrete placement for compliance with the moisture requirements and to confirm the adequacy of the bearing soil. Soft or loose soil present at the bottom of the excavation should be removed and replaced with engineered fill or lean concrete. To reduce the disturbance of the building pad once prepared, a “rat” or “mud” slab of lean concrete at least 2 inches thick can be used.

When buildings are constructed with concrete slab-on-grade, water vapor from beneath the slab will migrate through the slab and into the building. This water vapor can be reduced but not stopped. Vapor transmission can negatively affect floor coverings and lead to increased moisture within a building. When water vapor migrating through the slab would be undesirable, we recommend the following to reduce, but not stop, water vapor transmission upward through the slab-on-grade.

1. A moisture retarder system should be constructed directly beneath the slab-on-grade that consists of a vapor retarder membrane sealed at all seams and pipe penetrations and connected to all footings. Vapor retarders should conform to Class A vapor retarder in accordance with ASTM E 1745, latest edition, “Standard Specification for Plastic Water Vapor Retarders used in Contact with Soil or Granular Fill under Concrete Slabs”.

2. A concrete water-cement ratio for slabs-on-grade of no more than 0.50 should be used.

3. Inspection and testing during concrete placement should be performed to check that the proper concrete and water-cement ratio are used.

4. The slab should be moist cured for a minimum of 3 days or use other equivalent curing specified by the structural engineer.

If the option of spread footings and a slab-on-grade floor are used, we recommend placement of a 4-inch-thick layer of ¾-inch clean, crushed rock be placed below the vapor retarder membrane to act as a capillary break.

7.0 RETAINING WALLS

7.1 LATERAL SOIL PRESSURES

We understand that site retaining walls with maximum heights of up to 5 feet are proposed for the project.

Unrestrained site retaining walls should be designed to resist an active equivalent fluid-pressure of 45 pcf plus one-third of any surcharge loads located within a distance equal to the height of the wall. This lateral earth pressure assumes level backfill conditions and sufficient drainage behind the walls to prevent any build-up of hydrostatic pressures from surface water infiltration and/or a rise in the groundwater level. If adequate drainage is not provided, we recommend that an additional equivalent fluid pressure of 40 pcf be added to the value recommended. Damp-proofing of the walls should be included in areas where wall moisture would be problematic.

A drainage system, as recommended below, should be constructed behind the wall to reduce hydrostatic forces behind the retaining wall.

7.2 RETAINING WALL DRAINAGE

The design of retaining walls should include either graded rock drains or geosynthetic drainage composites behind the retaining walls to reduce hydrostatic lateral forces. For rock drain construction, we recommend two types of rock drain alternatives.

1. A minimum 12-inch-thick layer of Class 2 Permeable Filter Material (Caltrans Specification 68-2.02F) placed directly behind the wall, or

2. A minimum 12-inch-thick layer of washed, crushed rock with 100 percent passing the ¾-inch sieve and less than 5 percent passing the No. 4 sieve. Envelop rock in a minimum 6-ounce, nonwoven geotextile filter fabric.

For both types of rock drains:

1. The rock drain should be placed directly behind the walls of the structure.

2. The rock drains should extend from the wall base to within 12 inches of the top of the wall.

3. A minimum of 4-inch-diameter perforated pipe (glued joints and end caps) should be placed at the base of the wall, inside the rock drain and fabric, with perforations placed down.

4. The pipe should be placed at a gradient at least 1 percent to direct water away from the wall by gravity to a drainage facility.

We should review and approve geosynthetic composite drainage systems prior to use.

7.3 BACKFILL

Backfill behind retaining walls should be placed and compacted in accordance with Section 5.6. Light compaction equipment should be used within 5 feet of the wall face. If heavy compaction equipment is used, the walls should be temporarily braced to avoid excessive wall movement.

7.4 FOUNDATIONS

Retaining walls may be supported on footings designed in accordance with recommendations presented in Section 6.1.

8.0 PAVEMENT DESIGN

8.1 FLEXIBLE PAVEMENTS

We developed the following pavement sections for parking areas and access streets that will be used by passenger vehicles, using traffic indexes of 4 to 10, based on an assumed R-value of 5 and Topic 633 of the Caltrans Highway Design Manual (including the asphalt factor of safety).

TABLE 8.1-1: Recommended Asphalt Concrete Pavement Sections

TRAFFIC INDEX	SECTION	
	ASPHALT CONCRETE (inches)	CLASS 2 AB (inches)
4	3	8
5	3	10
6	3½	13
7	4	1
8	5	18
9	5 ½	21
10	4 ½	26

Note: AB is aggregate base Class 2 Material with minimum R = 78.

The civil engineer should evaluate the appropriate traffic indexes based on the estimated traffic loads and frequencies.

8.2 RIGID PAVEMENTS

Concrete pavement sections can be used to resist heavy loads and turning forces in areas such as fire lanes or trash enclosures. Final design of rigid pavement sections and reinforcement should be performed based on estimated traffic loads and frequencies. We recommend the following minimum design sections for rigid pavements.

- A minimum section of 6 inches of Portland Cement concrete over 6 inches of Caltrans Class 2 aggregate base should be used.
- Concrete used should have a minimum 28-day compressive strength of 3,500 psi.
- Control joint spacing should meet the minimums in accordance with Portland Cement Association guidelines.

8.3 PAVEMENT SUBGRADE PREPARATION

Pavement subgrade may consist of site soil, provided it is moisture conditioned and placed in accordance with the requirements described in Section 5.6.

Pavement subgrade preparation should comply with the following minimum requirements.

- All pavement subgrade should be scarified to a depth of 10 inches below finished subgrade elevation and compacted in accordance with Section 5.6.1. Pavement subgrade should also be prepared in accordance with City of San Jose requirements if necessary.

- Subgrade soil should be in a stable, non-pumping condition at the time aggregate baserock materials are placed and compacted.

- Proof-rolling the subgrade and aggregate baserock with a heavy wheel-loaded piece of construction equipment should be implemented. Yielding materials should be appropriately mitigated with suitable mitigation measures developed in coordination with the client, contractor, and our representative.

- Adequate provisions should be made such that the subgrade soil and aggregate baserock materials are not allowed to become overly wet.

- Aggregate baserock materials should meet current Caltrans specifications for Class 2 aggregate baserock and should be compacted in accordance with Section 5.6.

8.4 CUTOFF CURBS

Overly wet pavement subgrade or aggregate base can cause premature failure or increased maintenance of asphalt concrete pavements. This condition often occurs where landscape areas directly abut and drain towards pavement. If it is desired to install pavement cutoff barriers, they should be placed where pavement areas lie downslope of any landscape areas that are to be sprinklered or irrigated and should extend to a depth of at least 6 inches below the base rock layer. Cutoff barriers may consist of deepened concrete curbs or deep-root moisture barriers.

If reduced pavement life and greater-than-normal pavement maintenance are acceptable to the owner, the cutoff barrier may be eliminated.

8.5 EXTERIOR FLATWORK

Exterior flatwork includes items such as concrete sidewalks, steps, and outdoor plazas exposed to foot traffic only. Concrete flatwork should have a minimum thickness of 4 inches and include control and construction joints in accordance with current Portland Cement Association guidelines.

To improve long-term performance, secondary slabs-on-grade may be underlain by a 4-inch-thick layer of Class 2 aggregate base.

Exterior slabs should slope away from the building to prevent water from flowing toward the foundations. Site soil should remain moist prior to concrete placement.

We recommend that flatwork leading to the building entrance area be structurally independent of the structure foundation to allow for differential movement between the flatwork and the building. Where smooth transition to provide access is necessary (ADA ramps), a hinge-slab should be designed to accommodate movements of approximately 1 inch, including both static settlement and liquefaction-induced settlement. Flatwork should be reinforced to allow for the appropriate span in the event of settlement.

9.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents geotechnical recommendations for design of the improvements discussed in Section 1.3 for the Municipal Water Office Redevelopment project. If changes occur in the nature or design of the project, we should be allowed to review this report and provide additional recommendations, if any. It is the responsibility of the owner to transmit the information and recommendations of this report to the appropriate organizations or people involved in design of the project, including but not limited to developers, owners, buyers, architects, engineers, and designers. The conclusions and recommendations contained in this report are solely professional opinions and are valid for a period of no more than two years from the date of report issuance.

We strived to perform our professional services in accordance with generally accepted principles and practices currently employed in the area; there is no warranty, express or implied. There are risks of earth movement and property damages inherent in building on or with earth materials. We are unable to eliminate all risks; therefore, we are unable to guarantee or warrant the results of our services.

This report is based upon field and other conditions discovered at the time of report preparation. We developed this report with limited subsurface exploration data. We assumed that our subsurface exploration data are representative of the actual subsurface conditions across the site. Considering possible underground variability of soil and groundwater, additional costs may be required to complete the project. We recommend that the owner establish a contingency fund to cover such costs. If unexpected conditions are encountered, we must be notified immediately to review these conditions and provide additional and/or modified recommendations, as necessary.

Our services did not include excavation sloping or shoring, soil volume change factors, or flood potential. In addition, our geotechnical exploration did not include work to determine the existence of possible hazardous materials. If any hazardous materials are encountered during construction, the proper regulatory officials must be notified immediately.

This document must not be subject to unauthorized reuse, that is, reusing without written authorization of our firm. Such authorization is essential because it requires our firm to evaluate the document's applicability given new circumstances, not the least of which is passage of time.

Actual field or other conditions will necessitate clarifications, adjustments, modifications, or other changes to our documents. Therefore, we must be engaged to prepare the necessary clarifications, adjustments, modifications, or other changes before construction activities commence or further activity proceeds. If our scope of services does not include on-site construction observation, or if other persons or entities are retained to provide such services, we cannot be held responsible for any or all claims arising from or resulting from the performance of such services by other persons or entities, and from any or all claims arising from or resulting from clarifications, adjustments, modifications, discrepancies, or other changes necessary to reflect changed field or other conditions.

We determined the lines designating the interface between layers on the exploration logs using visual observations. The transition between the materials may be abrupt or gradual. The exploration logs contain information concerning samples recovered, indications of the presence of various materials such as clay, sand, silt, rock, existing fill, etc., and observations of groundwater encountered. The logs also contain our interpretation of the subsurface conditions between sample locations. Therefore, the logs contain both factual and interpretative information.

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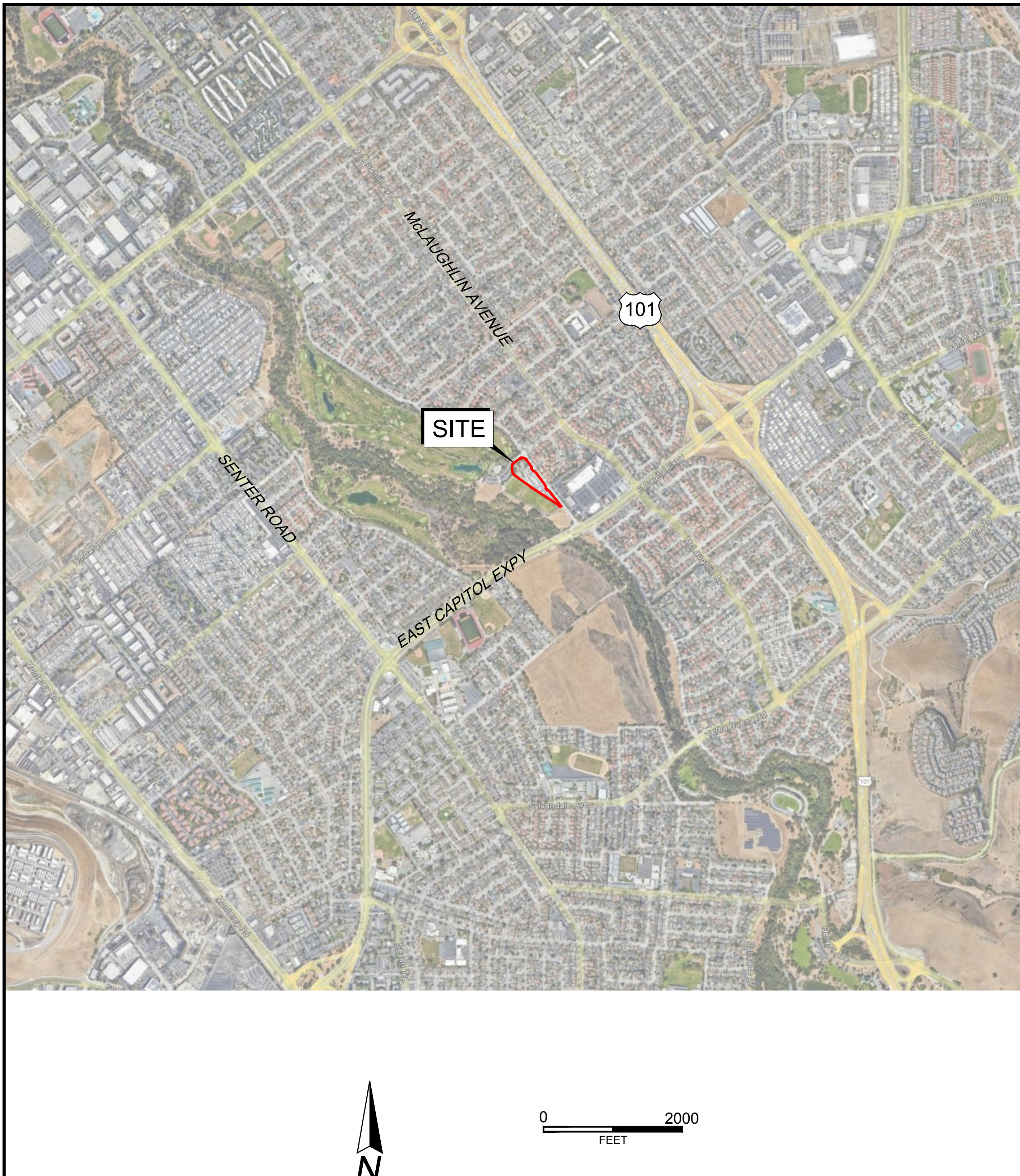
HMH; Utility Coordination, San Jose Municipal Water System, San Jose, California; July 19, 2021, HMH #5889.00.

TRC; Geotechnical Investigation, Four Photovoltaic Solar Arrays for the City of San Jose, San Jose, California, January 5, 2012, Report Number 188828

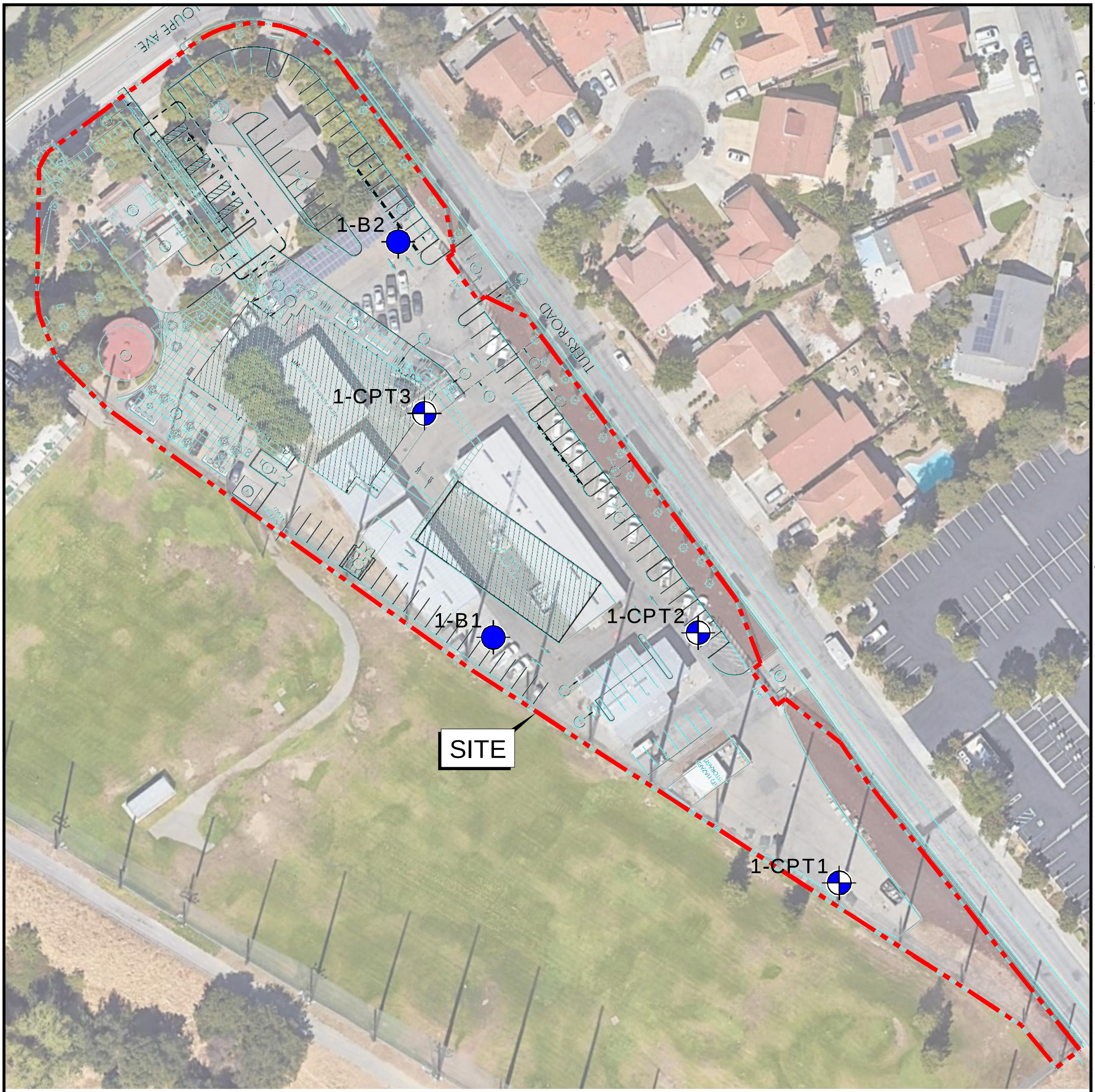
FIGURES



BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

ENGEO Expect Excellence	VICINITY MAP 3025 TUERS ROAD SAN JOSE, CALIFORNIA	PROJECT NO.: 19886.000.001		FIGURE NO. 1
		SCALE: AS SHOWN		
		DRAWN BY: CC	CHECKED BY: JAF	

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

1-B2 BORING (ENGEO, 2022)

1-CPT3 CONE PENETRATION TEST (ENGEO, 2022)

N

0 100 FEET

BASE MAP SOURCE: GOOGLE EARTH MAPPING SOURCE, HMH 2021

ENGEO Expect Excellence	SITE PLAN 3025 TUERS ROAD SAN JOSE, CALIFORNIA	PROJECT NO.: 19886.000.001		FIGURE NO. 2
		SCALE: AS SHOWN		
		DRAWN BY: CC	CHECKED BY: JAF	

ORIGINAL FIGURE PRINTED IN COLOR

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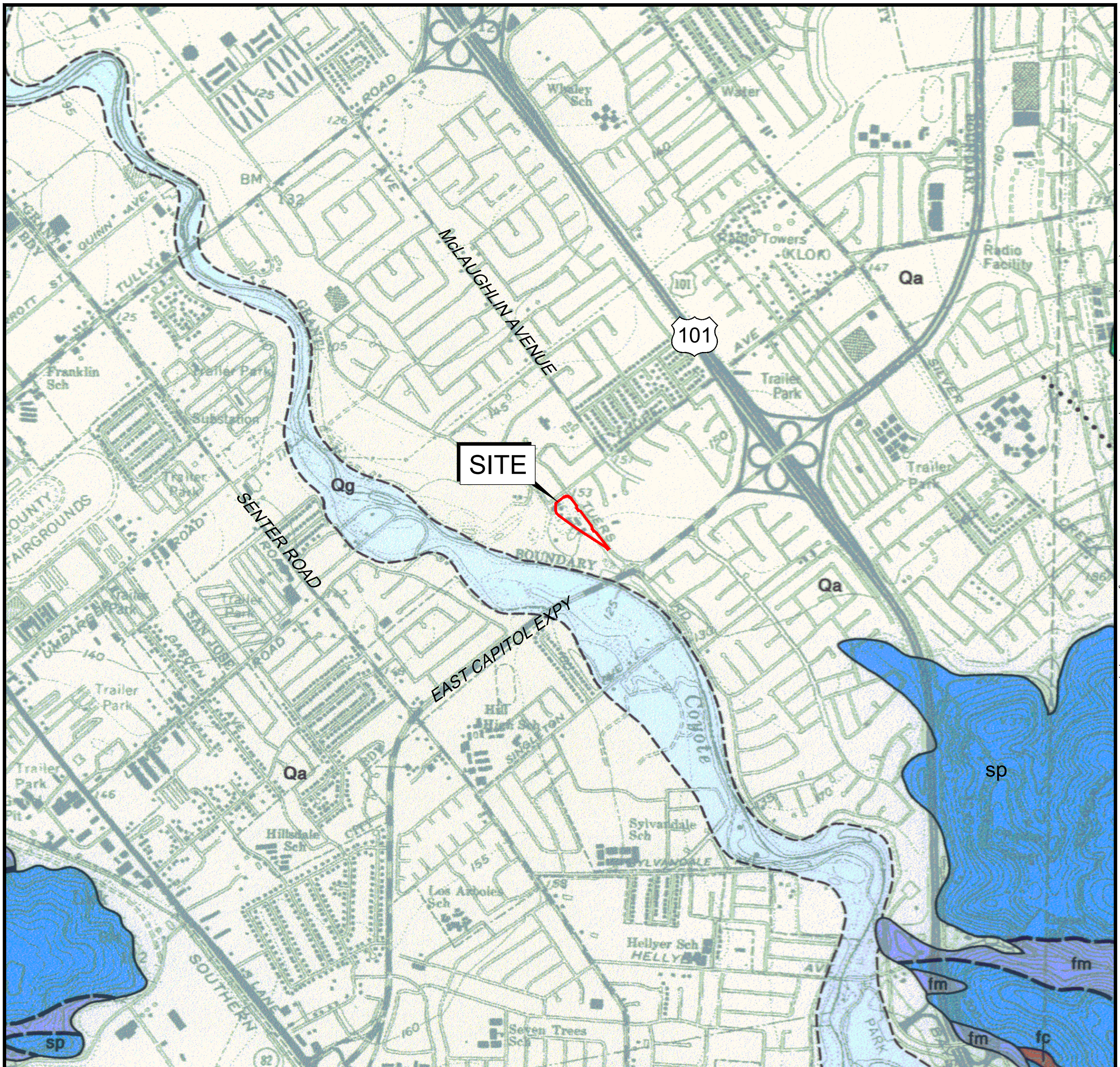


EXPLANATION

——— – – –	GEOLOGIC CONTACT-DASHED WHERE GRADATIONAL OR APPROXIMATELY LOCATED	Qg	SAND AND GRAVEL OF COYOTE CREEK CHANNEL
——▲—▲– – –·	FAULT-DASHED WHERE INFERRED, DOTTED WHERE CONCEALED, QUERIED WHERE EXISTENCE IS DOUBTFUL. SAWTEETH ARE ON UPPER PLATE OF LOW ANGLE THRUST FAULT	Qa	ALLUVIAL GRAVEL
		sp	SERPENTINITE
		fm	MELANGE
		fc	CHERT OR METACHERT

N

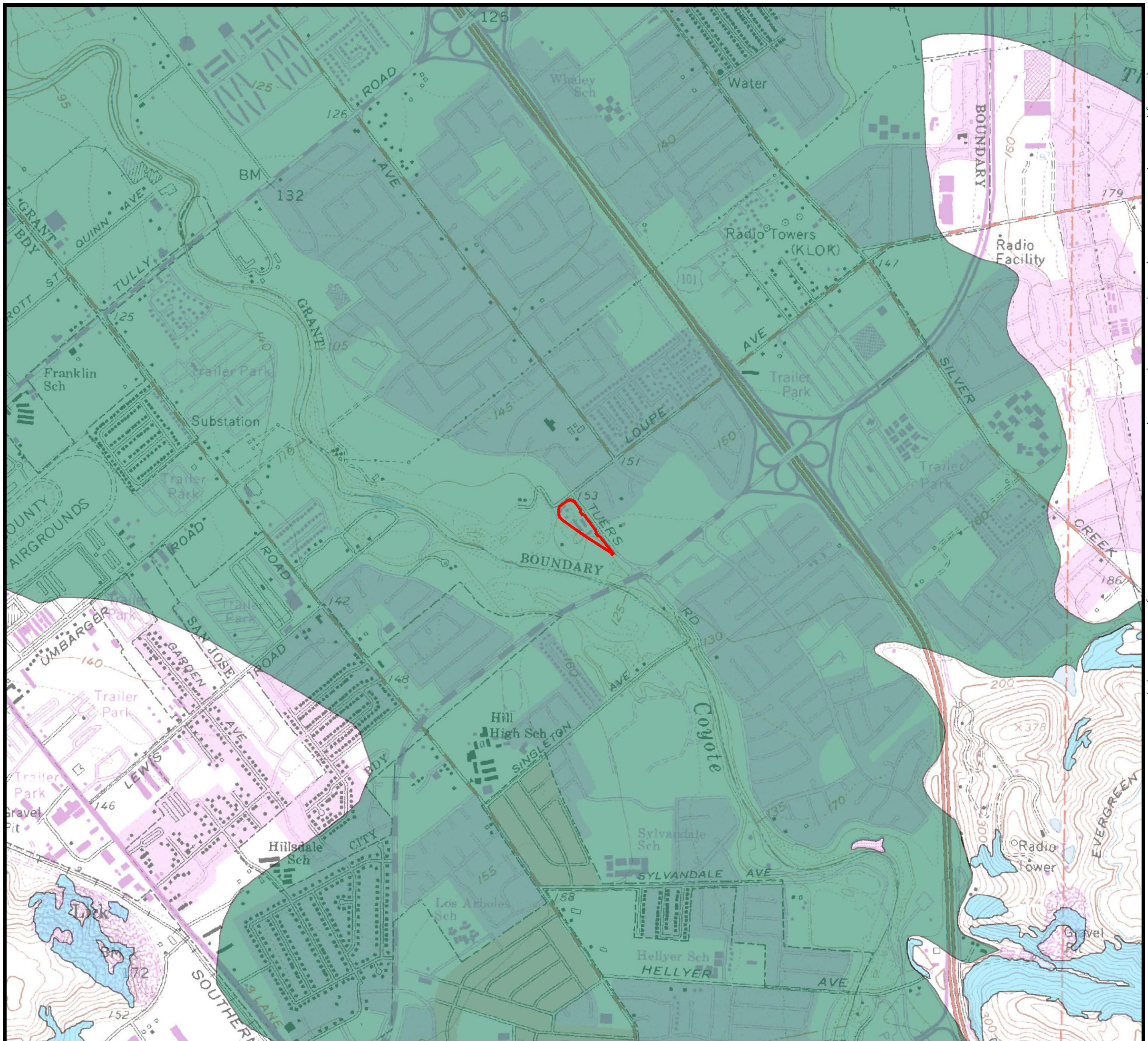
0 2000
FEET

BASE MAP SOURCE: DIBBLEE, 2005

ENGEO
Expect Excellence

REGIONAL GEOLOGIC MAP	PROJECT NO.: 19886.000.001	FIGURE NO. 3
3025 TUERS ROAD	SCALE: AS SHOWN	
SAN JOSE, CALIFORNIA	DRAWN BY: CC / CHECKED BY: JAF	

ORIGINAL FIGURE PRINTED IN COLOR



N

0 2000
FEET

EXPLANATION

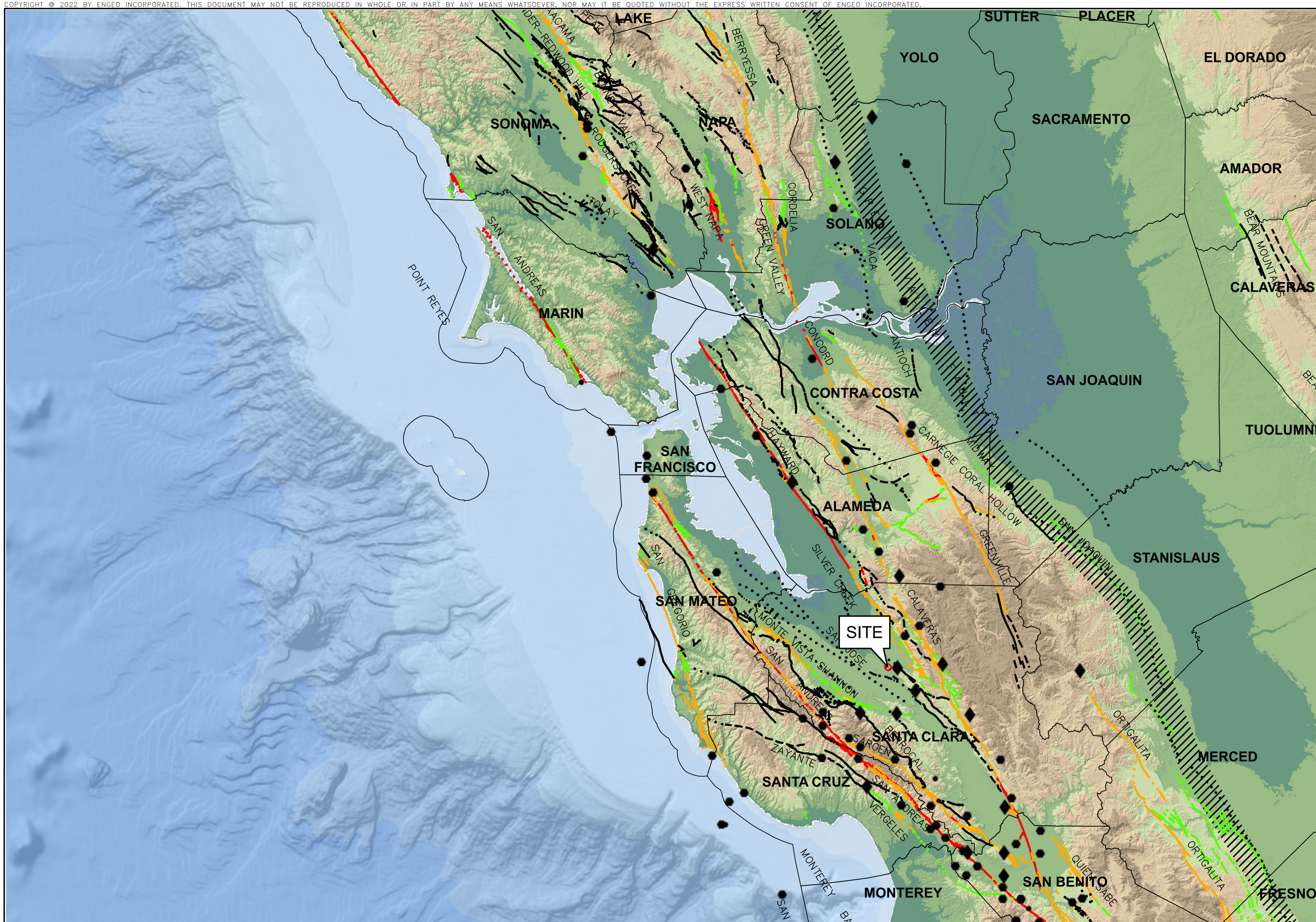
LIQUEFACTION

AREAS WHERE HISTORIC OCCURRENCE OF LIQUEFACTION, OR LOCAL GEOLOGICAL, GEOTECHNICAL AND GROUNDWATER CONDITIONS INDICATE A POTENTIAL FOR PERMANENT GROUND DISPLACEMENTS SUCH THAT MITIGATION AS DEFINED IN PUBLIC RESOURCES CODE SECTION 2693(c) WOULD BE REQUIRED

BASE MAP SOURCE: CALIFORNIA DEPARTMENT OF CONSERVATION, CALIFORNIA GEOLOGICAL SURVEY, 2006

ENGEO Expect Excellence	SEISMIC HAZARD ZONES MAP 3025 TUERS ROAD SAN JOSE, CALIFORNIA	PROJECT NO.: 19886.000.001		FIGURE NO. 4
		SCALE: AS SHOWN		
		DRAWN BY: CC	CHECKED BY: JAF	

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

EARTHQUAKE

- MAGNITUDE 7+
- MAGNITUDE 6-7
- MAGNITUDE 5-6

QUATERNARY FAULTS

BASED ON TIME OF MOST RECENT SURFACE DEFORMATION

- HISTORICAL (<150 YEARS), WELL CONSTRAINED LOCATION
- HISTORICAL (<150 YEARS), MODERATELY CONSTRAINED LOCATION
- HISTORICAL (<150 YEARS), INFERRED LOCATION
- LATEST QUATERNARY (<15,000 YEARS), WELL CONSTRAINED LOCATION
- LATEST QUATERNARY (<15,000 YEARS), MODERATELY CONSTRAINED LOCATION
- LATEST QUATERNARY (<15,000 YEARS), INFERRED LOCATION
- LATE QUATERNARY (<130,000 YEARS), WELL CONSTRAINED LOCATION
- LATE QUATERNARY (<130,000 YEARS), MODERATELY CONSTRAINED LOCATION
- LATE QUATERNARY (<130,000 YEARS), INFERRED LOCATION
- UNDIFFERENTIATED QUATERNARY(<1.6 MILLION YEARS), WELL CONSTRAINED LOCATION
- UNDIFFERENTIATED QUATERNARY(<1.6 MILLION YEARS), MODERATELY CONSTRAINED LOCATION
- UNDIFFERENTIATED QUATERNARY(<1.6 MILLION YEARS), INFERRED LOCATION
- GREAT VALLEY FAULT ZONE

BASE MAP SOURCE
ESRI, GEBCO, DELORME, NATURALVUE
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATA SET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, 2018
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800-PRESENT)

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REGIONAL FAULTING AND SEISMICITY
3025 TUERS ROAD
SAN JOSE, CALIFORNIA

PROJECT NO.: 19886.000.001	FIGURE NO. 5
SCALE: AS SHOWN	
DRAWN BY: CC	CHECKED BY: JAF

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LAYOUT: FAULT&SEISMICITY USER: CCULBERTSON

ORIGINAL FIGURE PRINTED IN COLOR

APPENDIX A

BORING LOG KEY
EXPLORATION LOGS

KEY TO BORING LOGS

MAJOR TYPES			DESCRIPTION
COARSE-GRAINED SOILS MORE THAN HALF OF MAT'L LARGER THAN #200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LESS THAN 5% FINES	GW - Well graded gravels or gravel-sand mixtures
			GP - Poorly graded gravels or gravel-sand mixtures
		GRAVELS WITH OVER 12 % FINES	GM - Silty gravels, gravel-sand and silt mixtures
			GC - Clayey gravels, gravel-sand and clay mixtures
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LESS THAN 5% FINES	SW - Well graded sands, or gravelly sand mixtures
			SP - Poorly graded sands or gravelly sand mixtures
		SANDS WITH OVER 12 % FINES	SM - Silty sand, sand-silt mixtures
			SC - Clayey sand, sand-clay mixtures
FINE-GRAINED SOILS MORE THAN HALF OF MAT'L SMALLER THAN #200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50 % OR LESS		ML - Inorganic silt with low to medium plasticity
			CL - Inorganic clay with low to medium plasticity
			OL - Low plasticity organic silts and clays
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50 %		MH - Elastic silt with high plasticity
			CH - Fat clay with high plasticity
			OH - Highly plastic organic silts and clays
	HIGHLY ORGANIC SOILS		PT - Peat and other highly organic soils

For fine-grained soils with 15 to 29% retained on the #200 sieve, the words "with sand" or "with gravel" (whichever is predominant) are added to the group name.

For fine-grained soil with >30% retained on the #200 sieve, the words "sandy" or "gravelly" (whichever is predominant) are added to the group name.

GRAIN SIZES

U.S. STANDARD SERIES SIEVE SIZE: 200, 40, 10, 4

CLEAR SQUARE SIEVE OPENINGS: 3/4 ", 3", 12"

SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

RELATIVE DENSITY

SANDS AND GRAVELS	BLOWS/FOOT (S.P.T.)
VERY LOOSE	0-4
LOOSE	4-10
MEDIUM DENSE	10-30
DENSE	30-50
VERY DENSE	OVER 50

CONSISTENCY

SILTS AND CLAYS	STRENGTH*
VERY SOFT	0-1/4
SOFT	1/4-1/2
MEDIUM STIFF	1/2-1
STIFF	1-2
VERY STIFF	2-4
HARD	OVER 4

MOISTURE CONDITION

DRY	Dusty, dry to touch
MOIST	Damp but no visible water
WET	Visible freewater

SAMPLER SYMBOLS

- Modified California (3" O.D.) sampler
- California (2.5" O.D.) sampler
- S.P.T. - Split spoon sampler
- Shelby Tube
- Dames and Moore Piston
- Continuous Core
- Bag Samples
- Grab Samples
- NR No Recovery

LINE TYPES

- Solid - Layer Break
- Dashed - Gradational or approximate layer break

GROUNDWATER SYMBOLS

- Groundwater level during drilling
- Stabilized groundwater level

(S.P.T.) Number of blows of 140 lb. hammer falling 30" to drive a 2-inch O.D. (1-3/8 inch I.D.) sampler

* Unconfined compressive strength in tons/sq. ft., asterisk on log means determined by pocket penetrometer

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LOG OF BORING 1-B1

LATITUDE: 37.3005 LONGITUDE: -121.8262

Geotechnical Exploration
Municipal Water Office Redevelopment
San Jose, California
19886.000.001

DATE DRILLED: 2/7/2022
HOLE DEPTH: Approx. 31½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS 84): Approx. 158 ft.

LOGGED / REVIEWED BY: E. Korogianos / JF
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	DESCRIPTION	Blow Count/Foot	Liquid Limit	Plastic Limit	Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
		ASPHALT								
		SILT WITH SAND (ML), brown, stiff to very stiff, moist, low plasticity, 10 - 20% fine-grained sand [FILL]	20	42	29	13		16	115	
	155		5							
5		CLAYEY GRAVEL (GC), brown, dense, moist, medium plasticity, 15-20% fines, 10-15% fine- to coarse-grained sand [FILL]								
		LEAN CLAY (CL), brown, stiff to very stiff, moist, medium plasticity, <15% fine-grained sand, contains silt fines [NATIVE]	18					17	111	
	150									
		SANDY LEAN CLAY (CL), brown, stiff to very stiff, moist, medium plasticity, 25-30% fine-grained sand, contains silt fines								
10			13				73			4.25*
		SILT WITH SAND (ML), brown, very stiff, moist, low plasticity, 15-25% fine-grained sand								
	145									
		LEAN CLAY WITH SAND (CL), brown, very stiff, moist, medium plasticity, 15-20% fine- to medium-grained sand								
15			16							3.0*
	140									
		LEAN CLAY (CL), brown, very stiff, moist, medium plasticity, <15% fine-grained sand								
20										

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LOG OF BORING 1-B1

LATITUDE: 37.3005 LONGITUDE: -121.8262

Geotechnical Exploration
Municipal Water Office Redevelopment
San Jose, California
19886.000.001

DATE DRILLED: 2/7/2022
HOLE DEPTH: Approx. 31½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS 84): Approx. 158 ft.

LOGGED / REVIEWED BY: E. Korogianos / JF
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
			LEAN CLAY (CL), brown, very stiff, moist, medium plasticity, <15% fine-grained sand			21							4.0*
	135		SANDY SILT (ML), brown, stiff, moist, low plasticity, 25-35% fine- to coarse-grained sand										
25			SILTY SAND (SM), brown, dense, moist, fine- to medium-grained sand, 10-20% fines			14				26			
	130		CLAYEY SAND (SC), brown, dense, moist, fine- to coarse-grained sand, 20-30% fines										
30			CLAYEY SAND WITH GRAVEL (SP-SC), brown, dense, moist, fine- to coarse-grained sand, 15-25% gravel, 10-15% fines			24							

Bottom of boring at 31 1/2 feet below ground surface. Groundwater was not encountered during drilling. Boring was backfilled with neat cement.

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LOG OF BORING 1-B2

LATITUDE: 37.3012 LONGITUDE: -121.8263

Geotechnical Exploration
Municipal Water Office Redevelopment
San Jose, California
19886.000.001

DATE DRILLED: 2/7/2022
HOLE DEPTH: Approx. 51½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS 84): Approx. 155 ft.

LOGGED / REVIEWED BY: E. Korogianos / JF
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: SFA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
			ASPHALT										
			SILT WITH SAND (ML), brown to grayish brown, moist, 15-20% fine-grained sand [FILL]			6	33	24	9		12	84	
			LEAN CLAY (CL), brown to grayish brown, moist, low plasticity, <15% fine-grained sand, contains silt fines [Native]			8							
5	150					23					12	101	
10	145					20	32	20	12				
			contains silt fines										
15	140												
			SILT WITH SAND (ML), brown, moist, medium plasticity, 15-20% fine-grained sand			19	30	24	6	82			
20	135												

ENGEO
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LOG OF BORING 1-B2

LATITUDE: 37.3012 LONGITUDE: -121.8263

Geotechnical Exploration
Municipal Water Office Redevelopment
San Jose, California
19886.000.001

DATE DRILLED:	2/7/2022	LOGGED / REVIEWED BY:	E. Korogianos / JF
HOLE DEPTH:	Approx. 51½ ft.	DRILLING CONTRACTOR:	Britton Exploration
HOLE DIAMETER:	4.0 in.	DRILLING METHOD:	SFA, Switch to Mud
SURF ELEV (WGS 84):	Approx. 155 ft.	HAMMER TYPE:	140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
			SILT WITH SAND (ML), brown, moist, medium plasticity, 15-20% fine-grained sand			23							
25	130		LEAN CLAY WITH SAND (CL), grayish brown, stiff to very stiff, moist, medium plasticity, 15-20% fine-grained sand										
			SILTY SAND (SM), brown, medium dense, moist, low plasticity, 30-35% fines, fine- to coarse-grained sand			22				33			
			CLAYEY SAND (SC), brown, medium dense, moist, medium plasticity, 10-15% fines, fine- to coarse-grained sand			23							
30	125		CLAYEY GRAVEL WITH SAND (GC), brown, medium dense, moist, medium plasticity, 10-20% fine- to coarse-grained sand, subangular to subrounded gravels			14				17			
			SILTY SAND (SM), brown, medium dense, moist, low plasticity, 10-20% fines, fine- to coarse-grained sand										
35	120		FAT CLAY (CH), olive brown, soft, moist, high plasticity, <10% fine-grained sand			9					33	90	
			LEAN CLAY (CL), dark grayish brown, soft to medium stiff, moist, medium plasticity, contains silt fines										
40	115		FAT CLAY WITH SAND (CH), olive brown with grayish brown, soft to medium stiff, moist, high plasticity, 15-20% fine- to coarse-grained sand										

LOG OF BORING 1-B2

ENGEO
Expect Excellence

LATITUDE: 37.3012 LONGITUDE: -121.8263

Geotechnical Exploration
Municipal Water Office Redevelopment
San Jose, California
19886.000.001

DATE DRILLED: 2/7/2022
HOLE DEPTH: Approx. 51½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS 84): Approx. 155 ft.

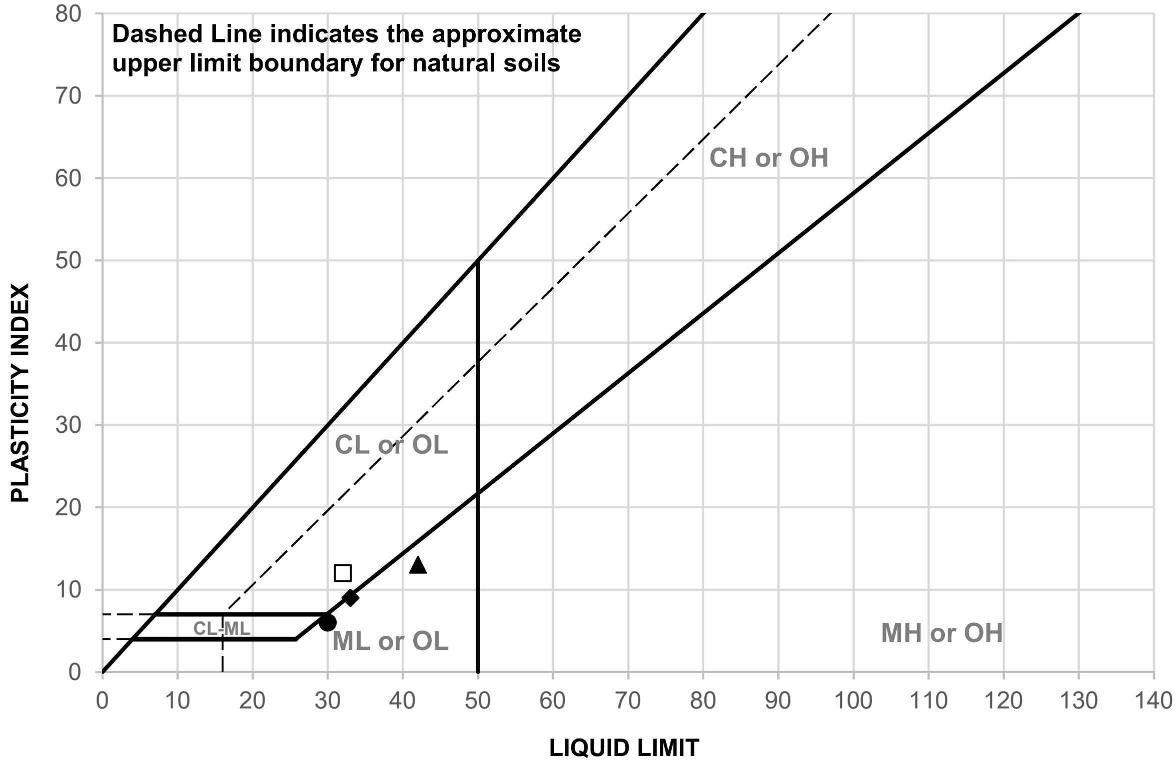
LOGGED / REVIEWED BY: E. Korogianos / JF
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: SFA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
			FAT CLAY WITH SAND (CH), olive brown with grayish brown, soft to medium stiff, moist, high plasticity, 15-20% fine- to coarse-grained sand			23					29		
			SANDY LEAN CLAY (CL), grayish brown, soft, moist, medium plasticity, 20-30% fine- to medium-grained sand										
45	110		LEAN CLAY WITH SAND (CL), grayish brown, soft, moist, medium plasticity, 10-15% fine-grained sand			12					25	100	
50	105		SANDY SILT (SM), brown to grayish brown, soft, moist, low plasticity, 25-30% fine-grained sand			7					25	101	
			Bottom of boring at 51 1/2 feet below ground surface. Groundwater was not observed due to drilling methods. Boring was backfilled with neat cement.										

APPENDIX B

LABORATORY TEST DATA

LIQUID AND PLASTIC LIMITS TEST REPORT
ASTM D4318



	SAMPLE ID	DEPTH (ft)	MATERIAL DESCRIPTION	LL	PL	PI
▲	1-B1@1.5	1.5 feet	See exploration logs	42	29	13
◆	1-B2@1	1 feet	See exploration logs	33	24	9
□	1-B2@10	10 feet	See exploration logs	32	20	12
●	1-B2@16	16 feet	See exploration logs	30	24	6

	SAMPLE ID	TEST METHOD	REMARKS
▲	1-B1@1.5	PI: ASTM D4318, Wet Method	
◆	1-B2@1	PI: ASTM D4318, Wet Method	
□	1-B2@10	PI: ASTM D4318, Wet Method	
●	1-B2@16	PI: ASTM D4318, Wet Method	

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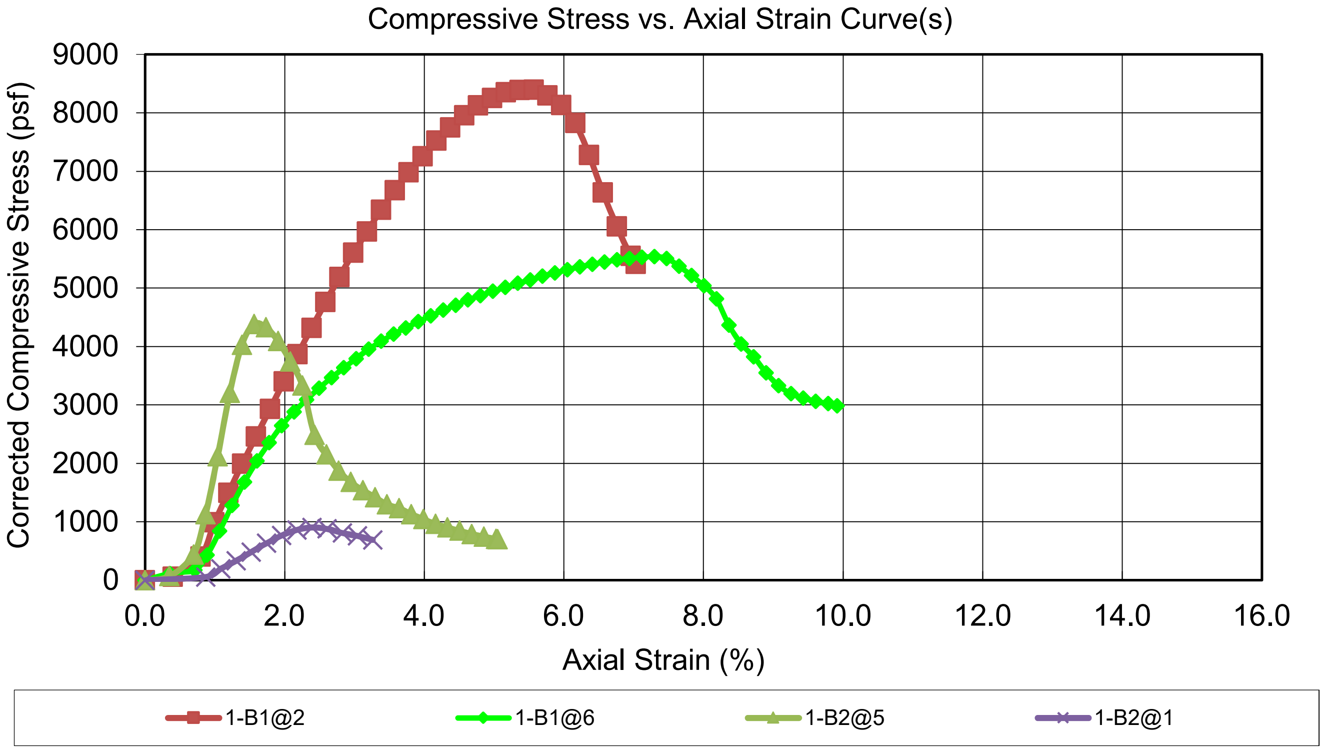
CLIENT: Environmental Science Associates
PROJECT NAME: Municipal Water Office Redevelopment
PROJECT NO: 19886.000.001 PH001
PROJECT LOCATION: San Jose, California
REPORT DATE: 2/17/2022
TESTED BY: M. Quasem
REVIEWED BY: W. Miller

UNCONFINED COMPRESSION TEST REPORT
(ASTM D2166)



BEFORE TEST	SPECIMEN 1-B1@2	SPECIMEN 1-B1@6	SPECIMEN 1-B2@5	SPECIMEN 1-B2@1
Test Moisture Content (%)	16.06	17.30	11.69	12.48
Dry Density (pcf)	115.0	110.8	100.6	83.8
Saturation (%)	91.6	88.3	46.2	33.1
Void Ratio	0.48	0.53	0.69	1.03
Diameter (in)	2.390	2.400	2.410	2.380
Height (in)	5.030	5.620	5.770	4.600
Height-To-Diameter Ratio	2.10	2.34	2.39	1.93
TEST DATA				
Unconfined Compressive Strength (psf)	8395.5	5537.8	4385.1	901.9
Undrained Shear Strength (psf)	4197.7	2768.9	2192.6	451.0
Strain Rate (in/min)	0.050	0.050	0.050	0.050
Specific Gravity (ASSUMED)	2.720	2.720	2.720	2.720
Strain at Failure(%)	5.57	7.30	1.56	2.39
Test Remarks				

SPECIMEN	DESCRIPTION
1-B1@2	See exploration logs
1-B1@6	See exploration logs
1-B2@5	See exploration logs
1-B2@1	See exploration logs

PROJECT NAME:	Municipal Water Office Redevelopment	**Test Date:**	2/15/2022
PROJECT NO:	19886.000.001 PH001	**Tested By:**	K. Nguyen
CLIENT:	Environmental Science Associates	**Reviewed By:**	G. Criste
LOCATION:	San Jose, California		

ENGEO
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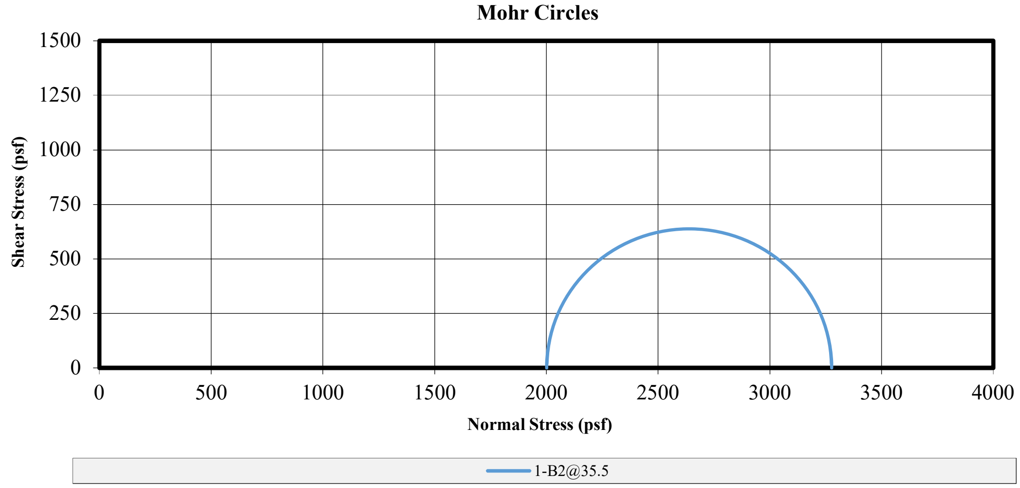
Isotropic Unconsolidated Undrained Triaxial Test
ASTM D2850

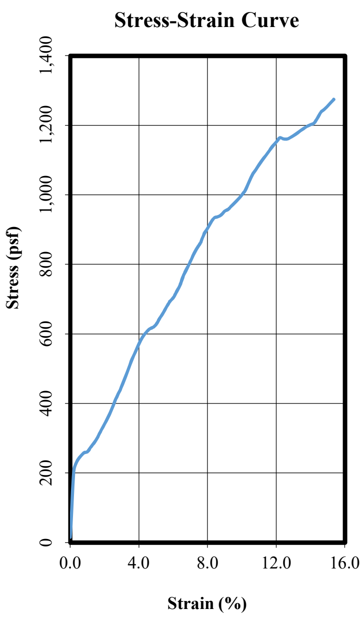
Date: 02/18/22

Checked By: W. Miller

Date: 2/18/2022

Tested By: M. Quasem





	Specimen			
Before Test	**1-B2@35.5**			
Water Content (%)	33.24			
Dry Density (pcf)	89.50			
Saturation (%)	99.61			
Void Ratio	0.92			
Diameter (in)	2.420			
Height (in)	5.000			
Height-to-Diameter Ratio	2.066			
ASTM D4318 - Wet Method				
Liquid Limit				
Plastic Limit				
ASTM D854 - Assumed				
Specific Gravity	2.750			
After Test	**1-B2@35.5**			
Water Content (%)	33.24			
Saturation (%)	99.61			
Strain Rate (%/min)	0.05			
Peak Deviator Stress (psf)	1274.9			
Axial Strain @ Failure (%)	15.353			
	Cell Pressure			
Cell (psf)	2001.6			
Back (psf)	n/a			
	Principle Stresses at Failure			
σ1 (psf)	3276.5			
σ3 (psf)	2001.6			
Corrected Peak Deviator Stress				

Mohr-Coulomb Parameters with a Non-zero Friction Angle (Ø≠0)			Cohesion at Failure with a Zero Friction Angle (Ø=0)			
Cohesion, c (psf)	n/a		637.4			
Friction Angle Ø	n/a		n/a			

Project Information	
Project Name:	Municipal Water Office Redevelopment
Project Number:	19886.000.001 PH001
Project Location:	San Jose, California
Client:	Environmental Science Associates
Description:	See exploration logs
Test Remarks:	

ENGEO
Expect Excellence

Isotropic Unconsolidated Undrained Triaxial Test
ASTM D2850

Tested By: M. Quasem Date: 2/18/2022 Checked By: W. Miller Date: 02/18/22

SPECIMEN PHOTOS

SAMPLE NUMBER: 1-B2@35.5	SAMPLE NUMBER:
	
SAMPLE NUMBER:	**SAMPLE NUMBER:**

Project Information	
Project Name:	Municipal Water Office Redevelopment
Project Number:	19886.000.001 PH001
Project Location:	San Jose, California
Client:	Environmental Science Associates
Description:	See exploration logs
Test Remarks:	

ENGEO Expect Excellence

MOISTURE CONTENT REPORT
ASTM D2216

SAMPLE ID	1-B2@41							
DEPTH (ft.)	41							
METHOD A OR B	B							
MOISTURE CONTENT (%)	28.5							

ENGEO
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CLIENT: Environmental Science Associates/City of San Jose
PROJECT NAME: Municipal Water Office Redevelopment
PROJECT NO: 19886.000.001 PH001
PROJECT LOCATION: San Jose, California
REPORT DATE: 2/16/2022
TESTED BY: K. Nguyen
REVIEWED BY: G. Criste

MOISTURE-DENSITY DETERMINATION REPORT
ASTM D7263

SAMPLE ID	1-B2@46	1-B2@51						
DEPTH (ft.)	46	51						
METHOD A OR B	B	B						
MOISTURE CONTENT (%)	25.3	25.3						
DRY DENSITY (pcf)	100.0	101.0						

ENGEO
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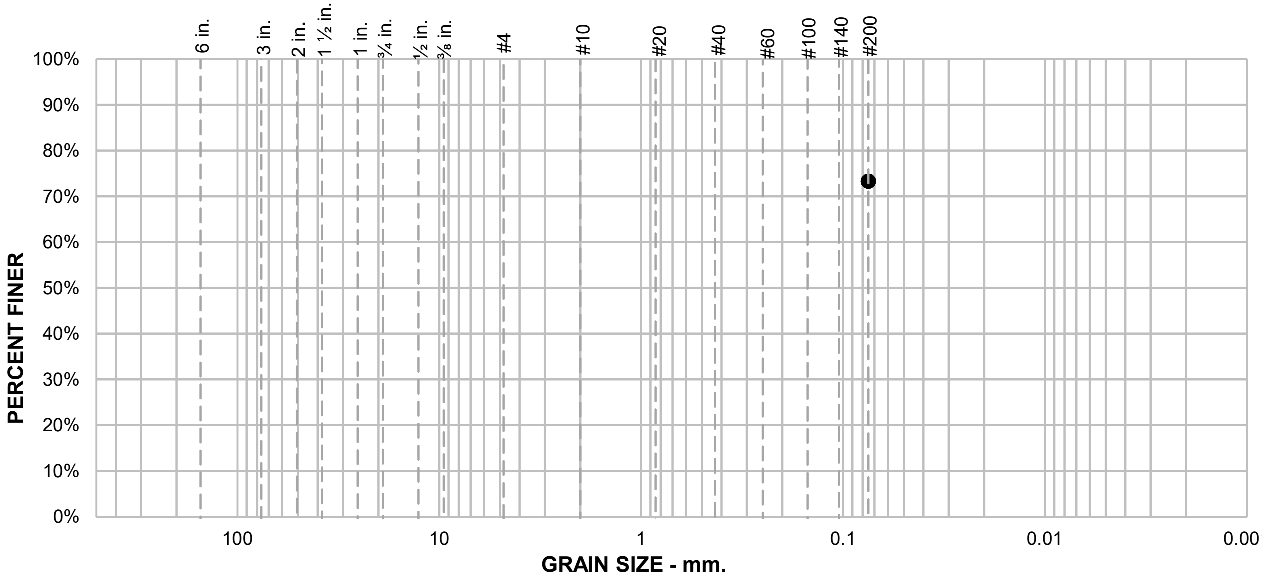
CLIENT: Environmental Science Associates
PROJECT NAME: Municipal Water Office Redevelopment
PROJECT NO: 19886.000.001 PH001
PROJECT LOCATION: San Jose, California
REPORT DATE: 2/16/2022
TESTED BY: K. Nguyen
REVIEWED BY: G. Criste

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 1-B1@10.5

DEPTH (ft): 10.5

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						73.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	73.3		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

REMARKS

Soak time = 180 min
Dry sample weight = 316.8 g

* (no specification provided)

CLIENT: Environmental Science Associates
PROJECT NAME: Municipal Water Office Redevelopment
PROJECT NO: 19886.000.001 PH001
PROJECT LOCATION: San Jose, California
REPORT DATE: 2/17/2022
TESTED BY: M. Quasem
REVIEWED BY: W. Miller

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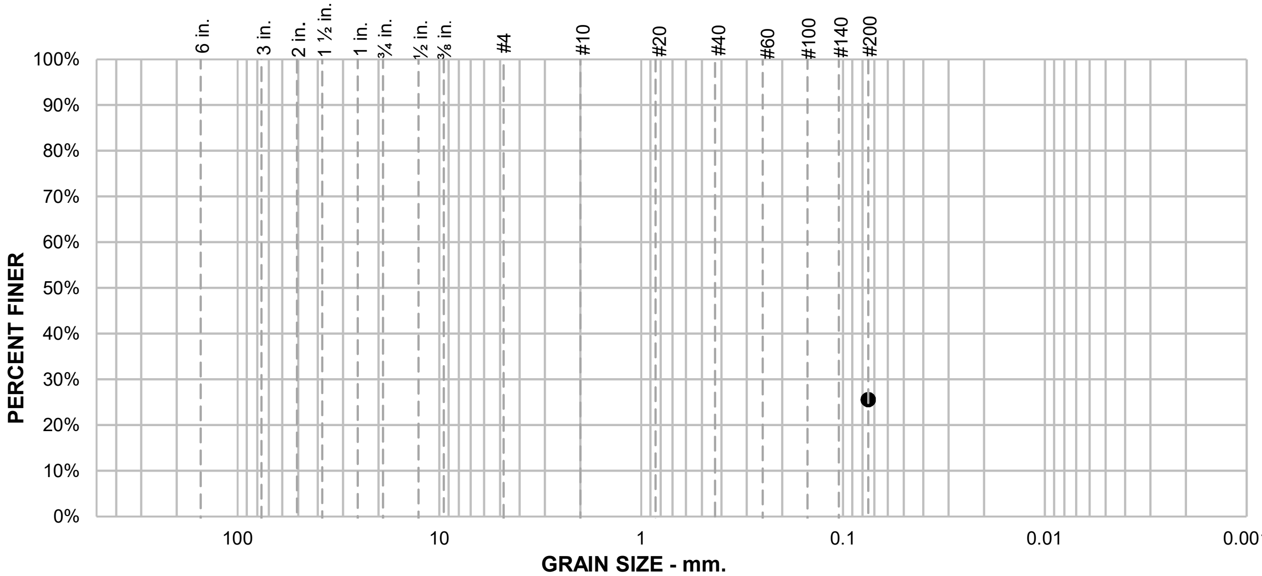
PARTICLE SIZE DISTRIBUTION REPORT
ASTM D1140, Method B



SAMPLE ID: 1-B1@26

DEPTH (ft): 26

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						25.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	25.6		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

REMARKS

Soak time = 180 min
Dry sample weight = 331.1 g

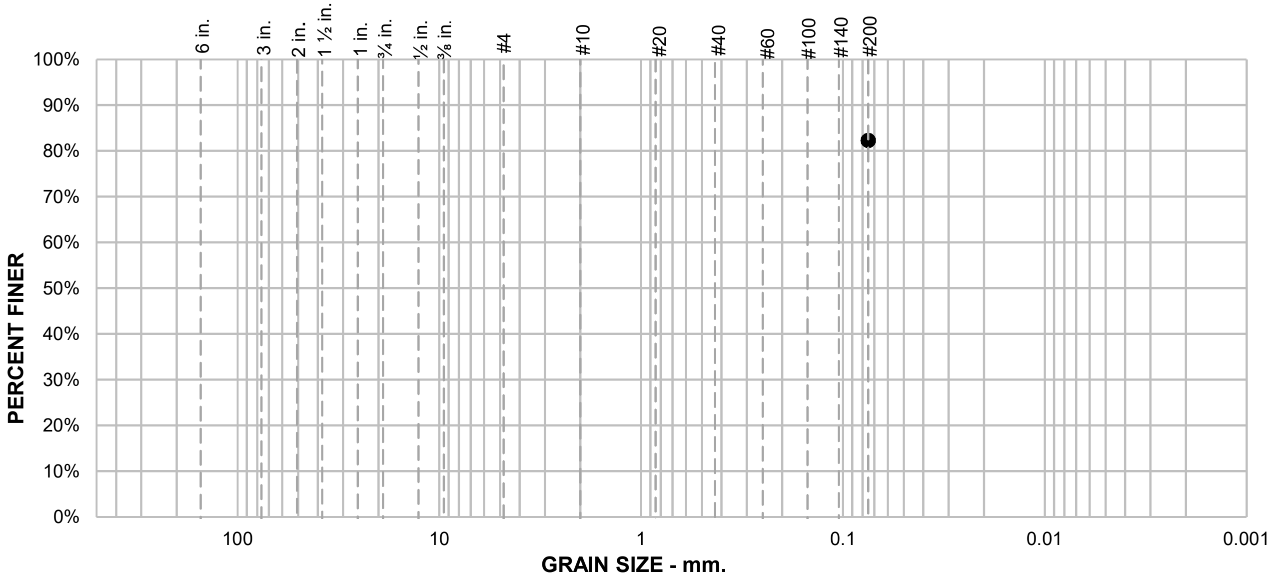
* (no specification provided)

ENGEO
Expect Excellence

CLIENT: Environmental Science Associates
PROJECT NAME: Municipal Water Office Redevelopment
PROJECT NO: 19886.000.001 PH001
PROJECT LOCATION: San Jose, California
REPORT DATE: 2/17/2022
TESTED BY: M. Quasem
REVIEWED BY: W. Miller

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 1-B2@16

DEPTH (ft): 16

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						82.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	82.3		

* (no specification provided)

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

REMARKS

Soak time = 180 min
Dry sample weight = 215.6 g

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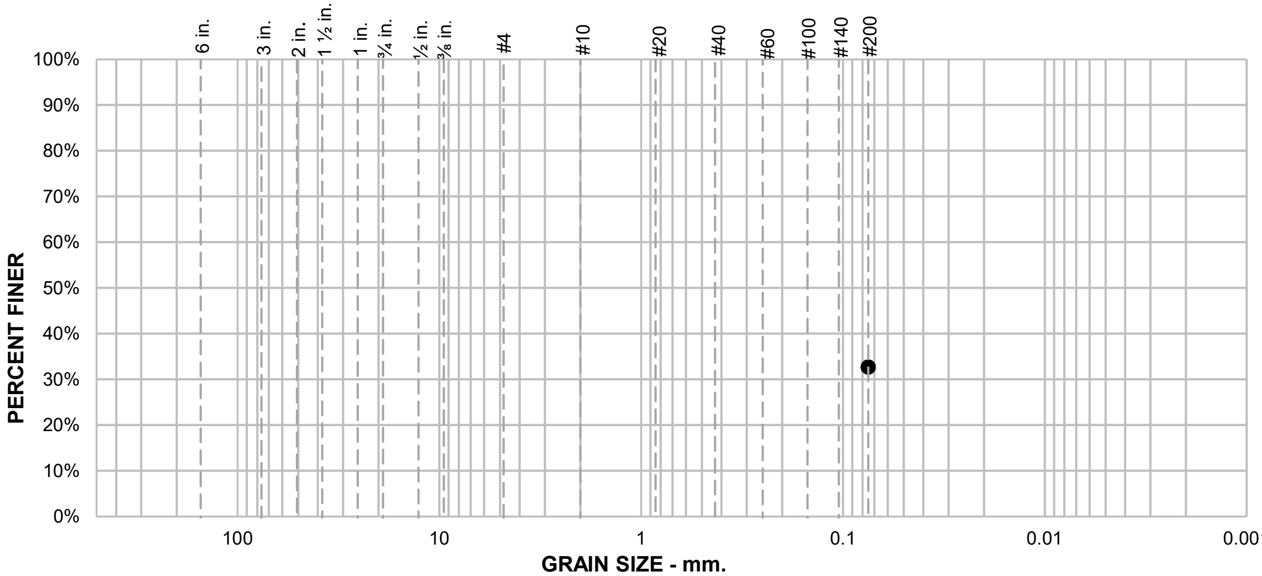
CLIENT: Environmental Science Associates
PROJECT NAME: Municipal Water Office Redevelopment
PROJECT NO: 19886.000.001 PH001
PROJECT LOCATION: San Jose, California
REPORT DATE: 2/17/2022
TESTED BY: M. Quasem
REVIEWED BY: W. Miller

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 1-B2@26

DEPTH (ft): 26

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						32.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	32.7		

* (no specification provided)

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

REMARKS

Soak time = 180 min
Dry sample weight = 395.3 g

ENGEO
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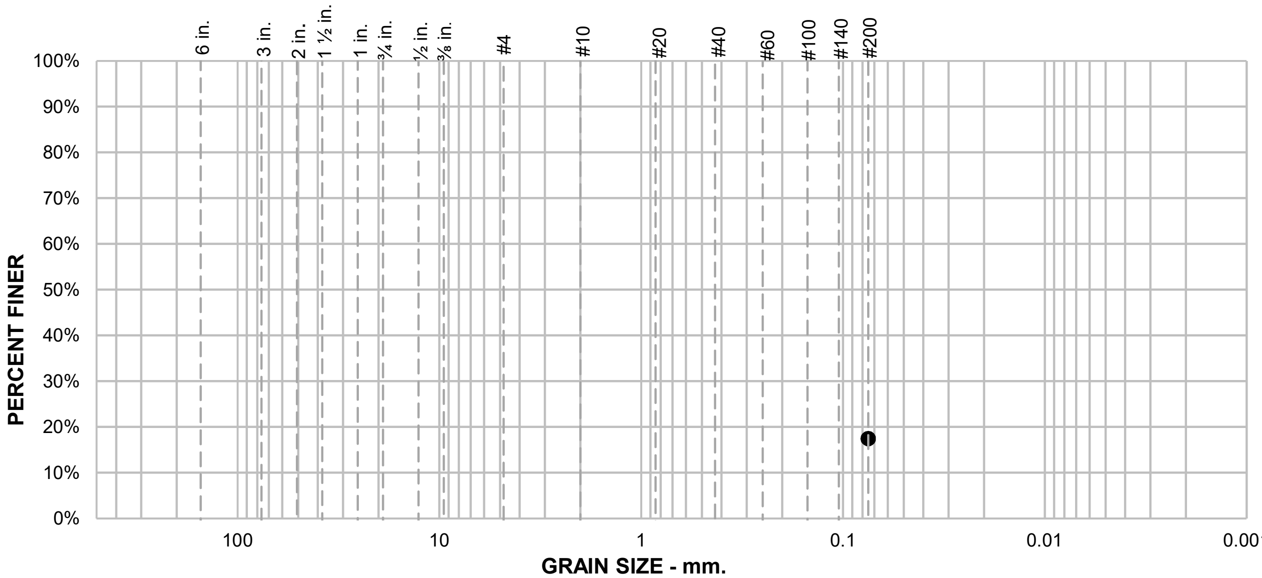
CLIENT: Environmental Science Associates
PROJECT NAME: Municipal Water Office Redevelopment
PROJECT NO: 19886.000.001 PH001
PROJECT LOCATION: San Jose, California
REPORT DATE: 2/17/2022
TESTED BY: M. Quasem
REVIEWED BY: W. Miller

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 1-B2@30.5

DEPTH (ft): 30.5

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						17.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	17.4		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

REMARKS

Soak time = 180 min
Dry sample weight = 523.4 g

* (no specification provided)

ENGEO Expect Excellence

CLIENT: Environmental Science Associates
PROJECT NAME: Municipal Water Office Redevelopment
PROJECT NO: 19886.000.001 PH001
PROJECT LOCATION: San Jose, California
REPORT DATE: 2/17/2022
TESTED BY: M. Quasem
REVIEWED BY: W. Miller

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 **462 2771** Fax. 925 **462 2775**
www.cercoanalytical.com

28 February, 2022

Job No. 2202035
Cust. No. 10169

Mr. Jonas Bauer
ENGEO Inc.
2010 Crow Canyon Place, Suite 250
San Ramon, CA 94583

Subject: Project No.: 19886.000.001
Project Name: Muni Water
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Bauer:

Pursuant to your request, CERCO Analytical has analyzed the soil sample submitted on February 17, 2022. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurement, the sample is classified as "corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentration is none detected with a reporting limit of 15 mg/kg.

The sulfate ion concentration is 64 mg/kg and is determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at this location.

The sulfide ion concentration reflects none detected with a reporting limit of 50 mg/kg.

The pH of the soil is 8.08 mg/kg, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potential is 280-mV, which is indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

Sherri Moore

for J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client: ENGEO, Incorporated
Client's Project No.: 19886.000.001
Client's Project Name: Muni Water
Date Sampled: 7-Feb-22
Date Received: 17-Feb-22
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 25-Feb-2022

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
2202035-001	1-B1 @ 2.5-4'	280	8.08	-	1,600	N.D.	N.D.	64

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	23-Feb-2022	23-Feb-2022	-	18-Feb-2022	24-Feb-2022	23-Feb-2022	23-Feb-2022

Sherri Moore

Sherri Moore
Chemist

* Results Reported on "As Received" Basis
N.D. - None Detected

Quality Control Summary - **All laboratory quality control parameters were found to be within established limits**

APPENDIX C

CPT DATA

ENGEO
San Jose, California
6399 San Ignacio Ave #150
https://www.engeo.com/

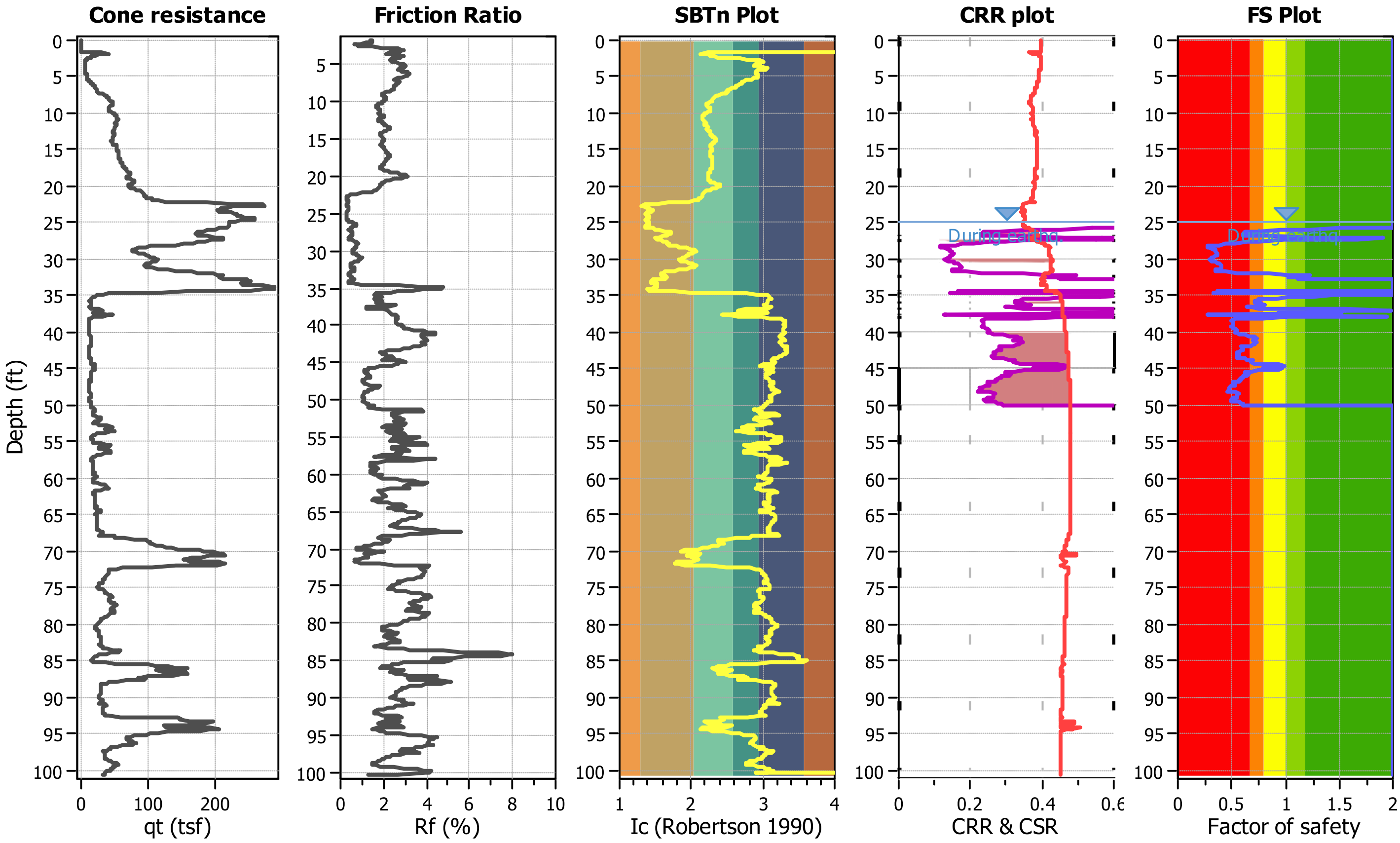
LIQUEFACTION ANALYSIS REPORT

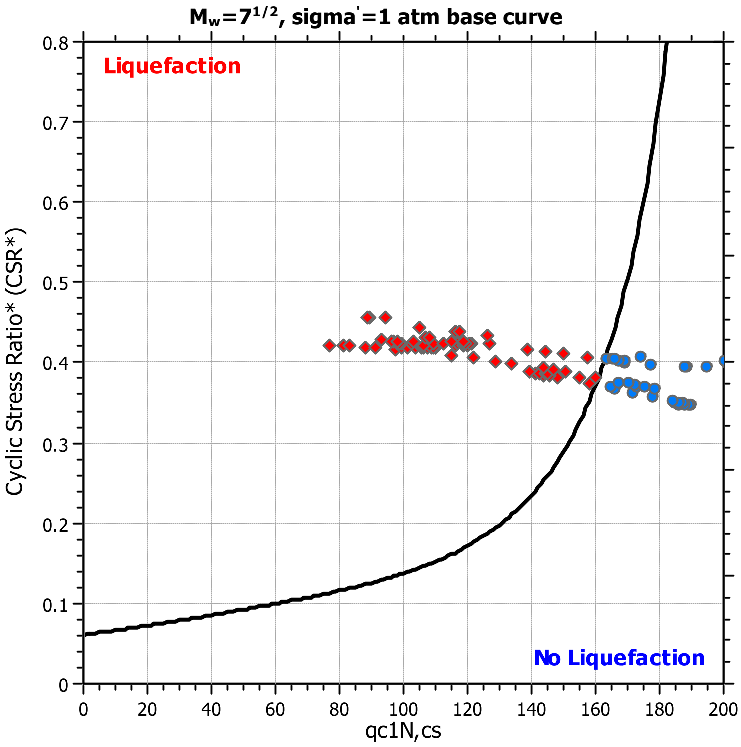
Project title : 3025 Tuers Road **Location : San Jose, California**

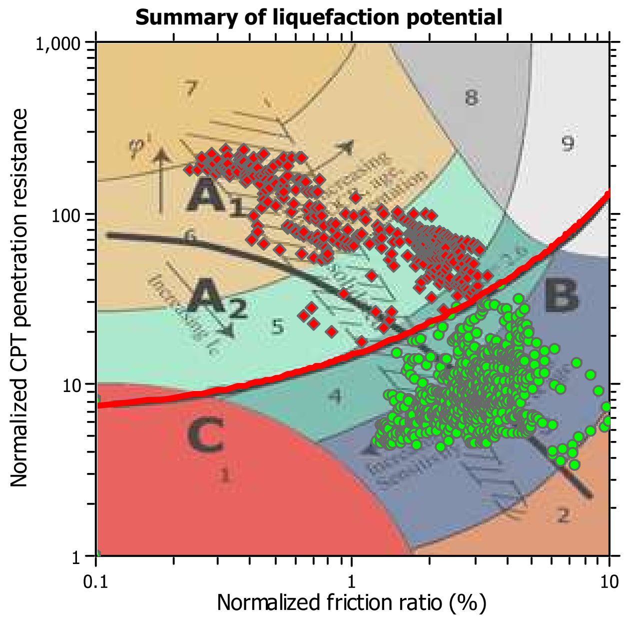
CPT file : 1-SCPT3

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	25.00 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	25.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method
Peak ground acceleration:	0.69	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

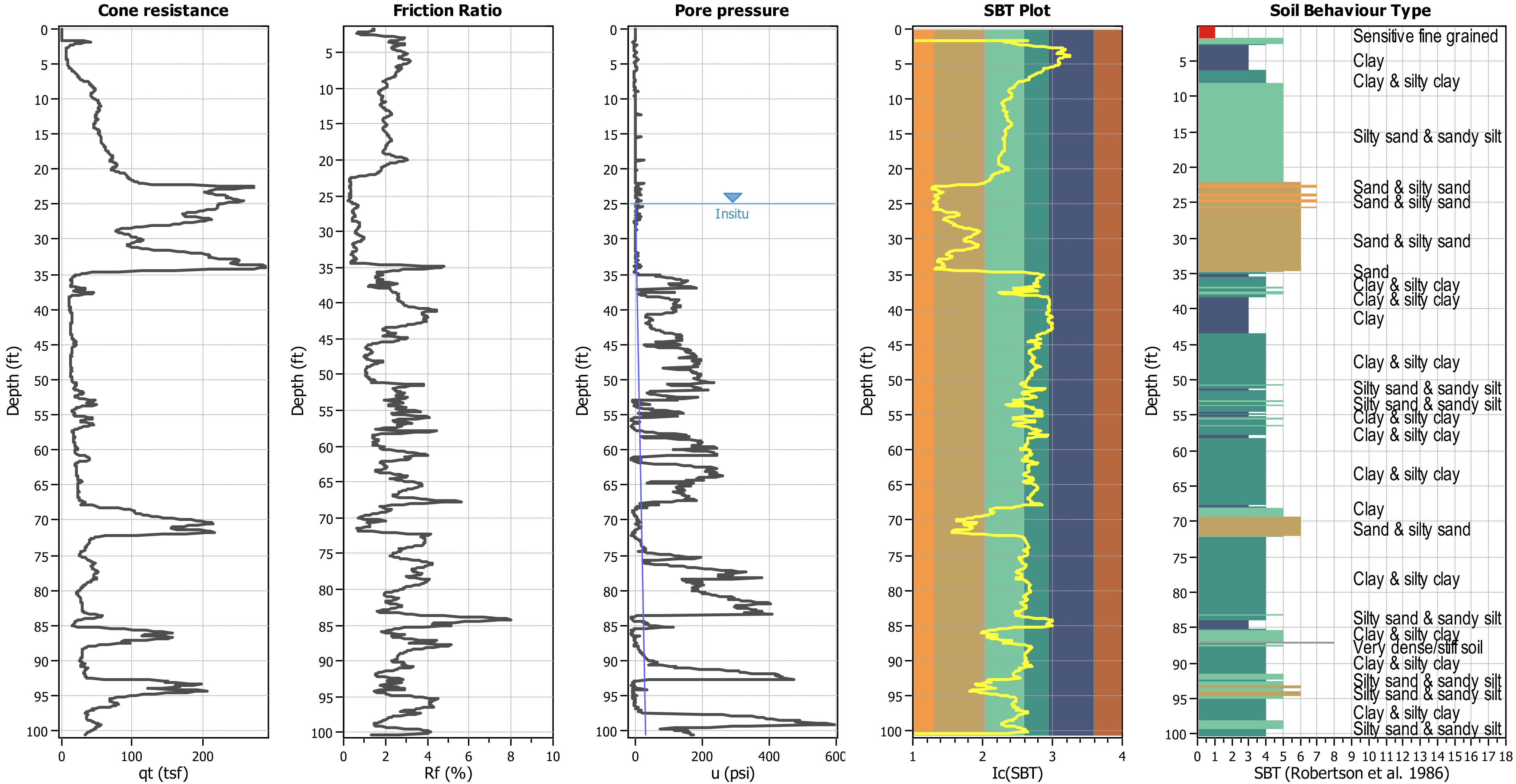






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



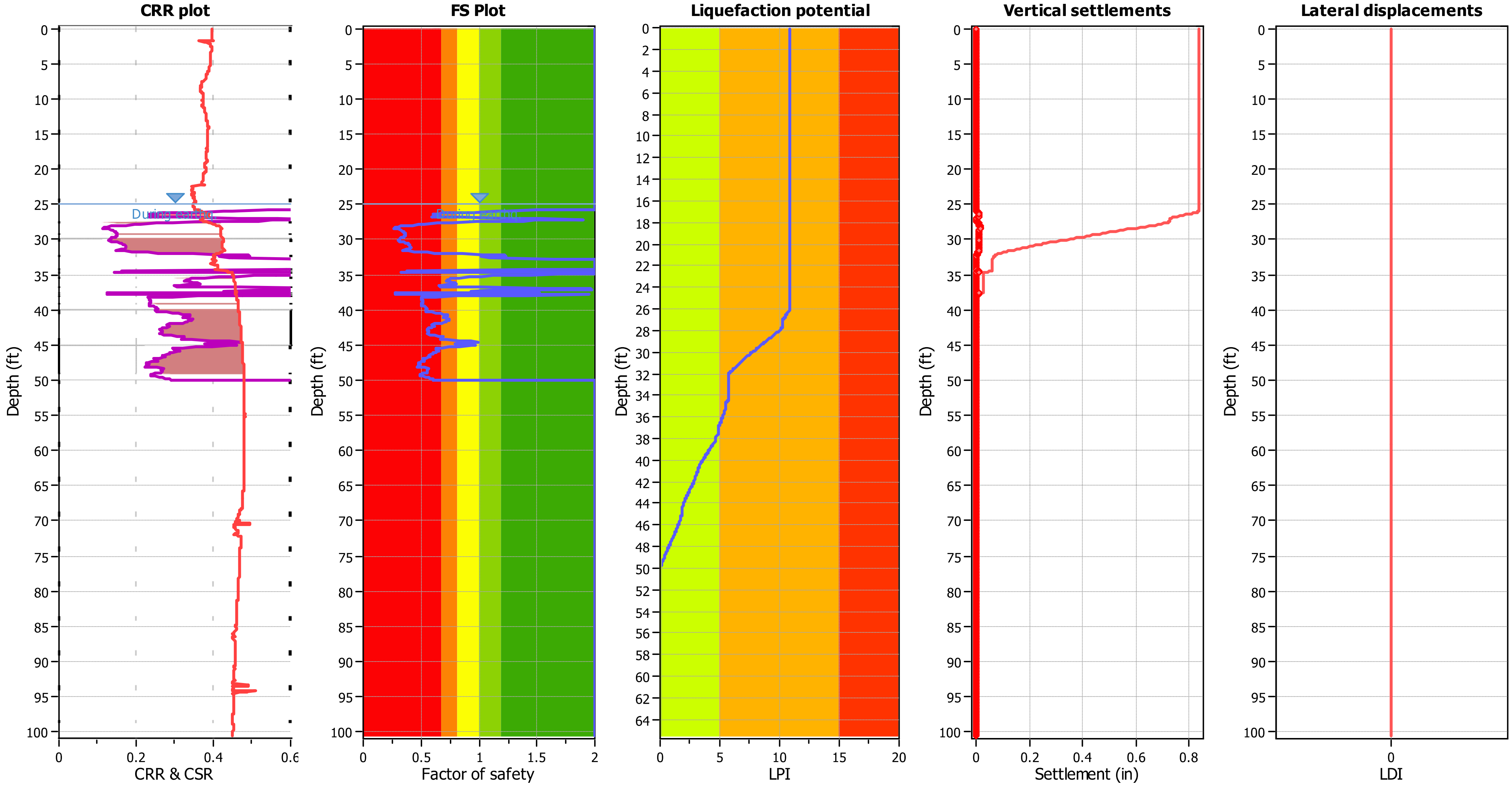
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	25.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.69	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	25.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	25.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.69	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	25.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

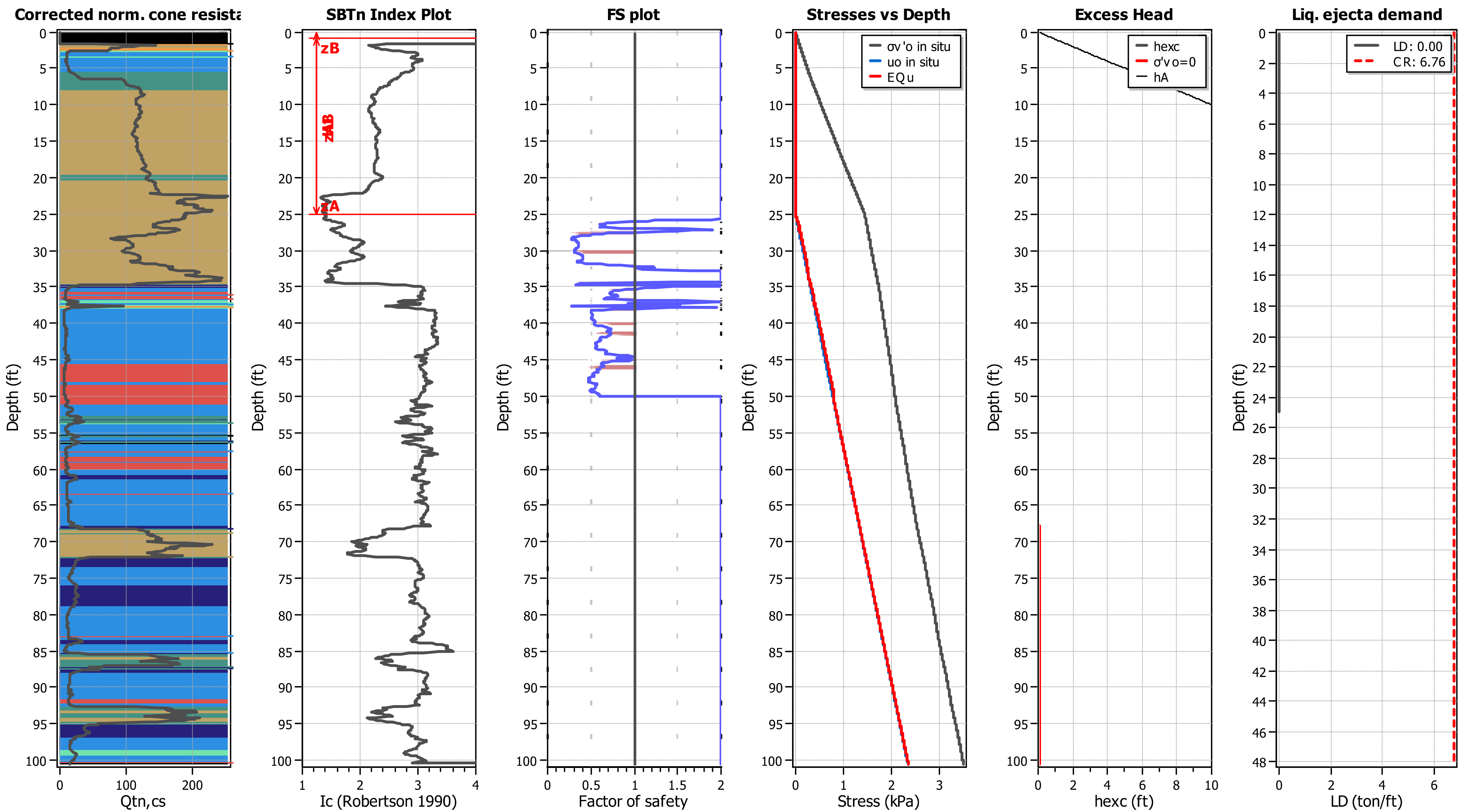
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

Ejecta Severity Estimation



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6399 San Ignacio Ave #150
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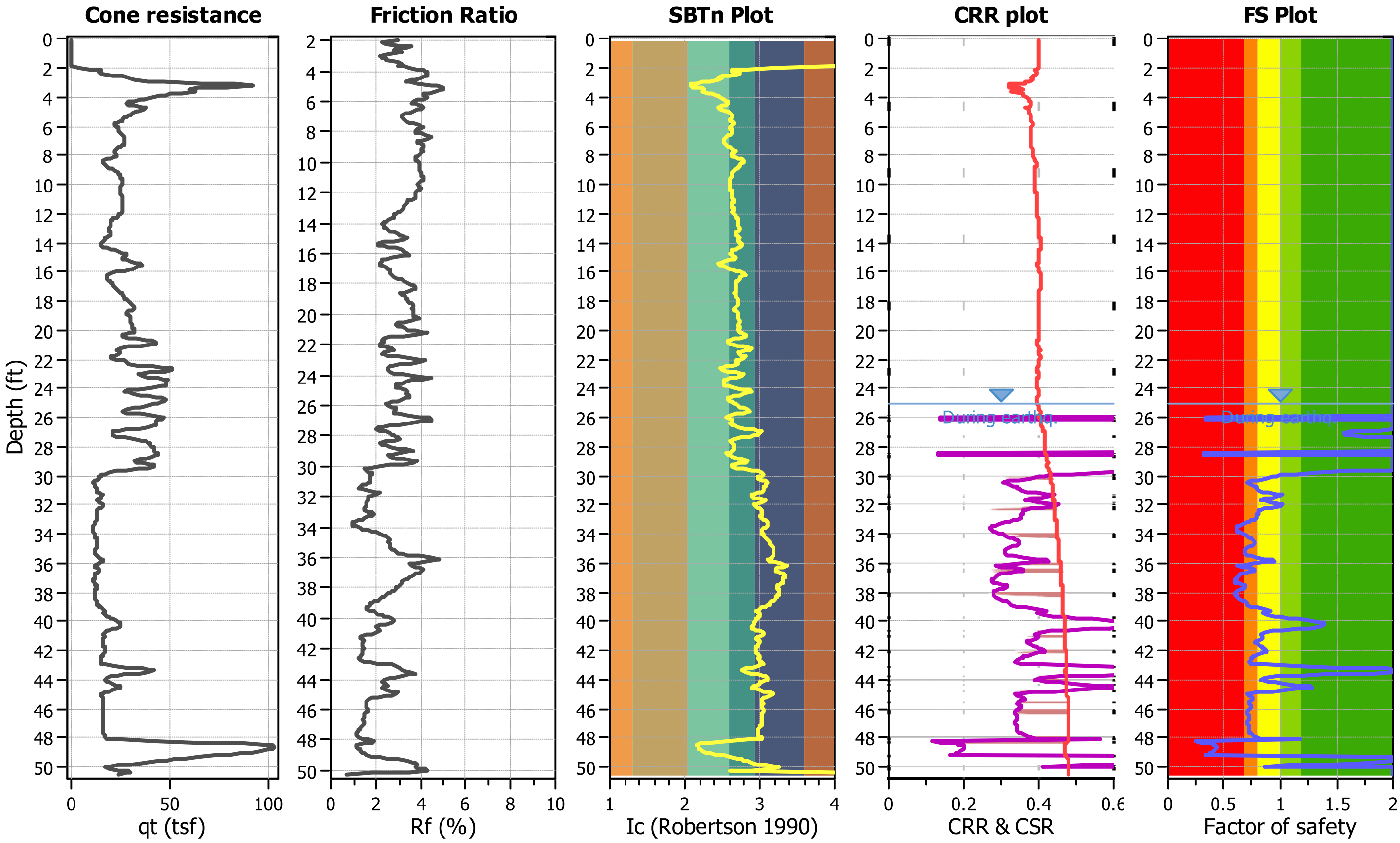
LIQUEFACTION ANALYSIS REPORT

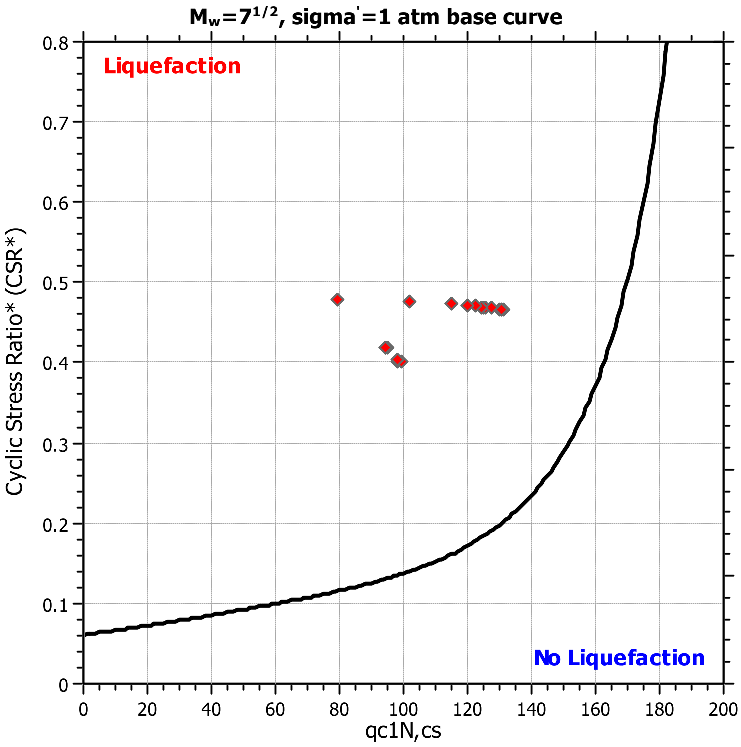
Project title : 3025 Tuers Road **Location : San Jose, California**

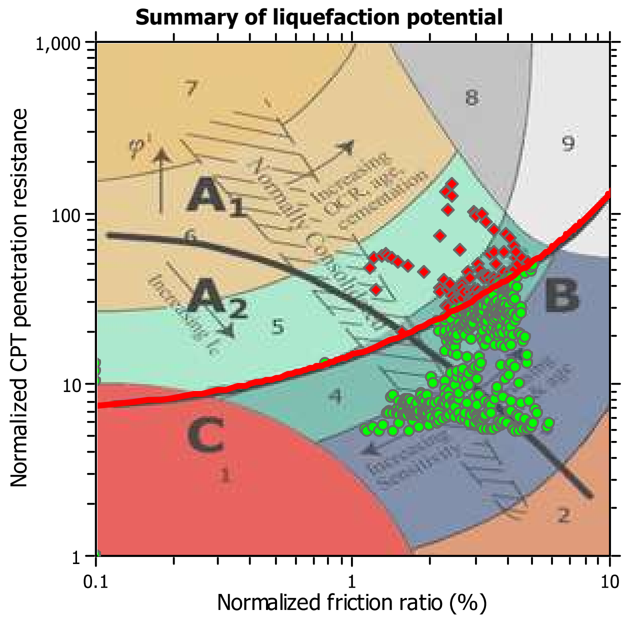
CPT file : 1-CPT1

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	25.00 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	25.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method
Peak ground acceleration:	0.69	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

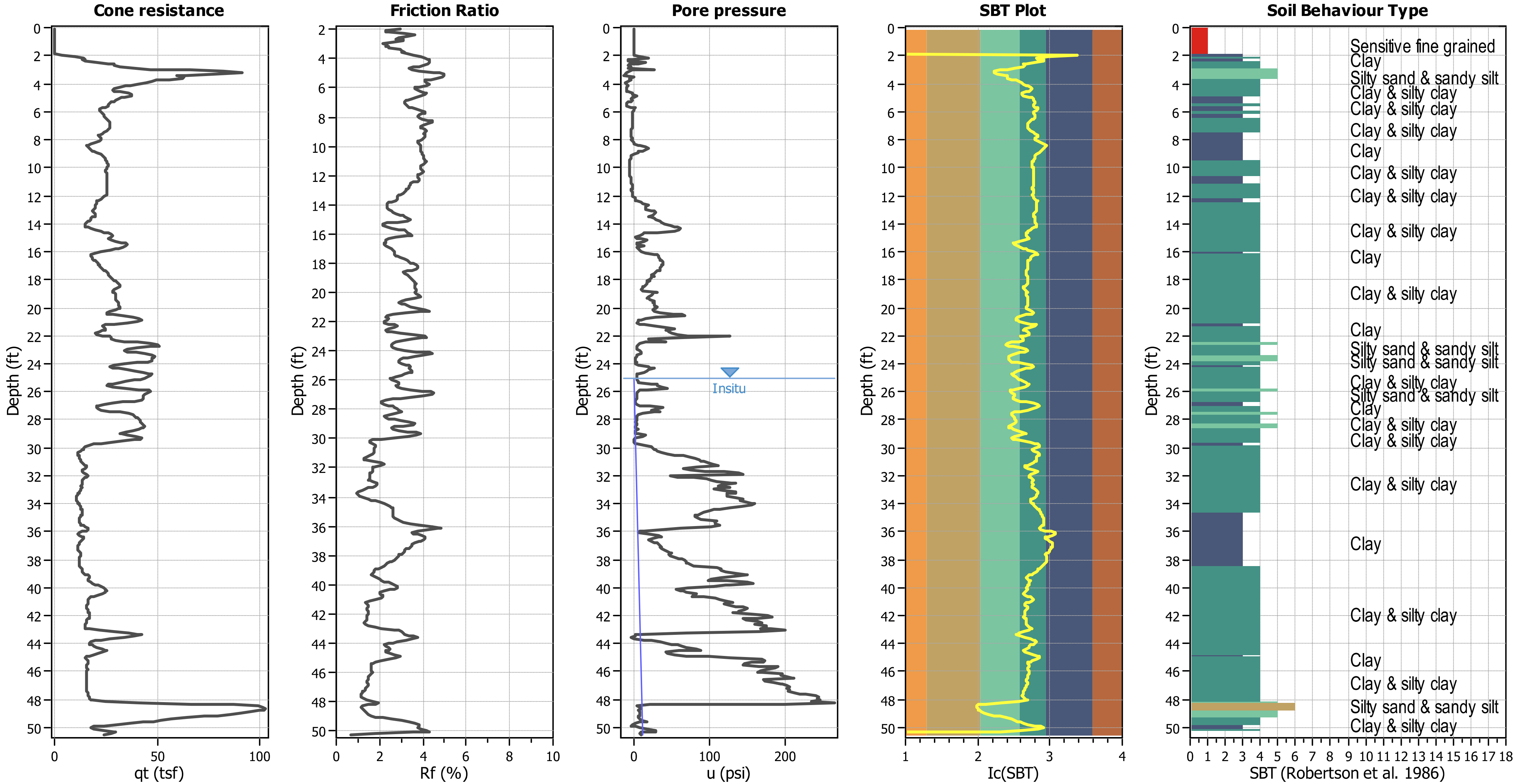






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



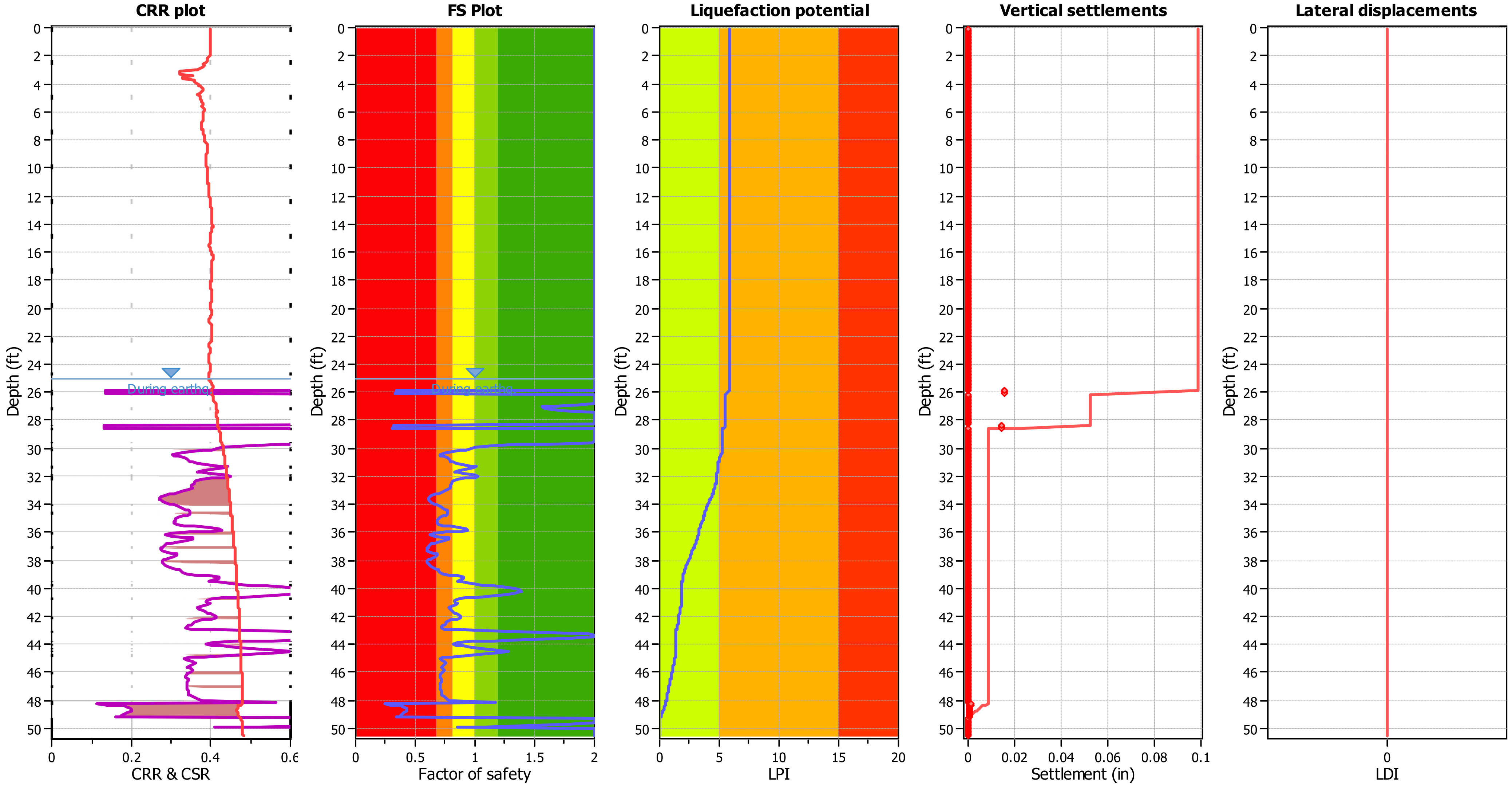
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	25.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.69	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	25.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	25.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.69	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	25.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

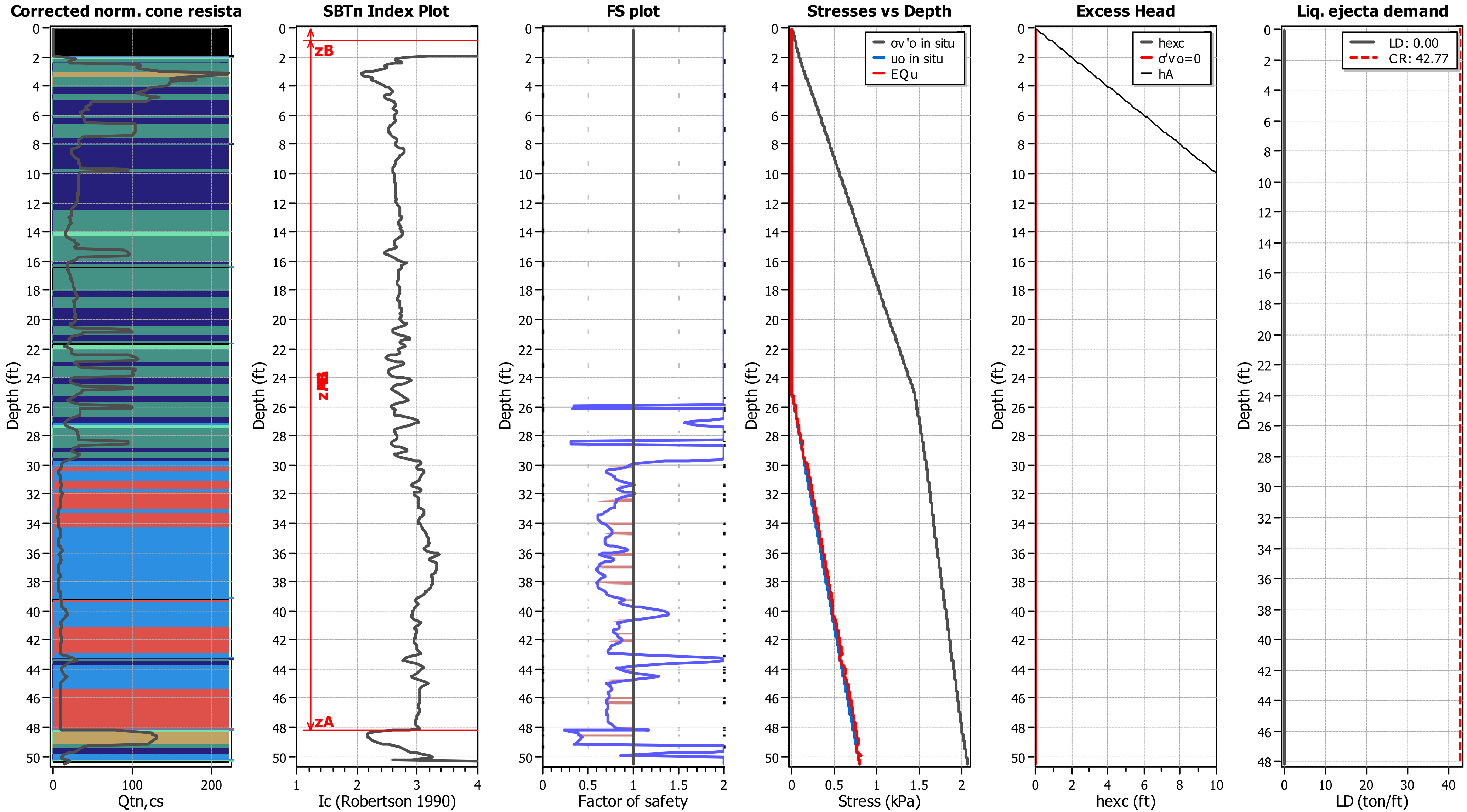
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

Ejecta Severity Estimation



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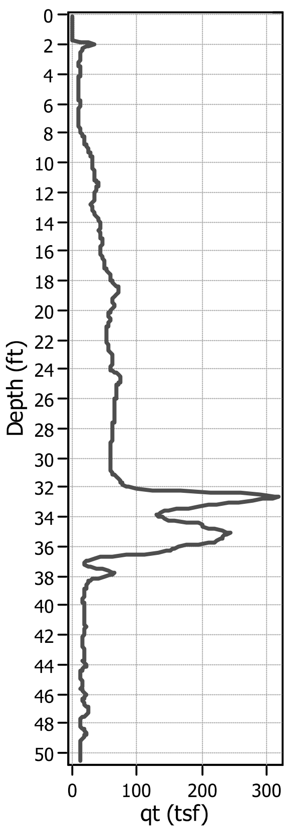
LIQUEFACTION ANALYSIS REPORT

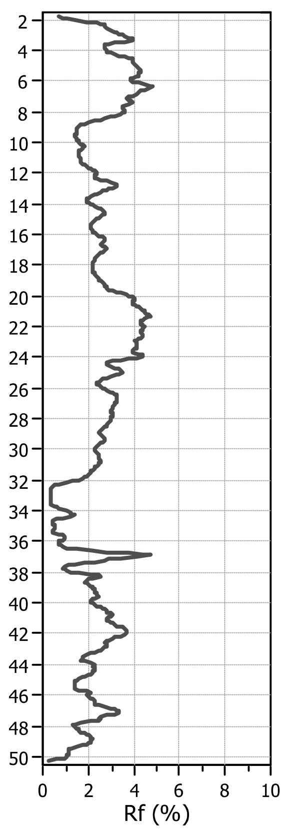
Project title : 3025 Tuers Road **Location : San Jose, California**

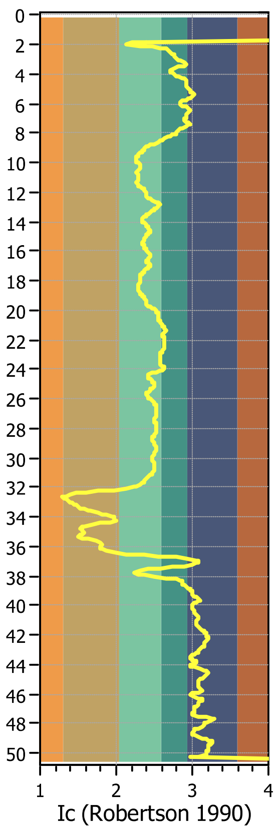
CPT file : 1-CPT2

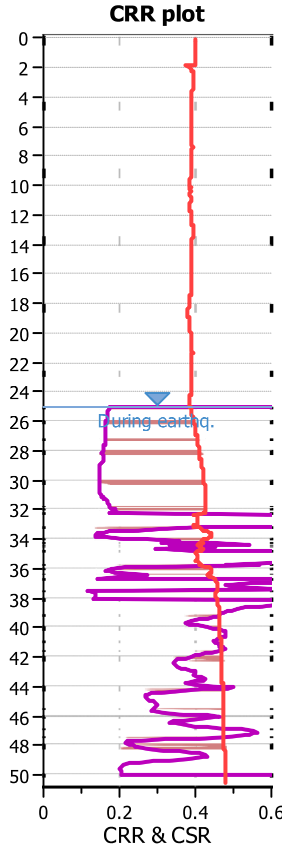
Input parameters and analysis data

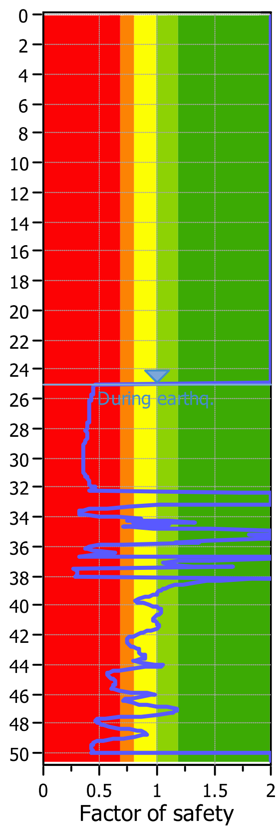
Analysis method:	B&I (2014)	G.W.T. (in-situ):	25.00 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	25.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method
Peak ground acceleration:	0.69	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

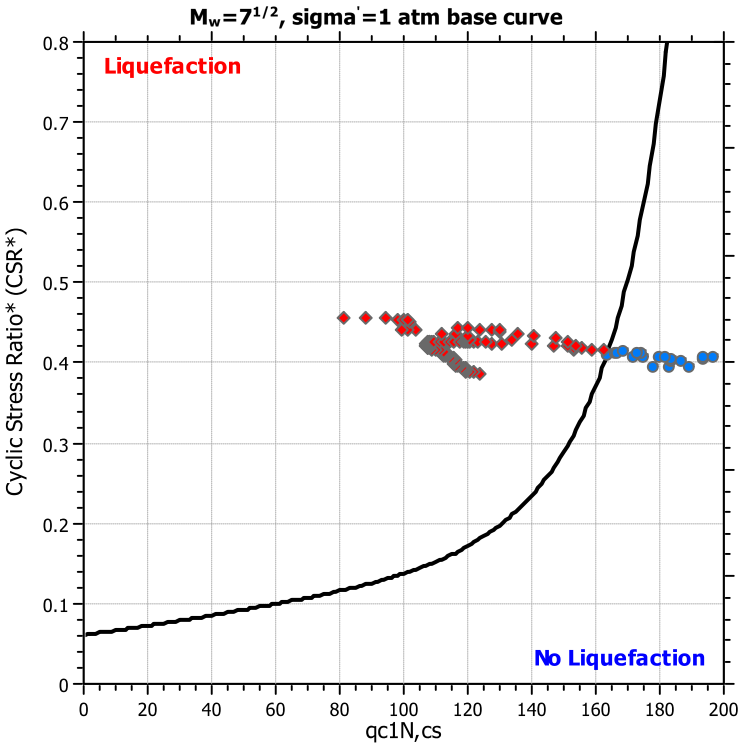


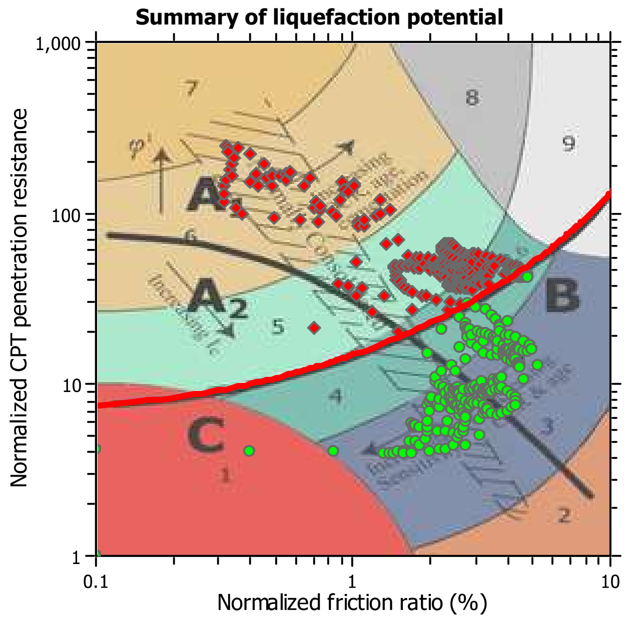












Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
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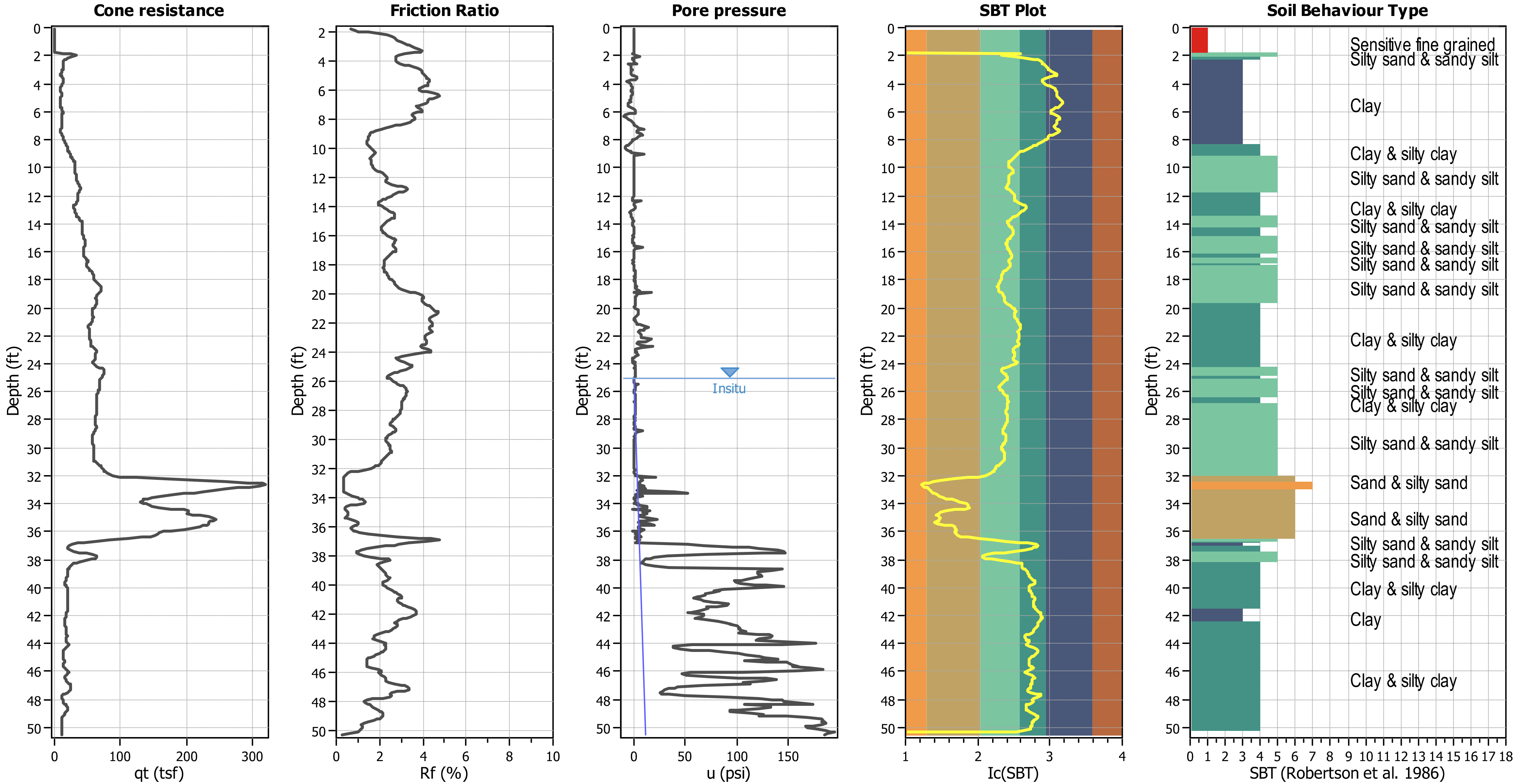
CPT basic interpretation plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	25.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
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Peak ground acceleration:	0.69	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	25.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

SBT legend

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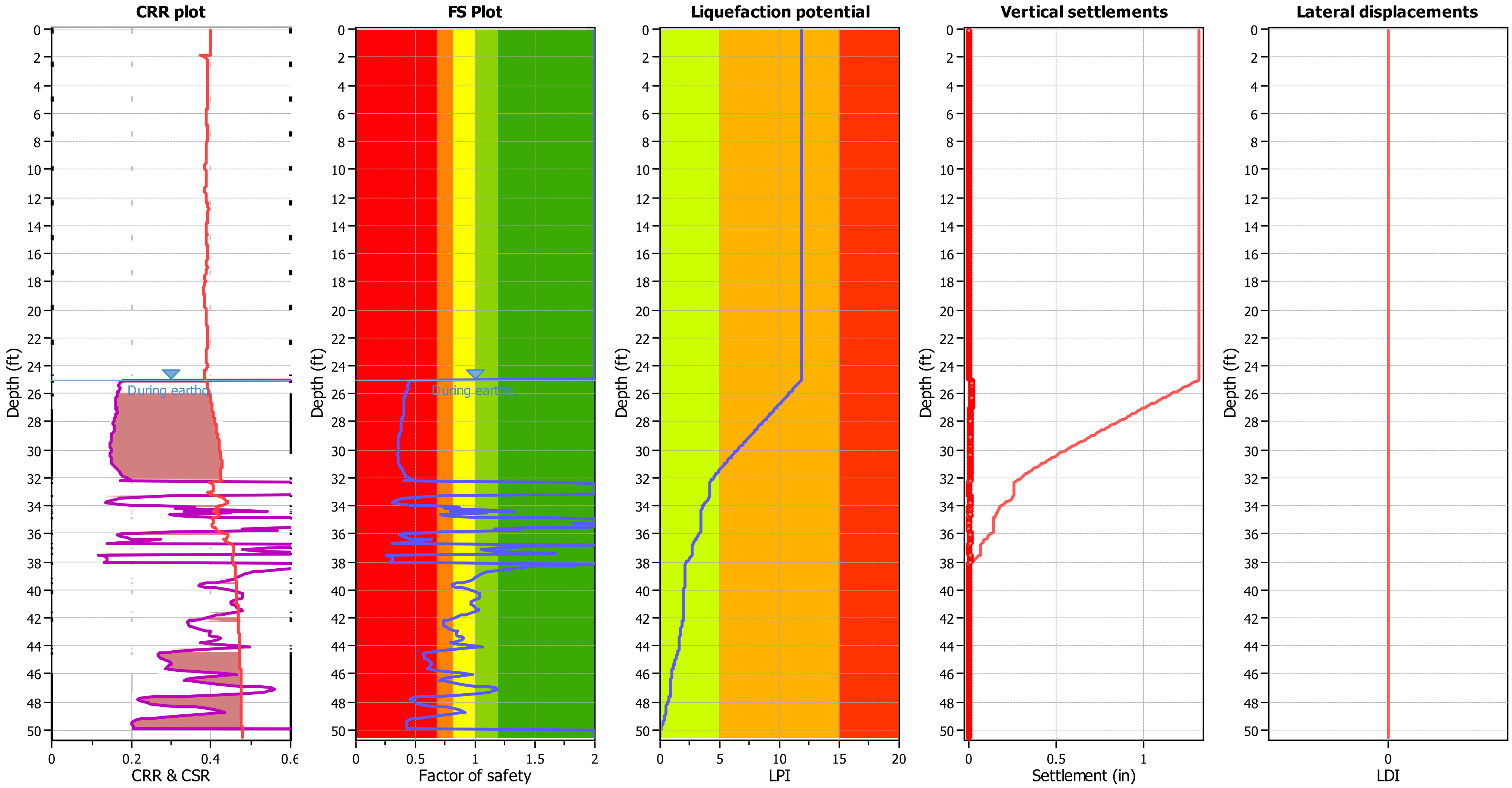
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Liquefaction analysis overall plots



Input parameters and analysis data

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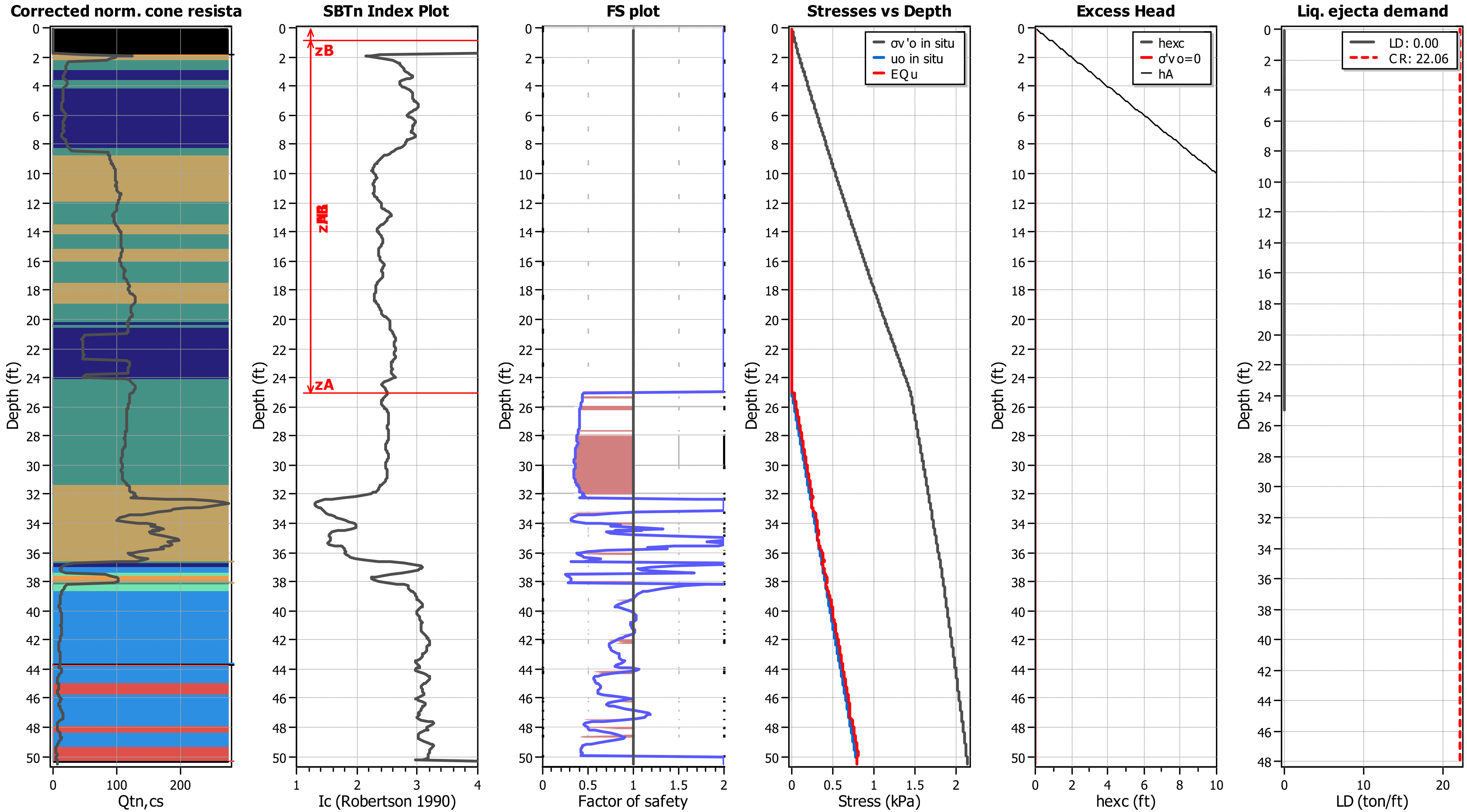
Ejecta Severity Estimation



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