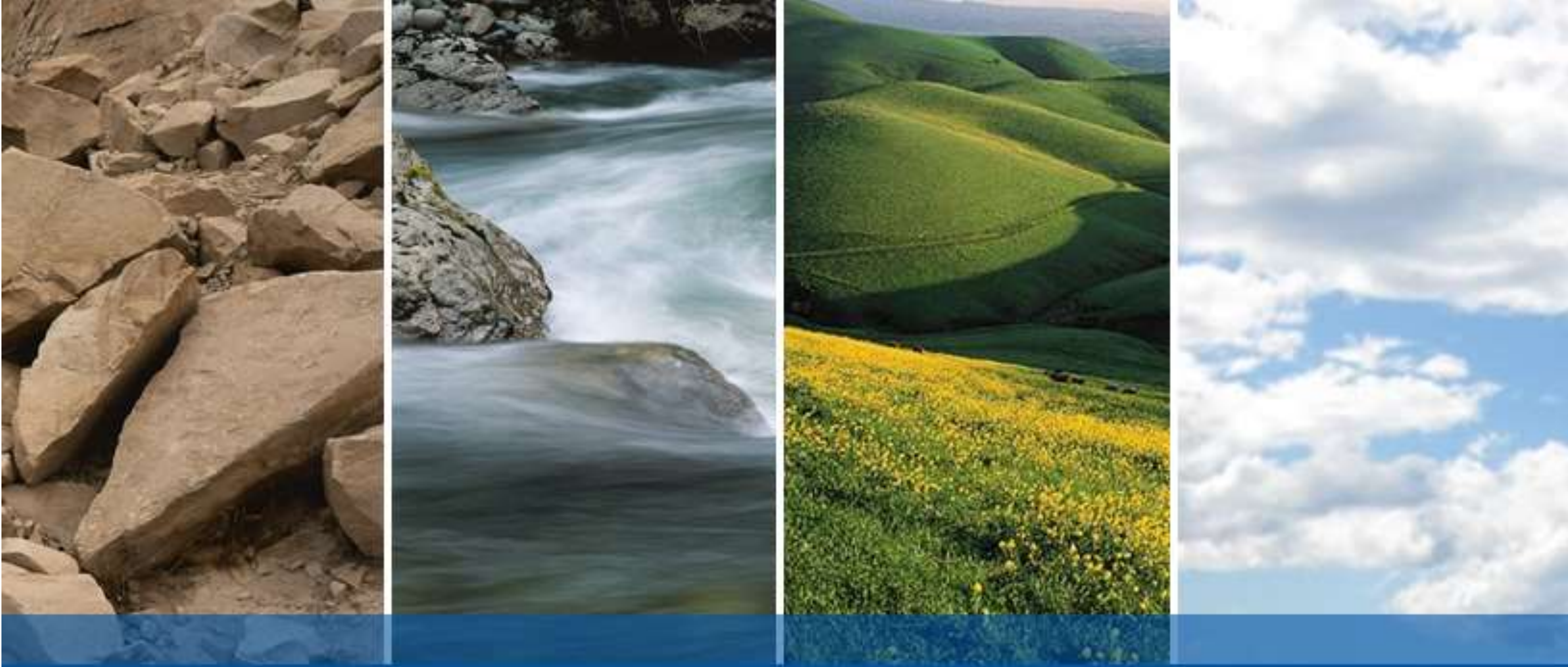
Appendix D
Basin 2 and 3 Collection System Improvements Underground Flow Equalization System and Diversion Sewers Geotechnical Interpretive Report



BASIN 2 AND 3 COLLECTION SYSTEM IMPROVEMENTS UNDERGROUND FLOW EQUALIZATION SYSTEM AND DIVERSION SEWERS

SAN MATEO, CALIFORNIA

GEOTECHNICAL INTERPRETIVE REPORT

SUBMITTED TO
City of San Mateo
℅ Margaret M. Regan
Stantec
2121 N. California Boulevard, Suite 600
Walnut Creek, CA 94596

PREPARED BY
ENGEO

May 21, 2018

PROJECT NO.
13231.000.001

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GEOTECHNICAL
ENVIRONMENTAL
WATER RESOURCES
CONSTRUCTION SERVICES

Project No.
13231.000.001

May 21, 2018

City of San Mateo
℅ Margaret M. Regan
Stantec
2121 N. California Boulevard, Suite 600
Walnut Creek, CA 94596

Subject: Basin 2 and 3 Collection System Improvements
Underground Flow Equalization System (UFES) and Diversion Sewers
San Mateo, California

GEOTECHNICAL INTERPRETIVE REPORT

Dear Ms. Regan:

With your authorization, we prepared this geotechnical interpretive report for the Underground Flow Equalization System (UFES) and Diversion Sewers of the Basin 2 and 3 Collection System Improvements in San Mateo, California. We submitted a draft geotechnical sampling data report the UFES and Diversion Sewers in March 2018. This report presents our conclusions and recommendations regarding the proposed storage tank facility and diversion sewers based on the data presented in the data report.

Based on our interpretation, the proposed UFES and Diversion Sewers are feasible from a geotechnical standpoint, provided the recommendations and design criteria presented in this report are incorporated into the project design plans and specification as well as implemented during construction.

We are pleased to have been of service to you on this project and are prepared to consult further with you and your design team as the project progresses.

Sincerely,

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1.0 INTRODUCTION

1.1 AUTHORIZATION

We performed the geotechnical sampling services in general accordance with the scope of services outlined in Change Order 1 of Task Order T10509240-104975-OM dated July 10, 2017, and the applicable Geotechnical and Environmental Exploration Work Plan.

Based on directions from Stantec, ENGEO prepared this report for the Underground Flow Equalization System (UFES) and Diversion Sewers. A draft geotechnical data report was prepared in March 2018, presenting geotechnical sampling procedures and results. This geotechnical interpretive report was prepared in general accordance with the City of San Mateo Collection System Design Standard CSDS13 V2.

1.2 PURPOSE AND SCOPE

ENGEO prepared this geotechnical interpretive report to present our geotechnical recommendations for design and construction of the UFES and Diversion Sewers. The scope of services completed includes the following:

- Geotechnical data analysis.
- Interpretation of geotechnical data.
- Report preparation summarizing our conclusions and recommendations.

This interpretive report was prepared for the exclusive use of the City of San Mateo and their consultants for design of this project. In the event that any changes are made in the character, design or layout of the project, the City of San Mateo or ENGEO must be contacted to review the conclusions and recommendations contained in this interpretive report to evaluate whether modifications are recommended.

1.3 PROJECT LOCATION

The project site for the UFES and Diversion Sewers is situated within the eastern trailer parking lot for the San Mateo County Event (Expo) Center. The Diversion Sewer Branches 1 to 3, which will be connected to the UFES, are situated within the Expo Center and in public roadways including Saratoga Drive and S. Delaware Street located in San Mateo, California.

Table 1.3-1 identifies the UFES (storage tank) and diversion sewer branch segments included in this interpretive report. The SST-14 Glendora and Shasta Relief segment, SST-3 Delaware Street Relief segment, and the remaining Conveyance Pipelines and Pump Stations are included under separate reports.

TABLE 1.3-1: Summary of UFES and Diversion Sewers

PROJECT ID	PROJECT LOCATION
UFES	San Mateo County Event Center (Expo), East Trailer Parking Lot
Diversion Sewer Branch 1	Saratoga Drive and S. Delaware Street
Diversion Sewer Branch 2	S. Delaware Street and San Mateo County Event Center
Diversion Sewer Branch 3	S. Delaware Street

1.4 PROJECT DESCRIPTION

According to the San Mateo Basin 2 and 3 Collection System project plan sheets prepared by Stantec, the UFES is approximately 205 feet long by 145 feet wide, and is conceptually planned to be an underground storage tank extending approximately 50 to 60 feet below existing ground surface (bgs). The locations of the UFES and the Diversion Sewers vary in surface elevation from approximately 100 feet (SM+100)[1] in the north project area to approximately 110 feet (SM+100) in the south project area.

Three diversion sewer branches (Diversion Sewer Branches 1 to 3) are planned to connect the UFES (storage facility) to the existing collection system. The proposed diversion sewer branches include the installation of new pipelines up to 36 inches in diameter. The three proposed diversion sewer branches are approximately between 1,120 to 3,000 lineal feet for each branch segment. The diversion pipeline inverts are currently planned to extend approximately 10 to 24 feet bgs with a slope gradient that varies from roughly 0.2 to 2.5 percent toward the UFES.

2.0 SUMMARY OF GEOLOGIC SAMPLING DATA

2.1 GEOTECHNICAL SAMPLING SUMMARY

The geotechnical field exploration for the UFES and Diversion Sewers was performed from October 12 to November 19, 2017. An ENGEO representative observed the drilling and Cone Penetration Test (CPT) probe activities and logged the subsurface conditions at each location. A truck-mounted drill rig and crew were retained to advance the borings using mud rotary and hollow stem auger drilling methods. The borings were advanced to depths ranging from approximately 33½ to 151½ feet bgs. For the CPTs, a truck-mounted vehicle with crew were retained to advance the probes to depths ranging from approximately 91½ to 100¼ feet bgs. Vibrating wire piezometers (VWP) were installed at select geotechnical borings during the field exploration program to monitor the groundwater fluctuations near the installed locations (Table 2.2.2-2). One standpipe well was installed at boring 1-EXPO-TNK-B2 within the UFES site.

2.1.1 Diversion Sewers Subsurface Profile

Through a combination of exploratory boreholes and review of published geologic information (Pampeyan, 1994), the following subsurface conditions at the site were identified. The soil conditions along the Diversion Sewers are anticipated to include artificial fill (Qf_1), bay mud (Qm), course-grained alluvium (Qac), medium-grained alluvium (Qam), and fine-grained alluvium (Qaf).

Table 2.1.1-1 provided below, summarizes the geologic stratigraphy encountered within the Diversion Sewer exploration locations from the ground surface to the bottom of the exploration. A description of the geologic units is included within the UFES and Diversion Sewers geotechnical data report.

TABLE 2.1.1-1: Geologic Units Encountered During Field Exploration

PROJECT ID	GEOLOGIC UNIT (PAMPEYAN, 1994)
Diversion Sewer Branch 1	Qf_1, Qm, Qam, Qaf
Diversion Sewer Branch 2	Qf_1, Qm, Qac, Qam, Qaf
Diversion Sewer Branch 3	Qf_1, Qam

[1] Project Datum is noted as City of San Mateo Datum + 100 feet (SM+100) in this report.

Based on laboratory testing on select soil samples, the general engineering properties of the soil stratum tested are summarized below. No samples were retrieved for artificial fill, so no lab testing was performed. Additionally, one sample of coarse-grained alluvium was retrieved within Boring 1-EXPO-BR2-B2 at a depth of 35 feet; however, no lab testing was performed on the sample due to the proposed shallower depth of the pipeline alignment.

TABLE 2.1.1-2: Summary of Soil Properties along Diversion Sewers

TESTED PROPERTIES	ASTM	NUMBER OF TESTS	RANGE OF RESULTS
Bay Mud (Qm)			
Moisture Content	D2216	1	87%
Total Unit Weight	D7263	1	94.1 pcf
Sieve Analysis	D422	0	Not tested
Plastic and Liquid Limits	D4318	1	Plastic Limit: 39 Liquid Limit: 127 Plasticity Index: 88
Unconfined Compressive Strength	D7012	0	Not tested
Undrained Shear Strength (Vane Shear Test)	D4648	1	152 psf
Medium-Grained Alluvium (Qam)			
Moisture Content	D2216	28	13.5 to 24.6%
Total Unit Weight	D7263	27	104.5 to 122 pcf
Sieve Analysis	D422	10	Fines (Clay & Silts): 12 to 83%
Plastic and Liquid Limits	D4318	8	Plastic Limit: 21 to 51 Liquid Limit: 14 to 19 Plasticity Index: 5 to 32
Unconfined Compressive Strength	D7012	6	0.95 to 3.01 tsf
Undrained Shear Strength (Isotropic Unconsolidated Undrained Triaxial Test)	D2850	5	473.5 to 3482 psf
Undrained Shear Strength (Vane Shear Test)	D4648	1	966 psf
Fine-Grained Alluvium (Qaf)			
Moisture Content	D2216	1	20.1%
Total Unit Weight	D7263	1	106.4 pcf
Sieve Analysis	D422	1	Fines (Clay & Silts): 57%
Plastic and Liquid Limits	D4318	0	Not tested
Unconfined Compressive Strength	D7012	0	Not tested
Undrained Shear Strength	D2850, D4648	0	Not tested

The geologic units, associated thickness, and approximate geologic contacts are presented on the diversion pipeline profiles shown on Sheets 8 through 17.

2.1.2 UFES Subsurface Profile

Exploratory borings were drilled within the proposed tank site within the RV parking lot on the eastern side of the San Mateo Event Center. In general, the borings within the tank site encountered approximately 5 feet of artificial fill, which consisted of sandy clay and clayey sand. Beneath the artificial fill, approximately 1½ to 2 feet of Bay Mud was encountered. Underlying the Bay Mud, the borings encountered natural alluvial soil deposits consisting of medium stiff-to-stiff lean clays and sandy clays to a depth of approximately 35 feet bgs, followed by stiff to very stiff lean and fat clays to a depth of approximately 50 feet bgs. A layer of medium dense to very dense clayey sand, and very stiff to hard sandy to gravelly clay was encountered in each of the borings starting at about 50 feet bgs, and varied in thickness ranging from 15 to 26 feet. Below the more granular layer, hard lean and sandy clay was encountered to the maximum depth explored of 151½ feet. The CPT probes indicated similar subsurface profiles.

The UFES exploration locations and associated cross-sections are presented on Sheets 5 to 7 and the geotechnical laboratory test results are presented in Appendix G of the UFES and Diversion Sewers geotechnical sampling data report.

2.2 GROUNDWATER CONDITIONS

2.2.1 At Time of Drilling

The following table summarizes groundwater measurements taken when groundwater was encountered during hollow-stem auger drilling operations and CPT probe operations. For the mud-rotary boreholes drilled for the UFES (1-EXPO-TNK-B1 through B4), groundwater was not measured at the time of drilling due to the drilling method.

TABLE 2.2.1-1: Groundwater Level Encountered at Time of Drilling/Probing

BOREHOLE ID	APPROXIMATE DEPTH TO GROUNDWATER AT TIME OF DRILLING (feet)	INTERPRETED GROUNDWATER ELEVATION AT TIME OF DRILLING (feet, SM+100)
1-EXPO-BR1-B1	10.5	92.5
1-EXPO-BR1-B2	10	91
1-EXPO-BR1-B3	9	96
1-EXPO-BR1-B4	10	95
1-EXPO-BR2-B1	10	95
1-EXPO-BR2-B2	14	91
1-EXPO-BR2-B3	20	87
1-EXPO-BR3-B1	15	92
1-EXPO-BR3-B2	10	100
1-EXPO-TNK-CPT3	7.5	93.5

2.2.2 Post-Installation Groundwater Monitoring

In February 2018, data from the standpipe well installed at Boring 1-EXPO-TNK-B2, and from the vibrating wire piezometers installed at select borings within the UFES and Diversion sewers were obtained.

Table 2.2.2-1 presents groundwater measurements from the standpipe well and the interpreted groundwater measurements for select vibrating wire piezometers. The remaining data collected from the VWPs will be presented in a separate report.

TABLE 2.2.2-1: Groundwater measurements from standpipe well and vibrating wire piezometer (November 17, 2017 to February 27, 2018)

BOREHOLE ID	SENSOR DEPTH OR SCREENED DEPTH (feet, bgs)	RANGE OF GROUNDWATER DEPTH* (feet)	INTERPRETED RANGE OF GROUNDWATER ELEVATION (feet, SM+100)	AVERAGE TEMPERATURE AT SENSOR DEPTH (°C)
1-EXPO-BR1-B3	34	4.8 to 6.1	98.9 to 100.2	19.3
1-EXPO-BR2-B2	30	5.7	99.3	20.3
1-EXPO-BR3-B1	30	9.6 to 11.9	95.1 to 97.4	19.0
1-EXPO-TNK-B1	65	13.2 to 15.1	85.9 to 87.8	20.2
1-EXPO-TNK-B2	55 to 70	4.3 to 6	96.7 to 95	--
1-EXPO-TNK-B3	32	2.7 to 3.9	97.1 to 98.3	19.7
1-EXPO-TNK-B4	20	4.9 to 6.8	94.2 to 96.1	21.6
1-EXPO-TNK-B4	45	3.3 to 4.6	96.4 to 97.7	19.8

GeoTracker, a website maintained by the State of California, identified wells located within a 1-mile radius of the property. The wells reported depths to groundwater between approximately 2 to 40 feet bgs, with groundwater flow direction generally to the north and northeast. Groundwater levels in borings from projects in the vicinity ranged from 7.5 to 20 feet below the ground surface.

Fluctuations in groundwater levels occur seasonally and over a period of years because of variations in tidal action, precipitation, temperature, irrigation, and other factors. In addition, the measurements performed on the days of our exploration may not represent a fully equilibrated groundwater level due to the less permeable clayey soils encountered.

3.0 SUMMARY OF GEOTECHNICAL CONSIDERATIONS

3.1 SEISMIC DESIGN

The subject project site was evaluated with respect to known geologic hazards common to the San Francisco Bay Region. 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, ground lurching, soil liquefaction, and lateral spreading. The following discussion of these hazards, as they apply to the subject storage tank and diversion pipelines, is based on our understanding of the regional seismicity, review of readily available geologic reports and maps, and subsurface conditions.

3.1.1 Ground Rupture

The project site is not located within a currently designated State of California Earthquake Fault Zone, and no known active faults are mapped on the site. The nearest known active fault is the San Andreas fault, located about 3.3 miles west of the project site limits. Major active faults in the region are shown on Sheet 4. Based on these findings, the risk of faulting occurring within the project limits is considered low.

3.1.2 Design Ground Motion

An earthquake of moderate to high magnitude generated by the nearby active faults, similar to those that have occurred in the past, could cause considerable ground shaking at the site. To mitigate the ground shaking effects, the proposed UFES and Diversion Sewers should be designed using sound engineering judgment and the latest California Building Code (CBC) and State of California Department of Transportation (Caltrans) requirements as a minimum, when applicable.

The 2016 CBC utilizes design criteria set forth in the 2010 ASCE 7 Standard. Based on the subsurface conditions encountered, we characterized the site as Site Class D in accordance with the 2016 CBC.

Additionally, in-situ shear wave velocity measurements from a seismic cone penetrometer testing for the upper 100-ft of the site profile resulted in an average shear wave velocity of approximately 945 feet per second, which classifies as a Site Class D soil. A Risk Category III was assigned to the site, as provided by Stantec. We provide the 2016 CBC seismic design parameters in Table 3.1.2-1 below, which include design spectral response acceleration parameters based on the mapped Risk-Targeted Maximum Considered Earthquake (MCER) spectral response acceleration parameters.

TABLE 3.1.2-1: 2016 CBC Seismic Design Parameters, Latitude: 37.54703 **Longitude:** -122.2981

PARAMETER	VALUE
Site Class	D
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	1.86
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.86
Site Coefficient, F_A	1.0
Site Coefficient, F_V	1.5
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	1.86
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	1.30
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	1.24
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	0.86
Mapped MCE Geometric Mean (MCE_G) Peak Ground Acceleration, PGA (g)	0.73
Site Coefficient, F_{PGA}	1.0
MCE_G Peak Ground Acceleration adjusted for Site Class effects, PGA_M (g)	0.73

3.1.3 Liquefaction and Seismic Settlement

Liquefaction is a phenomenon wherein saturated cohesionless soils lose their inherent shear strength due to increased pore water pressures, which may be induced by reversing cyclic shear stresses associated with earthquakes. Low-relative-density cohesionless soils, shallow groundwater, and long-duration and high-acceleration seismic shaking are some of the factors that cause liquefaction. Surface manifestation of liquefaction is generally observed when saturated liquefiable material is present within about 50 feet from the ground surface.

Based on our review of the liquefaction hazards map by the California Geological Survey (CGS), the UFES and Diversion Sewers are located in an area identified as having a potential susceptibility to liquefaction. The liquefaction hazards map is included on Sheet 3.

We performed our analyses using a peak ground acceleration value (PGA) of 0.73g as outlined in the 2016 California Building Code, and a moment magnitude of 7.9 based on the theoretical rupture of the San Andreas Fault. The design groundwater table was established between 3 to 5 feet below existing grades depending on location, shown in Section 3.2.

According to Bray and Sancio (2006), fine-grained soils with a plasticity index (PI) less than or equal to 12 and moisture content and liquid limit ratio (wc/LL) of greater than 0.85 can undergo cyclic mobility and are susceptible to liquefaction. Based on our laboratory results, fine-grained soils with a PI less than or equal to 12 yielded a liquid limit ratio less than 0.85. Therefore, the risk of cyclic softening and liquefaction of fine-grained soils is considered low.

3.1.3.1 Liquefaction Analysis for Diversion Sewers

For Diversion Sewers Branches 1 to 3, liquefaction analysis of the borings was performed using triggering and settlement analysis methodologies outlined by Youd et al. (2001) and Idriss and Boulanger (2008), respectively. The analyses indicated that the well-graded sand layer in Diversion Sewer Branch 1 is potentially liquefiable, while the soil profile along Branches 2 and 3 do not appear to be liquefiable. Based on our analysis, we estimated the following potential liquefaction induced settlements for the susceptible Diversion Sewer.

TABLE 3.1.3.1-1: Estimated Potential Settlement Due to Liquefaction – Diversion Sewers

PROJECT ID	SOIL TYPE	POTENTIAL LIQUEFIABLE LAYER DEPTH RANGE (FEET)	ESTIMATED TOTAL SETTLEMENT (INCHES)	APPROXIMATE PIPE INVERT DEPTH (FEET, BGS)
Diversion Sewer Branch 1	SW	12 to 15	¼	17 to 24

As noted in Table 3.1.3.1-1, the proposed pipe invert depth of Diversion Sewer Branch 1 is deeper than the bottom of the liquefiable soil; therefore, the risk of liquefaction-induced settlement under Diversion Sewer Branch 1 pipeline will be low.

3.1.3.2 Liquefaction Analysis for UFES

To evaluate the liquefaction potential for the UFES site, liquefaction analyses utilizing the data obtained from the CPT probes was performed. Considering the planned excavation depth of 50 to 60 feet for UFES construction, any potentially liquefiable soils within the upper 50 feet of the tank footprint will be mitigated.

For depths below 50 feet, the liquefaction potential at the UFES site was evaluated using the CPT data and the computer program, CLiq Version 2.1.6.11, assuming an I_c cutoff of 2.60, and using methods developed by Idriss and Boulanger (2008) and Robertson (2009). The liquefaction analysis using CLiq indicates that some approximately 1 to 3 feet thick medium dense layers of silty and clayey sand and sandy and clayey silt below 50 feet bgs and below the groundwater table are considered potentially liquefiable when subject to strong ground shaking. Confirmation samples in the potentially liquefiable layers were collected and laboratory testing was performed, including Plasticity Index (PI), Fines Content, and Moisture Content to further evaluate the liquefaction potential based on methods developed by Bray and Sancio (2006). The test results indicated that the silty and clayey sand/sandy and clayey silt generally contains over 35 percent of fines (Passing #200) and the fines exhibit PIs ranging from 12 to 33. Based on these factors, the risk of cyclic softening and liquefaction of the silty and clayey sand/sandy and clayey silt layers is considered low. The results of the liquefaction analysis are presented in Appendix A.

3.2 GROUNDWATER CONDITIONS

As discussed previously, the groundwater levels encountered in boreholes, CPTs, vibrating wire piezometers, and the standpipe piezometer ranges from approximately 5 to 12 feet below ground surface within the Diversion Sewers and between 3 to 15 feet below ground surface within the UFES. We recommend the following design groundwater levels, ranging from 3 to 5 feet below grade, for the UFES and Diversion Sewers.

TABLE 3.2-1: Design Groundwater Level within UFES and Diversion Sewers

PROJECT ID	DESIGN DEPTH TO GROUNDWATER (FEET, BGS)
UFES	3
Diversion Sewer Branch 1	5
Diversion Sewer Branch 2	5
Diversion Sewer Branch 3	5

3.2.1 Artesian Conditions

An assessment for artesian conditions was also performed as part of this study. Artesian conditions occur when groundwater is confined under pressure between two layers of relatively impermeable strata. When the upper confining layer is penetrated, the water will rise above the level at which it was first encountered. If the gradient is sufficiently high, the groundwater may rise above the ground surface.

Based on the vibrating wire piezometer readings, potentially semi-confined artesian conditions were encountered at 1-EXPO-TNK-B1 at 65 feet bgs (El. 36 feet, SM+100), which is installed within the clayey sand, sandy clay and gravelly clay layer between two less permeable clay layers. The pressure head within this semi-confined sandy and gravelly clay layer (El. 27 to 52 feet, SM+100) is approximately 3 to 4 feet lower than the local groundwater level. Therefore, the local groundwater level is recommended in Table 3.2-1 to be used as the design groundwater level.

3.2.2 Soil Permeability and Groundwater Flow

As mentioned in the geotechnical data report for the UFES and Diversion Sewers, packer tests were performed at Borehole 1-EXPO-TNK-B4. Two single packer tests were performed at depth intervals 15 to 20 feet bgs and 41 to 50 feet bgs to measure groundwater flow rates.

Based on the results of the packer tests in Borehole 1-EXPO-TNK-B4, the clayey sand to sandy clay deposits encountered between 15 and 20 feet bgs had a field measured flow rate of approximately 0.35 gallons per minute (gal/min) or 1.9 cubic meters per day (m^3/day), and a horizontal permeability of approximately 2.4×10^{-4} centimeters per second (cm/s). The clayey sand deposits encountered between 41 and 50 feet bgs had a field measured flow rate range of approximately 1.0 to 1.5 gal/min or 5.5 to 8.2 m^3/day, and a horizontal permeability range of approximately 2×10^{-4} cm/s to 1.6 $\times 10^{-4}$ cm/s.

Permeability laboratory tests were performed on three samples obtained from the upper 20 feet. Soils encountered in the upper 20 feet of the UFES consisted of artificial fill (clayey sands and sandy clay) and Bay Mud. The vertical permeability measured is approximately 10^{-6} to 10^{-7} cm/s.

Based on the laboratory permeability test results and the in-situ packer test results, the vertical and horizontal permeability and the groundwater flow rate of site soils are low.

3.3 EXISTING FILL

As previously discussed, the UFES and Diversion Sewers are underlain by existing artificial fill extending from the ground surface to depths of approximately 5 feet overlying alluvial or bay mud deposits. The existing artificial fill typically consists of soft to medium stiff sandy clays (CL) and clayey gravelly sand (SC).

3.4 EXPANSIVE SOILS

Soils samples from the upper 10 feet were tested for Plasticity Index (PI) with values ranging from 5 to 32, indicating that these materials ranged from low to high expansion potential. Highly expansive soils are most prevalent within the norther portion of the project site, including Diversion Sewer Branches 1 and 2, and the UFES site, associated with bay mud (Qm). Expansive soils tend to shrink and swell when subject to fluctuations in moisture.

3.5 COMPRESSIBLE SOILS

As previously discussed, Diversion Sewer Branches 1 and 2, and the UFES site are underlain by very soft to stiff clay Bay Mud deposits up to 10 feet in thickness. At this time, the proposed Diversion Sewer Branch 1 and 2 pipelines and the proposed bottom of the UFES are planned to extend below the compressible Bay Mud soils. Since the compressible soils will be excavated and removed during construction within the alignment of the improvements, the risk of load-induced settlement on the improvements are considered low.

3.6 CORROSIVE SOILS

A total of seven samples were collected and transported under proper chain-of-custody to CERCO Analytical, Inc. for corrosivity testing. Samples were tested for redox potential, pH, resistivity, sulfide, soluble sulfate, and chloride ion concentrations. The results of each of these tests, organized by depth, are summarized below.

TABLE 3.6-1: Summary of Corrosivity Testing Results

BOREHOLE ID AND DEPTH	USCS SOIL TYPE	REDOX POTENTIAL (mV)	pH	RESISTIVITY* (ohms-cm)	SULFIDE (mg/kg)	SOLUBLE SULFATE * (mg/kg)	CHLORIDE ION* (mg/kg)
1-EXPO-BR2-B2 @ 5.5’	CL	380	7.59	1,100	N.D.	19	110
1-EXPO-TNK-B3 @ 6’	CH	260	6.78	380	N.D.	N.D.	580
1-EXPO-BR3-B1 @ 15	CL	400	7.96	390	N.D.	160	500
1-EXPO-BR1-B4 @15.5’	CH	470	8.04	730	N.D.	100	34
1-EXPO-TNK-B2 @ 21’	CH	370	7.71	130	N.D.	330	2,100
1-EXPO-TNK-B4 @ 36’	CH	280	7.15	220	N.D.	250	1,400
1-EXPO-TNK-B1 @ 55.5’	CH	380	7.67	470	N.D.	28	430

*Results reported on a wet weight basis
N.D. – None detected above reporting limits

Based on the resistivity measurements on samples obtained along the pipeline alignment and within the UFES site, the soils are considered to be “corrosive” to “very corrosive” to buried metal piping (NCHRP, 1978). 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 pipelines should be protected against corrosion. A corrosion consultant should provide specific design recommendations on corrosion protection for the buried storage tank and diversion branch pipelines.

The reported sulfate concentration result ranged from non-detect to 330 mg/kg. The 2016 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 guidelines to characterize the potential exposure for sulfate attack and associated recommendations for concrete in contact with soil based upon the exposure risk. In accordance with the criteria presented in Table 19.3.1.1 of the ACI 318-14, the test results are classified in the “not applicable” sulfate exposure range. Considering the “not applicable” sulfate exposure, the building code specifies a minimum concrete compressive strength of 2,500 psi. Additionally, for hydraulic structures, ACI 350-06 is the governing standard. In accordance with Table 4.3.1 of ACI 350-06, the test results are classified in the “negligible” sulfate exposure, and specifies a maximum water-cement ratio of 0.45. Although there is no requirement for cement type at this exposure range, a Type II (MH) and Type V cement can also be used. It should be noted, however, that the structural engineering design requirements for concrete may result in more stringent concrete specifications.

It should be noted that testing was not completed in near-surface soils, nor was it completed for all depths of potential embedment. Once more specifics of the proposed improvements are known, additional testing and/or guidance regarding the exposure risk for sulfates can be provided. Steel reinforcement in concrete should be provided with adequate cover in accordance with the CBC, as a minimum, and the structural engineering design requirements, which might result in more stringent concrete specifications once the final disposition of potential concrete elements are known.

4.0 ENGINEERING CONSIDERATIONS – PIPELINE

4.1 OPEN CUT TRENCHING METHOD

Open-cut trenching is a conventional method to install pipelines. This method consists of excavating a trench along the pipeline alignment, placing the pipe on stable base subgrade material, dewatering and trench supporting (as necessary), and backfilling the excavation. Open cut pipeline installation is feasible for the Diversion Sewers.

The main disadvantages of open cut pipeline installation are the need for shoring, dewatering static or perched groundwater, and offhaul of dewatering liquids and soil along the alignment. Significant disturbance and potential settlement to overlying streets or surface conditions along the alignment may occur.

If this method is selected, the pipelines should be installed by a qualified Contractor experienced in such installation methods. Additional recommendations can be provided once a final alignment has been designed and if this method is selected.

4.2 TRENCHLESS PIPELINE INSTALLATION

It is our understanding that a portion of Diversion Branch 1 pipeline, near the intersection of South Delaware Boulevard and Saratoga Drive, is proposed to be installed using microtunneling, which is a trenchless pipeline installation method.

As shown on Sheet 17, and Civil Plans for the In-System Storage Package, prepared by Stantec and dated January 30, 2018, the proposed section of Diversion Sewer Branch 1 will be installed within the public right-of-way and below an existing culvert crossing under Borel Creek, parallel to Saratoga Drive. The proposed pipeline section will be installed in variable fine-grained and alluvial deposits (Qaf) and medium-grained alluvial deposits (Qam) beneath the existing artificial fill and Bay Mud layers and groundwater table.

As shown on the plan sheets, the pipe invert is at an elevation of approximately 85 feet (SM+100). Based on the soil conditions encountered within nearby borings 1-EXPO-BR1-B4, 1-EXPO-BR3-B1, and 1-B12 and 1-B21, the proposed trenchless pipeline section will go through silty lean clay with variable amounts of sand, and silty sand. The results from our laboratory testing indicate the fines (clays and silts) within the silty sand is approximately 12 percent and within the lean clay is approximately 64 percent. The plasticity index of the lean clay ranged from 10 to 31, indicating a range of low to high expansion potential. We recommend a total unit weight of 130 pounds per square foot (psf) for the lean clay and silty sand soil. Additionally, the undrained shear strength of the lean clay generally ranged from 500 to 800 psf.

Mixed-face and change-in-face conditions between fine-grained silts and clays and granular soils with varying amounts of gravel should be anticipated. There is a risk of the microtunnel boring machine (MTBM) becoming stuck at these transition zones. However, selection of an appropriate MTBM cutter head to handle these soil conditions should minimize this risk.

Microtunneling is a trenchless installation method where a guided pipe advancement tunneling process is used. The pipeline is advanced directly behind and attached to a remotely controlled, laser-guided, slurry-based microtunnel boring machine (MTBM) that provides continuous support to the excavation face. This method requires construction of launching and receiving pits and the launching pits must be designed to accommodate specified jacking loads. Microtunneling is feasible for this area of Diversion Sewer Branch 1.

The launching and receiving pits for the trenchless installation can be designed for active lateral equivalent fluid pressures provided in the table below.

TABLE 4.2-1: Trenchless Installation Design Parameters for Diversion Sewer Branch 1

LATERAL EARTH PRESSURE	DESIGN PARAMETER
Active Earth Pressure:	60 pcf (drained conditions). Active earth pressures should be used where existing buildings and critical utilities are situated outside a 1:1 line of projection extending up from the bottom of the wall.
Passive Earth Pressure:	250 pcf, acting as equivalent fluid weight.

The trenchless technology used should maintain line and grade for the pipeline within tolerances desired for this project and should avoid impacts to overlying existing improvements. The actual improvements and selected trenchless installation method should be designed and installed by a qualified Contractor and designer experienced in such installation methods.

4.3 DIVERSION SEWER PIPELINES

The Diversion Sewer pipeline inverts are currently proposed at approximately 10 to 24 feet below existing grade. Below is a summary of subsurface conditions on the Diversion Sewers.

TABLE 4.3-1: Summary of Subsurface Conditions for Diversion Sewers

PROJECT ID	STATION	PIPE INVERT ELEVATION (FEET,SM+100)	SOIL TYPE
Diversion Sewer Branch 1	10+00 to 14+60, 18+00 to 20+00	80.5 to 81	Fine-grained alluvium
	14+60 to 18+00, 20+00 to 40+07	81 to 86.5	Medium-grained alluvium
Diversion Sewer Branch 2	10+00 to 30+60	80.5 to 94.5	Medium-grained alluvium
Diversion Sewer Branch 3	10+00 to 13+00, 16+45 to 18+00	86 to 87	Fine-grained alluvium
	13+00 to 16+45, 18+00 to 21+90.11	86 to 87.5	Medium-grained alluvium

4.3.1 Soil Loads

The proposed pipeline should be designed to resist loads imposed by overlying soil cover and from vehicle or construction traffic. Soil loads may be calculated using a total unit weight up to 135 pounds per cubic foot (pcf) and a buoyant unit weight of 75 pcf for fill and alluvial soils.

4.3.2 Modulus of Soil Reaction

Provided the site earthwork is conducted in accordance with the recommendations in this report, the modulus of soil reaction in the table below can be used for the pipeline design. The modulus of soil reaction given in Table 4.3.2-1 is based on soil conditions encountered during the field exploration and also assumes a required relative compaction of not less than 85 percent.

TABLE 4.3.2-1: Modulus of Soil Reaction

SOIL BACKFILL TYPE	DEPTH OF COVER (FEET)	MODULUS OF SOIL REACTION (PSI)
Site Soils	2-5	700
	5-10	1,000
	10-15	1,050
	15-20	1,100
Import Granular Material	--	1,000

4.4 MANHOLES AND JUNCTION BOXES

Based on the soil conditions encountered along the planned diversion sewer pipeline depths, manholes and junction boxes are anticipated to be bottomed/supported on fine- to medium-grained alluvium. An allowable bearing capacity of 2,500 pounds per square foot (psf) can be considered in the design of manholes and junction boxes founded on the alluvial soils. Earth pressures for the design of walls are presented in Section 6.2.1.

4.5 BUOYANCY

The pipeline, manholes, and junction boxes should be designed for buoyancy effects considering a design groundwater depth of 5 feet. Where buoyancy effects are determined to be high, concrete collars or tie downs should be used to resist uplift.

5.0 ENGINEERING CONSIDERATIONS – UFES

5.1 EXCAVATION AND GROUNDWATER

Groundwater is relatively shallow throughout the UFES site. Design considerations addressed later in this report include construction dewatering, hydrostatic uplift forces, waterproofing, and wall drainage.

5.2 BUOYANCY

We understand that the UFES will go through cycles of filling and emptying. The UFES will be subject to buoyant uplift forces when tank water levels are low. The structural engineer may consider the following forces to resist buoyancy upload forces:

- Weight of the empty UFES structure.
- Weight of the soil projected vertically from the edge of tank wall footings. Estimate a unit weight of wall backfill of 125 pcf.
- Skin friction on piles constructed at the bottom of the tank (See Section 7.0 for details)

6.0 CONSTRUCTION AND EARTHWORK CONSIDERATIONS

Provided below are general construction recommendations for the project.

6.1 PRECONSTRUCTION AND CONSTRUCTION SETTLEMENT SURVEYS

A preconstruction survey and construction surveys are recommended to monitor for potential movements of existing structures or improvements that may be affected by construction activities. Existing structures and improvements may experience movement as a result of shoring installation, dewatering, or pipeline installation. For this project, a minimum frequency of at least weekly is suggested during construction. If excess movement is noted, work should be stopped immediately and the Engineer should be notified.

Moreover, the locations and depths of the existing utilities located adjacent to or over the proposed pipeline should be evaluated such that they are not undermined or damaged during construction. Protection of existing utility crossings in trenches should also be considered. Critical utilities should be protected through cradling while less critical utilities could span trenches unprotected.

6.2 EXCAVATION AND SHORING

Shoring is required for sections of the sanitary sewer pipes with vertical excavations greater than 4 feet and for the UFES excavation. The Contractor should be familiar with applicable local, state, and federal regulations, including the current Occupational Safety and Health Administration

(OSHA) Excavation and Trench Safety Standards. It is the responsibility of the Contractor to provide stable, safe trench and construction slope conditions and to follow OSHA safety requirements. Since excavation procedures may be dangerous, it is also the responsibility of the Contractor to provide a trained "competent person" as defined by OSHA to supervise all excavation operations, ensure that all personnel are working in safe conditions and have thorough knowledge of OSHA excavation safety requirements.

Shoring systems should be designed by a qualified registered engineer. Variation in hydrostatic pressures or surcharges may require an increase in design pressures and distribution. The design of the shoring should be sufficiently rigid to prevent detrimental movement of the temporary shoring and possible damage of pavements, sidewalks, or adjacent utilities. Appropriate safety factors against overturning and sliding should be incorporated into the design calculations.

Excavated soils, construction materials or other items imposing a surcharge should be stockpiled at least 20 feet away or at least a 1:1 setback, whichever is greater, from the edge of excavations to reduce potential adverse effect on slope or trench stability. We recommend that no vertical trench excavations be left open overnight without adequate shoring. Once shoring has been removed, the contractor should backfill the excavation to within 4 feet of the ground surface before the end of the day.

6.2.1 Diversion Sewers Excavation

Excavations ranging from 10 to 24 feet deep are anticipated along the diversion sewer pipeline alignment within roadways. The specified clearance between the Diversion Sewer pipeline alignments and other utilities is 3 feet in several locations. Based on soil and groundwater conditions, the use of trench boxes, hydraulic shoring, shields with plates, or a cross-brace strut and lagging system appear to be suitable shoring options for the Diversion Sewers.

The temporary shoring design may be designed for active lateral equivalent fluid pressures provided in the table below.

TABLE 6.2.1-1: Temporary Shoring Design Parameters for Diversion Sewers 1, 2 and 3

TEMPORARY SHORING DESIGN ELEMENT	DESIGN PARAMETER
Active Earth Pressure:	60 pcf (drained conditions). Active earth pressures should be used where existing buildings and critical utilities are situated outside a 1:1 line of projection extending up from the bottom of the wall.
Passive Earth Pressure:	250 pcf, acting as equivalent fluid weight.

Surcharge loads from structures, stockpiles, and vehicles should be included in shoring design if the surcharge loading is situated within 20 feet of the top of the trench or within a 1:1 line of projection extending from the bottom of the trench, whichever is farther. The surcharge should be taken as one-half of any vertical surcharge loads and should be applied as a uniform lateral load. A minimum lateral surcharge load equal to 72 psf, as prescribed in the Caltrans Trenching and Shoring Manual, should be considered for traffic loading, where applicable.

The final temporary shoring design will be based on the contractor's means and methods of construction, including equipment and available shoring materials, as well as other general conditions defined by the project team.

6.2.2 UFES Excavation

The UFES excavation is expected to be approximately 145 feet wide, 205 feet long and 50 to 60 feet below existing grade. Typical shoring for large and deep excavations including driven sheet piles, cross-lot/internal braces and anchored soldier piles and lagging walls. For the proposed UFES excavation, an anchored soldier piles and lagging wall system is anticipated to be more cost effective.

The temporary shoring may be designed for active lateral equivalent fluid pressures provided in the table below. When permanent shoring systems are planned, at-rest pressures provided below should be considered. For thickness and depth of soil layers presented in Table 6.2.2-1, refer to Sheets 6 and 7.

TABLE 6.2.2-1: Temporary and Permanent Shoring Design Parameters for UFES

SOIL LAYER	AT-REST UNDRAINED PRESSURES (pcf)	ACTIVE UNDRAINED PRESSURES (pcf)
Artificial Fill /Young Bay Mud	110	100
Lean Clay and Sandy Clay (medium stiff to very stiff)	100	90
Lean Clay and Fat Clay (stiff to very stiff)	100	80
Clayey Sand, Sandy to Gravelly Clay (medium dense/very stiff to very dense/hard)	90	80
Lean Clay and Sandy Clay (hard)	100	60

6.2.2.1 Anchored Soldier Beam and Lagging Wall

Anchored soldier beam and lagging shoring walls are commonly designed and constructed in accordance with the Federal Highway Administration (FHWA) Geotechnical Engineering Circular No. 4 (FHWA-IF-99-015). Soldier beams usually consist of steel beams such as wide flange sections installed in drilled shafts. The drilled shaft diameter and spacing will depend on the structural shape and diameter of the ground anchor. The spacing between drilled shafts (center to center) will depend on capacity requirements. The drilled shafts should be backfilled with lean-mix concrete from the level of the excavation subgrade to the existing ground surface to allow for easy removal, which will be required for lagging and anchor installation. Unless the structural engineer determines otherwise, lean-mix concrete is commonly used to backfill the portion of the shafts from the bottom of the hole to the excavation subgrade depending on the capacity requirements of the embedded portion of the shoring wall. Photographs 6.2.2.1-1 and 6.2.2.1-2 below show an anchored soldier beam and lagging wall system being installed in San Francisco for a 55 feet deep basement. A cement deep soil mixing (CDSM) cut-off wall described in Section 6.3.2 below was installed at this San Francisco site prior to installation of soldier beam, lagging and tieback anchors.

PHOTOGRAPH 6.2.2.1-1: A solider pile and lagging shoring system with tieback anchors for a 55 feet deep excavation in San Francisco. CDSM columns were pre-installed to control water inflow. Interior dewatering wells are installed within the excavation to keep the excavation dry.

Lagging for a temporary shoring wall may consist of timber and should be placed from the top-down as soon as possible after excavation to minimize erosion of materials into them excavation.

Ground anchors, also commonly referred to as tiebacks, are structural elements installed in grout-filled holes drilled into soil and are used to transmit applied tensile loads into the ground. The drilling method used for the installation of ground anchors should consider the potential for caving of the drilled holes. Typical tieback inclinations range between 15 and 30 degrees below the horizontal. Ground anchor inclinations up to 45 degrees below the horizontal can generally be installed by most contractors. For preliminary design and cost estimate, the bonded zone of the ground anchors cab be assumed to locate behind a potential failure plane, drawn from the heel of the wall at a 30-degree angle from vertical. This plane roughly corresponds to the active earth pressure wedge for the site alluvial deposits. The vertical position of ground anchors will depend on capacity requirements and constructability. The horizontal spacing of the ground anchors should be large enough to avoid group effects of anchors.

For preliminary design and cost estimating purposes, an ultimate (unfactored) bond strength of 2.0 ksf for gravity-grouted anchors in soil (fill and alluvium) may be assumed. Also, a minimum of 15 feet of overburden soil should be present at the center of the ground anchor bond zone for the development of the ground anchor strength for gravity-grouted anchors. If this minimum coverage cannot be maintained, the ultimate bond strength should be reduced accordingly. Ground anchor bond strengths will depend on the construction method used for ground anchor installation.

PHOTOGRAPH 6.2.2.1-2: Installation of tieback anchors within a solider pile and lagging shoring system.

Construction activities should also include sacrificial and proof anchor testing. The contractor should consider at least eight sacrificial tiebacks for the UFES excavation to confirm the ultimate bond strengths. The procedures for this testing should generally conform to those discussed in FHWA-IF-99-015. Additional proof testing should be performed on a minimum of 5 percent of the production anchors (tiebacks). It is typical for contract specifications to allow for modification of the design based on higher demonstrated ultimate bond strengths from field verification testing.

When tiebacks extend beyond the property limits, authorization from neighboring property owners will be required prior to construction. Neighboring property owner may request de-tensioning of tieback anchors upon completion of the final structural wall. Alternatively, internal bracing systems can be installed in areas when tieback anchors cannot be installed, similar to a system shown on Photograph 6.2.2.1-3.

PHOTOGRAPH 6.2.2.1-3: Internal braces installed at the corner of the excavation where tiebacks cannot be installed due to utility conflicts.

6.3 TEMPORARY DEWATERING

As discussed in Section 3.2, the design groundwater levels for the UFES and Diversion Sewers segments range from 3 to 5 feet below grade. Dewatering systems implemented within the project should be selected so as to impose minimal impact on the groundwater level surrounding the proposed excavations. The dewatering system should be designed to prevent pumping soil fines with the discharge water. Uncontrolled dewatering could cause settlement of the general area and affect existing improvements in the vicinity of the site. It should be noted that existing utilities may be bedded in gravel, which may conduct groundwater to the trench excavation.

6.3.1 Diversion Sewers Trench Dewatering

The groundwater level at the Diversion Sewer trench locations should be maintained below the bottom of the trenches for the duration of utility installation. The selection of equipment and method should be determined by the contractor. Moist to saturated subgrade conditions should be anticipated at the bottom of the utility trench.

6.3.2 UFES Excavation Dewatering

The high groundwater at the UFES site has been recently measured at approximately 3 feet below the ground surface at Elevation 98 feet (SM+100). It is likely that groundwater levels could vary from these elevations.

Laboratory test results indicated measured vertical permeability is approximately 10^{-6} to 10^{-7} cm/s. Field packer tests performed at 41 to 50 feet below the ground surface yielded a horizontal permeability in the range of 1.0 to 1.5 gallons per minute. The recorded flow rates within the tank excavation are expected to be low and can be controlled by perimeter well points. Alternatively, a slurry cut off wall can be constructed along the excavation perimeter to reduce the amount of groundwater seepage into the excavation. Slurry cut off walls for deep excavation commonly utilize Cement Deep Soil Mixing (CDSM) construction methods. We anticipate the slurry cut-off wall to extend 15 to 25 feet below the bottom of the excavation.

Dewatering should be performed in a manner such that water levels are maintained not less than two feet below the bottom of excavation prior to and continuously during shoring installation. As the excavation progresses, it may be necessary to dewater the soils ahead of the excavation, such as by continuous pumping from sumps, to control the tendency for the bottom of the excavation to heave under hydrostatic pressures and to reduce inflow of water or soil beneath temporary shoring.

Groundwater levels outside of the shoring system should not be allowed to drop significantly. Lowering of groundwater levels outside of the excavation could result in settlement of surrounding improvements. Special attention should be given to the dewatering efforts to minimize potential groundwater impacts to the nearby ponds and wetlands within the adjacent Bay Meadows Park. Piezometers should be installed outside the shoring system to monitor groundwater drawdown.

6.4 TRENCH AND EXCAVATION BACKFILL

Utility trenches and excavations should be constructed in accordance with the City of San Mateo Standard Trench Detail and recommendations provided in this report, as appropriate. Where conflict occurs, please consult with the Geotechnical Engineer for clarification.

6.4.1 Selection of Materials

With the exception of construction debris (wood, brick, asphalt, concrete, metal, etc.), trees, organically contaminated materials (soil which contains more than 3 percent organic content by weight), and environmentally impacted soils (if any), the site soils are suitable for use as engineered fill within the trench zone or for backfilling the annulus outside the storage tank. Oversized soil or rock materials (those exceeding two-thirds of the lift thickness or 3 inches in dimension, whichever is less) should be removed from the fill and broken down to meet this requirement or otherwise off-hauled.

For import material used for Diversion Sewer pipe zone backfill, we recommend it consist of quarry fines, fine- to medium-grained sand, or a well-graded mixture of sand and gravel and that this material not be used within 2 feet of finish subgrades. This material should be compacted to at least 90 percent relative compaction at a moisture content of not less than optimum and comply with the grading requirements in the following table.

TABLE 6.4.1-1: Pipe Zone Backfill

BACKFILL TYPE	GRADATION (ASTM D-421)	
	SIEVE SIZE	PERCENT PASSING
Quarry Fines*	3-inch	100
	No. 4	35-100
	No. 30	20-100

BACKFILL TYPE	GRADATION (ASTM D-421)	
	SIEVE SIZE	PERCENT PASSING
Sand	No. 4	90-100
	No. 200	0-5
Sand and Gravel Mix	2-inch	100
	No. 50	0-100
	No. 100	0-8
	No. 200	0-4

*Sand equivalent shall be not less than 20

Trench zone backfill (i.e. material placed between the pipe zone backfill and the ground surface) may consist of excavated soil or, if required, imported aggregate base compacted in accordance with the recommendations for engineered fill. Control density fill is also suitable for pipe zone and trench zone backfill. Engineered fill and backfill shall comply with the grading requirements shown in the following table.

TABLE 6.4.1-2: Trench Zone Backfill - Engineered Fill

	US STANDARD SIEVE	PERCENTAGE PASSING
GRADATION (ASTM D-421)	3-inch	100
	No. 4	35-100
	No. 30	20-100
PLASTICITY (ASTM D-4318)		Plasticity Index < 12
ORGANIC CONTENT (ASTM D-2974)		Less than 2 percent

The Geotechnical Engineer should be informed when import soil materials are planned for the site. Import materials should be submitted to, and approved by, the Geotechnical Engineer prior to delivery at the site and should conform to the requirements provided in the Supplemental Recommendations (Appendix C).

If multiple backfill types are used for the project, consideration should be given to using materials with similar unit weights to reduce potential settlement due to difference in material weight.

6.4.2 Fill Placement and Compaction

Loose soils found in excavation trenches should be removed to expose a firm undisturbed bottom, moisture conditioned and recompacted. If a yielding or soft bottom is encountered, the contractor may consider overexcavating 12 inches, placing stabilization fabric such as Mirafi 600X or geogrid such as BX1200 or TX160, and backfilling with compacted ¾- to 1½-inch clean crushed rock wrapped in a 6-ounce filter fabric. Other approaches may be acceptable and ENGEO should be consulted if alternative approaches are desired. Once a suitable firm base is achieved, fills should be placed in thin lifts with the lift thickness not to exceed 10 inches or the depth of penetration of the compaction equipment used, whichever is less. Lightweight equipment should be used when working in soft to medium stiff materials.

The following compaction control requirements should be applied to general fills comprised of onsite soils:

Test Procedures:	ASTM D-1557
Required Moisture Content:	Not less than 3 percentage points above optimum moisture content
Required Relative Compaction:	Not less than 90 percent

The following compaction control requirements should be applied to import fill material (quarry fines, sand), soil fill materials with low expansion potential (PI<12), or chemically treated soils:

Test Procedures:	ASTM D-1557
Required Moisture Content:	Not less than optimum moisture content
Required Relative Compaction:	Not less than 92 percent

The following compaction control requirements should be applied to Caltrans Class 2 aggregate base:

Test Procedures:	ASTM D-1557
Required Moisture Content:	Not less than optimum moisture content
Required Relative Compaction:	Not less than 95 percent

Backfill materials placed within the upper 12 inches below roadway subgrade should be compacted to at least 95 percent relative compaction at a moisture content of at least optimum moisture. Relative compaction refers to in-place dry density of the fill material expressed as a percentage of the maximum dry density based on ASTM D-1557. Optimum moisture is the moisture content corresponding to the maximum dry density.

Compaction of trench backfill by jetting should not be allowed.

6.4.3 Construction Monitoring and Testing

It is important that all construction activities be done under the observation of the Geotechnical Engineer's field representative, in accordance with the recommendations contained herein and in the Supplemental Recommendations in Appendix C.

7.0 FOUNDATION RECOMMENDATIONS – UFES

We recommend the proposed UFES structure to be supported on a stiff structural mat foundation. Piles can be included to resist buoyant uplift forces, as discussed in Section 5.3.

7.1 STRUCTURAL REINFORCED MAT FOUNDATION

Depending on the final design depth of the UFES, the mat foundation may be founded on lean clay and fat clay (El. 56 to 70) or Clayey Sand, Sandy Clay and Gravelly Clay (El. 48 to 58). Average bearing pressure for these two founding soil layers are shown below.

TABLE 7.1-1: Mat Foundation Design Parameters

ANTICIPATED SOIL CONDITIONS AT BOTTOM OF FOUNDATION	ALLOWABLE BEARING PRESSURE (PSF)	COEFFICIENT OF FRICTION	PASSIVE PRESSURE (PCF)
Lean Clay and Fat Clay (El. 56 to 70, SM+100)	2,500	0.30	300
Clayey Sand, Sandy to Gravelly Clay (El. 48 to 58, SM+100)	3,000	0.35	350

Resistance to lateral loads may be provided by frictional resistance between the foundation concrete and the subgrade soils, passive earth pressure acting against the side of the foundation and passive earth pressure against the below grade perimeter walls.

Prior to foundation construction, the upper 12 inches of the foundation subgrade should be scarified and recompacted in accordance with Section 6.4.2.

7.1.1 Waterproofing

As stated previously, we recommend the design groundwater level for the UFES to be 3 feet bgs (El. 98 feet, SM+100). Because the proposed foundation will extend below the groundwater level, waterproofing the base of the mat and the perimeter walls are recommended. The waterproofing should be designed by a consultant that specialized in permanent waterproofing construction and placed in accordance with manufacturer's specifications.

7.2 PILE FOUNDATIONS

To resist uplift forces, the proposed UFES structure can be supported on precast, prestressed concrete piles driven to competent soils as recommended below. Precast, pre-stressed concrete piles will derive their vertical capacity primarily from skin friction within the stiff soil layers at the proposed base of the UFES. The following recommendations were based on an estimated top of pile at El. 54 feet (SM+100).

Alternatively, drilled in-place piles such as auger cast piles (ACP), Fundex or Tubex piles may be considered for uplift resistance if noise and vibration from pile driving is not acceptable. These low vibration piles are proprietary and should be designed by a design-build or specialty contractor. ENGEO should be provided the opportunity to review the pile design to confirm assumed soil profile, soil shear strengths and downdrag forces are in conformance with site conditions.

7.2.1 Vertical Pile Capacities

For precast concrete piles, the analysis performed assumed two pile types (14- and 16-inch-square piles). A chart showing the allowable vertical pile capacity vs. depth of each pile type from 50 feet bgs (El. 51 feet, SM+100) is provided in Appendix B. For piles in cohesive soils, the FHWA recommends to calculate vertical pile capacities using the alpha method. Based on the soil conditions encountered and laboratory test results, the following adhesion values can be used to calculate the vertical pile capacities.

TABLE 7.2.1-1: Adhesion Parameters at UFES Site

APPROXIMATE DEPTH TO BOTTOM OF SOIL LAYER BELOW GROUND SURFACE (FEET)	SOIL TYPE	ADHESION (PSF)
20 to 40	Lean Clay and Fat Clay (stiff to very stiff)	950
40 to 65	Clayey Sand, Sandy to Gravelly Clay (medium dense/very stiff to very dense/hard)	1,300
65+	Lean Clay and Sandy Clay (hard)	1,300

The vertical allowable capacities and embedment lengths in the table below include a Factor of Safety of 2.0 for skin friction, and the uplift allowable capacities include a Factor of Safety of 1.5.

TABLE 7.2.1-2: Allowable Vertical Capacities and Embedment Lengths

PILE TYPE	RECOMMENDED PILE LENGTH (PILE TIP ELEVATION*), FEET	ALLOWABLE VERTICAL CAPACITY (KIPS) DEAD PLUS LIVE LOADS	ALLOWABLE UPLIFT CAPACITY (KIPS) DEAD PLUS LIVE LOADS
14-inch Diameter	17 (El. 34 ft.)	100	140
	22 (E. 29 ft.)	150	200
	27 (El. 24 ft.)	200	265
16-inch Diameter	15 (El. 36 ft.)	100	130
	20 (El. 31 ft.)	150	200
	25 (El. 26 ft.)	200	275

* Datum = City of San Mateo Datum + 100 feet (SM+100), where top of pile is assumed at El. 51 feet, SM+100

7.2.2 Corrosion Protection

As discussed above, some site soils are considered "very corrosive" to buried metal and steel embedded in a concrete mortar coating. For preliminary design and planning purposes, all concrete located at or below grade be designed for "moderate" sulfate exposure conditions. A corrosion consultant should be retained to provide specific design recommendations for corrosion protection. In addition, the structural engineering design requirements may result in more stringent concrete specifications.

7.2.3 Pile Load Tests

When a large number of piles are planned, performing a pile load test prior to production pile installation can aid in optimizing pile foundation design and likely reduce foundation costs by reducing pile lengths. Pile load tests are optional and can be performed if desired by the owner to further optimize the pile foundation design.

The load test should be performed in accordance with ASTM D1143 (Reapproved 1994) *Standard Test Method for Piles Under Static Axial Compressive Load*, *Standard Loading Procedure.* The contractor is responsible for the design, operation, and safety of the load test system. This includes supplying and installing the necessary components including the dial gauges and reference beams.

ENGEO and the structural engineer should be retained to review the load test program prior to mobilization of pile test equipment to the site. We should also be retained to monitor and evaluate the entire pile load test, including test pile installation. Load test piles should not be used as production piles. Following our analysis of the load testing, we will consult with you and the structural engineer to establish the minimum pile lengths necessary to achieve the desired pile capacities.

7.2.4 Production Pile Installation

Production piles should be driven using the same hammer and system as the indicator and load test piles. The data obtained from the indicator pile program, load tests, wave equation analysis, and this geotechnical report will be used to develop pile-driving criteria for production piles. ENGEO should be retained to observe and record the results of all production pile driving.

8.0 TANK WALL RECOMMENDATIONS - UFES

8.1 LATERAL SOIL PRESSURES

Based on the soil conditions encountered and laboratory test results, the following lateral earth pressures can be used for the permanent UFES perimeter walls, assuming a permanent shoring system is not constructed. For thickness and depth of soil layers presented in the table below, refer to Sheets 6 and 7.

TABLE 8.1-1: Lateral Earth Pressures for UFES Perimeter Walls

SOIL LAYER	AT-REST UNDRAINED PRESSURES (pcf)
Artificial Fill /Young Bay Mud	110
Lean Clay and Sandy Clay (medium stiff to very stiff)	100
Lean Clay and Fat Clay (stiff to very stiff)	100
Clayey Sand, Sandy to Gravelly Clay (medium dense/very stiff to very dense/hard)	90
Lean Clay and Sandy Clay (hard)	100

8.2 SEISMIC DESIGN CONSIDERATIONS

Where seismic evaluation is performed, the tank should be designed with an additional dynamic increment combined with active equivalent pressures and can be calculated as follows:

$$\Delta P = 15 \times H^2$$

We developed the dynamic increment formula using site soil conditions and methodologies outlined by Seed and Whitman (1970) and Monobe-Okabe (1926, 1929). A groundwater level corresponding to a depth of 3 feet below final grade should be assumed for the seismic condition. H is the retained height of the tank wall (in feet) and ΔP is the active incremental seismic force in pounds per foot of wall. The dynamic increment should be added in an inverted triangular distribution loading pattern.

8.3 TANK BACKFILL PLACEMENT

All backfill should be placed in accordance with recommendations provided previously for fill placement. Light equipment should be used during backfill compaction adjacent to tank walls to minimize possible overstressing of the walls. Provided that the fill placement and compaction specifications provided in Section 6.4.2 are followed, we estimate that settlement of the engineered backfill around the UFES will be small and therefore a downward drag coefficient of backfill on the tank wall can be neglected.

9.0 PAVEMENT DESIGN

Preliminary pavement design is provided based on assumed Traffic Index and subgrade resistance values (R-value). The Traffic Index should be determined by the Civil Engineer or appropriate public agency. The following preliminary pavement sections for new construction have been determined based on an assumed R-value of 5 and in accordance with the design methods contained in Topic 633 of Caltrans Highway Design Manual (including the asphalt factor of safety).

TABLE 9.0-1: Flexible Pavement Design

TRAFFIC INDEX (TI)	R-VALUE OF 5 (UNTREATED SUBGRADE)	
	AC (INCHES)	AB (INCHES)
5.0	3.0	10.0
6.0	3.5	13.0
7.0	4.0	16.0
8.0	5.0	18.0

Notes: AC is asphalt concrete
AB is aggregate base Class 2 Material with minimum R = 78

For pavement repairs in trenches, refer to the City Standard Details for minimum pavement sections.

Pavement construction and all materials (hot mix asphalt and aggregate base) should comply with the requirements of the Standard Specifications of the State of California Division of Highways, City of San Mateo requirements and the following minimum requirements.

- All pavement subgrades should be scarified to a depth of 10 to 12 inches below finished subgrade elevation, moisture conditioned to at least optimum moisture content, and compacted to at least 95 percent relative compaction and in accordance with City of San Mateo requirements.

- Aggregate base materials should meet current Caltrans Standard Specifications for Class 2 aggregate base and should be compacted to at least 95 percent of maximum dry density at a moisture content of at least optimum.

- Subgrade soils should be in a stable, non-pumping condition at the time aggregate base materials are placed and compacted. Proof-rolling with a heavy wheel-loaded piece of construction equipment should be implemented after preparation and compaction of the subgrade soils and again after placement and compaction of the aggregate base. Yielding materials should be appropriately mitigated, with suitable mitigation measures developed in coordination with the client, contractor and Geotechnical Engineer.

- Adequate provisions must be made such that the subgrade soils and aggregate base materials are not allowed to become saturated.

- All vertical concrete curbs separating pavement and irrigated landscaped areas should extend into the subgrade and below the bottom of adjacent aggregate base materials. An undercurb drain could also be considered to help collect and transport subsurface seepage.

10.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents a broad characterization of subsurface conditions. 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 limited environmental exploration performed was intended to provide preliminary testing to determine potential presence of hazardous materials that may be encountered during pipeline trenching activities.

ENGEO strived to perform its professional services in accordance with generally accepted geotechnical and environmental engineering principles and practices currently employed in the area; no warranty is expressed or implied. There are risks of earth movement and property damages inherent in building on or with earth materials. ENGEO is unable to eliminate all risks or provide insurance; therefore, is unable to guarantee or warrant the results of its services.

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

Actual field or other conditions may necessitate clarifications, adjustments, modifications or other changes to this document. Therefore, ENGEO should be engaged to prepare the necessary clarifications, adjustments, modifications or other changes before construction activities commence or further activity proceeds. If ENGEO's scope of services does not include onsite construction observation, or if other persons or entities are retained to provide such services, ENGEO 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.

SELECTED REFERENCES

1. Brabb, E.E., Graymer, R.W., and Jones, D.L., 1998, Geology of the Onshore Part of San Mateo County, California; U.S. Geological Survey, Open-File Report 98-137, scale 1:62,500.

2. Brabb, E.E., and Pampeyan, E.H., 1983, Geologic Map of San Mateo County, California; U.S. Geologic Survey, scale 1:62,500.

3. Bray, J.D. and Sancio, R. B., 2006, Assessment of Liquefaction Susceptibility of Fine-Grained Soils.

4. California Department of Conservation, Division of Mines and Geology (CDMG), 1974, State of California Special Studies Zones, San Mateo Quadrangle. Scale 1:24,000.

5. California Geologic Survey, 2008, Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California.

6. Division of Mines and Geology, 1997, Special Publication 117, Guidelines for Evaluation and Mitigating Seismic Hazards in California, Adopted March 13.

7. ENGEO Incorporated, In-System Storage Facility and Diversion Sewers Geotechnical Sampling Data Report DRAFT, March 1, 2018.

8. Google Earth (Website), Historical Aerials, accessed March 2018.

9. Historical Aerials (Website), Nationwide Environmental Title Research, LLC, accessed October 2017. https://www.historicaerials.com.

10. Idriss, I.M. and Boulanger, R.W., 2008, Soil Liquefaction During Earthquakes, Earthquake Engineering Research Institute, Monograph MNO-12.

11. Ishihara, K. (1985), Stability of Natural Deposits During Earthquakes, Proc 11th International Conference on Soil Mechanics and Foundation Engineering, Vol 1, A. A. Balkema, Rotterdam, The Netherlands, 321-376.

12. Hart, E.W. and Bryant W.A., 1997. Fault Rupture Hazard in California: Alquist-Priolo earthquake fault zoning act with index to earthquake fault zone maps: California Division of Mines and Geology Special Publication 42.

13. Mononobe, N, and Matsuo M., 1929. "On the Determination of Earth Pressures during Earthquakes," Proceedings, World Engineering Congress, Vol. 9, 179-187.

14. National Cooperative Highway Research Program (NCHRP), 1978, Durability of Drainage Pipe, NCHRP Synthesis of Highway Practice Issue 50, Transportation Research Board, Washington, DC.

15. Okabe, S., 1926, General Theory of Earth Pressure, Journal of the Japanese Society of Civil Engineers, Tokyo, Japan, Vol. 12 , No. 1.

SELECTED REFERENCES (Continued)

16. Pampeyan, E.H., 1994, Geologic Map of the Montara Mountain and San Mateo 7-1/2' Quadrangles, San Mateo County, California: U.S. Geological Survey Library, Reston, Virginia, scale 1:24,000.

17. Seed, H.B. and Whitman, R.V., 1970, Design of Earth Retaining Structures for Dynamic Loads, ASCE Specialty Conference, Lateral Stresses in the Ground and Design of Earth Retaining Structures, Cornell Univ., Ithaca, New York, 103–147.

18. State of California, 2018, Seismic Hazard Zone Report for the San Mateo 7.5-Minute Quadrangle, San Mateo County.

19. United States Geological Survey and California Geological Survey, 2006, Quaternary fault and fold database for the United States, accessed October 2017.

20. Wentworth, C.M, Graham, S.E., Pike, R.J., Beukelman, G.S., Ramsey, D.W., and Barron, A.D., 1997, Summary Distribution of Slides and Earth Flows in San Mateo County; U.S. Geological Survey, Open-File Report 97-745C, scale 1:125,000.

21. Youd, T. L. and I. M. Idriss, 2001, Liquefaction Resistance of Soils: Summary Report from the ER and 1998 NCEER/NSF Workshop on Evaluation of Liquefaction Resistance of Soils.

FIGURES

BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

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1	COVER SHEET
2	REGIONAL GEOLOGIC MAP
3	SEISMIC HAZARD ZONE MAP
4	REGIONAL FAULTING AND SEISMICITY
5	EXPO CENTER - STORAGE TANK SITE
6	CROSS SECTION A-A'
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15	DIVERSION SEWER - BRANCH 3
16	DIVERSION SEWER - BRANCH 3
17	DIVERSION SEWER - BRANCH 1 - TRENCHLESS

HORIZONTAL SCALE: 1"=150'

0 150 300

SCALE IN FEET

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CITY OF SAN MATEO

EXPO CENTER STORAGE FACILITY - EAST AND DIVERSION SEWERS

GEOTECHNICAL PLAN SET

COVER SHEET

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SHEET 1 OF 17

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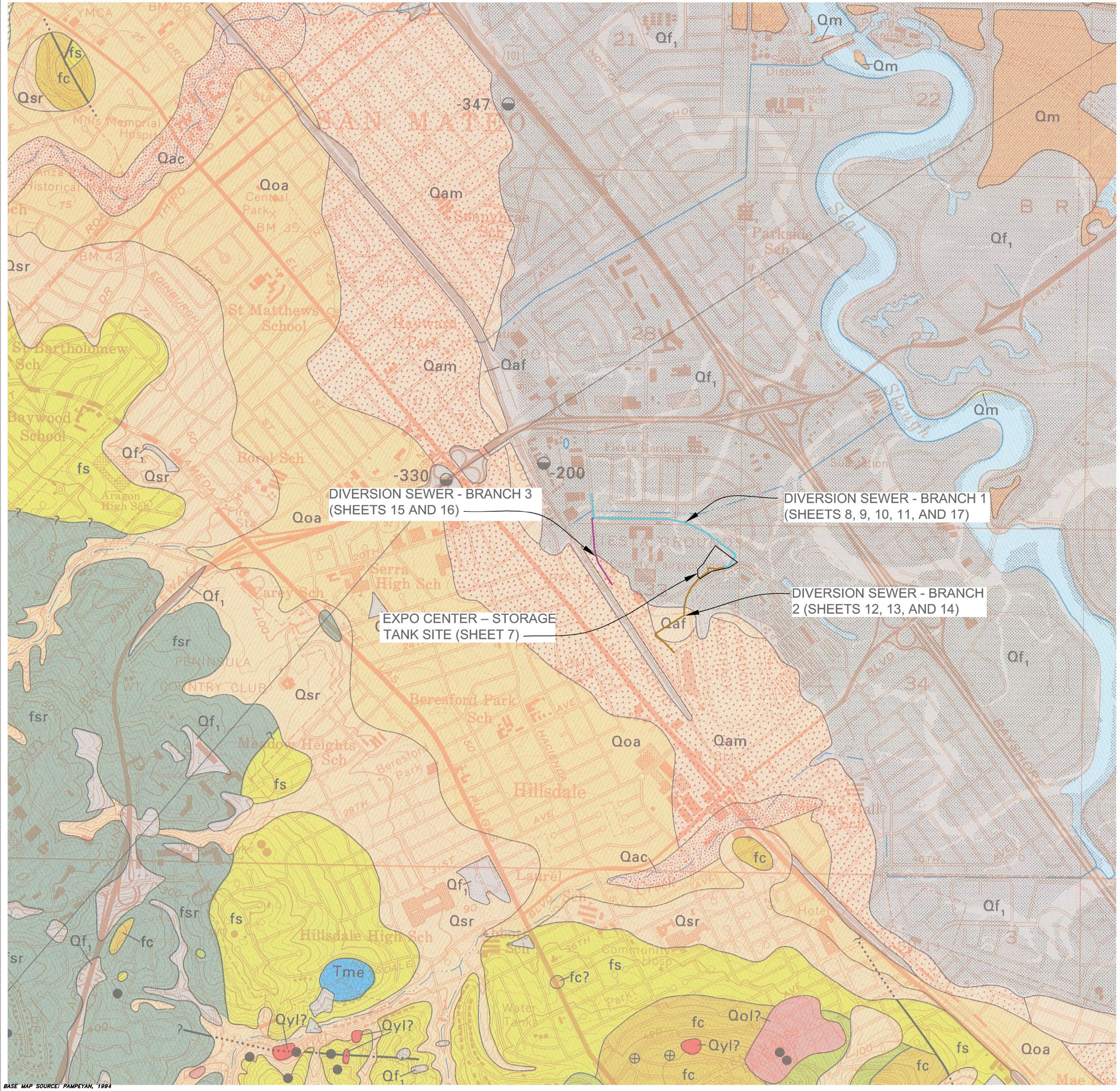

EXPLANATION

GEOLOGIC CONTACT-DASHED WHERE GRADATIONAL OR APPROXIMATELY LOCATED

Qf1	ARTIFICIAL FILL
Qyl	YOUNGER LANDSLIDE DEPOSITS (HOLOCENE)
Qm	BAY MUD (HOLOCENE)
Qsr	SLOPE WASH, RAVINE FILL AND COLLUVIUM (HOLOCENE)
Qac	COURSE-GRAINED ALLUVIUM (HOLOCENE)
Qam	MEDIUM-GRAINED ALLUVIUM (HOLOCENE)
Qaf	FINE-GRAINED ALLUVIUM (HOLOCENE)

HORIZONTAL SCALE: 1"=800'

0 800 1,600

SCALE IN FEET

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EXPO CENTER STORAGE FACILITY - EAST AND DIVERSION SEWERS

GEOTECHNICAL PLAN SET

REGIONAL GEOLOGIC MAP

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SHEET 2 OF 17

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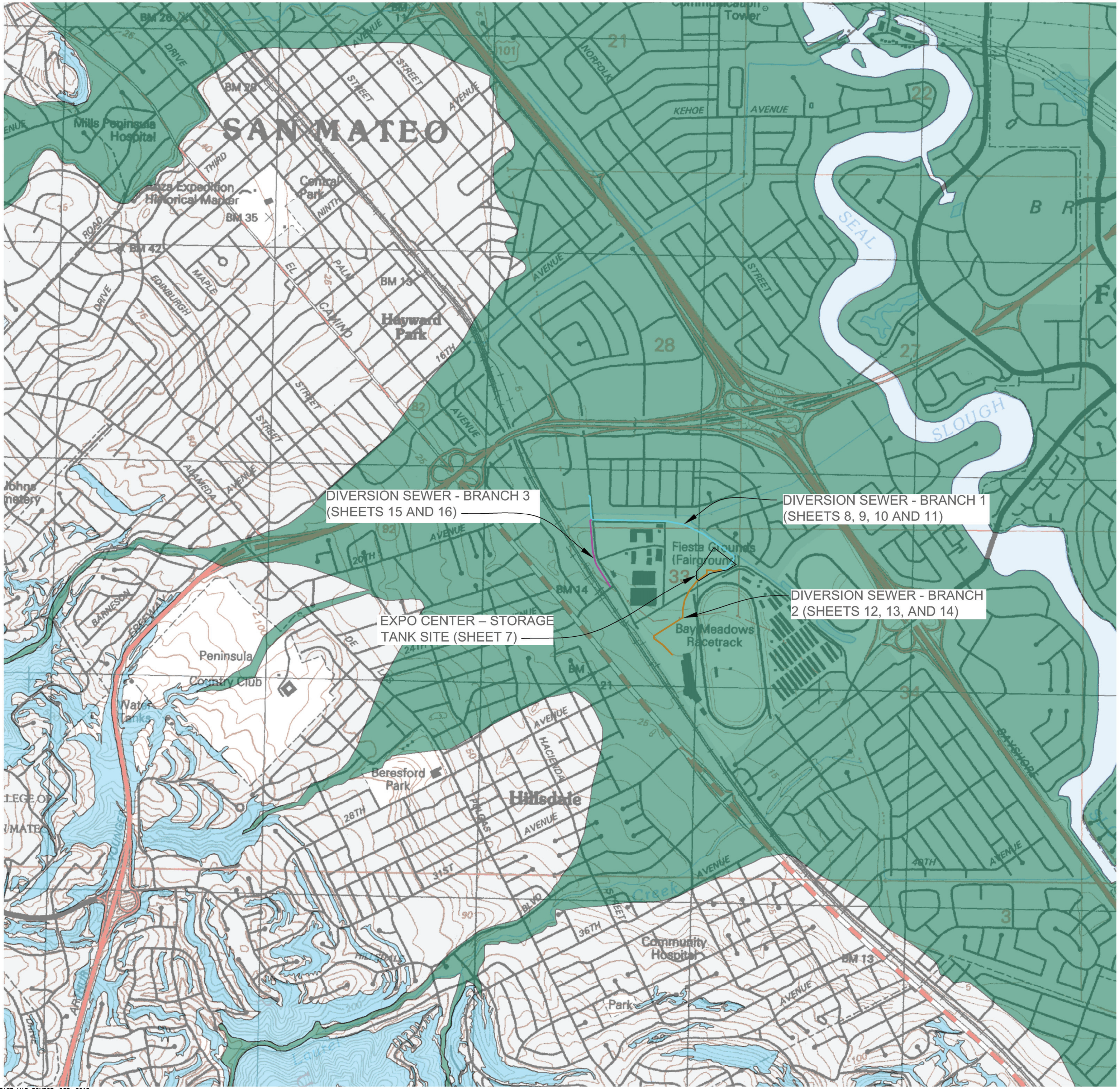

EXPLANATION

SEISMIC HAZARD ZONES

Liquefaction Zones
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslide Zones
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

HORIZONTAL SCALE: 1"=800'

0 800 1,600
SCALE IN FEET

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EXPO CENTER STORAGE FACILITY - EAST AND DIVERSION SEWERS

GEOTECHNICAL PLAN SET

SEISMIC HAZARD ZONE MAP

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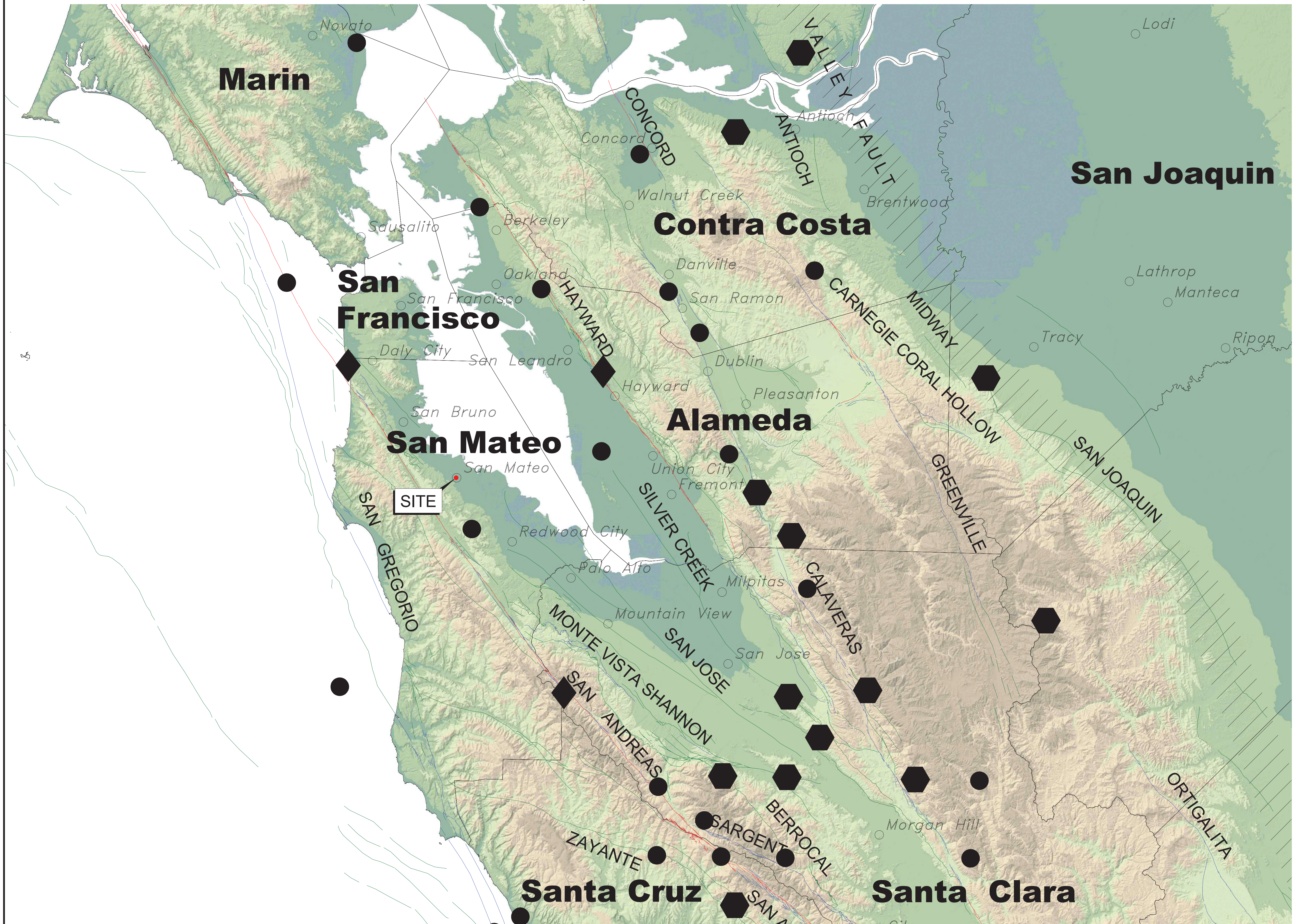

EXPLANATION

- MAGNITUDE 7+
- MAGNITUDE 6-7
- MAGNITUDE 5-6
- HISTORIC FAULT
- HOLOCENE FAULT
- QUATERNARY FAULT
- HISTORIC BLIND THRUST FAULT ZONE

HORIZONTAL SCALE: 1" = 7 MILES

0 7 14

SCALE IN MILES

BASE MAP SOURCE:
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATASET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, NOVEMBER, 2010
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800–2000)

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CITY OF SAN MATEO INCORPORATED 1894 CALIFORNIA

EXPO CENTER STORAGE FACILITY - EAST AND DIVERSION SEWERS

GEOTECHNICAL PLAN SET

REGIONAL FAULTING AND SEISMICITY

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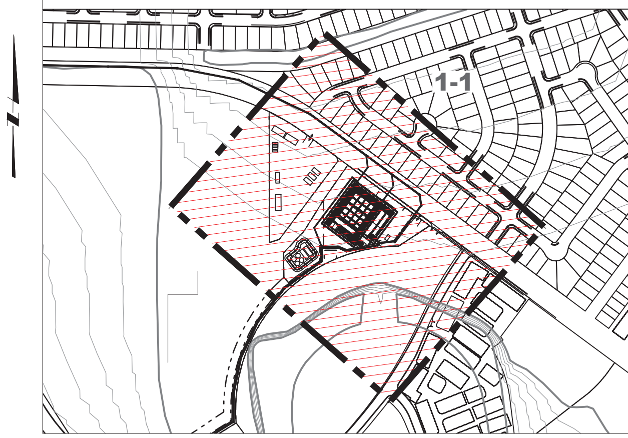
SHEET 4 OF 17

KEY PLAN



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

1-EXPO-TNK-B4 — BORING (ENGEO, 2018)

1-EXPO-TNK-CPT4 — CONE PENETRATION TEST (ENGEO, 2018)

PREVIOUS BOREHOLE OR CONE PENETRATION TEST - ADVANCED FOR BAY MEADOWS (LANGAN, 1995, 1996, 1999, 2000, 2002, 2007)

VWP — VIBRATING WIRE PIEZOMETER

SP — STANDPIPE WELL

B — B' — CROSS SECTION LOCATION

HORIZONTAL SCALE:

0 30 60

SCALE IN FEET

PLAN 1-1

STORAGE TANK SITE

REV. NO.	DESCRIPTION	BY	DATE
1			
2			
3			
4			
5			

DIRECTOR OF PUBLIC WORKS — DATE

ENGINEERING MANAGER — DATE

SENIOR ENGINEER — DATE

PROJECT ENGINEER — DATE

CITY OF SAN MATEO CALIFORNIA INCORPORATED

EXPO CENTER STORAGE FACILITY - EAST AND DIVERSION SEWERS

GEOTECHNICAL PLAN SET

TANK EAST OPTION 1 CONSTRUCTION LAYOUT

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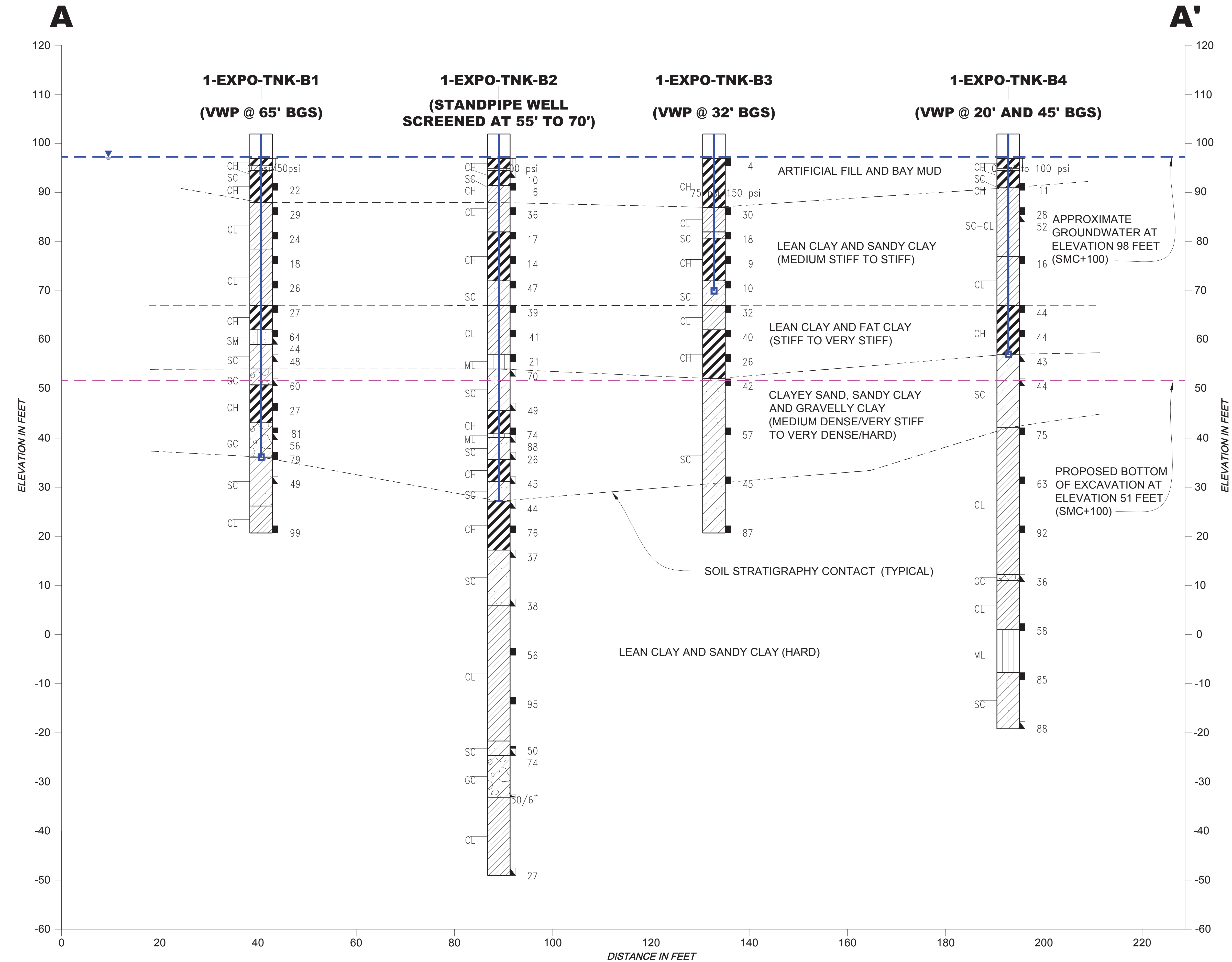

SECTION A-A'

SCALE: 1"=10'

VERTICAL SCALE: 0 10 20 SCALE IN FEET

HORIZONTAL SCALE: 0 10 20 SCALE IN FEET

ENGEO Expect Excellence

EXPO CENTER STORAGE FACILITY - EAST AND DIVERSION SEWERS

GEOTECHNICAL PLAN SET

CROSS SECTION A-A'

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SCALE: AS SHOWN

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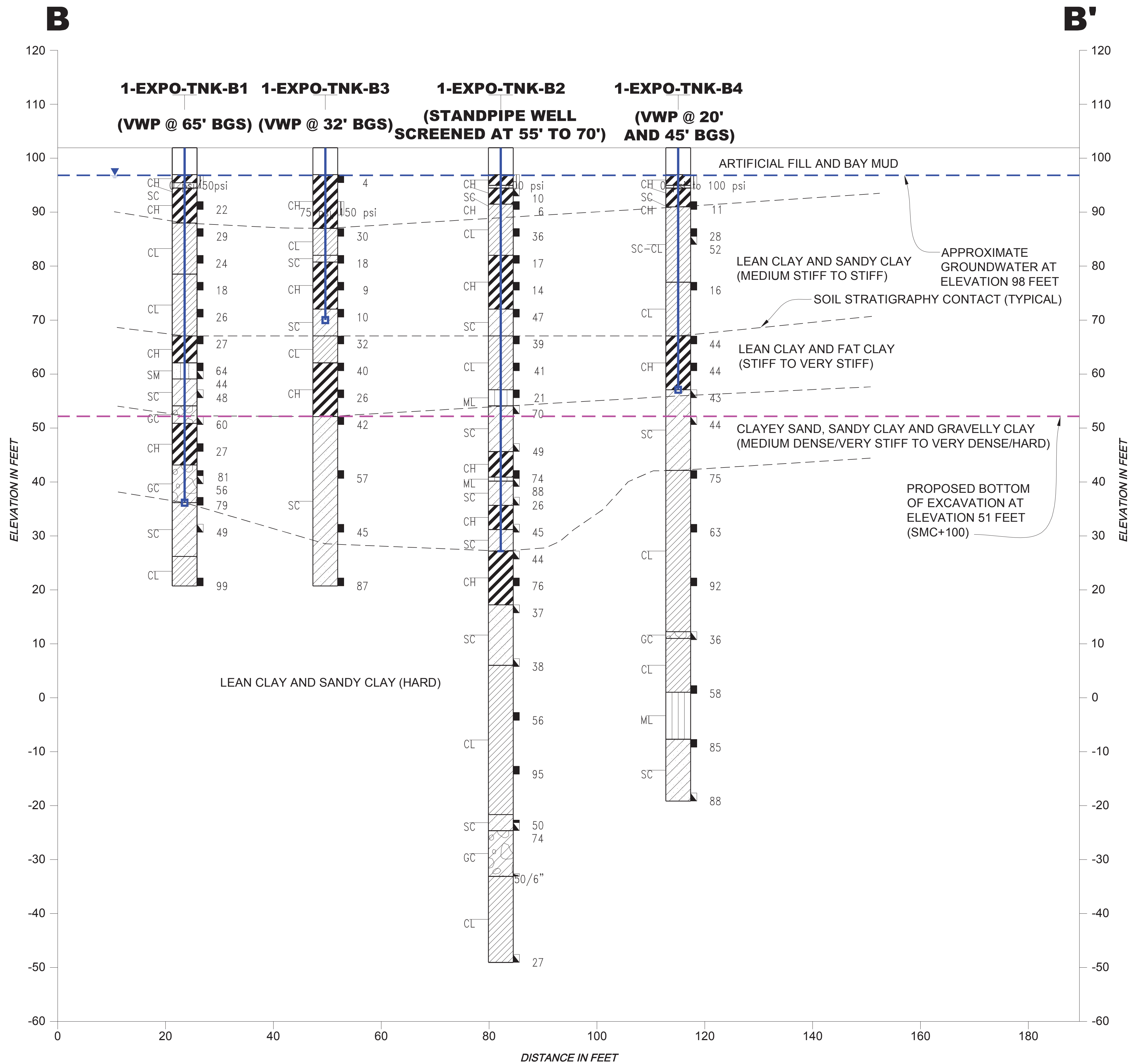

SECTION B-B'

SCALE: 1"=10'

VERTICAL SCALE: 0 10 20 SCALE IN FEET

HORIZONTAL SCALE: 0 10 20 SCALE IN FEET

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EXPO CENTER STORAGE FACILITY - EAST AND DIVERSION SEWERS

GEOTECHNICAL PLAN SET

CROSS SECTION B-B'

CITY OF SAN MATEO

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SCALE: AS SHOWN

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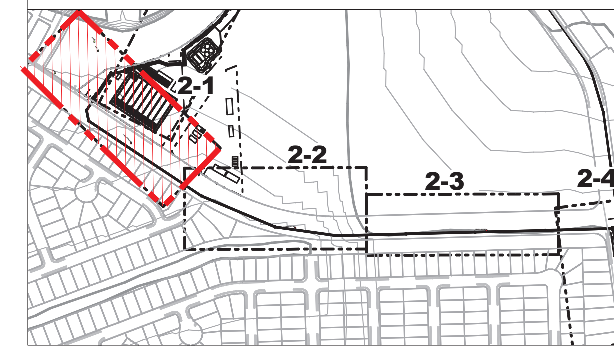
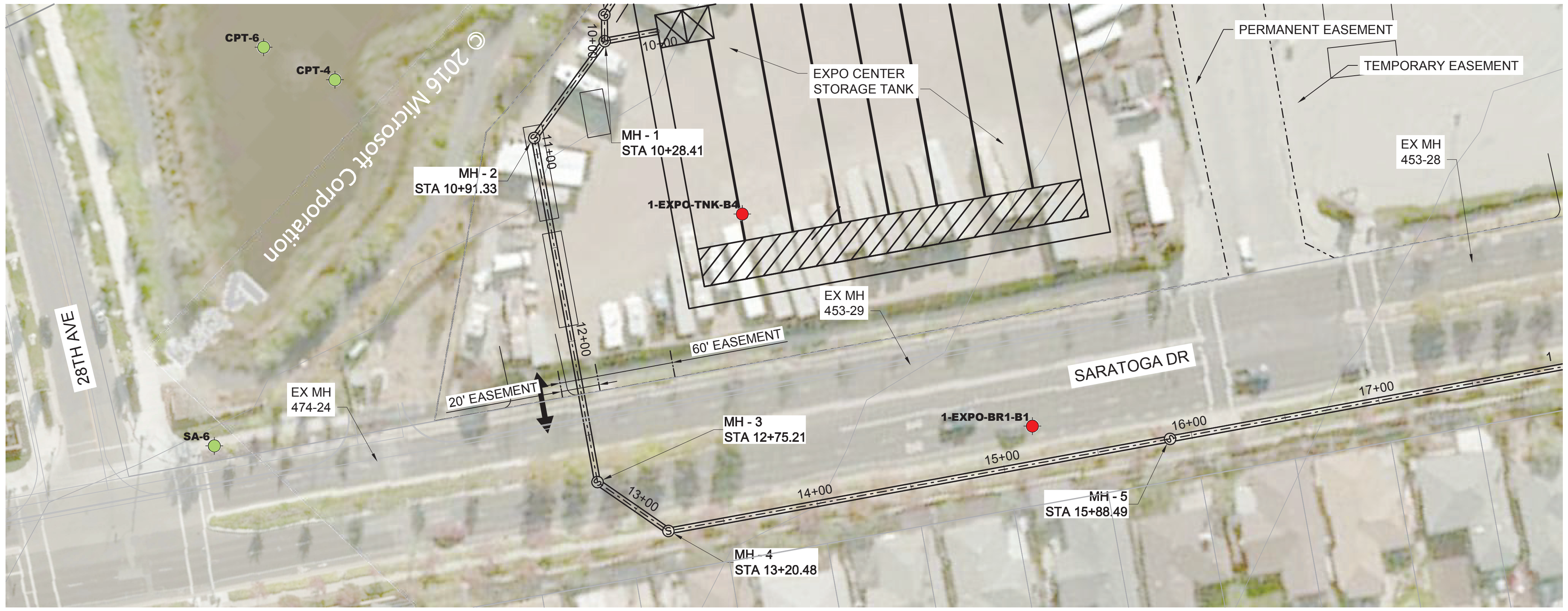




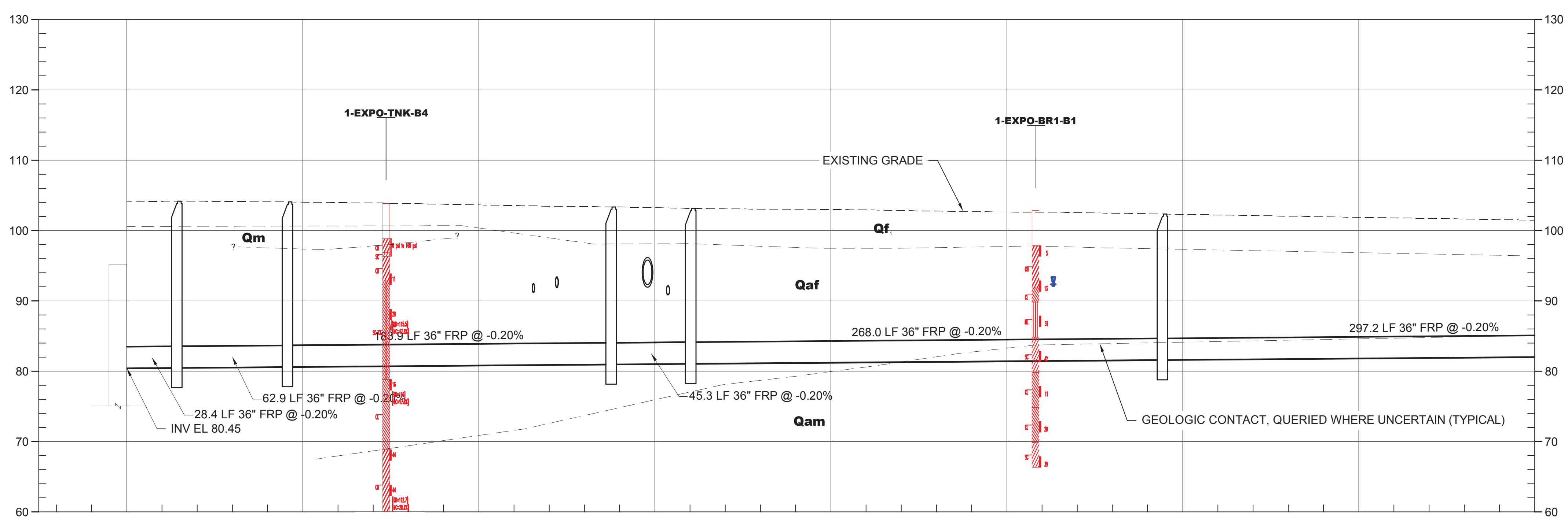
PLAN 2-1

DIVERSION SEWER BRANCH 1 - STATION 10+00 TO 18+00

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

1-EXPO-BR1-B1	BORING (ENGEO, 2018)
	PREVIOUS BOREHOLE OR CONE PENETRATION TEST - ADVANCED FOR BAY MEADOWS (LANGAN, 1995, 1996, 1999, 2000, 2002, 2007)
Qf	ARTIFICIAL FILL
Qm	BAY MUD (HOLOCENE)
Qam	MEDIUM-GRAINED ALLUVIUM (HOLOCENE)
Qaf	FINE-GRAINED ALLUVIUM (HOLOCENE)



PROFILE

SCALE: 1"=40' HORIZONTAL

SCALE: 1"=10' VERTICAL

VERTICAL SCALE: 1" = 10'

0 10 20 SCALE IN FEET

HORIZONTAL SCALE: 1" = 40'

0 40 80 SCALE IN FEET

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CITY OF SAN MATEO INCORPORATED 1894 CALIFORNIA

EXPO CENTER STORAGE FACILITY - EAST AND DIVERSION SEWERS

GEOTECHNICAL PLAN SET

BRANCH 1 PLAN AND PROFILE STA 10+00 TO 18+00

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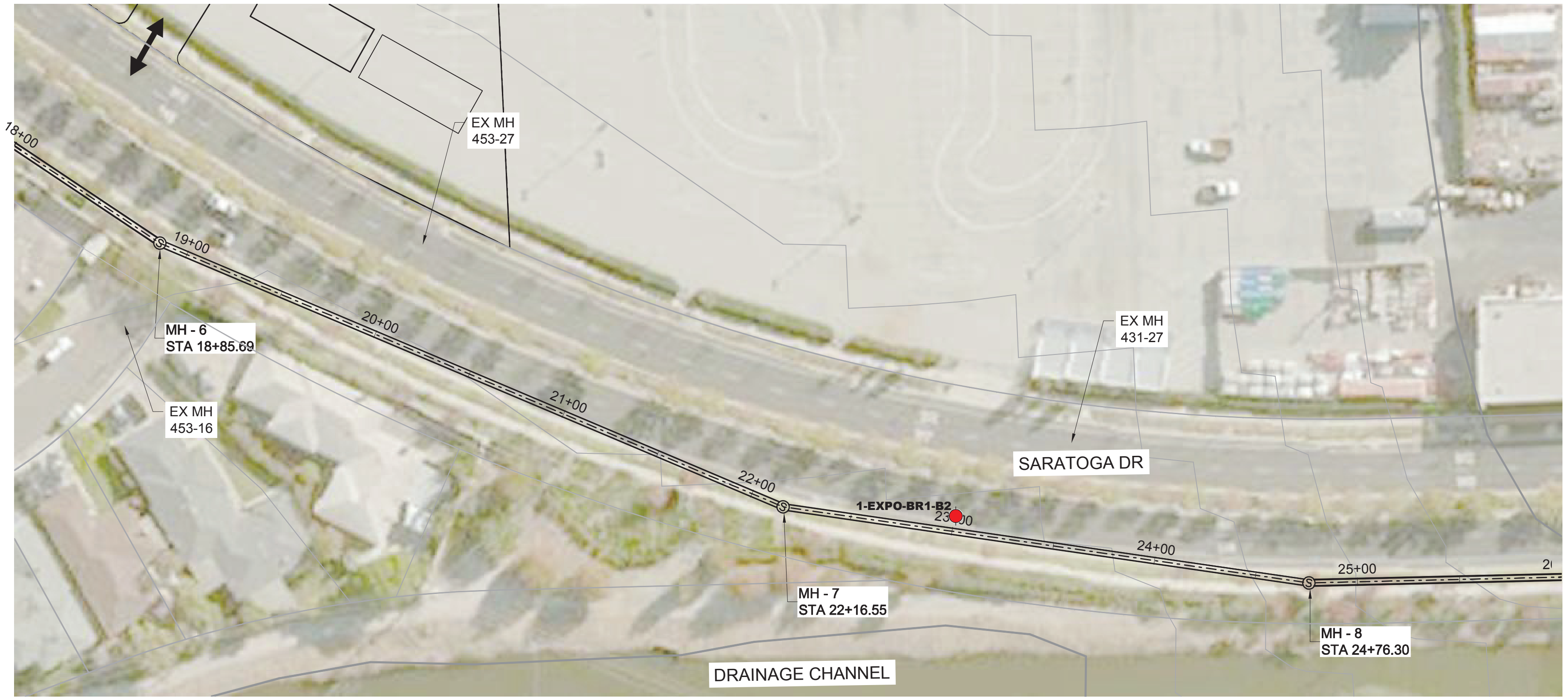
SCALE: AS SHOWN

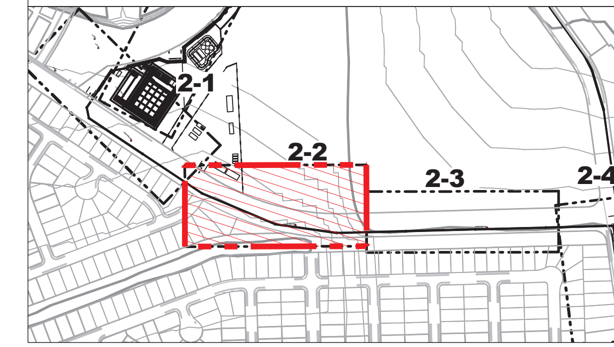
PROJECT NO: 13231.000.001

SHEET 8 OF 17



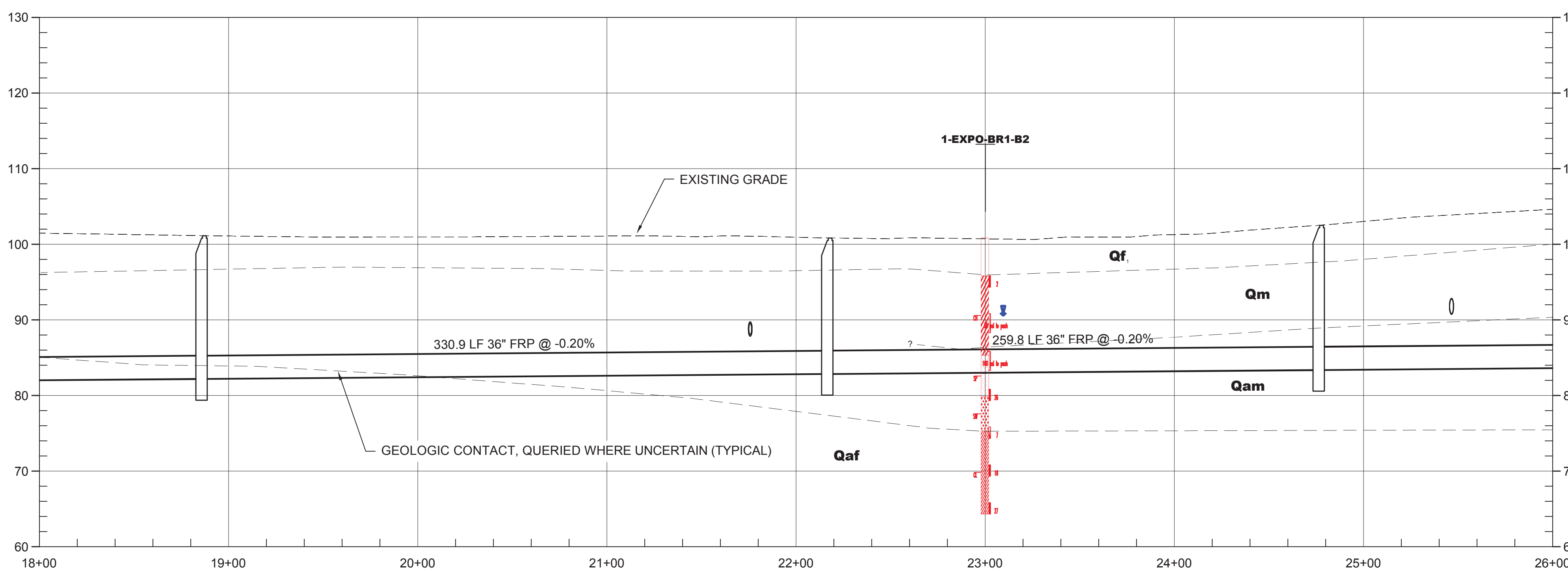
PLAN 2-2
DIVERSION SEWER BRANCH 1 STATION 18+00 TO 26+00

KEY PLAN



EXPLANATION
ALL LOCATIONS ARE APPROXIMATE

1-EXPO-BR1-B2	BORING (ENGEO, 2018)
Qf	ARTIFICIAL FILL
Qm	BAY MUD (HOLOCENE)
Qam	MEDIUM-GRAINED ALLUVIUM (HOLOCENE)
Qaf	FINE-GRAINED ALLUVIUM (HOLOCENE)



PROFILE
SCALE: 1"=40' HORIZONTAL
SCALE: 1"=10' VERTICAL

VERTICAL SCALE: 1" = 10'
0 10 20
SCALE IN FEET

HORIZONTAL SCALE: 1" = 40'
0 40 80
SCALE IN FEET

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CITY OF SAN MATEO INCORPORATED 1894 CALIFORNIA

EXPO CENTER STORAGE FACILITY - EAST AND DIVERSION SEWERS

GEOTECHNICAL PLAN SET

BRANCH 1 PLAN AND PROFILE STA 18+00 TO 26+00

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SCALE: AS SHOWN
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SHEET 9 OF 17

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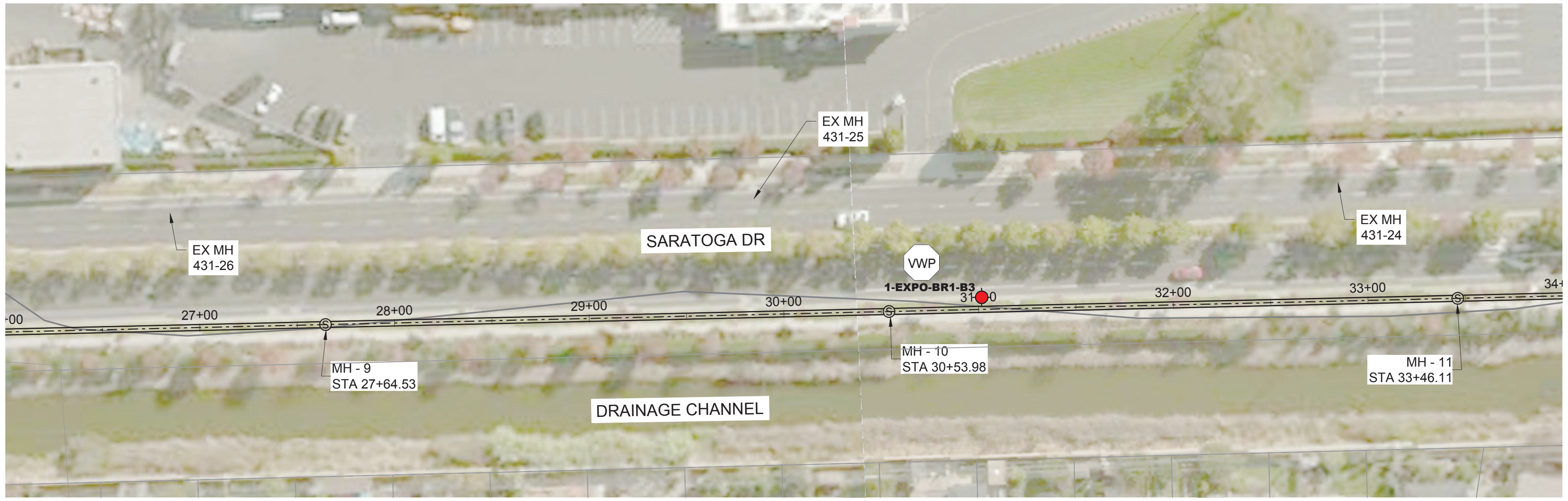
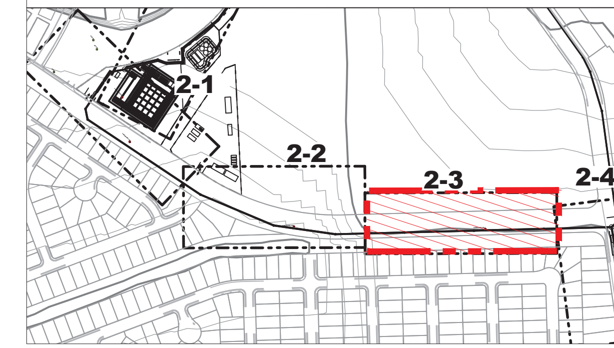


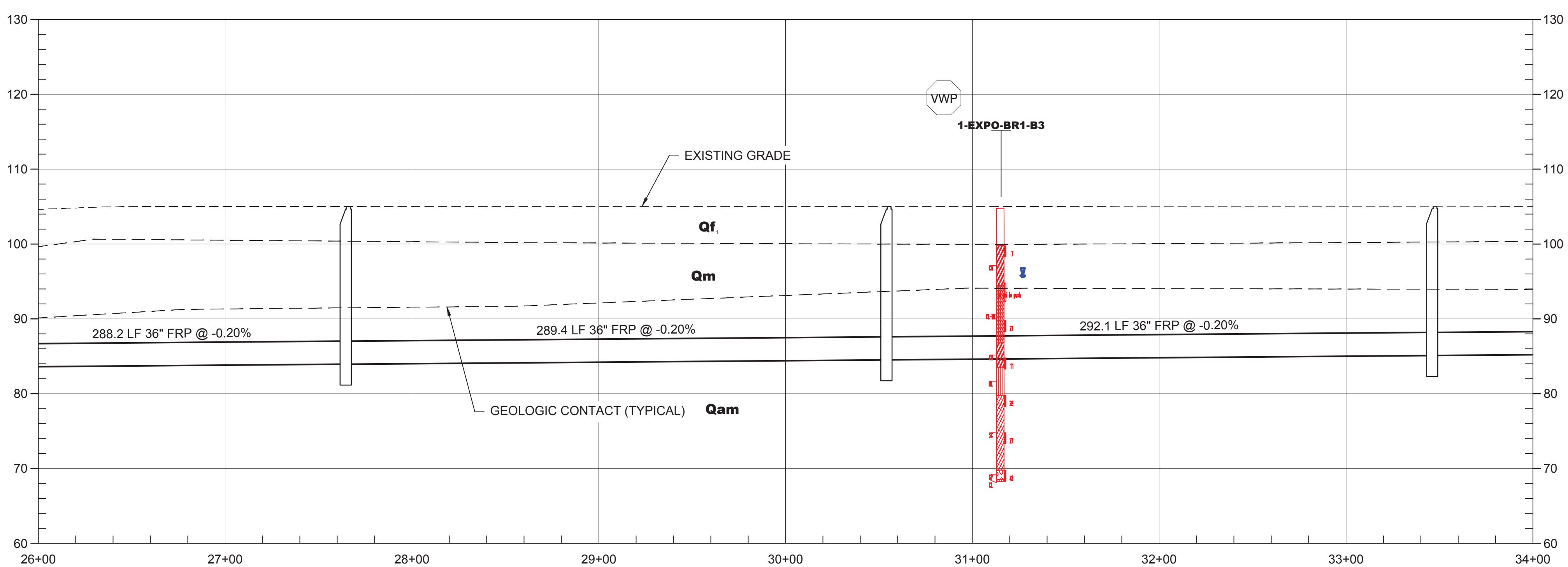
PLAN 2-3
DIVERSION SEWER BRANCH 1 STATION 26+00 TO 34+00

KEY PLAN



EXPLANATION
ALL LOCATIONS ARE APPROXIMATE

1-EXPO-BR1-B3	BORING (ENGEO, 2017)
VWP	VIBRATING WIRE PIEZOMETER
Qf	ARTIFICIAL FILL
Qm	BAY MUD (HOLOCENE)
Qam	MEDIUM-GRAINED ALLUVIUM (HOLOCENE)



PROFILE
SCALE: 1"=40' HORIZONTAL
SCALE: 1"=10' VERTICAL

VERTICAL SCALE: 1" = 10'
0 10 20
SCALE IN FEET

HORIZONTAL SCALE: 1" = 40'
0 40 80
SCALE IN FEET

ENGEO
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REV. NO. | DESCRIPTION | BY | DATE

DIRECTOR OF PUBLIC WORKS DATE
ENGINEERING MANAGER DATE
SENIOR ENGINEER DATE
PROJECT ENGINEER DATE

CITY OF SAN MATEO CALIFORNIA

EXPO CENTER STORAGE FACILITY - EAST
AND DIVERSION SEWERS

GEOTECHNICAL PLAN SET

BRANCH 1 PLAN AND PROFILE STA 26+00 TO 34+00

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CHECKED BY: S. BARUA DATE:
SCALE: AS SHOWN
PROJECT NO: 13231.000.001

SHEET 10 OF 17

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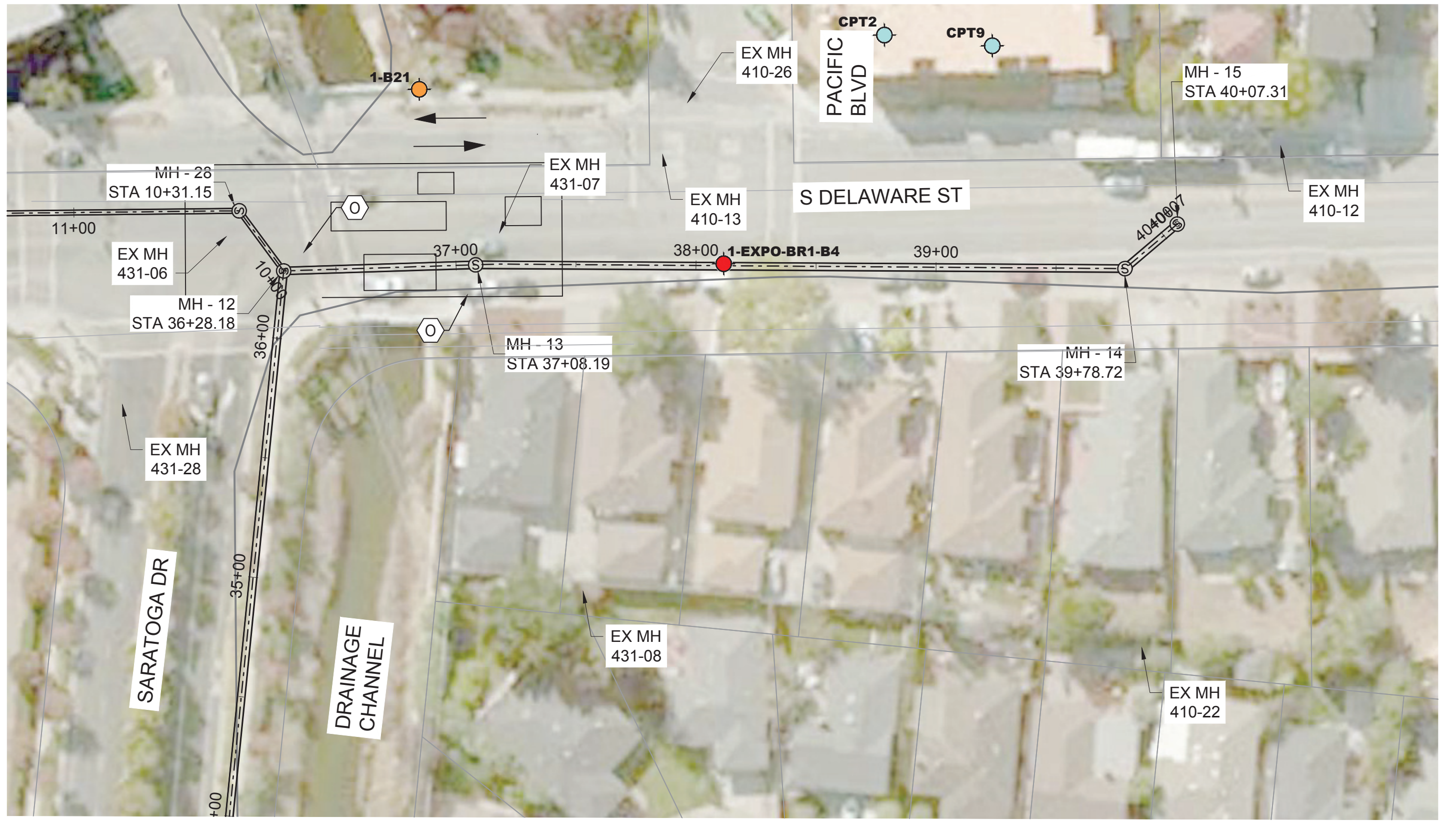
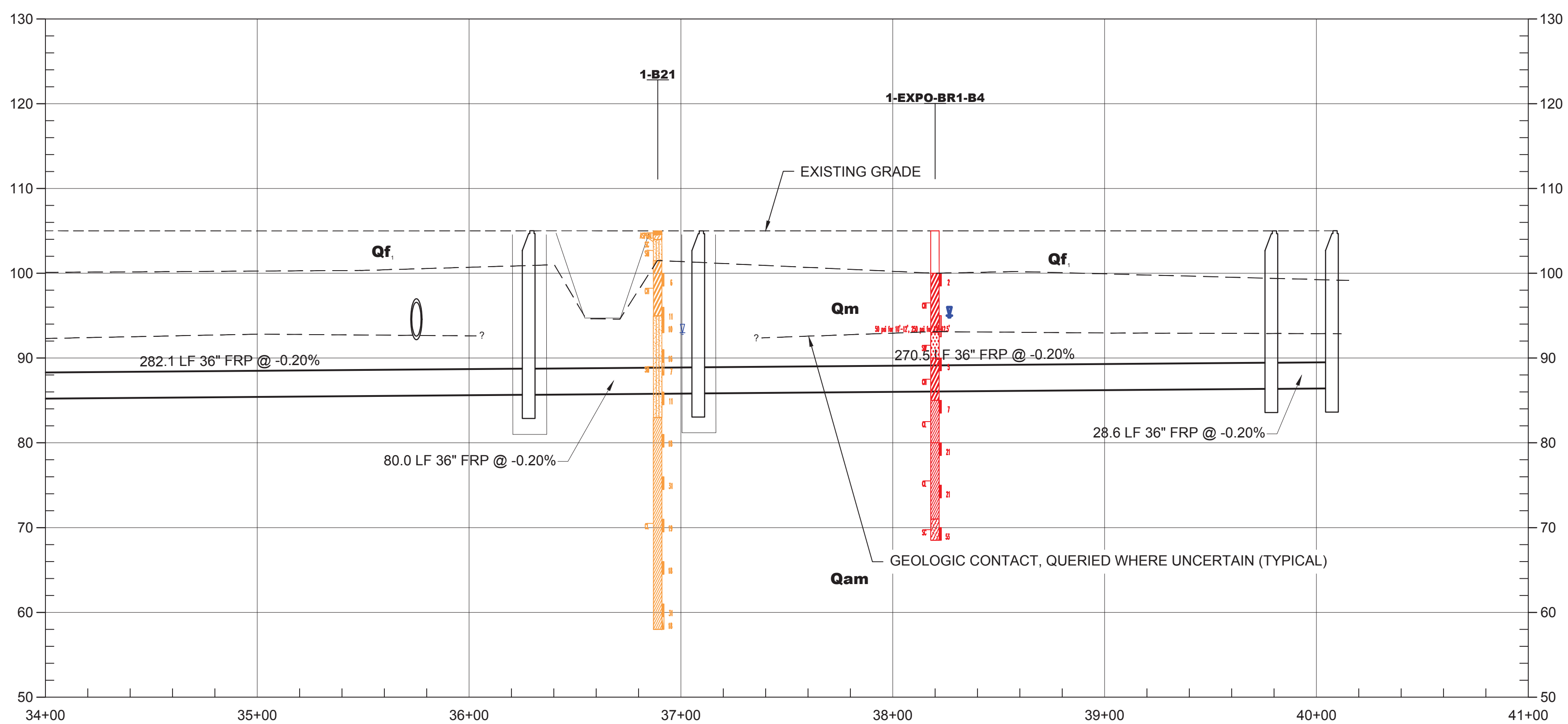


PLAN 2-4

DIVERSION SEWER BRANCH 1 STATION 34+00 TO 40+00

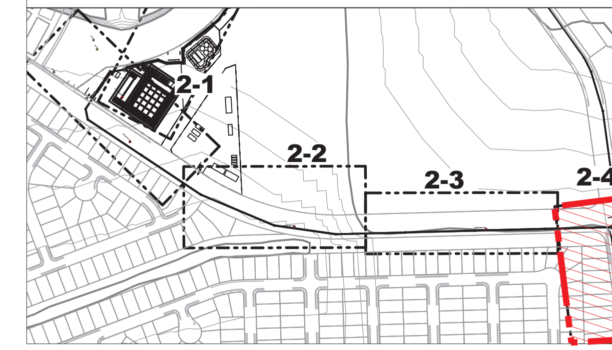


PROFILE

SCALE: 1"=40' HORIZONTAL

SCALE: 1"=10' VERTICAL

KEY PLAN



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

1-EXPO-BR1-B4	BORING (ENGEO, 2017)
1-B21	PREVIOUS BORING - ADVANCED FOR SOUTH TRUNK SANITARY SEWER RELIEF LINE (ENGEO, 2010)
CPT9	PREVIOUS CONE PENETRATION TEST - ADVANCED FOR 2090 SOUTH DELAWARE STREET (ENGEO, 2012)
Qf	ARTIFICIAL FILL
Qm	BAY MUD (HOLOCENE)
Qam	MEDIUM-GRAINED ALLUVIUM (HOLOCENE)

VERTICAL SCALE: 1" = 10'

0 10 20 SCALE IN FEET

HORIZONTAL SCALE: 1" = 40'

0 40 80 SCALE IN FEET

ENGEO

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DIRECTOR OF PUBLIC WORKS DATE

ENGINEERING MANAGER DATE

SENIOR ENGINEER DATE

PROJECT ENGINEER DATE

CITY OF SAN MATEO CALIFORNIA INCORPORATED 1894

EXPO CENTER STORAGE FACILITY - EAST AND DIVERSION SEWERS

GEOTECHNICAL PLAN SET

BRANCH 1 PLAN AND PROFILE STA 34+00 TO 40+00

SURVEYED BY: DATE:

BOOK NO: DATE:

DRAWN BY: G. JAFFE DATE:

CHECKED BY: S. BARUA DATE:

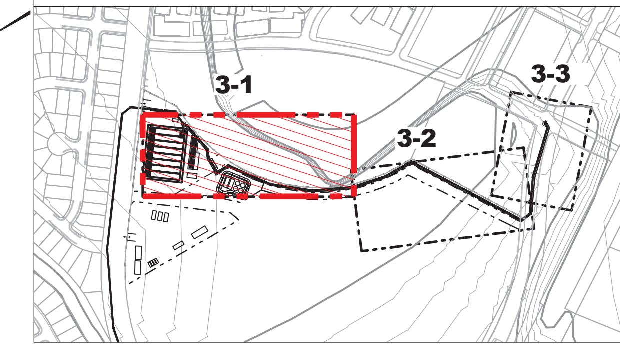
SCALE: AS SHOWN

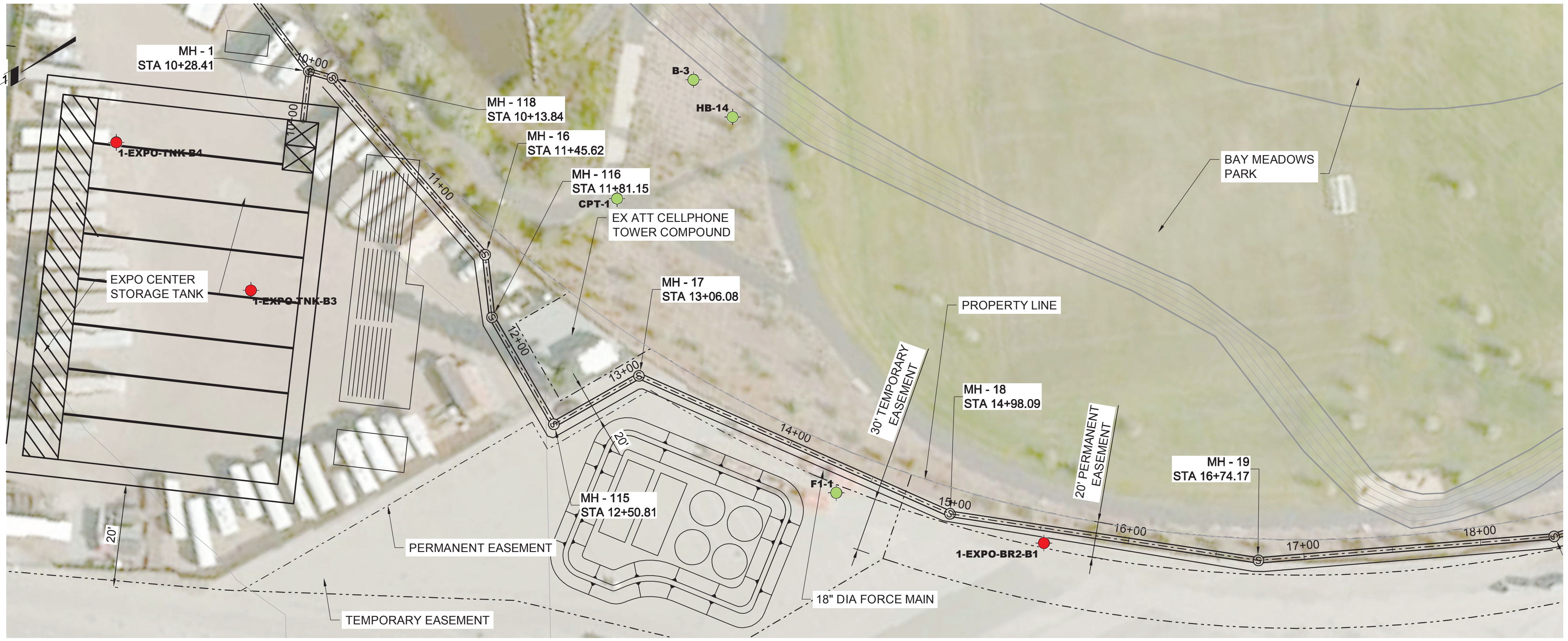
PROJECT NO: 13231.000.001

SHEET 11 OF 17

DWG FILE: G:\Active Projects_12000 To 13999\13231\13231000001\DRAFTING\CAD Files\EXPO Tank And Diversion Sewers\ENGEO-SM-2-2-C-EXPO_EAST-0118.dwg

KEY PLAN





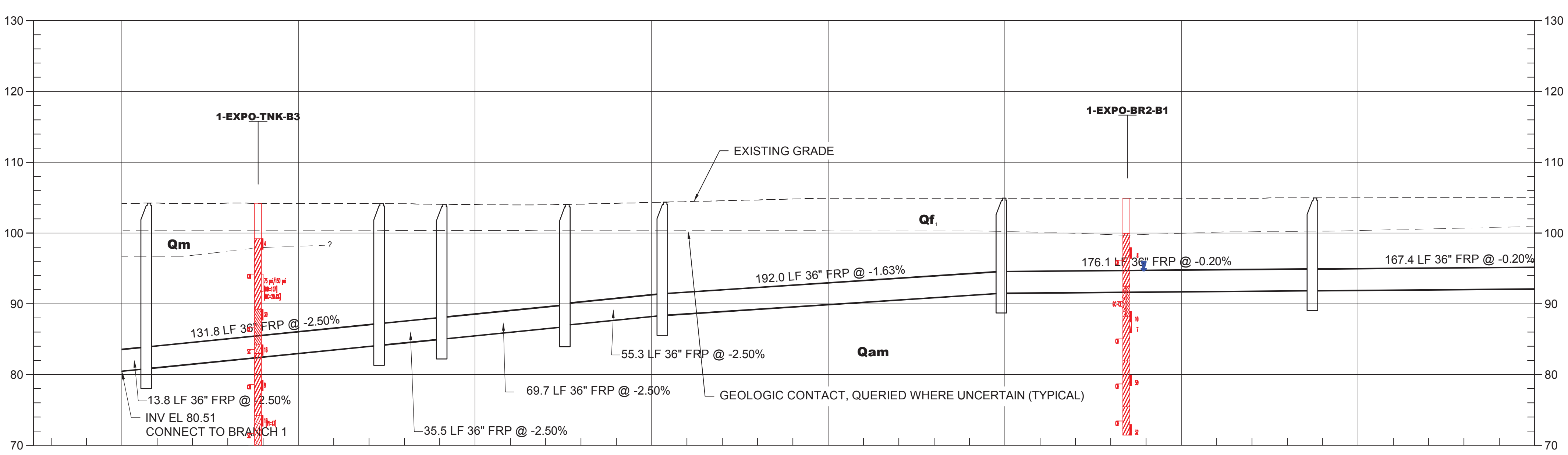
EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

1-EXPO-BR2-B1	BORING (ENGEO, 2018)
	PREVIOUS BOREHOLE OR CONE PENETRATION TEST - ADVANCED FOR BAY MEADOWS (LANGAN, 1995, 1996, 1999, 2000, 2002, 2007)
Qf	ARTIFICIAL FILL
Qm	BAY MUD (HOLOCENE)
Qam	MEDIUM-GRAINED ALLUVIUM (HOLOCENE)

PLAN 3-1

DIVERSION SEWER BRANCH 2 STATION 10+00 TO 19+00



PROFILE

SCALE: 1"=40' HORIZONTAL

SCALE: 1"=10' VERTICAL

VERTICAL SCALE: 1" = 10'

0 10 20

SCALE IN FEET

HORIZONTAL SCALE: 1" = 40'

0 40 80

SCALE IN FEET

ENGEO

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BY: GARY JAFFE

DWG FILE: G:\Active Projects_12000 To 13999\13231\132310000001\DRAFTING\CAD Files\EXPO Tank And Diversion Sewers\ENGEO-SM-2-3-C-EXPO_EAST-0118.dwg

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DIRECTOR OF PUBLIC WORKS DATE

ENGINEERING MANAGER DATE

SENIOR ENGINEER DATE

PROJECT ENGINEER DATE

CITY OF SAN MATEO INCORPORATED

EXPO CENTER STORAGE FACILITY - EAST AND DIVERSION SEWERS

GEOTECHNICAL PLAN SET

BRANCH 2 PLAN AND PROFILE STA 10+00 TO 19+00

SURVEYED BY: DATE:

BOOK NO: DATE:

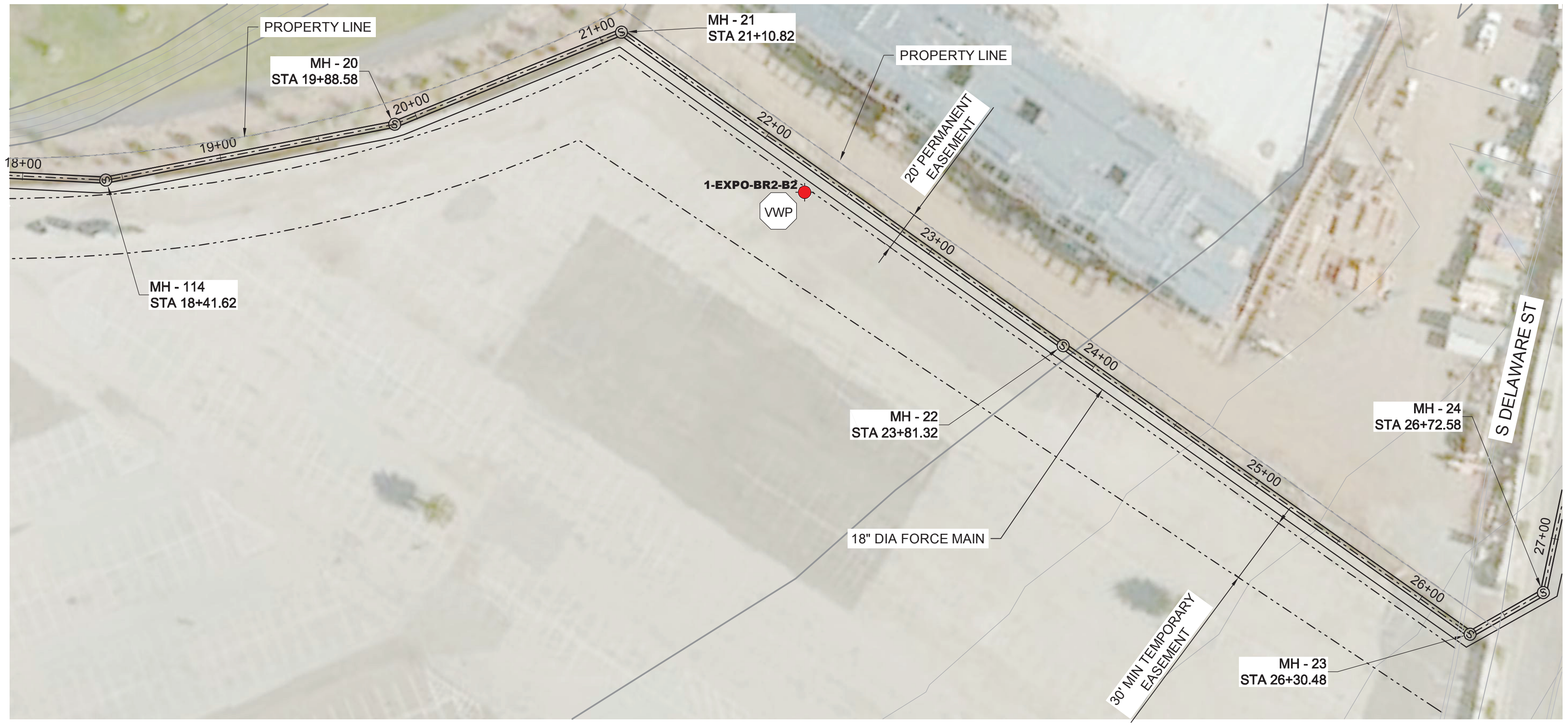
DRAWN BY: G. JAFFE DATE:

CHECKED BY: S. BARUA DATE:

SCALE: AS SHOWN

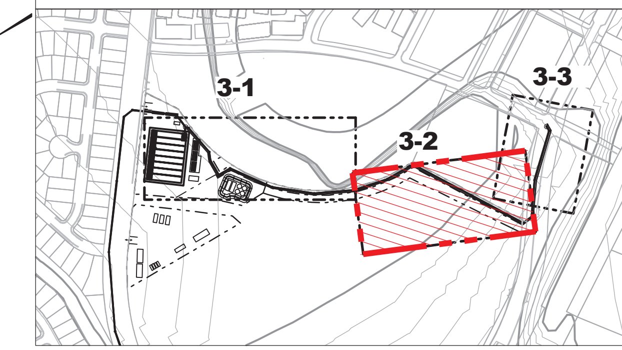
PROJECT NO: 13231.000.001

SHEET 12 OF 17



PLAN 3-2

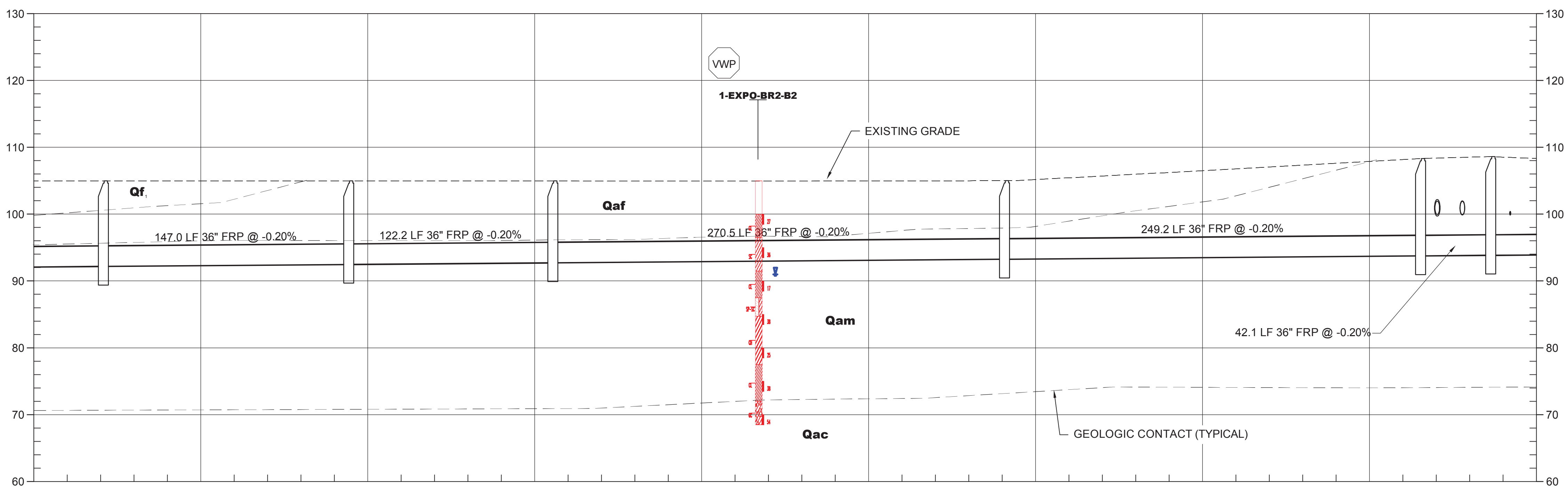
DIVERSION SEWER BRANCH 2 STATION 18+00 TO 27+00



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

1-EXPO-BR2-B2	BORING (ENGEO, 2018)
VWP	VIBRATING WIRE PIEZOMETER
Qf	ARTIFICIAL FILL
Qac	COARSE-GRAINED ALLUVIUM (HOLOCENE)
Qam	MEDIUM-GRAINED ALLUVIUM (HOLOCENE)
Qaf	FINE-GRAINED ALLUVIUM (HOLOCENE)



PROFILE

SCALE: 1"=40' HORIZONTAL

SCALE: 1"=10' VERTICAL

VERTICAL SCALE: 1" = 10'
0 10 20
SCALE IN FEET

HORIZONTAL SCALE: 1" = 40'
0 40 80
SCALE IN FEET

ENGEO
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BY: GARY JAFFE

DWG FILE: G:\Active Projects_12000 To 13999\13231\13231000001\DRAFTING\CAD Files\EXPO Tank And Diversion Sewers\ENGEO-SM-2-3-C-EXPO_EAST-0118.dwg

REV. NO. | BY | DATE | DESCRIPTION

DIRECTOR OF PUBLIC WORKS DATE
ENGINEERING MANAGER DATE
SENIOR ENGINEER DATE
PROJECT ENGINEER DATE

CITY OF SAN MATEO INCORPORATED CALIFORNIA

EXPO CENTER STORAGE FACILITY - EAST AND DIVERSION SEWERS

GEOTECHNICAL PLAN SET

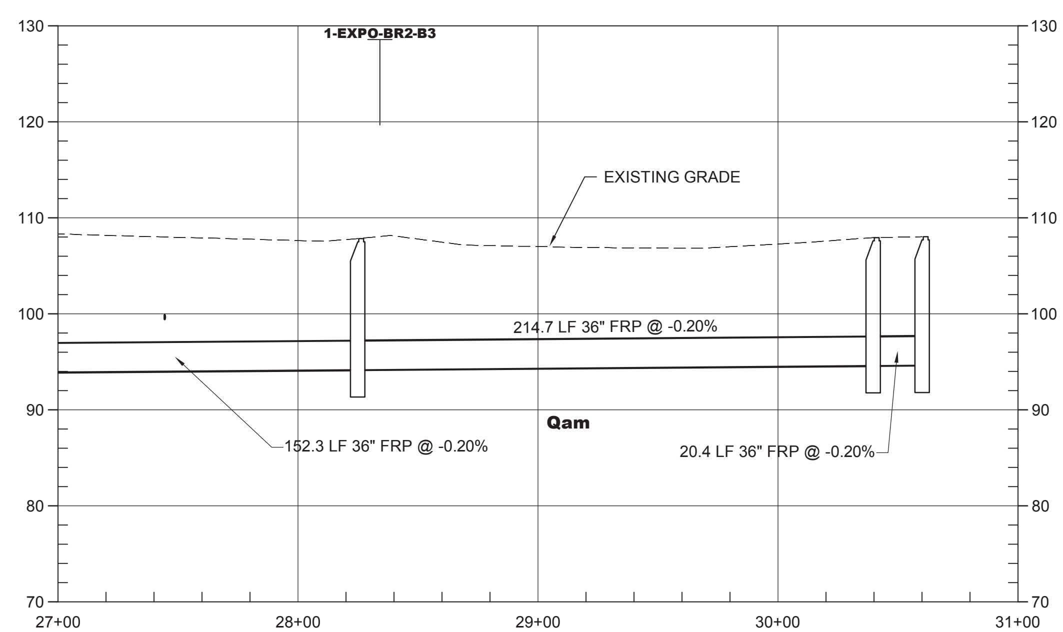
BRANCH 2 PLAN AND PROFILE STA 18+00 TO 27+00

SURVEYED BY: DATE:
BOOK NO: DATE:
DRAWN BY: G. JAFFE DATE:
CHECKED BY: S. BARUA DATE:
SCALE: AS SHOWN
PROJECT NO: 13231.000.001

SHEET 13 OF 17

PLAN 3-3

DIVERSION SEWER BRANCH 2 STATION 27+00 TO 31+00

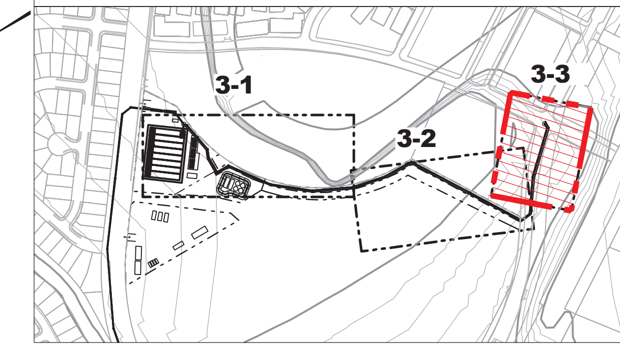


PROFILE

SCALE: 1"=40' HORIZONTAL

SCALE: 1"=10' VERTICAL

KEY PLAN



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

1-EXPO-BR2-B1 — BORING (ENGEO, 2018)

PREVIOUS BOREHOLE OR CONE PENETRATION TEST - ADVANCED FOR BAY MEADOWS (LANGAN, 1995, 1996, 1999, 2000, 2002, 2007)

Qam — MEDIUM-GRAINED ALLUVIUM (HOLOCENE)

VERTICAL SCALE: 1" = 10'

0 10 20

SCALE IN FEET

HORIZONTAL SCALE: 1" = 40'

0 40 80

SCALE IN FEET

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GEOTECHNICAL PLAN SET

BRANCH 2 PLAN AND PROFILE STA 27+00 TO 31+00

CITY OF SAN MATEO

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DIRECTOR OF PUBLIC WORKS DATE

ENGINEERING MANAGER DATE

SENIOR ENGINEER DATE

PROJECT ENGINEER DATE

SURVEYED BY: DATE:

BOOK NO: DATE:

DRAWN BY: G. JAFFE DATE:

CHECKED BY: S. BARUA DATE:

SCALE: AS SHOWN

PROJECT NO: 13231.000.001

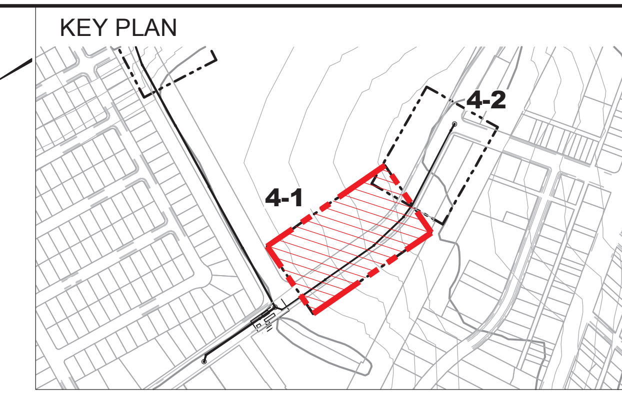
SHEET 14 OF 17

BY: GARY JAFFE

DWG FILE: G:\Active Projects_12000 To 13999\13231\132310000001\DRAFTING\CAD Files\EXPO Tank And Diversion Sewers\ENGEO-SM-2-3-C-EXPO_EAST-0118.dwg

PLAN 4-1

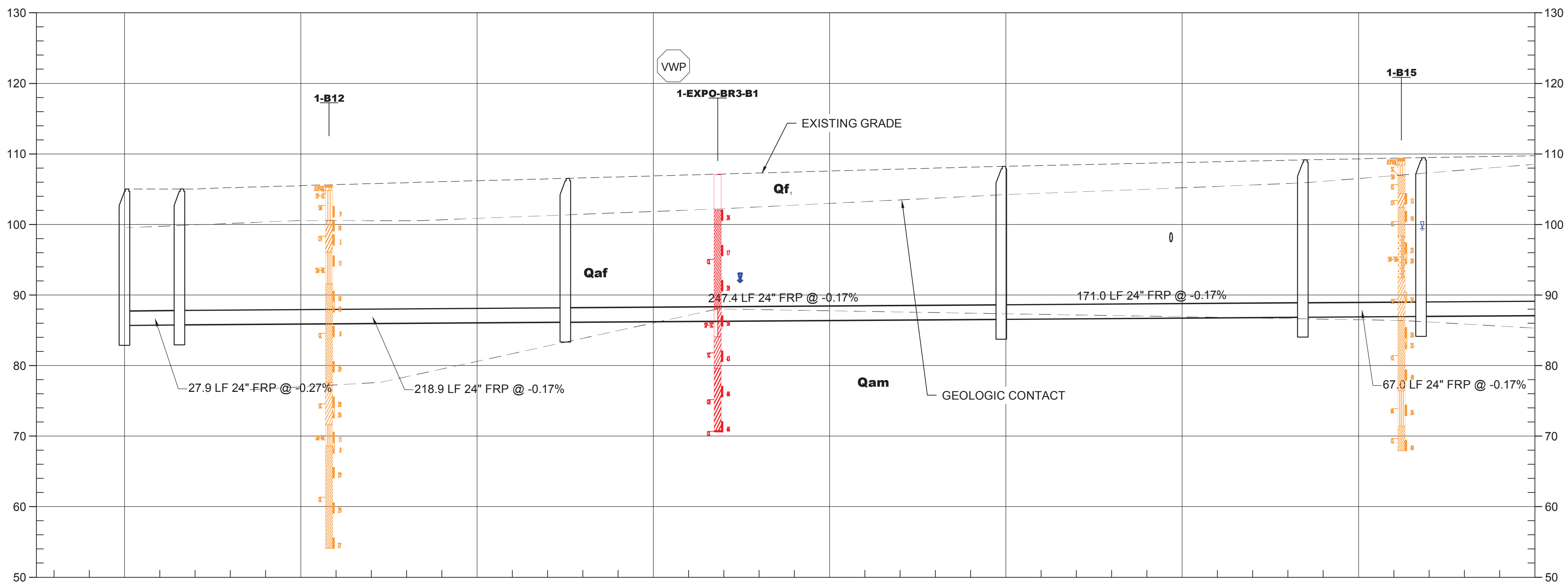
DIVERSION SEWER BRANCH 3 STATION 10+00 TO 18+00



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

1-EXPO-BR3-B1	BORING (ENGEO, 2017)
	PREVIOUS BORING OR CONE PENETRATION TEST - ADVANCED FOR SOUTH TRUNK SANITARY SEWER RELIEF LINE (ENGEO, 2010)
VWP	VIBRATING WIRE PIEZOMETER
Qf_t	ARTIFICIAL FILL
Qam	MEDIUM-GRAINED ALLUVIUM (HOLOCENE)
Qaf	FINE-GRAINED ALLUVIUM (HOLOCENE)



PROFILE

SCALE: 1"=40' HORIZONTAL

SCALE: 1"=10' VERTICAL

VERTICAL SCALE: 1" = 10'

0 10 20

SCALE IN FEET

HORIZONTAL SCALE: 1" = 40'

0 40 80

SCALE IN FEET

BY: GARY JAFFE

DWG FILE: G:\Active Projects_12000 To 13999\13231\13231000001\DRAFTING\CAD Files\EXPO Tank And Diversion Sewers\ENGEO-SM-2-4-C-EXPO_EAST-0118.dwg

REV. NO. | DESCRIPTION | BY | DATE

DIRECTOR OF PUBLIC WORKS DATE

ENGINEERING MANAGER DATE

SENIOR ENGINEER DATE

PROJECT ENGINEER DATE

CITY OF SAN MATEO INCORPORATED CALIFORNIA

EXPO CENTER STORAGE FACILITY - EAST AND DIVERSION SEWERS

GEOTECHNICAL PLAN SET

BRANCH 3 PLAN AND PROFILE STA 10+00 TO 18+00

SURVEYED BY: DATE:

BOOK NO: DATE:

DRAWN BY: G. JAFFE DATE:

CHECKED BY: S. BARUA DATE:

SCALE: AS SHOWN

PROJECT NO:

13231.000.001

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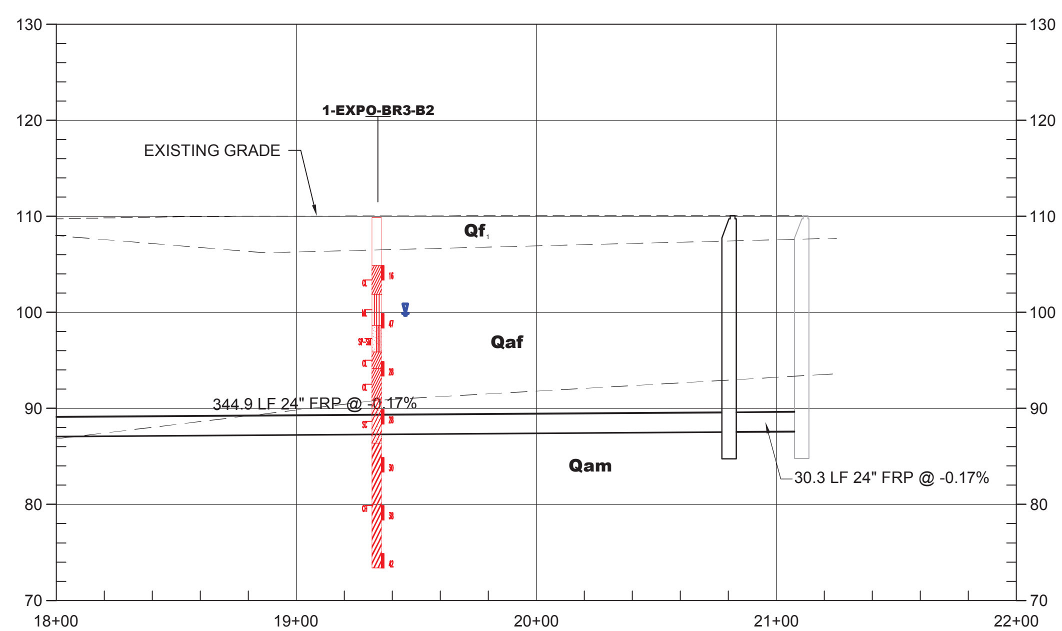
SHEET 15 OF 17

BY: GARY JAFFE

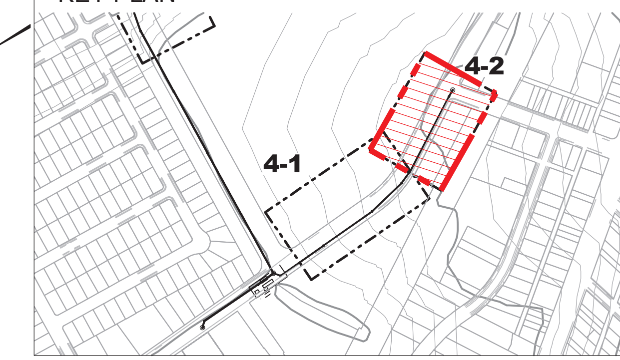
DWG FILE: G:\Active Projects_12000 To 13999\13231\13231000001\DRAFTING\CAD Files\EXPO Tank And Diversion Sewers\ENGEO-SM-2-4-C-EXPO_EAST-0118.dwg

PLAN 4-2
DIVERSION SEWER BRANCH 3 STATION 18+00 TO 22+00



PROFILE
SCALE: 1"=40' HORIZONTAL
SCALE: 1"=10' VERTICAL

KEY PLAN



EXPLANATION
ALL LOCATIONS ARE APPROXIMATE

1-EXPO-BR3-B2	BORING (ENGEO, 2017)
Qf	ARTIFICIAL FILL
Qam	MEDIUM-GRAINED ALLUVIUM (HOLOCENE)
Qaf	FINE-GRAINED ALLUVIUM (HOLOCENE)

VERTICAL SCALE: 1" = 10'
0 10 20
SCALE IN FEET

HORIZONTAL SCALE: 1" = 40'
0 40 80
SCALE IN FEET

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PROJECT ENGINEER DATE

CITY OF SAN MATEO

EXPO CENTER STORAGE FACILITY - EAST AND DIVERSION SEWERS

GEOTECHNICAL PLAN SET

BRANCH 3 PLAN AND PROFILE STA 18+00 TO 22+00

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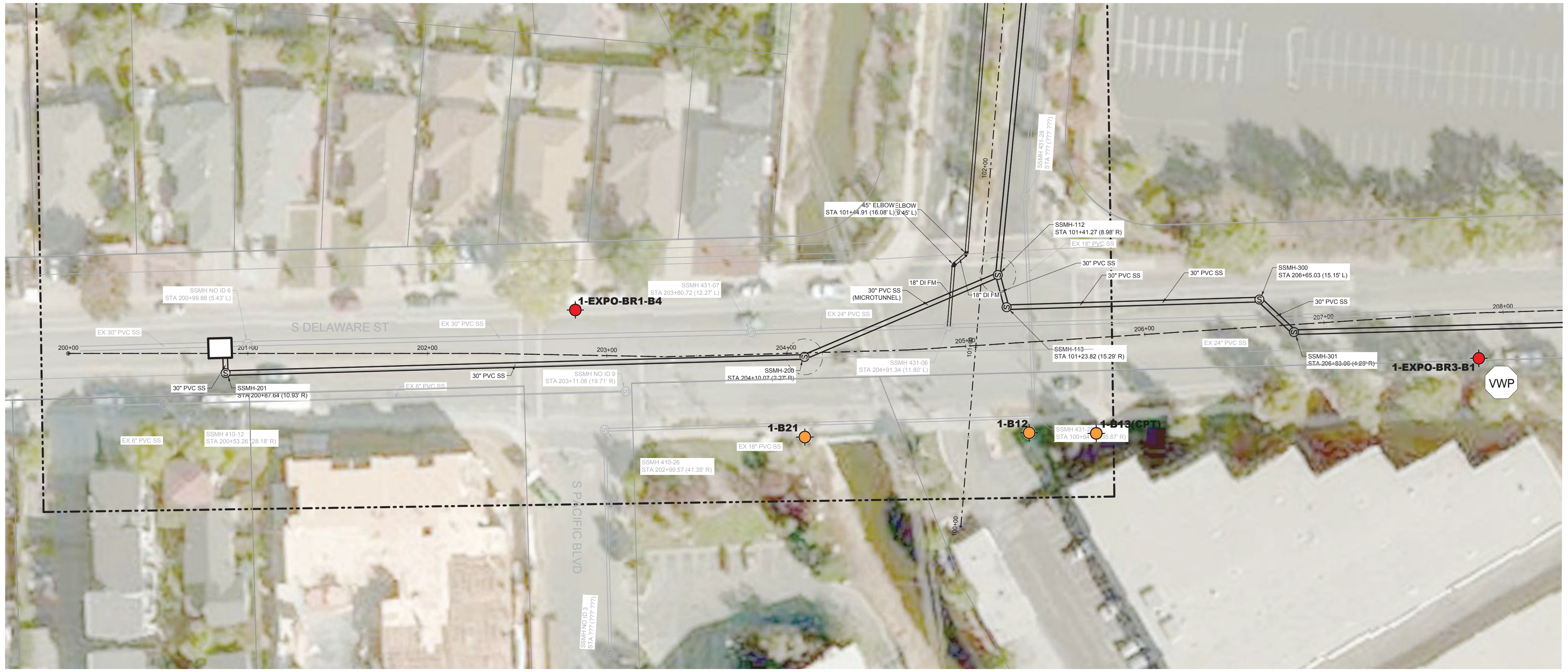
BOOK NO: DATE:
DRAWN BY: G. JAFFE DATE:
CHECKED BY: S. BARUA DATE:
SCALE: AS SHOWN
PROJECT NO: 13231.000.001

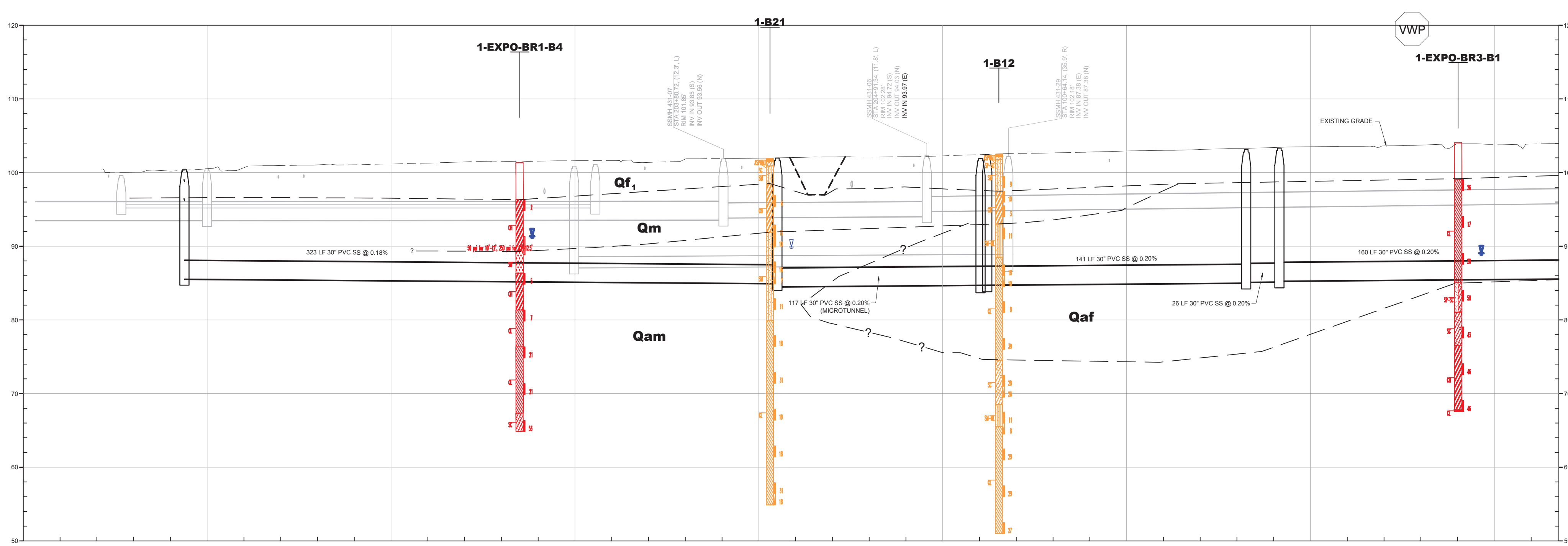
SHEET 16 OF 17

BY: STEPHEN PATTERSON



PLAN 5-1

DIVERSION SEWER BRANCH 1 STATION 200+00 TO 208+35



PROFILE

SCALE: 1"=40' HORIZONTAL

SCALE: 1"=10' VERTICAL

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

1-EXPO-BR1-B4	BORING (ENGEO, 2017)
1-B21	PREVIOUS BORING - ADVANCED FOR SOUTH TRUNK SANITARY SEWER RELIEF LINE (ENGEO, 2010)
VWP	VIBRATING WIRE PIEZOMETER
Qf_1	ARTIFICIAL FILL
Qm	BAY MUD (HOLOCENE)
Qam	MEDIUM-GRAINED ALLUVIUM (HOLOCENE)
Qaf	FINE-GRAINED ALLUVIUM (HOLOCENE)

VERTICAL SCALE: 1" = 10'

0 10 20 SCALE IN FEET

HORIZONTAL SCALE: 1" = 40'

0 40 80 SCALE IN FEET

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EXPO CENTER STORAGE FACILITY - EAST AND DIVERSION SEWERS

GEOTECHNICAL PLAN SET

BRANCH 1 TRENCHLESS PLAN AND PROFILE STA 200+00 TO 208+35

CITY OF SAN MATEO CALIFORNIA

SURVEYED BY:	DATE:
BOOK NO:	DATE:
DRAWN BY: G. JAFFE	DATE:
CHECKED BY: S. BARUA	DATE:
SCALE: AS SHOWN	
PROJECT NO:	

13231.000.001

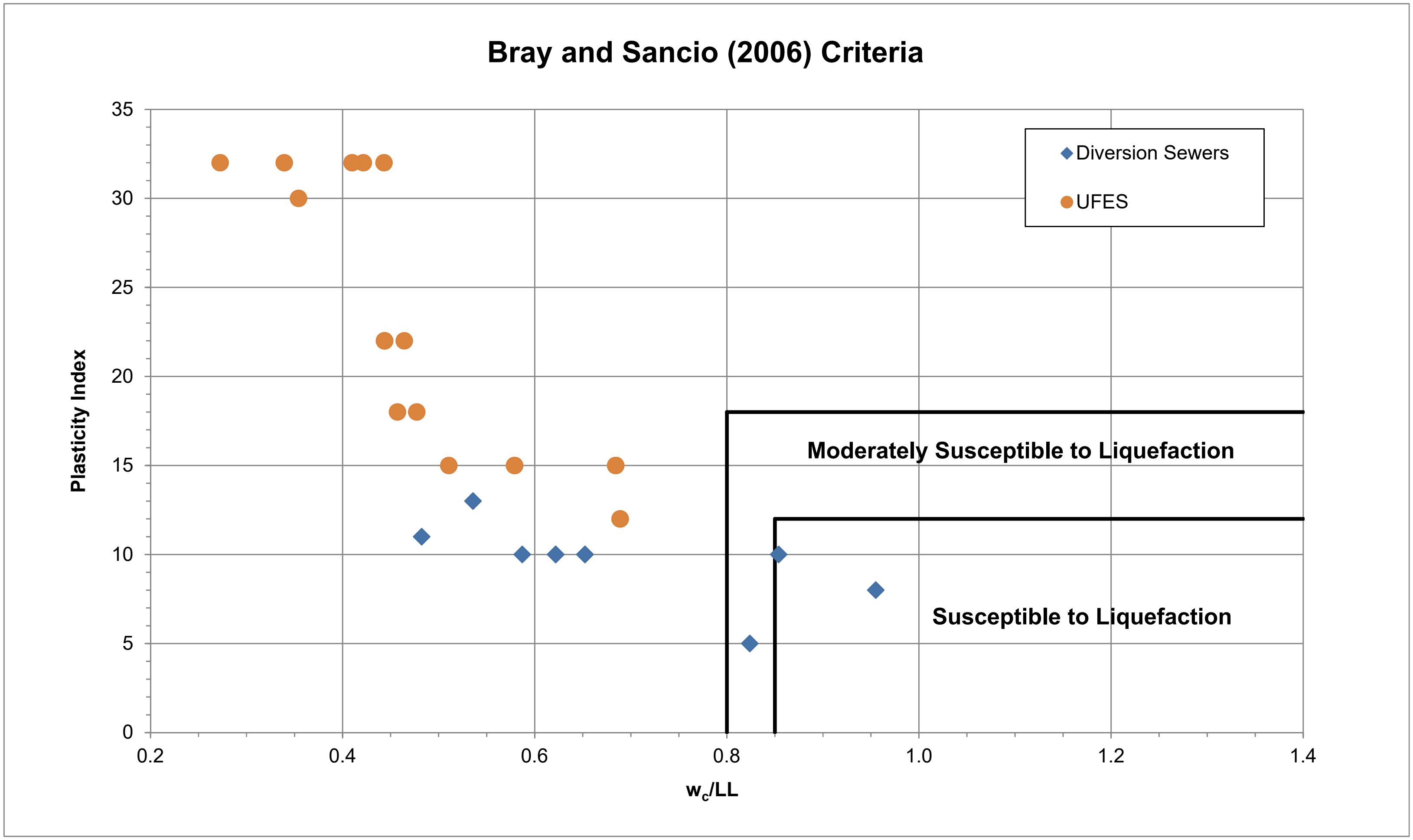
SHEET 17 OF 17

DWG FILE: G:\Active Projects_12000 To 13999\13231\13231000001\DRAFTING\CAD Files\EXPO Tank And Diversion Sewers\ENGEO-SM-2-5-C-EXPO_EAST-0518.dwg

APPENDIX A

LIQUEFACTION ANALYSIS

Appendix A: Bray and Sancio Methodology for Liquefaction Susceptibility



Engeo, Inc.
www.engeo.com

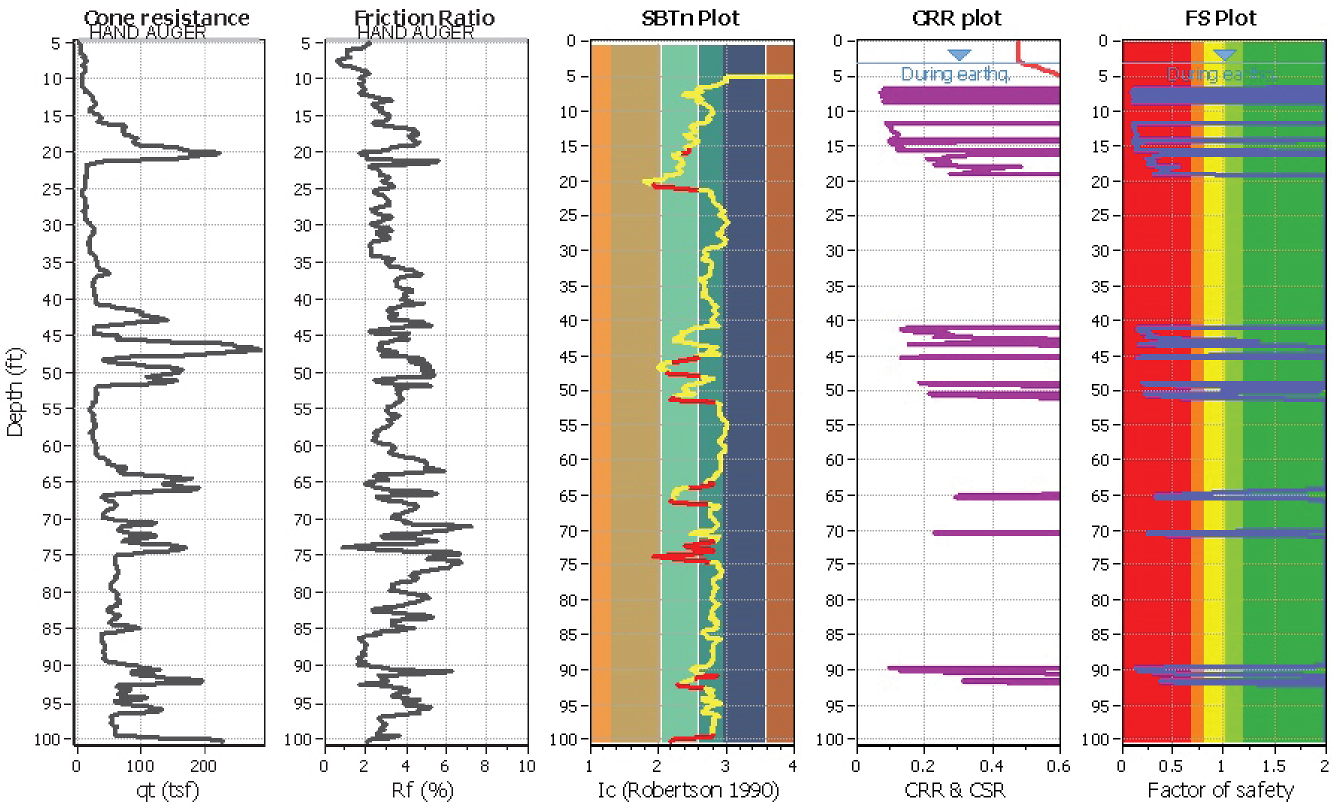
LIQUEFACTION ANALYSIS REPORT

Project title : San Mateo Basin 2&3 **Location :**

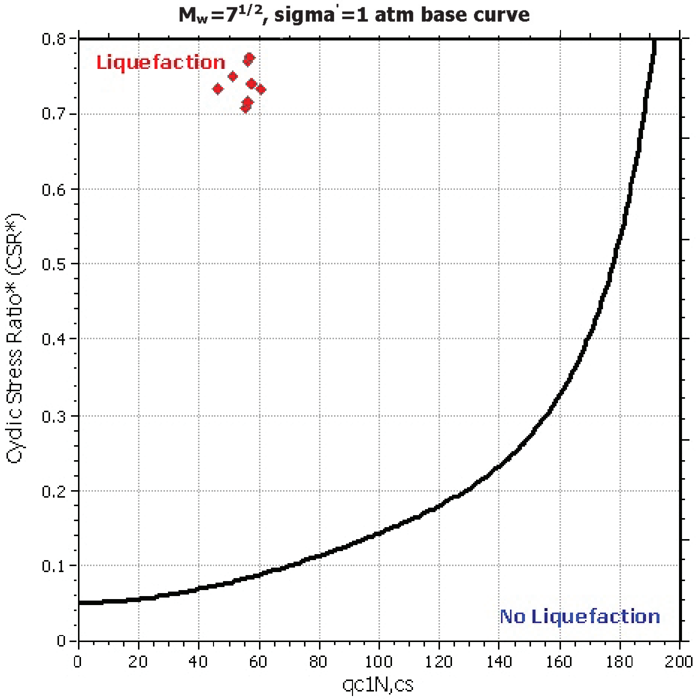
CPT file : CPT-01

Input parameters and analysis data

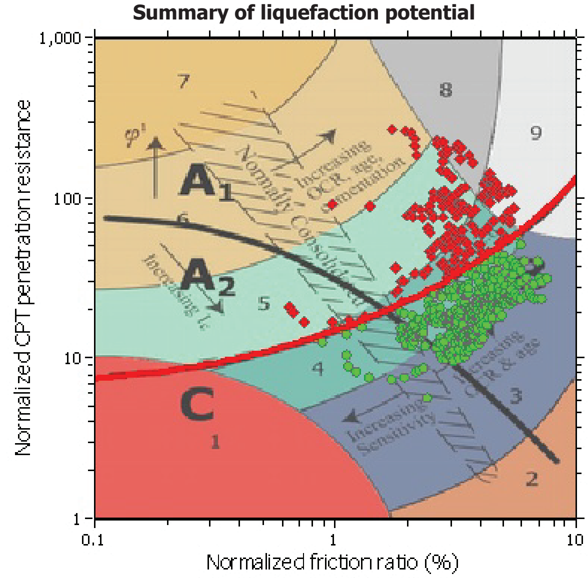
Analysis method:	I&B (2008)	G.W.T. (in-situ):	5.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	I&B (2008)	G.W.T. (earthq.):	3.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.73	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



$M_w=7^{1/2}$, sigma'=1 atm base curve

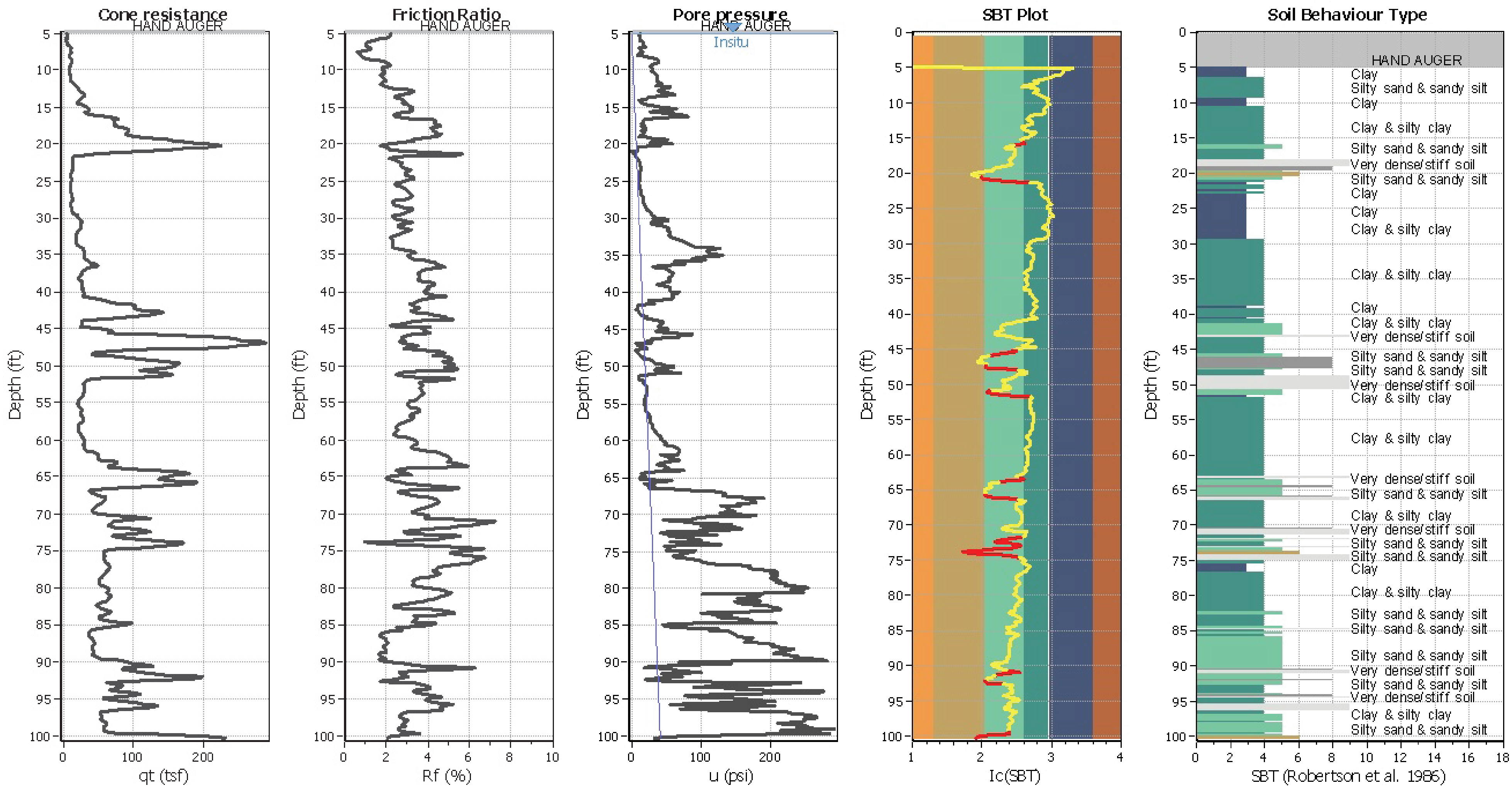


Summary of liquefaction potential



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 plo



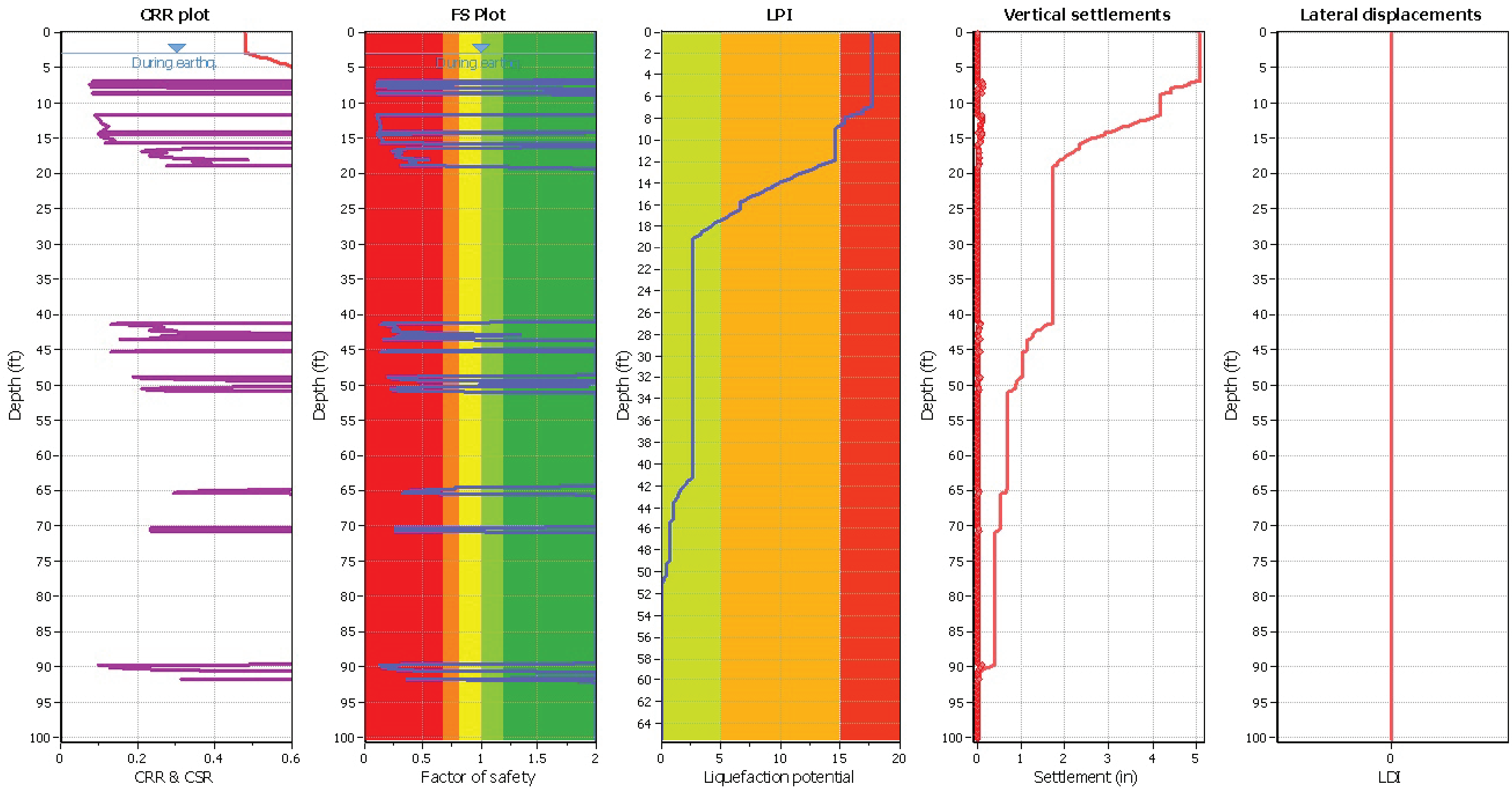
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.73	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

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 plot



Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.73	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

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

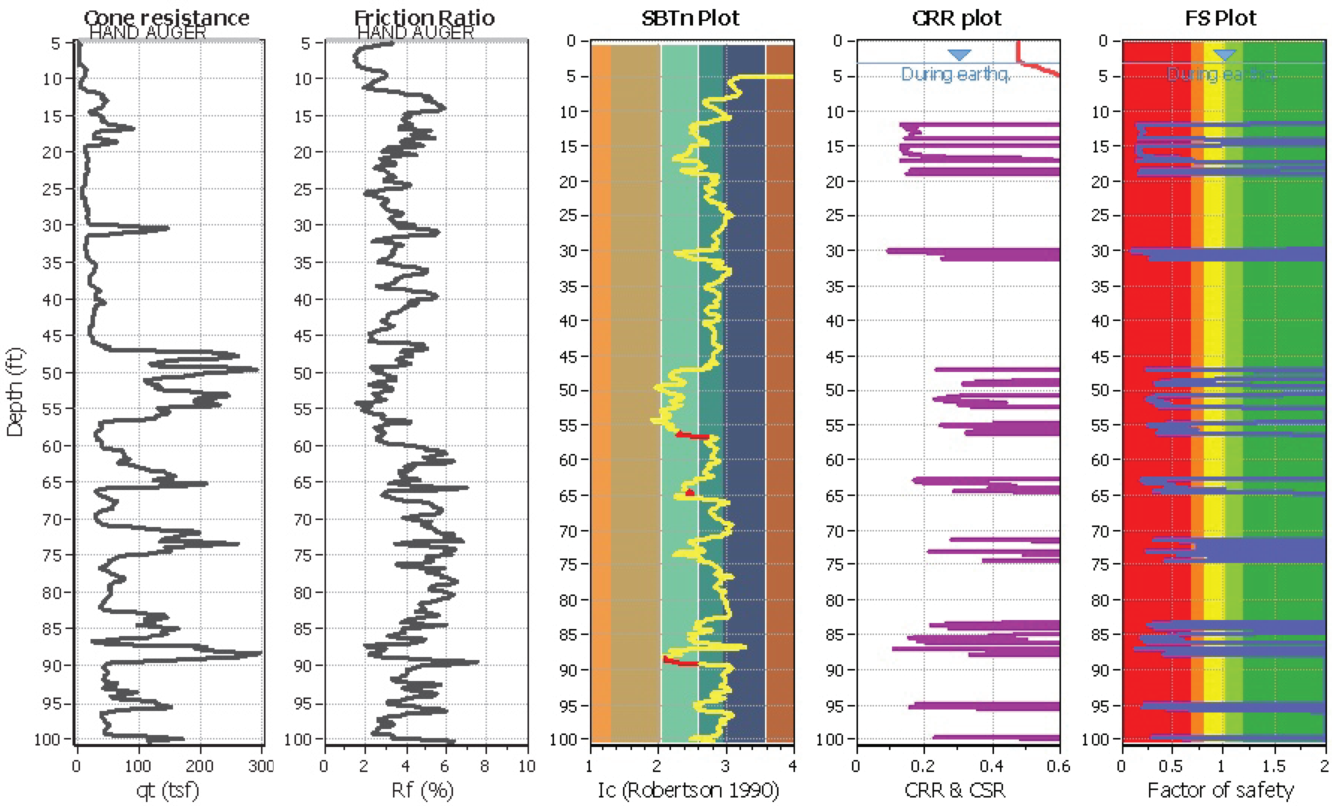
LIQUEFACTION ANALYSIS REPORT

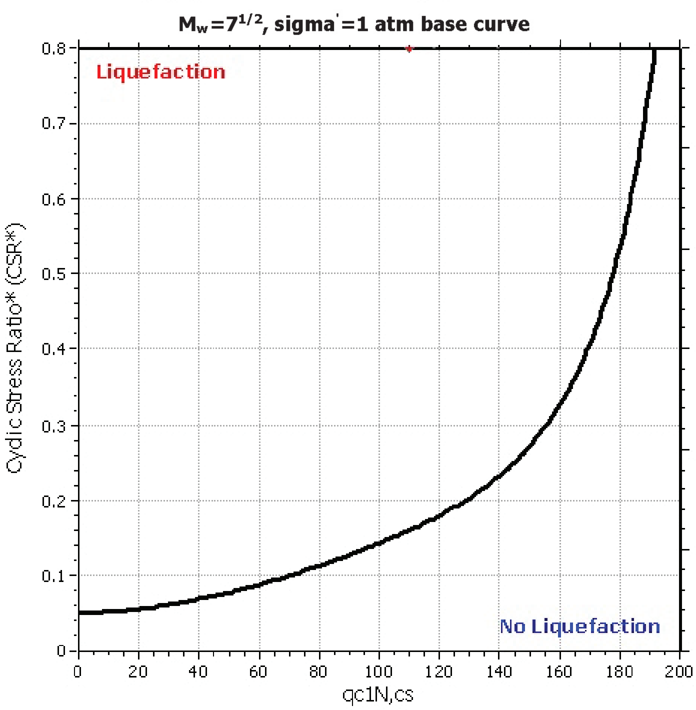
Project title : San Mateo Basin 2&3 **Location :**

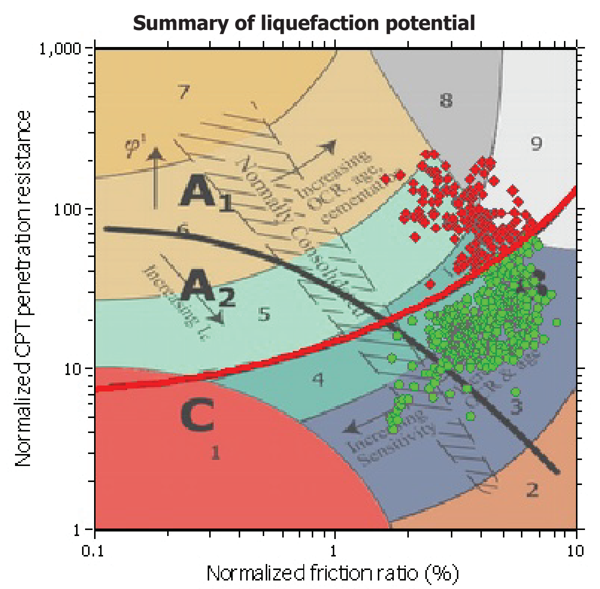
CPT file : CPT-02

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	5.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	I&B (2008)	G.W.T. (earthq.):	3.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.73	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

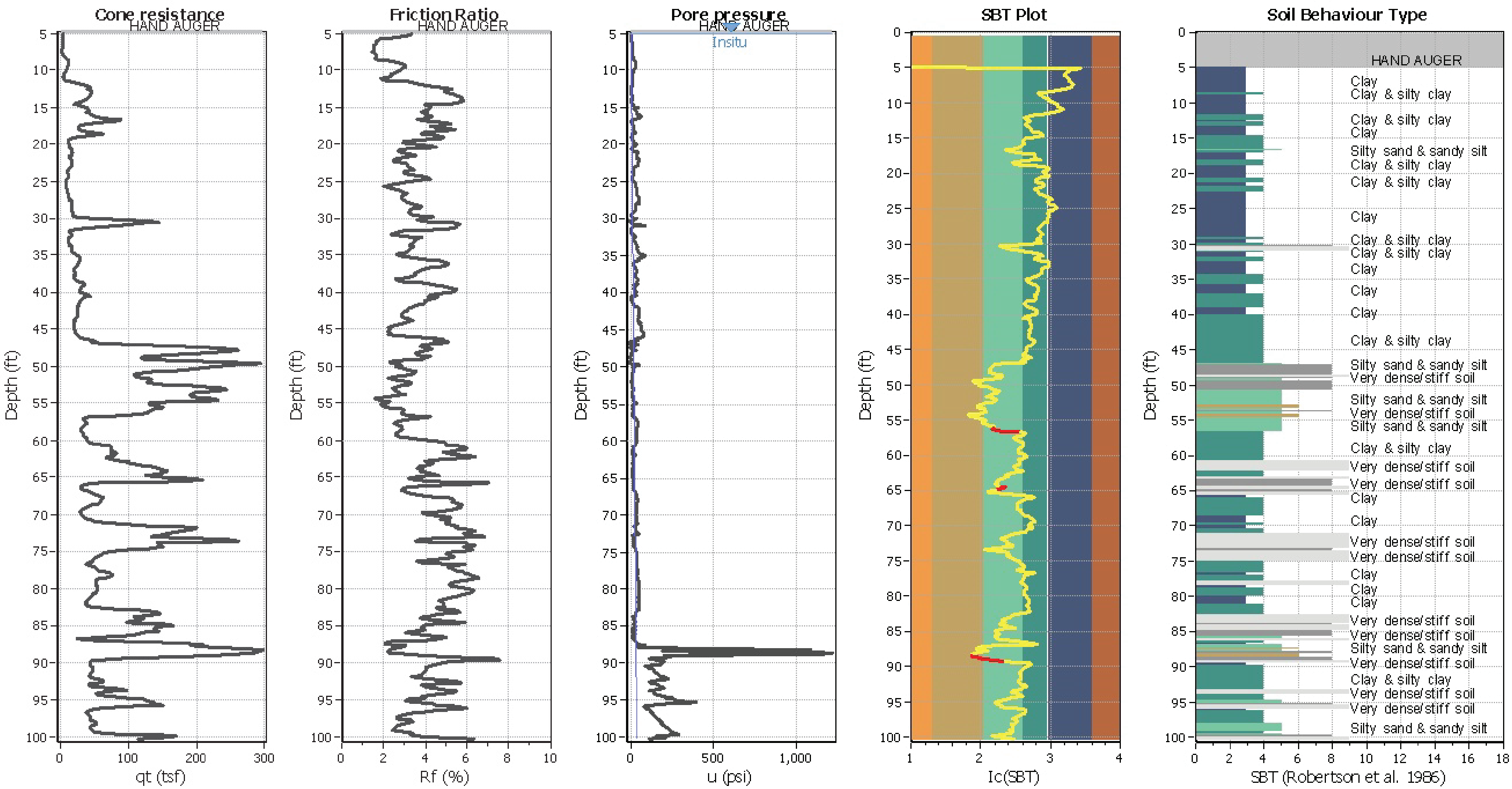






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plo



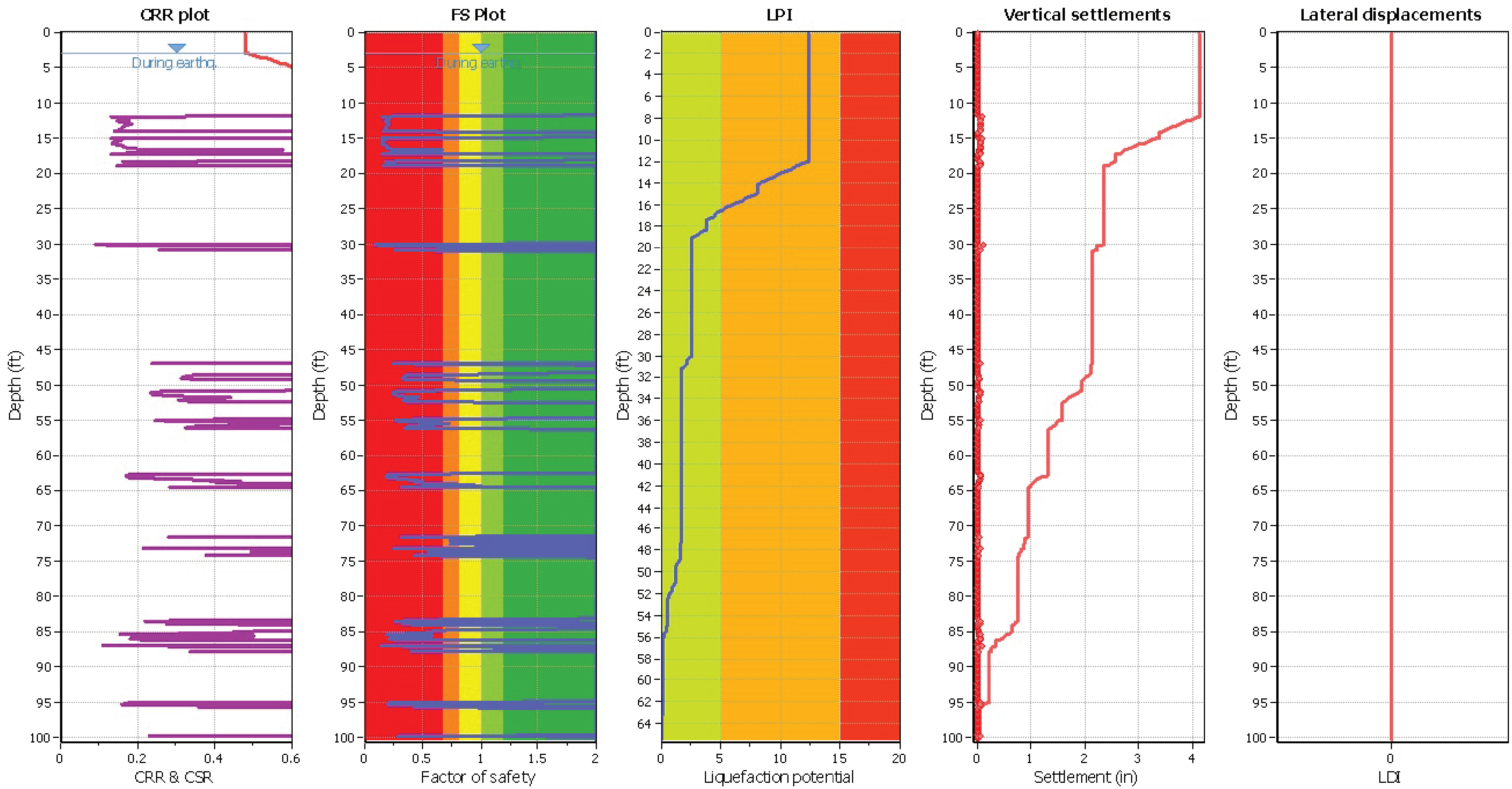
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.73	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

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 plot



Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.73	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

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

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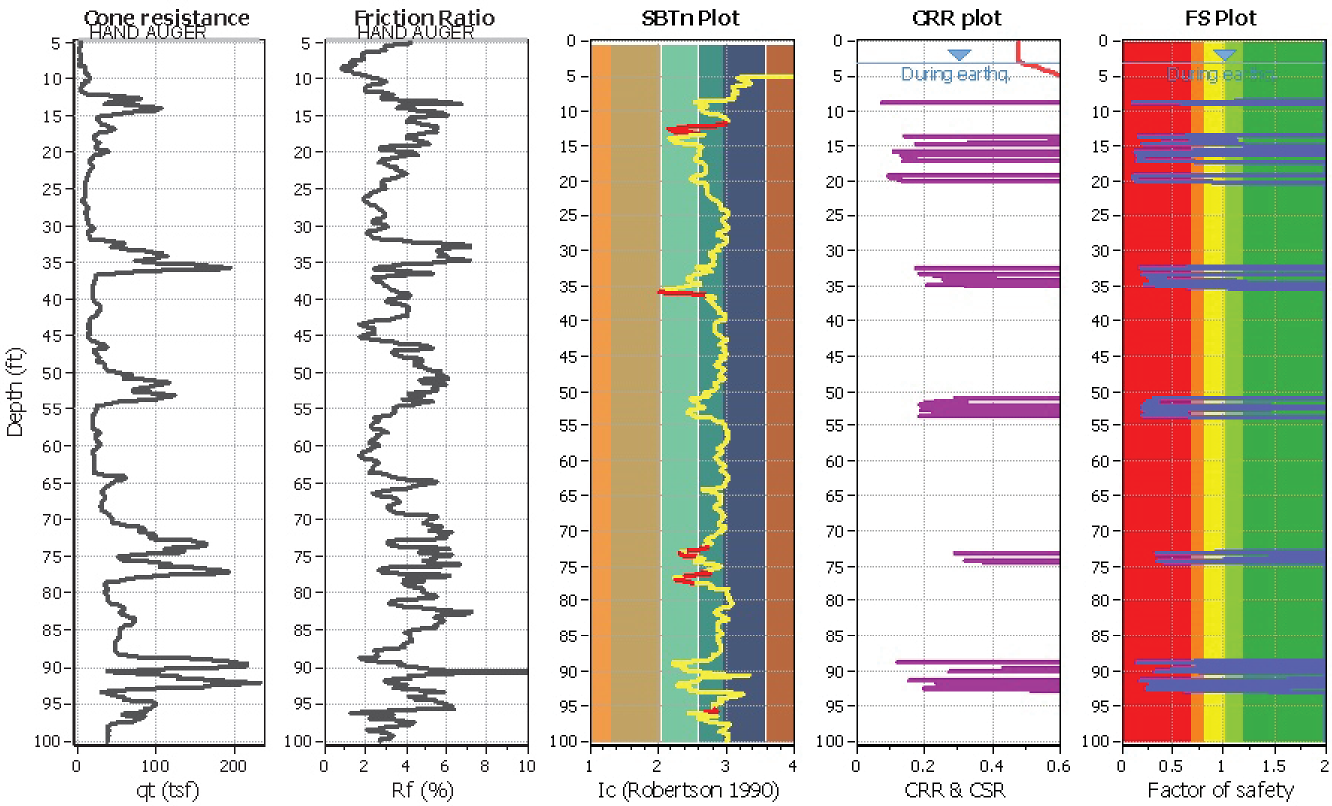
LIQUEFACTION ANALYSIS REPORT

Project title : San Mateo Basin 2&3 **Location :**

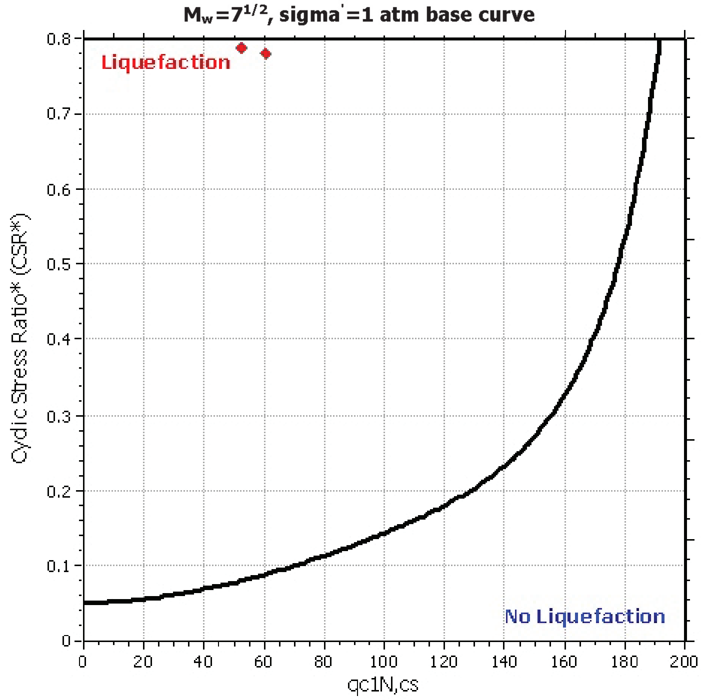
CPT file : CPT-03

Input parameters and analysis data

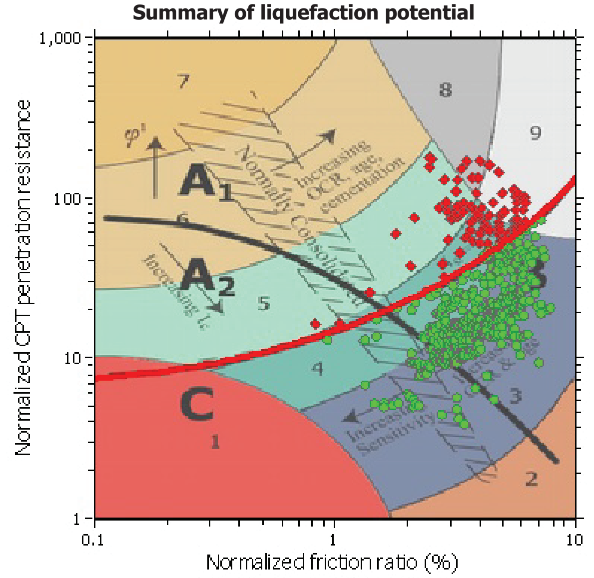
Analysis method:	I&B (2008)	G.W.T. (in-situ):	5.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	I&B (2008)	G.W.T. (earthq.):	3.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.73	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



$M_w=7^{1/2}$, sigma'=1 atm base curve

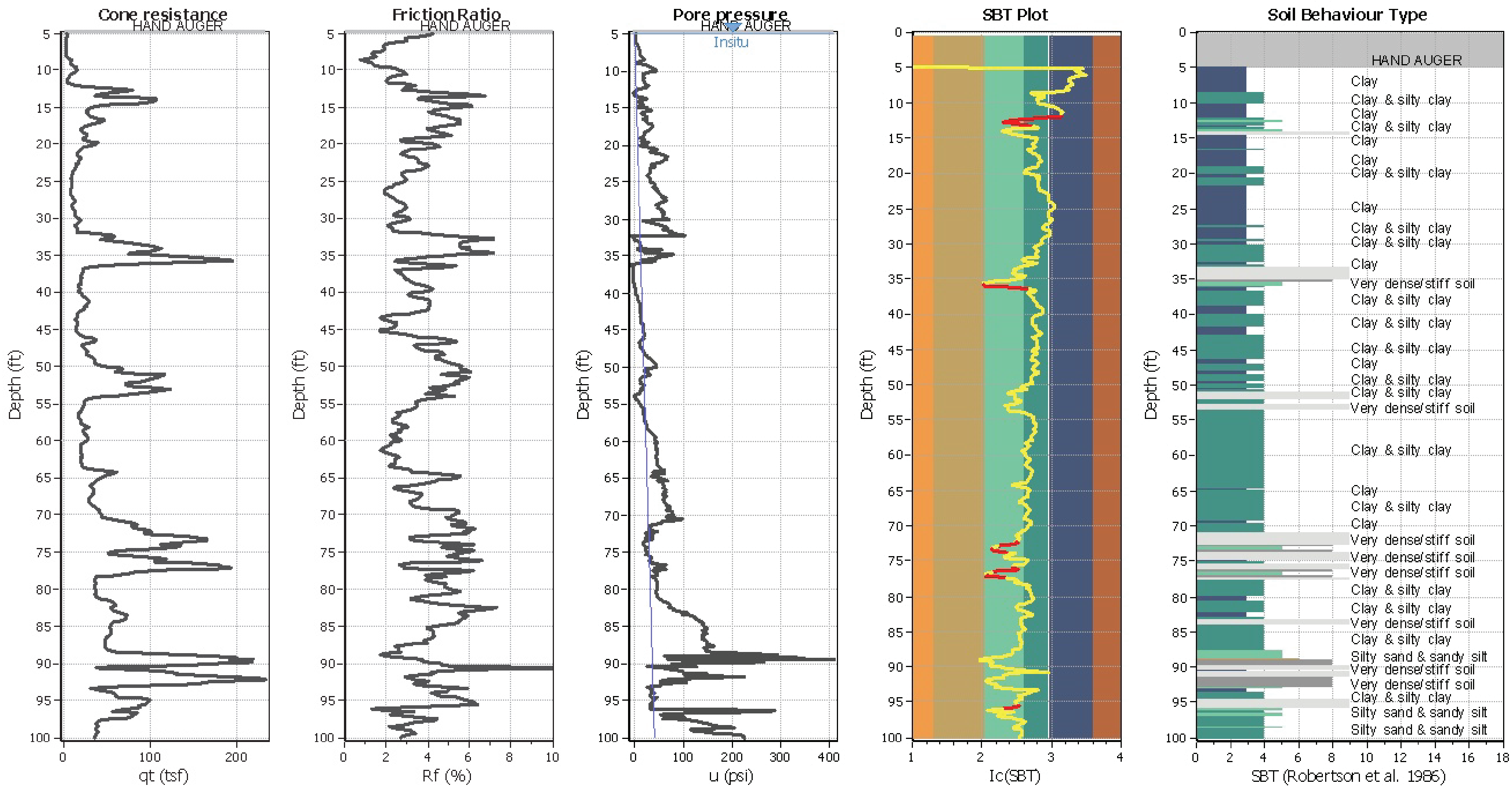


Summary of liquefaction potential



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 plo



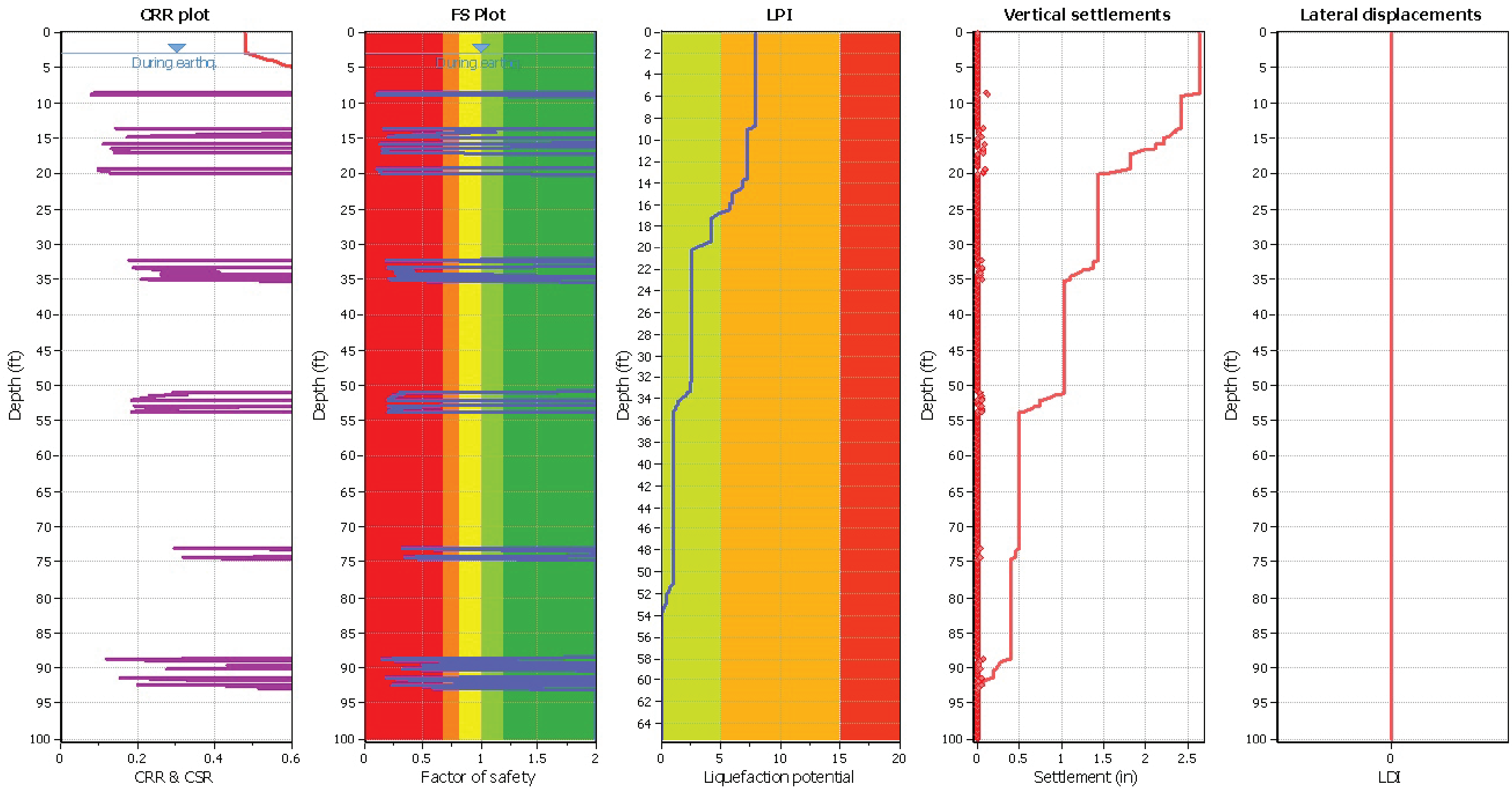
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.73	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

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 plot



Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.73	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

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

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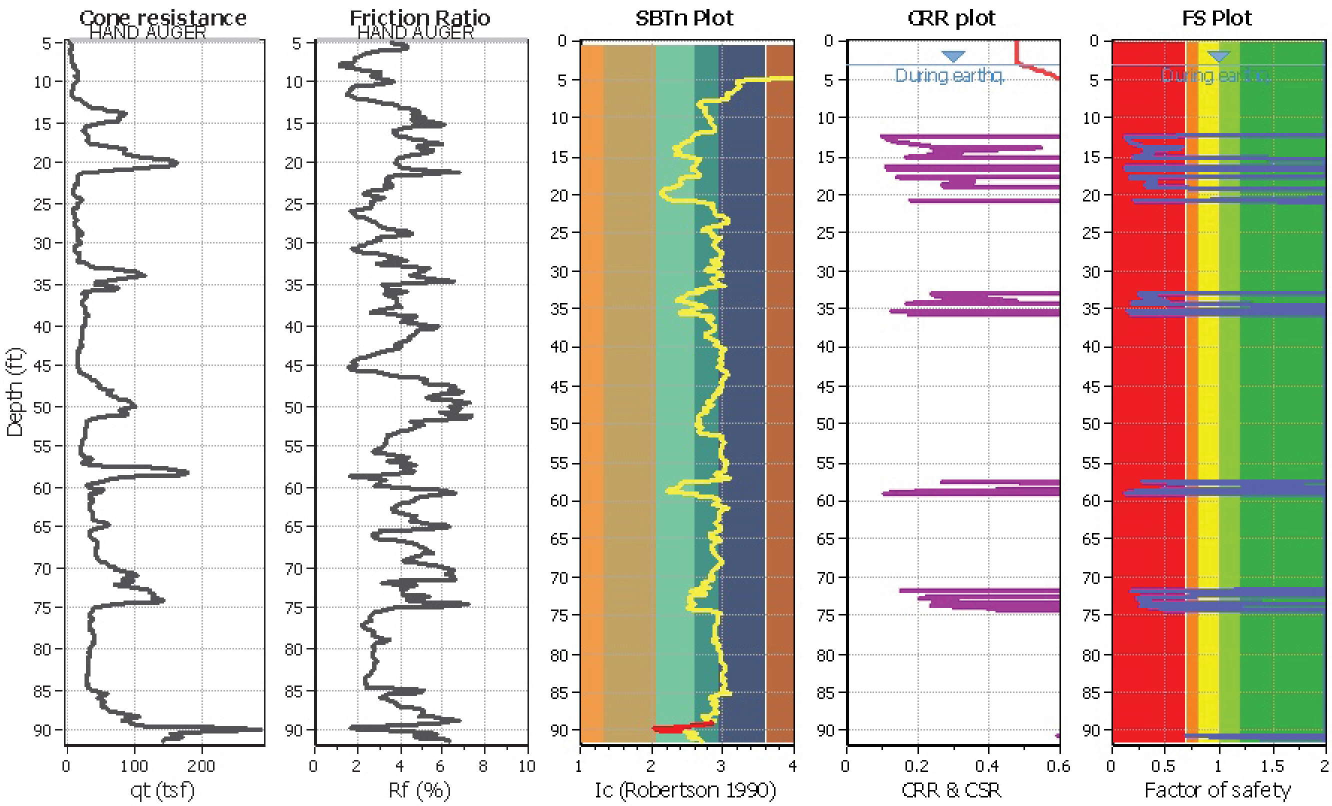
LIQUEFACTION ANALYSIS REPORT

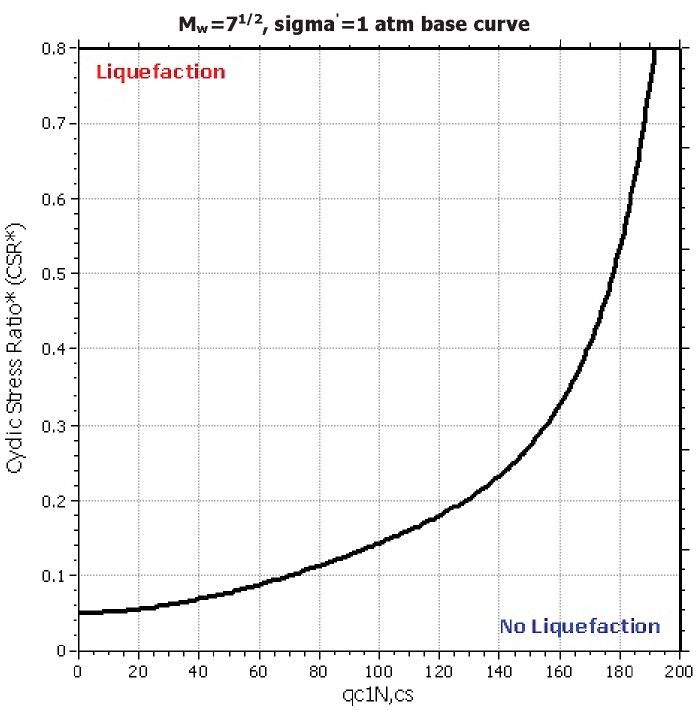
Project title : San Mateo Basin 2&3 **Location :**

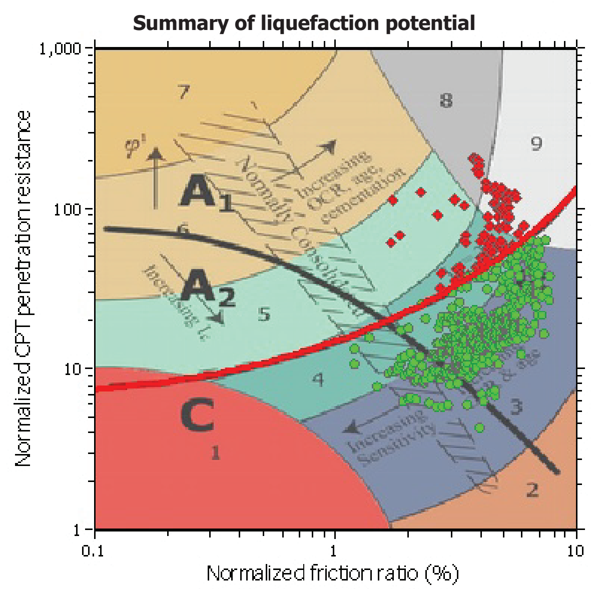
CPT file : CPT-04

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	5.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	I&B (2008)	G.W.T. (earthq.):	3.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.73	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

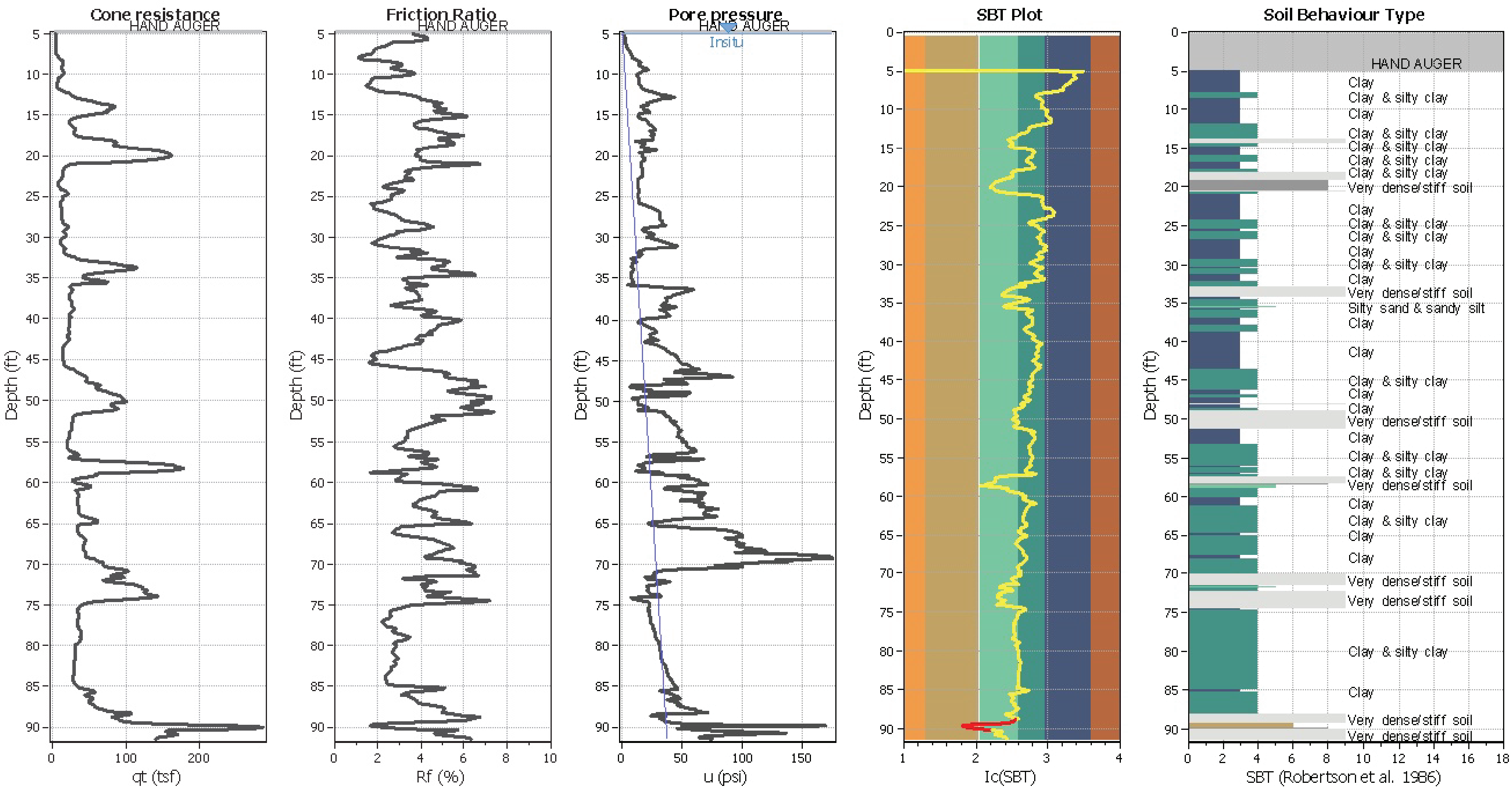






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plo



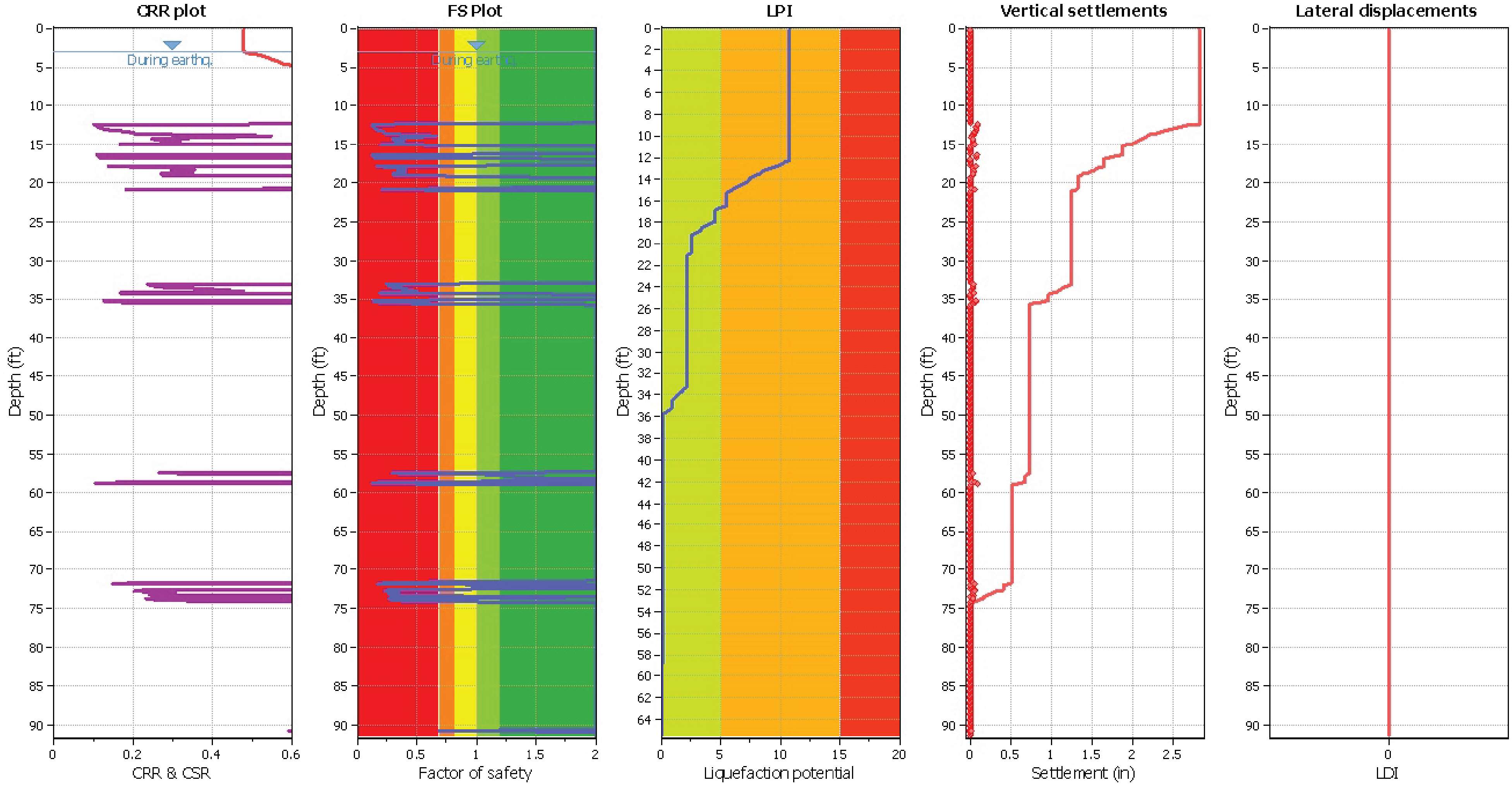
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.73	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

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 plot



Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.73	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

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

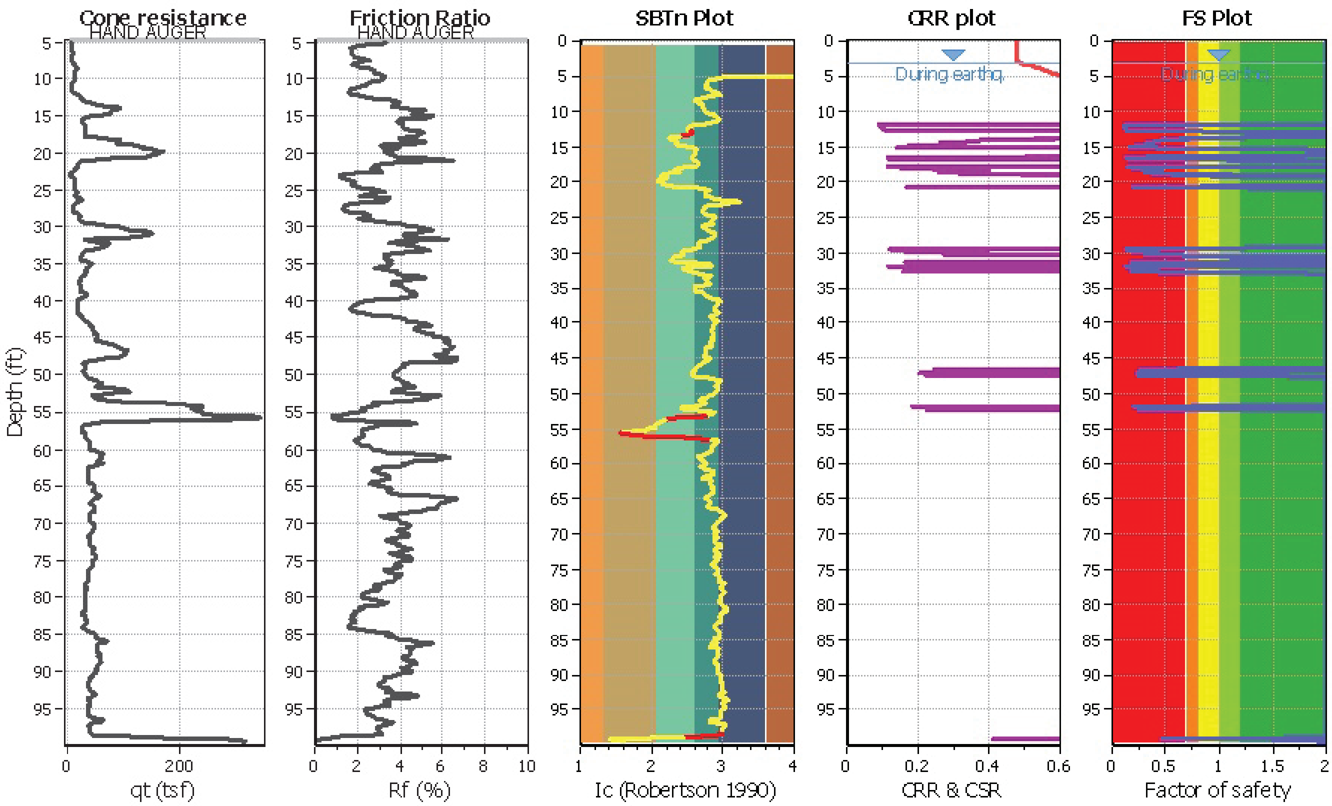
LIQUEFACTION ANALYSIS REPORT

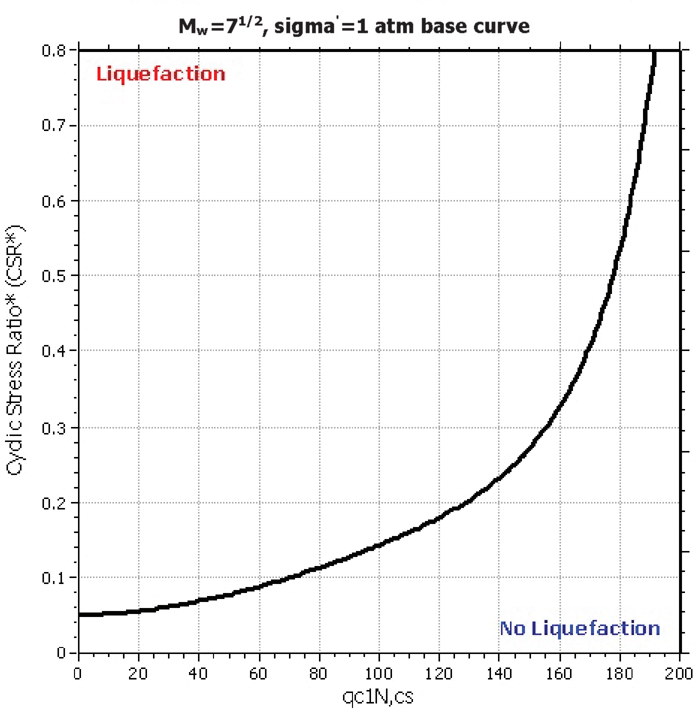
Project title : San Mateo Basin 2&3 **Location :**

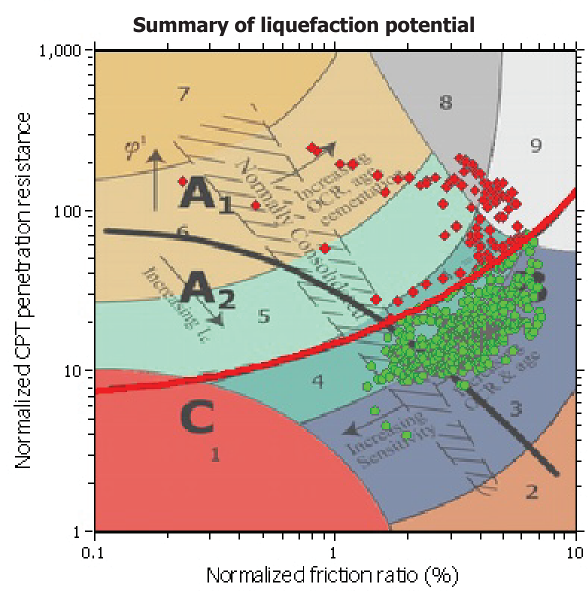
CPT file : CPT-04a

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	5.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	I&B (2008)	G.W.T. (earthq.):	3.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.73	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

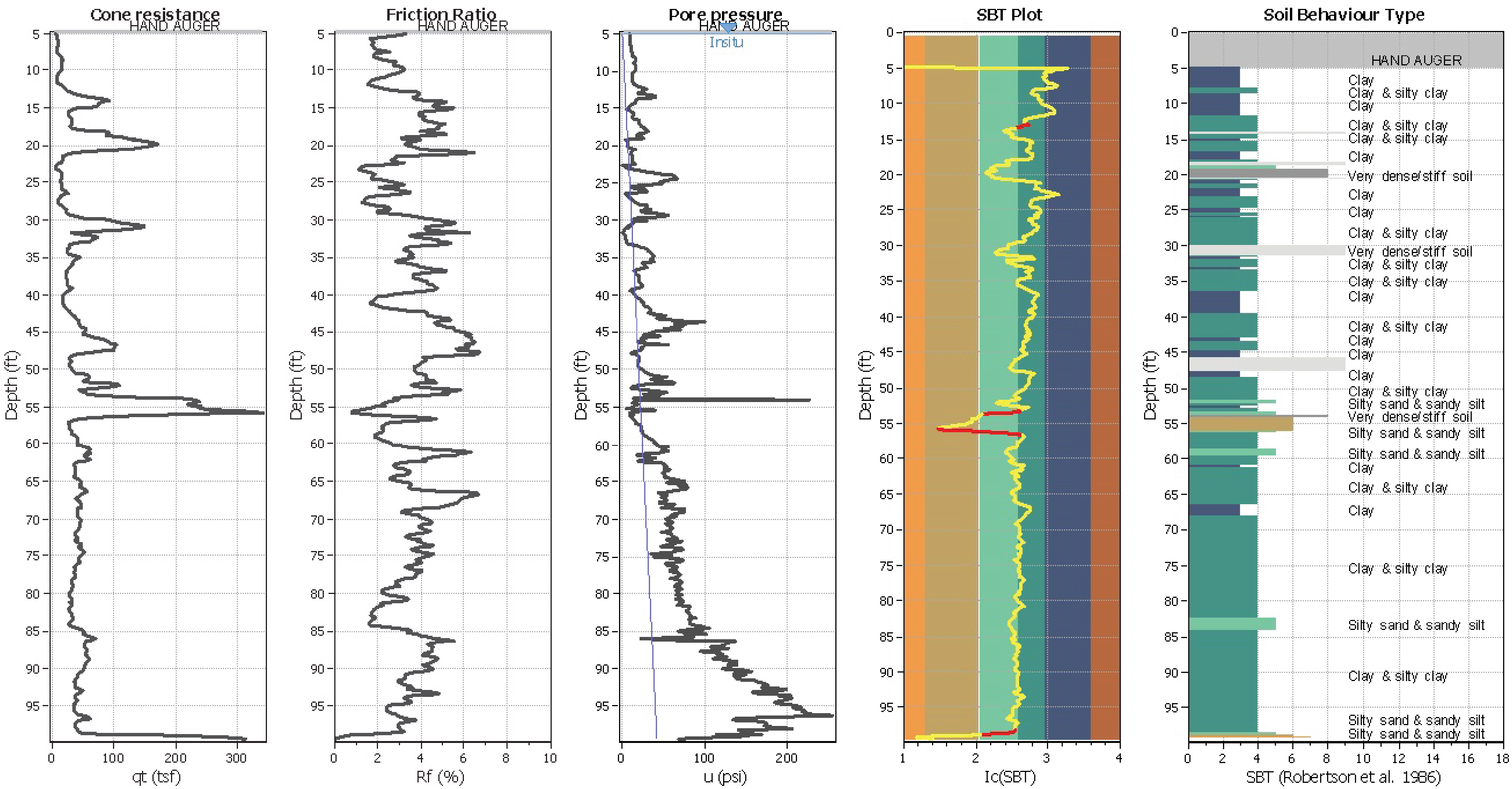






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plo



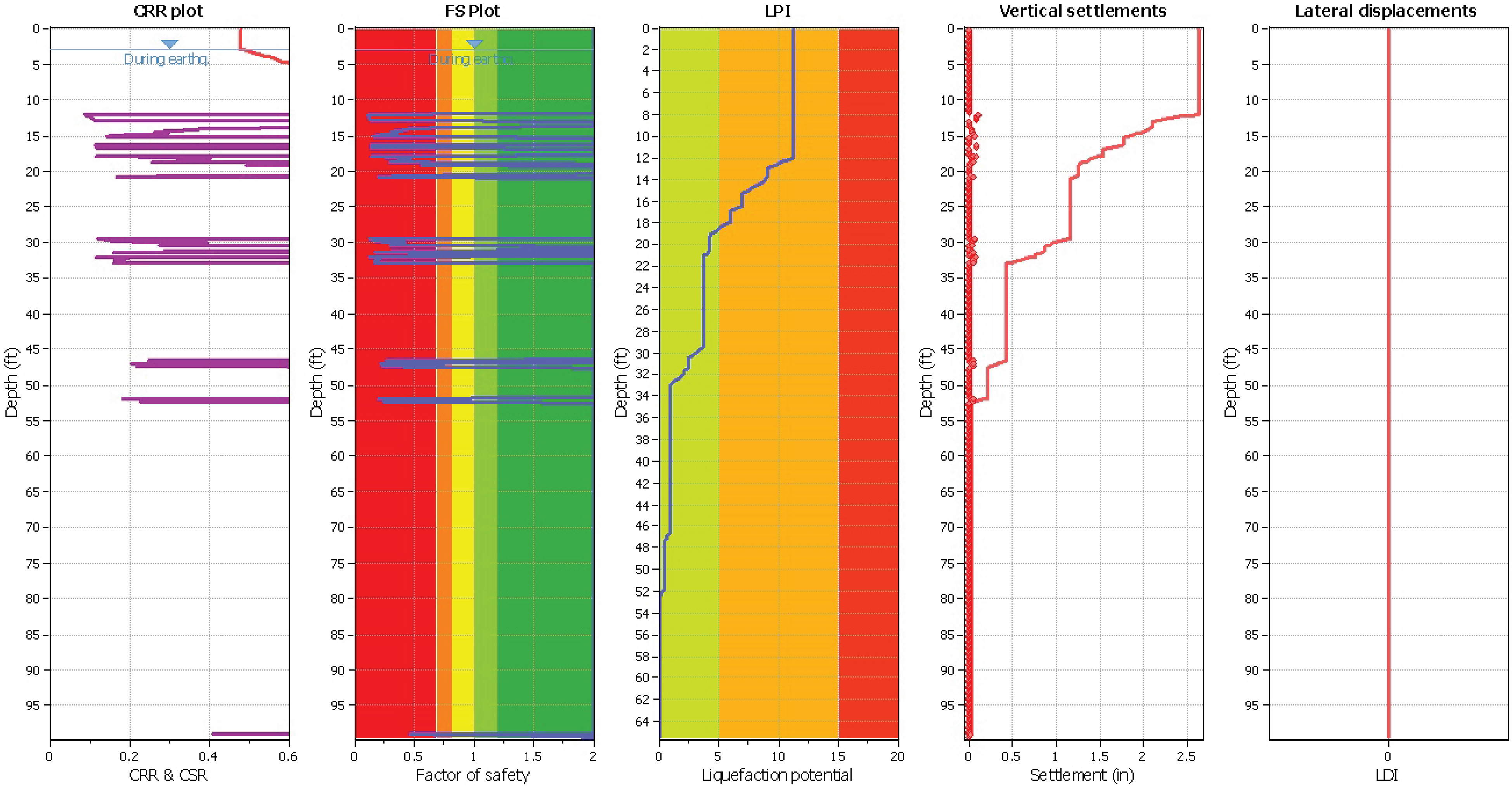
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.73	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

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 clayey sand
9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.73	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

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

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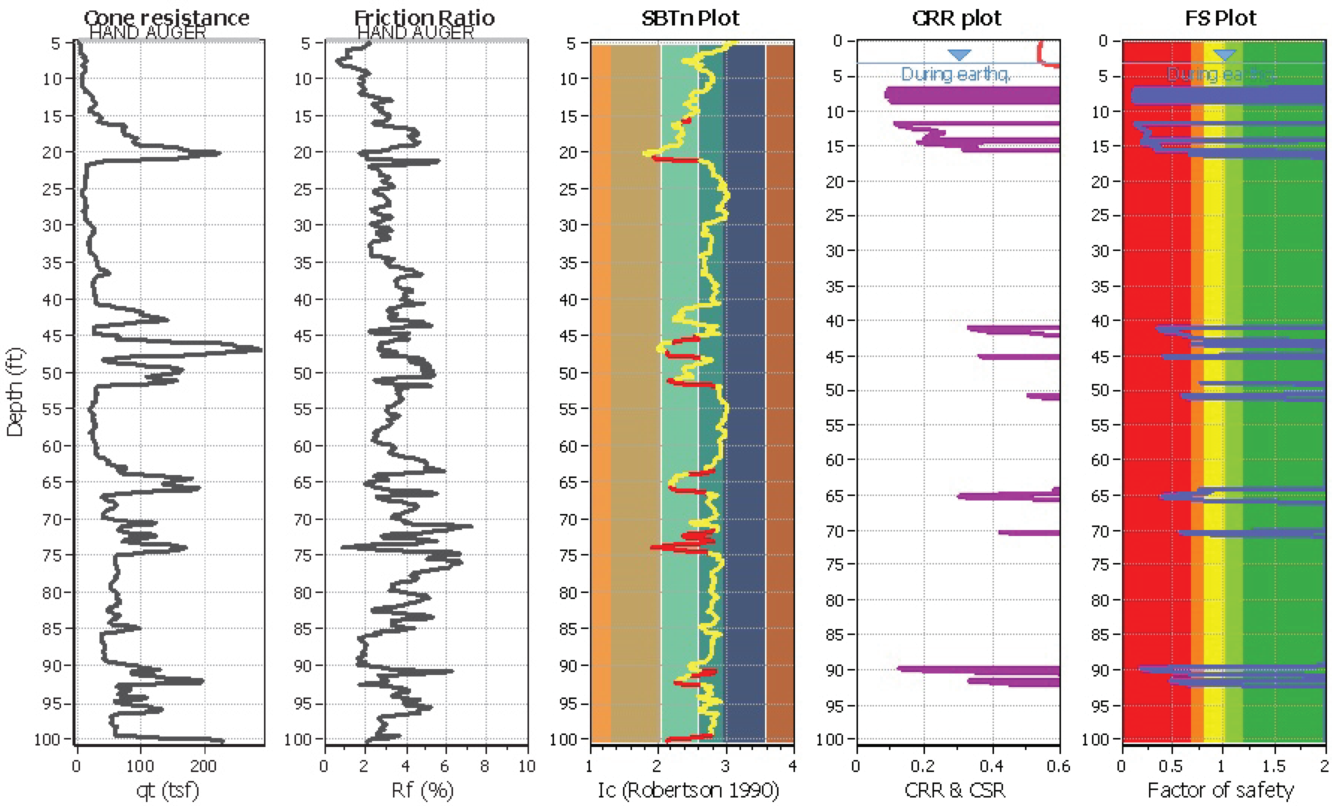
LIQUEFACTION ANALYSIS REPORT

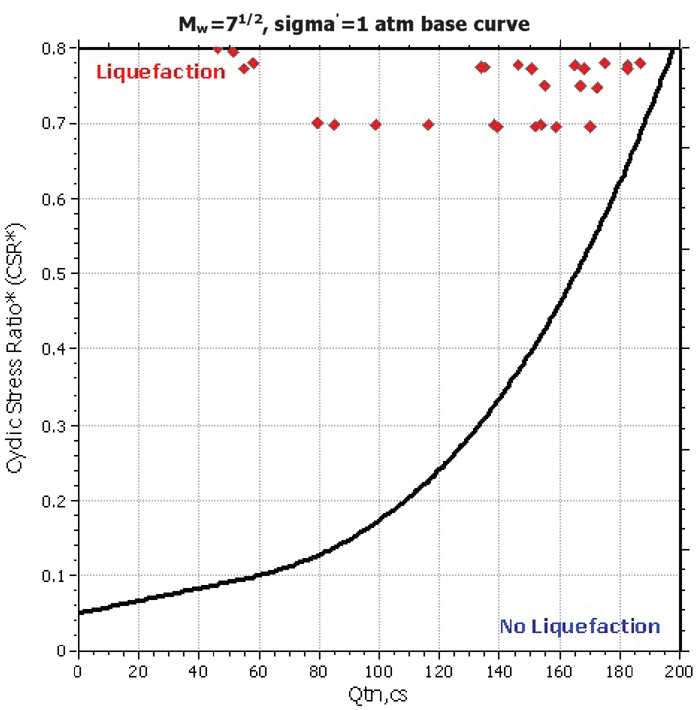
Project title : San Mateo Basin 2&3 **Location :**

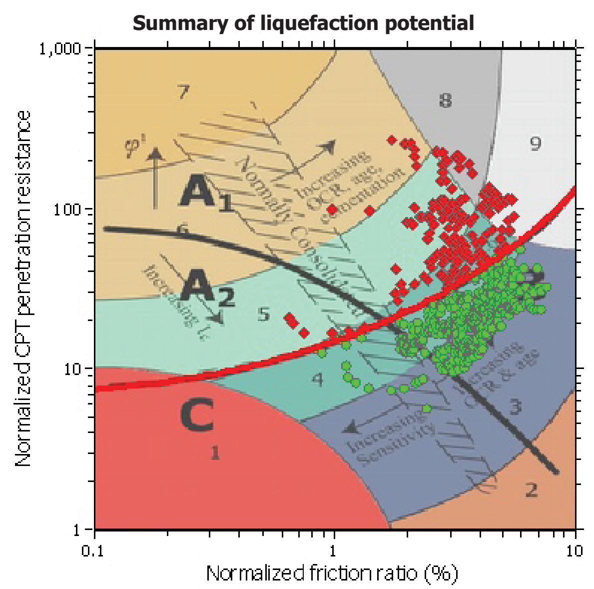
CPT file : CPT-01

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	5.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	3.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.73	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

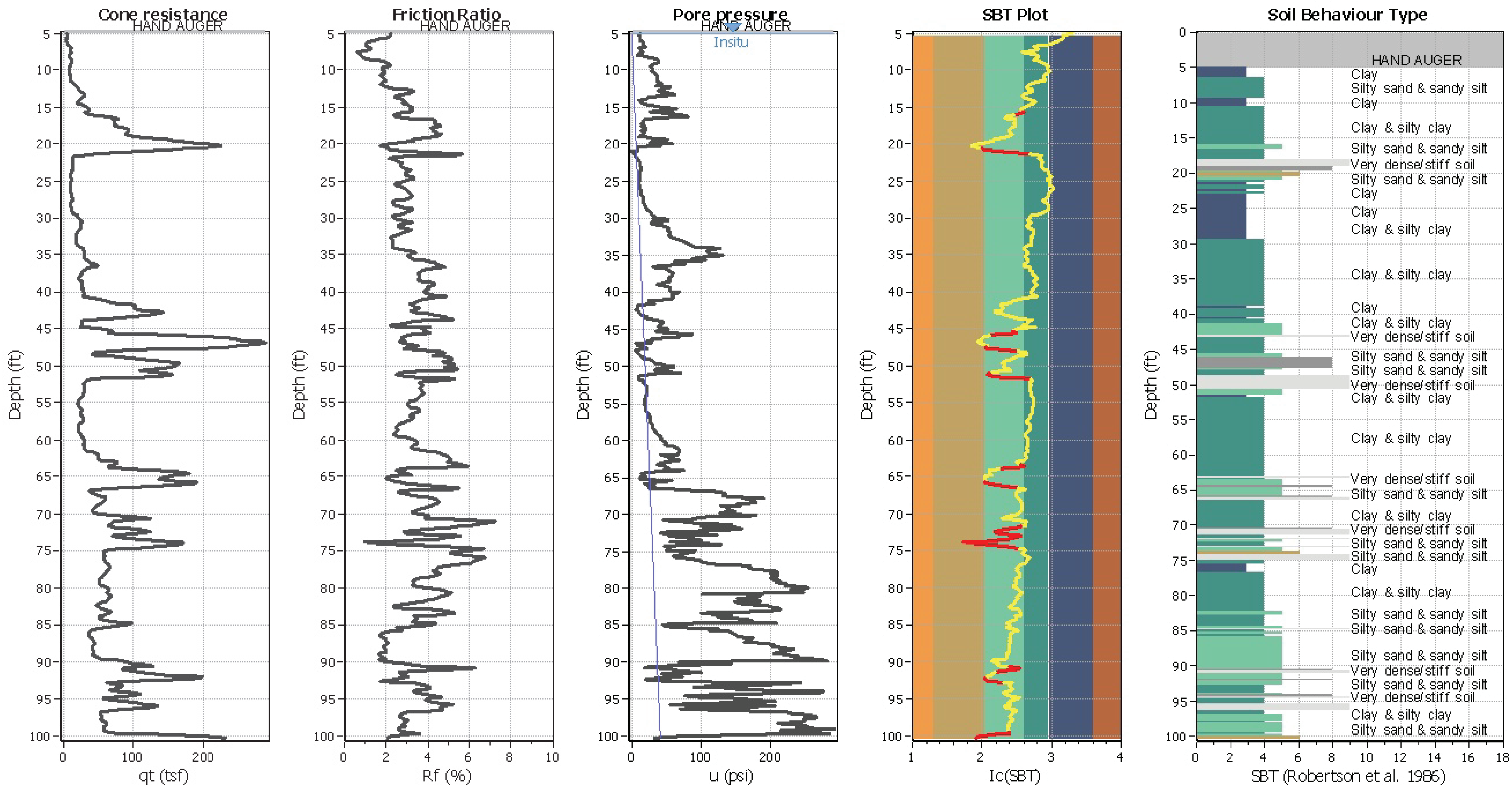






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plo



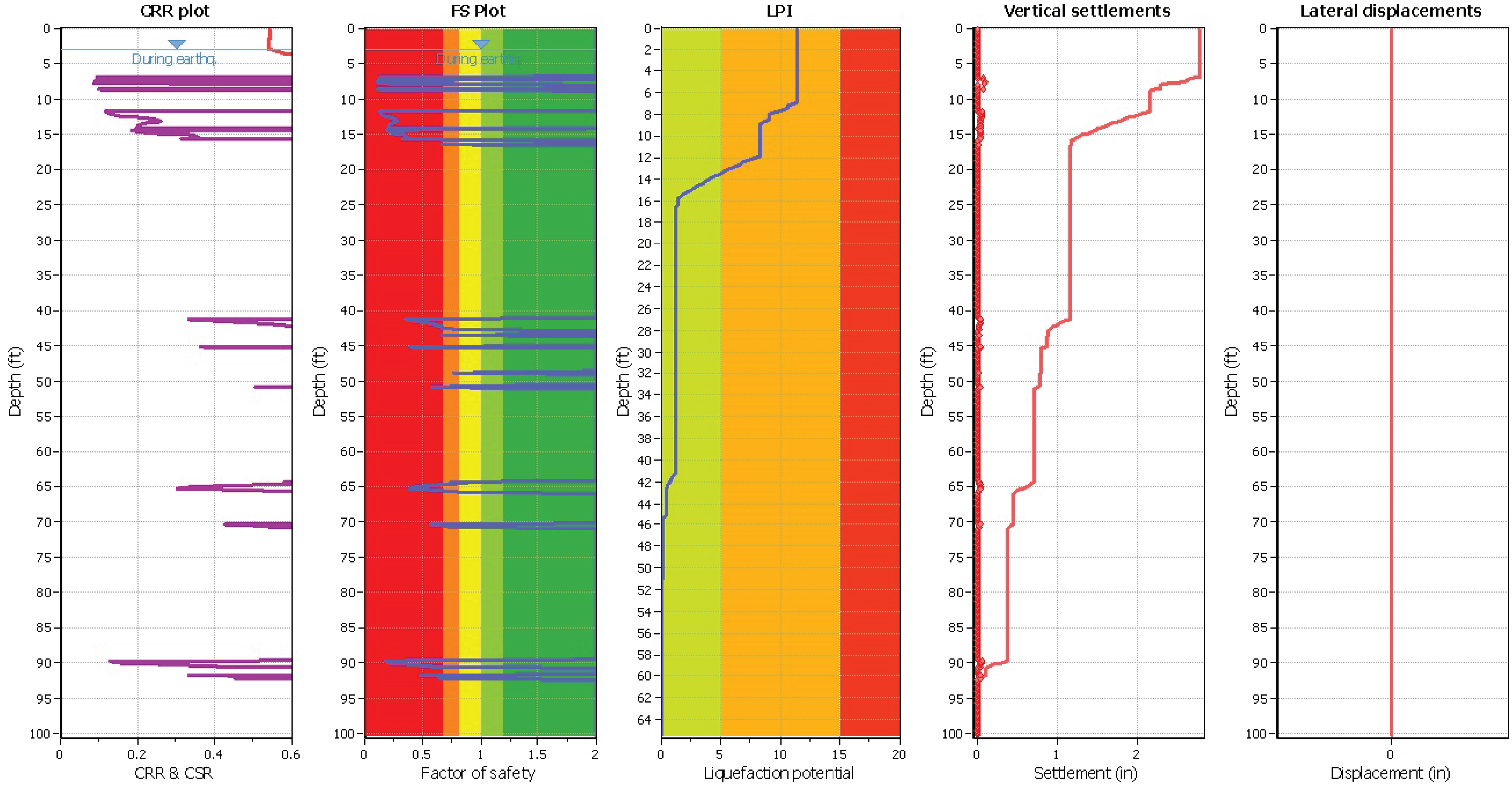
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.73	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

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 clayey sand
9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.73	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

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

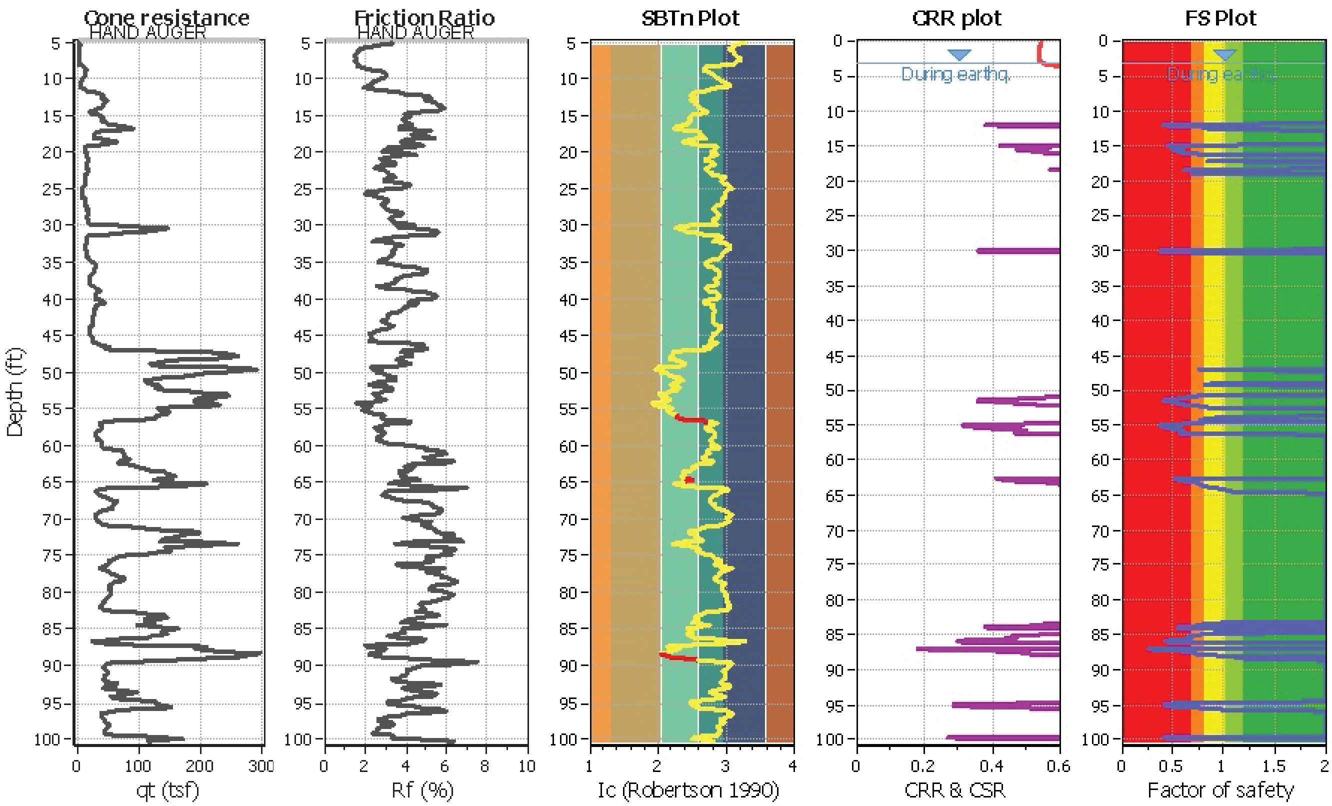
LIQUEFACTION ANALYSIS REPORT

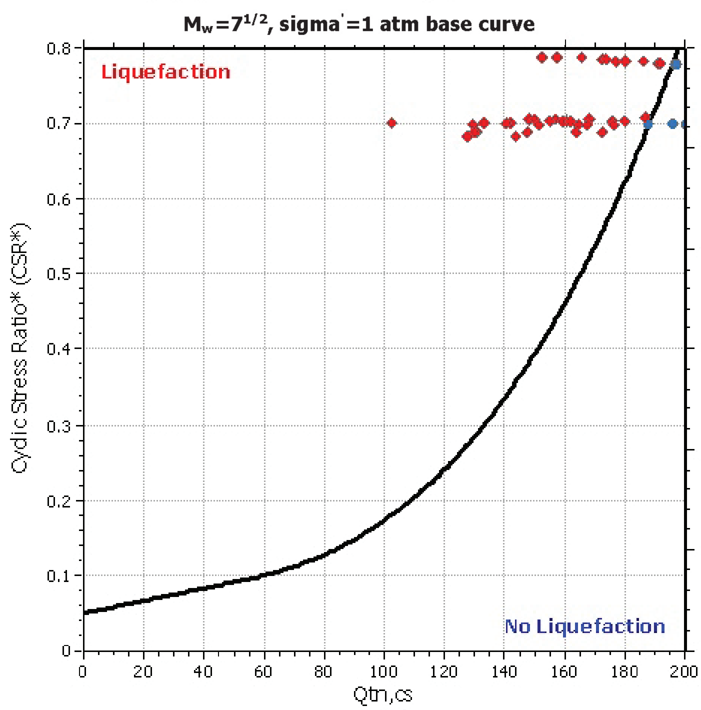
Project title : San Mateo Basin 2&3 **Location :**

CPT file : CPT-02

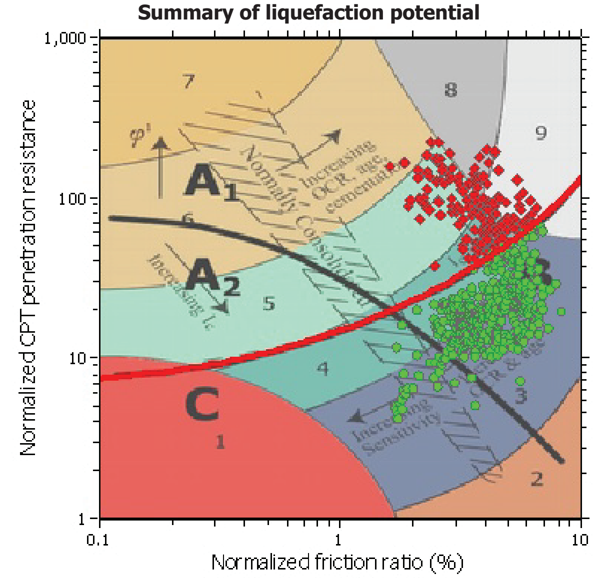
Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	5.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	3.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.73	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



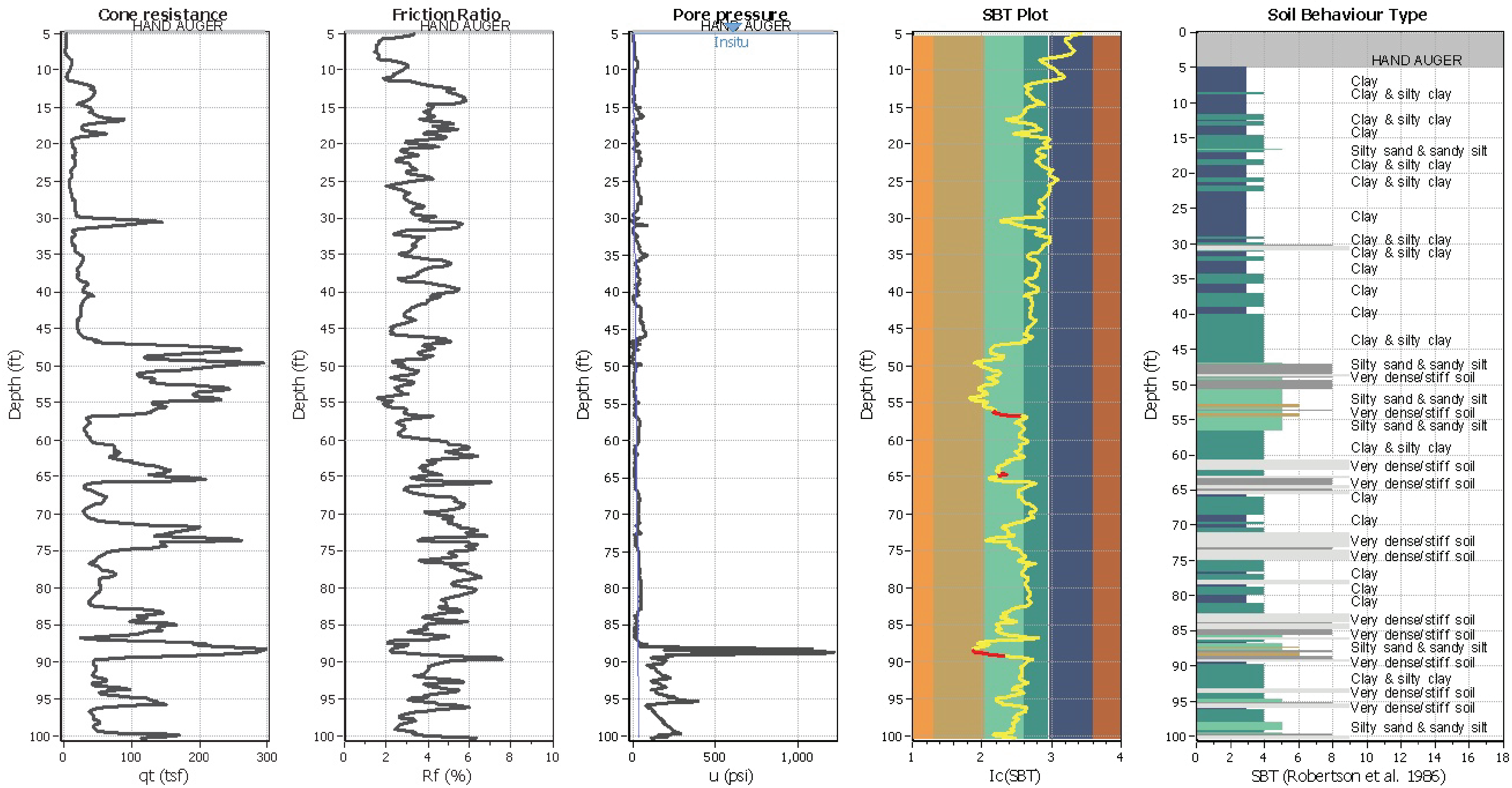


Summary of liquefaction potential



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 plo



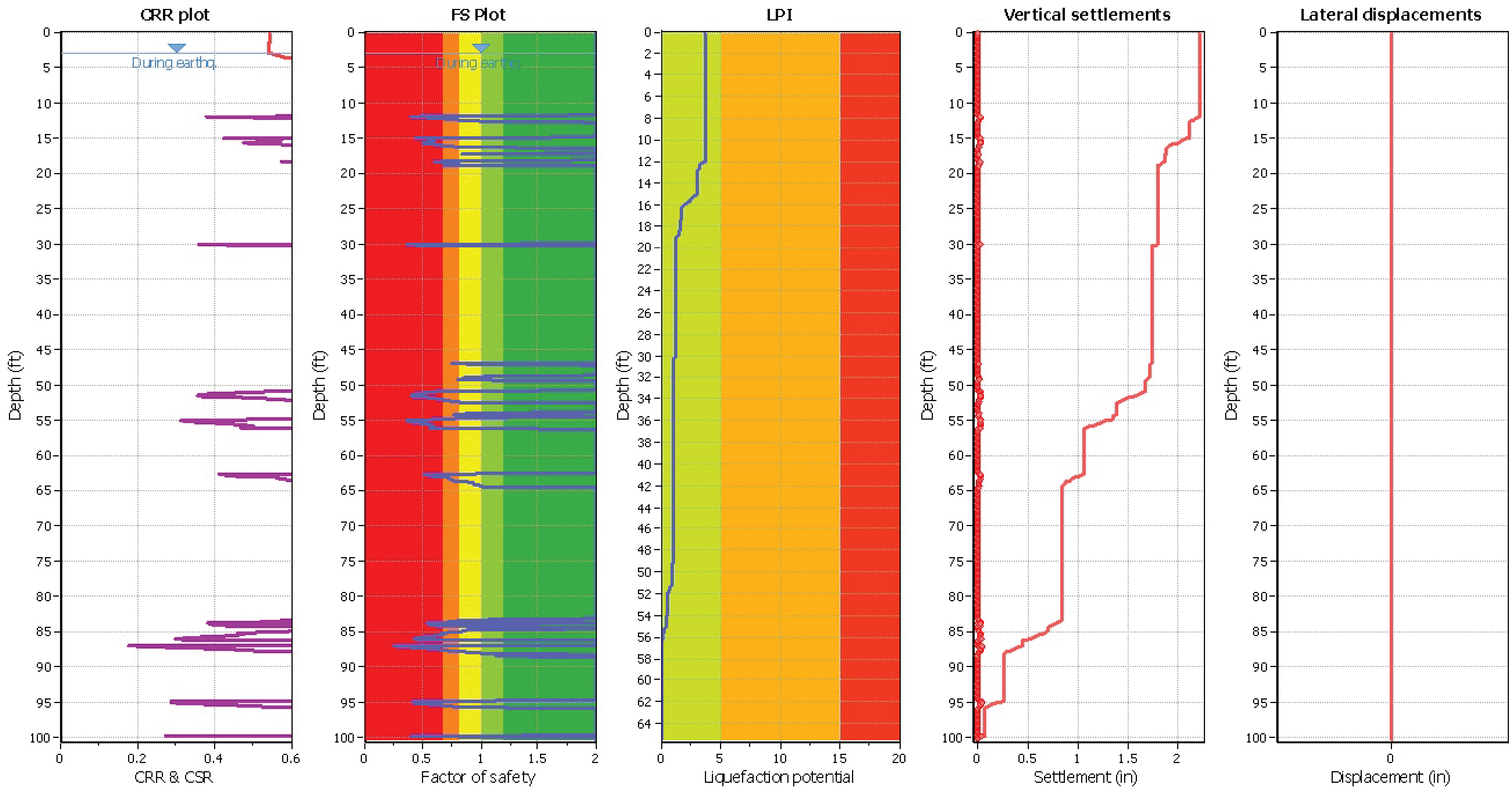
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.73	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

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 plot



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.73	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
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LPI color scheme

- Very high risk
- High risk
- Low risk

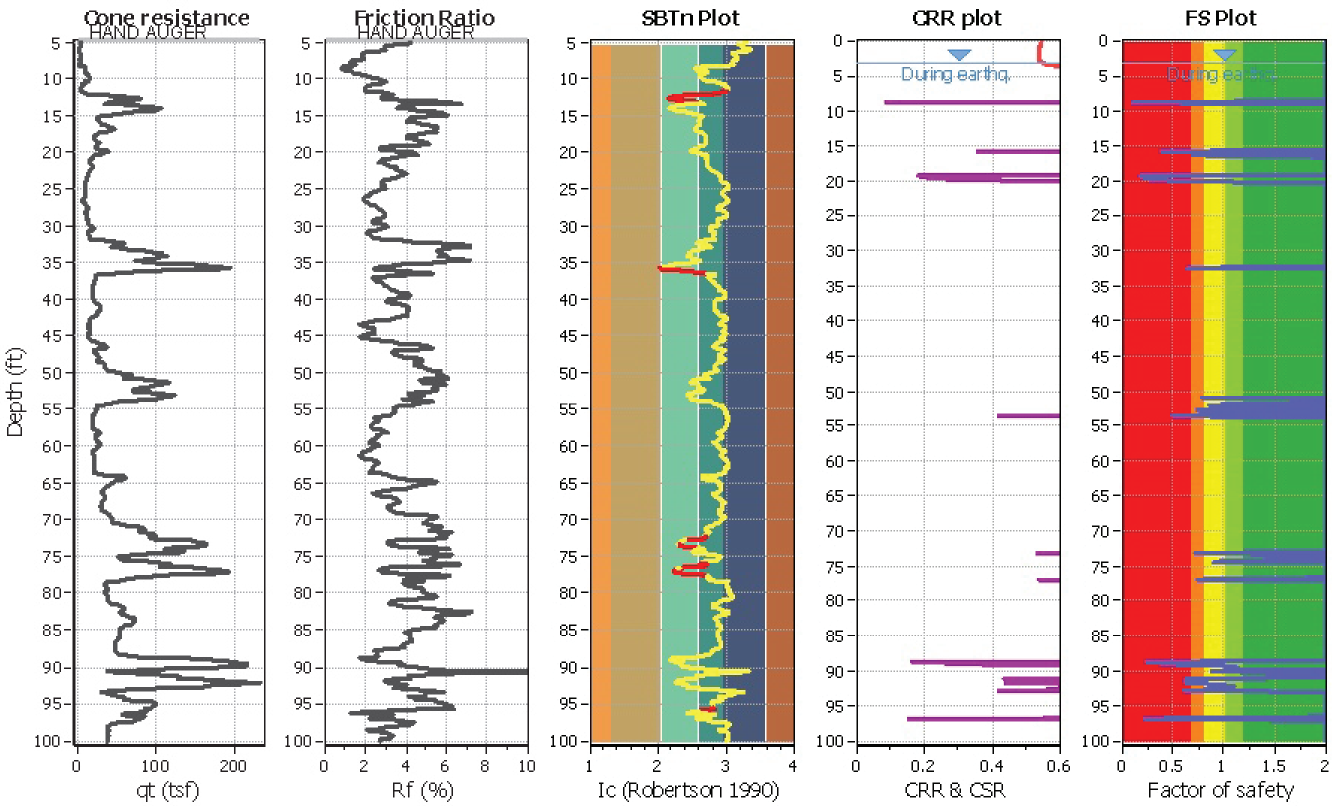
LIQUEFACTION ANALYSIS REPORT

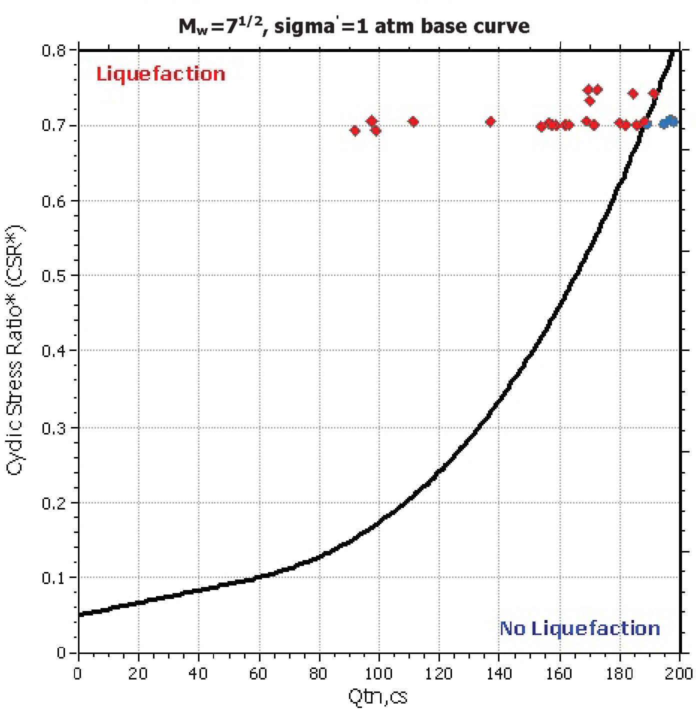
Project title : San Mateo Basin 2&3 **Location :**

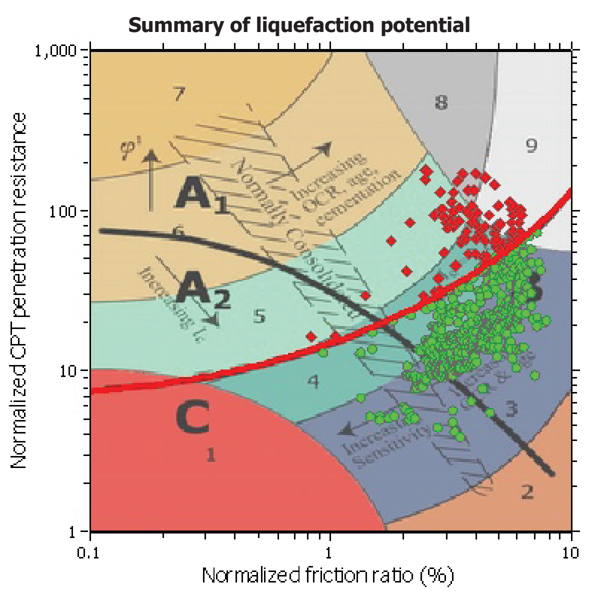
CPT file : CPT-03

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	5.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	3.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.73	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

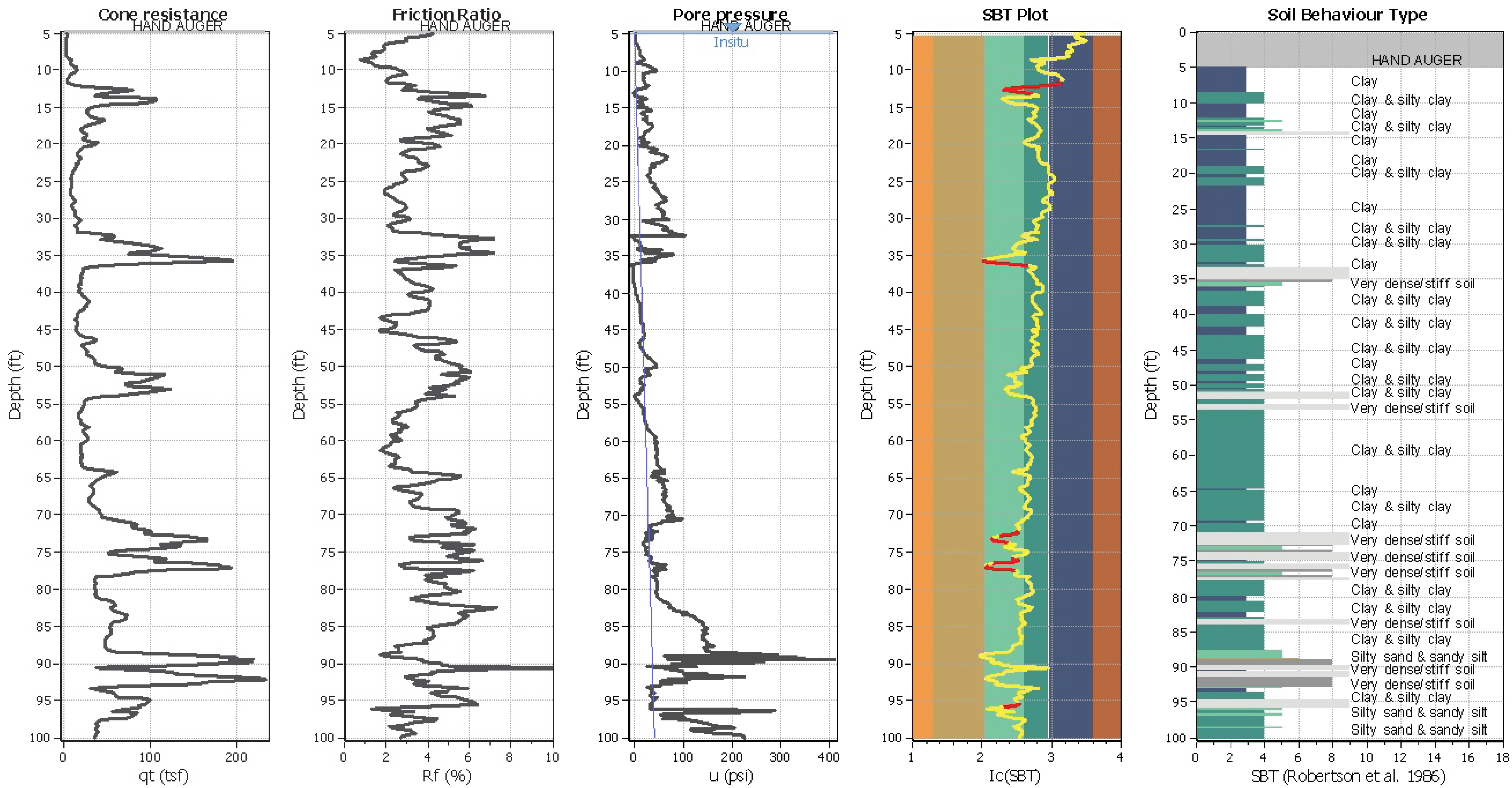






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plo



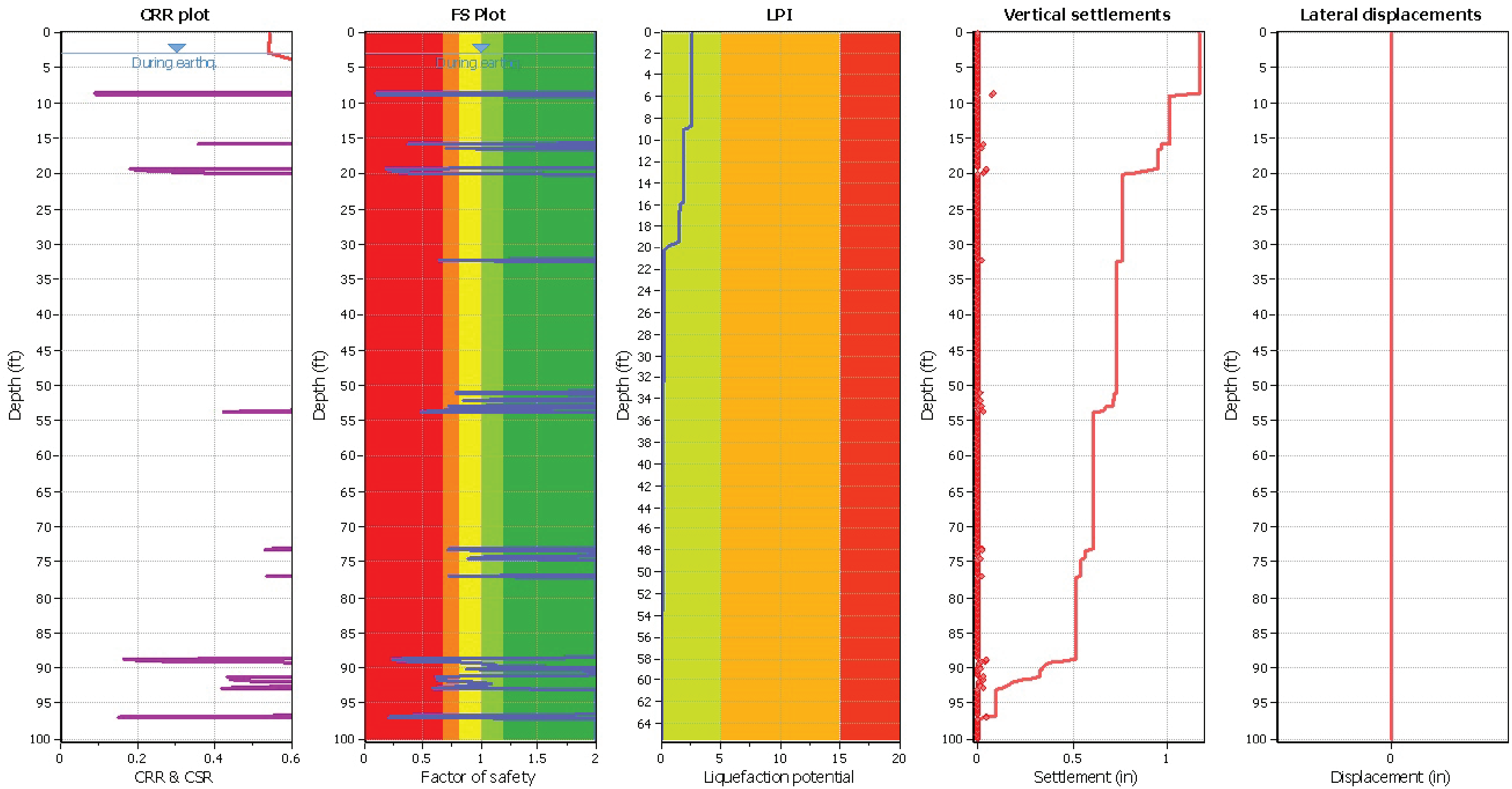
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.73	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

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 plot



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.73	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

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

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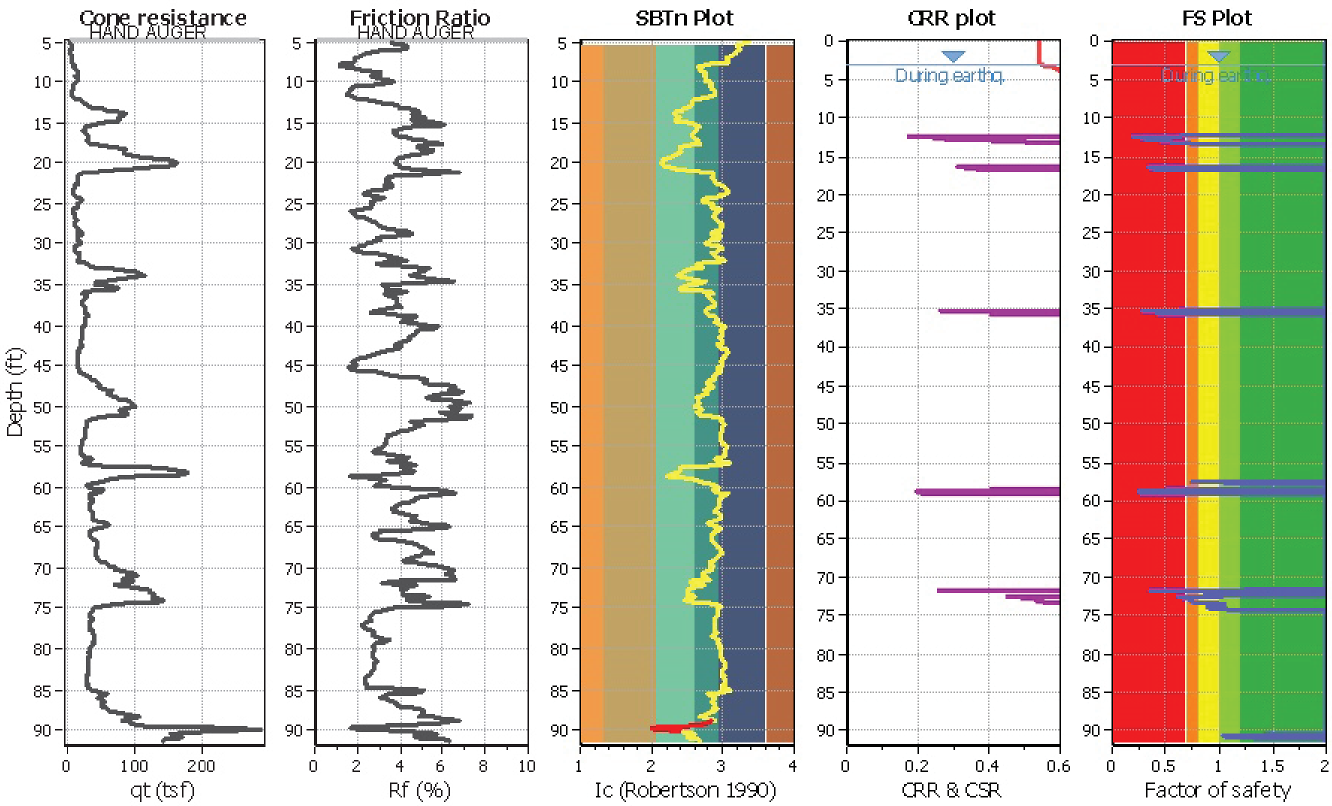
LIQUEFACTION ANALYSIS REPORT

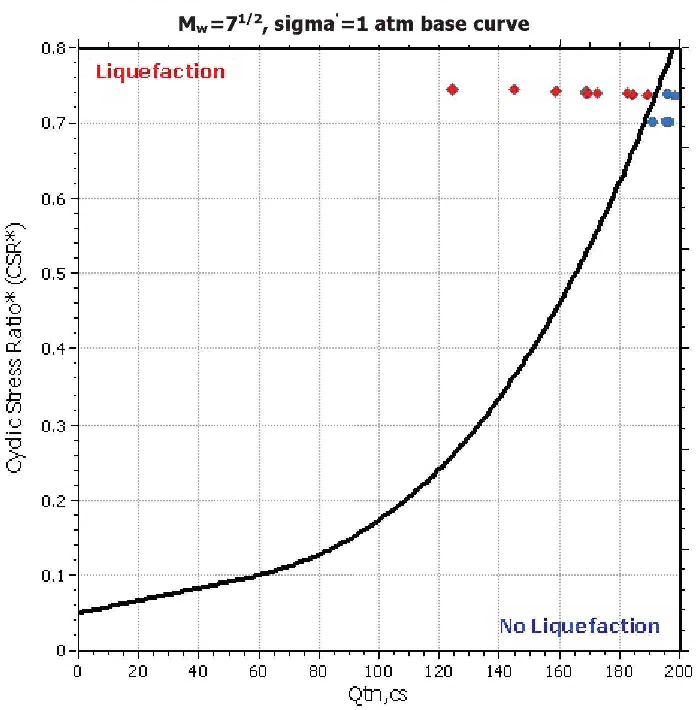
Project title : San Mateo Basin 2&3 **Location :**

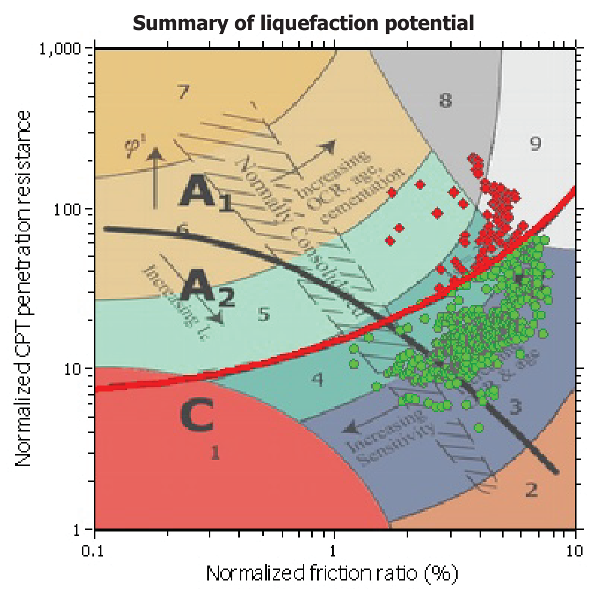
CPT file : CPT-04

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	5.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	3.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.73	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

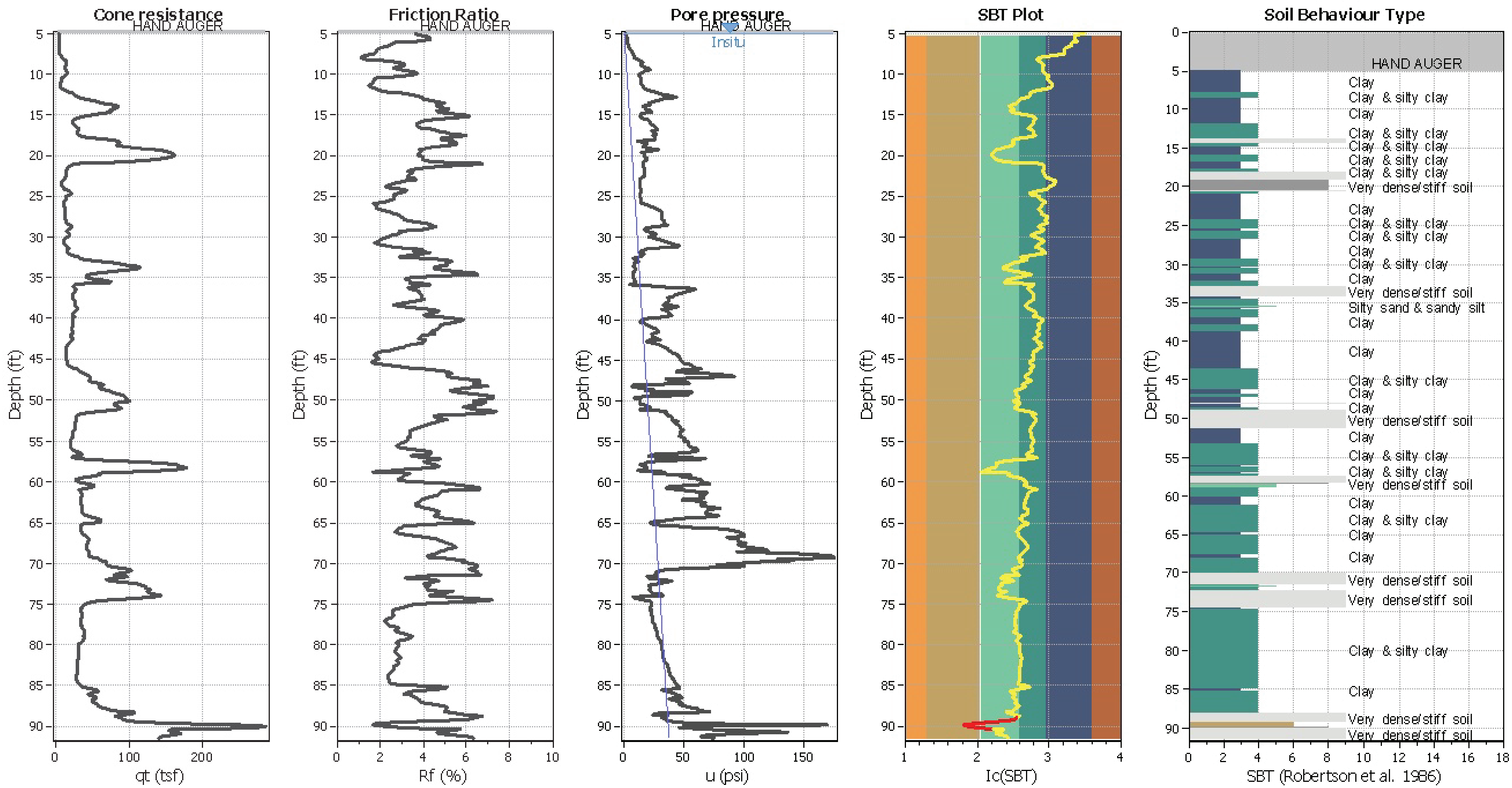






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plo



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.73	Use fill:	No	Limit depth applied:	No
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SBT legend

1. Sensitive fine grained
2. Organic material
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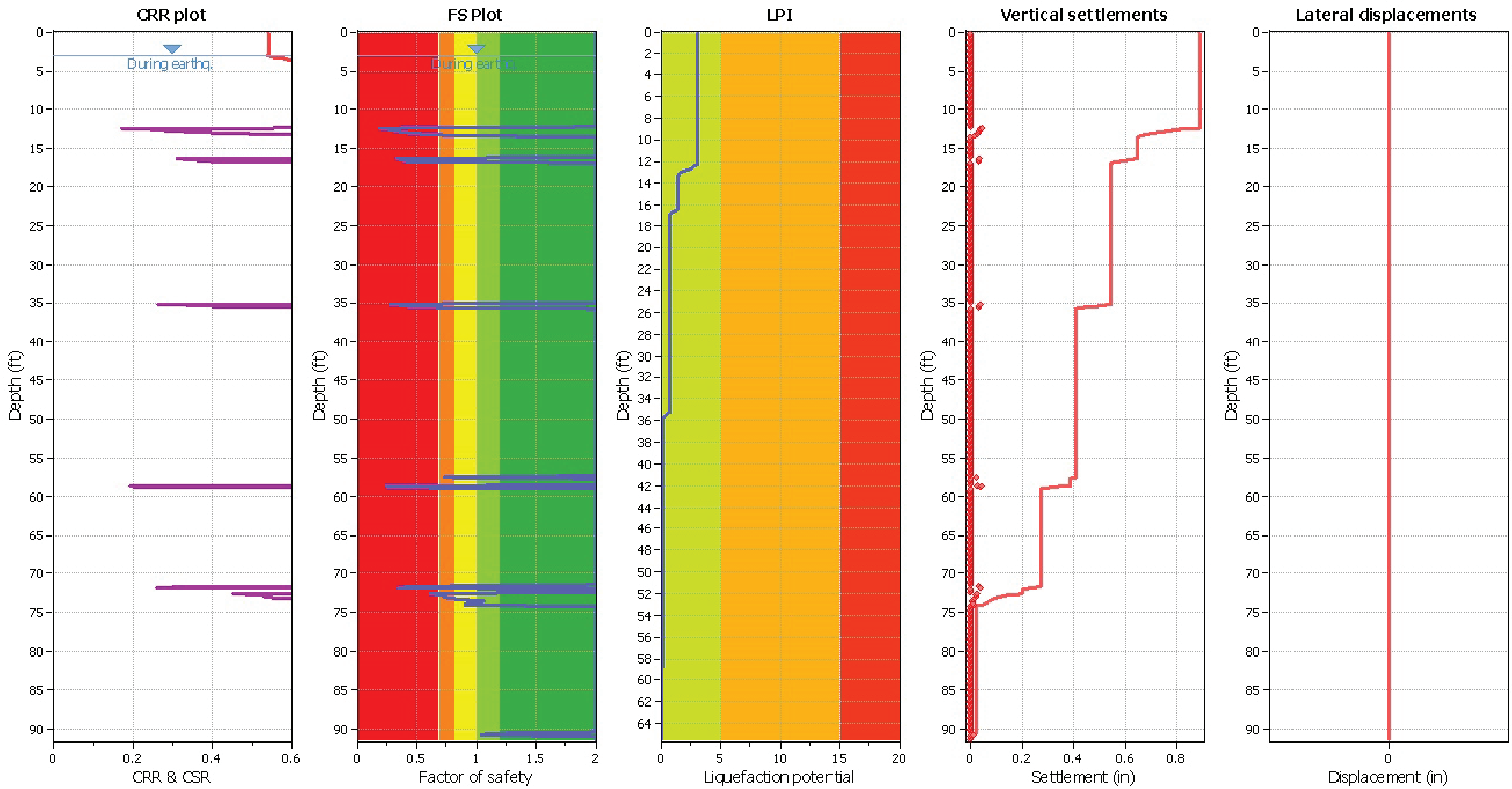
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.73	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

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

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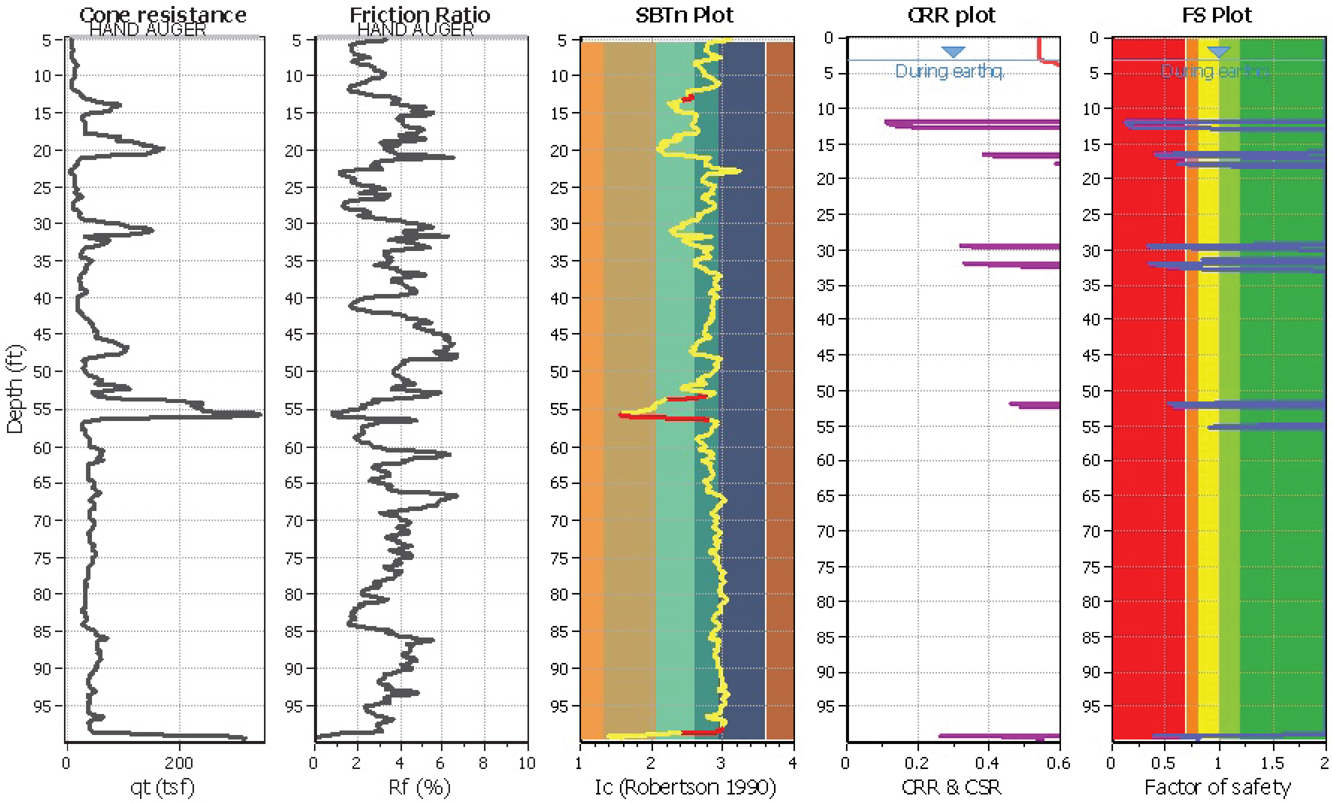
LIQUEFACTION ANALYSIS REPORT

Project title : San Mateo Basin 2&3 **Location :**

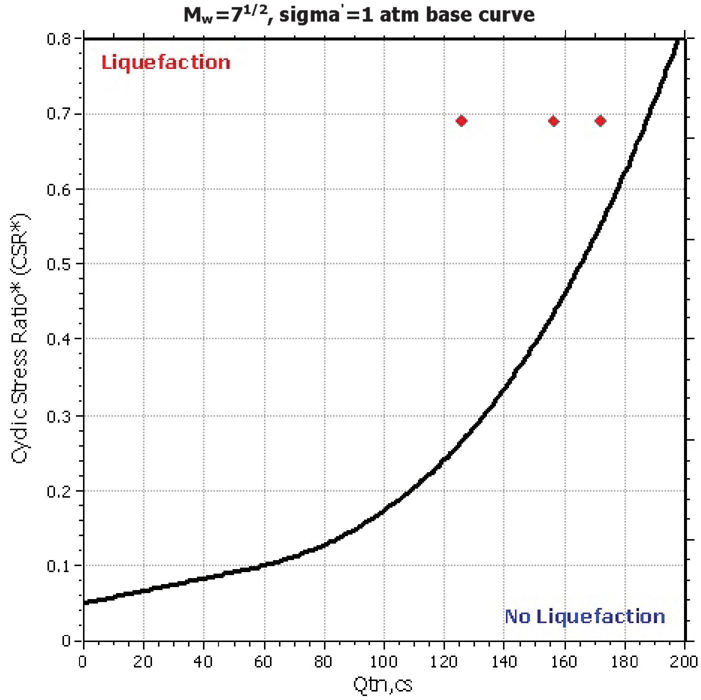
CPT file : CPT-04a

Input parameters and analysis data

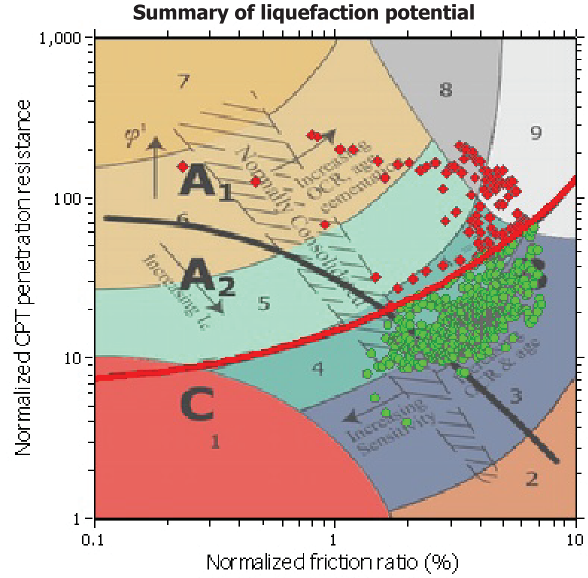
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	5.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	3.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.73	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



$M_w=7^{1/2}$, sigma'=1 atm base curve



Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
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Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
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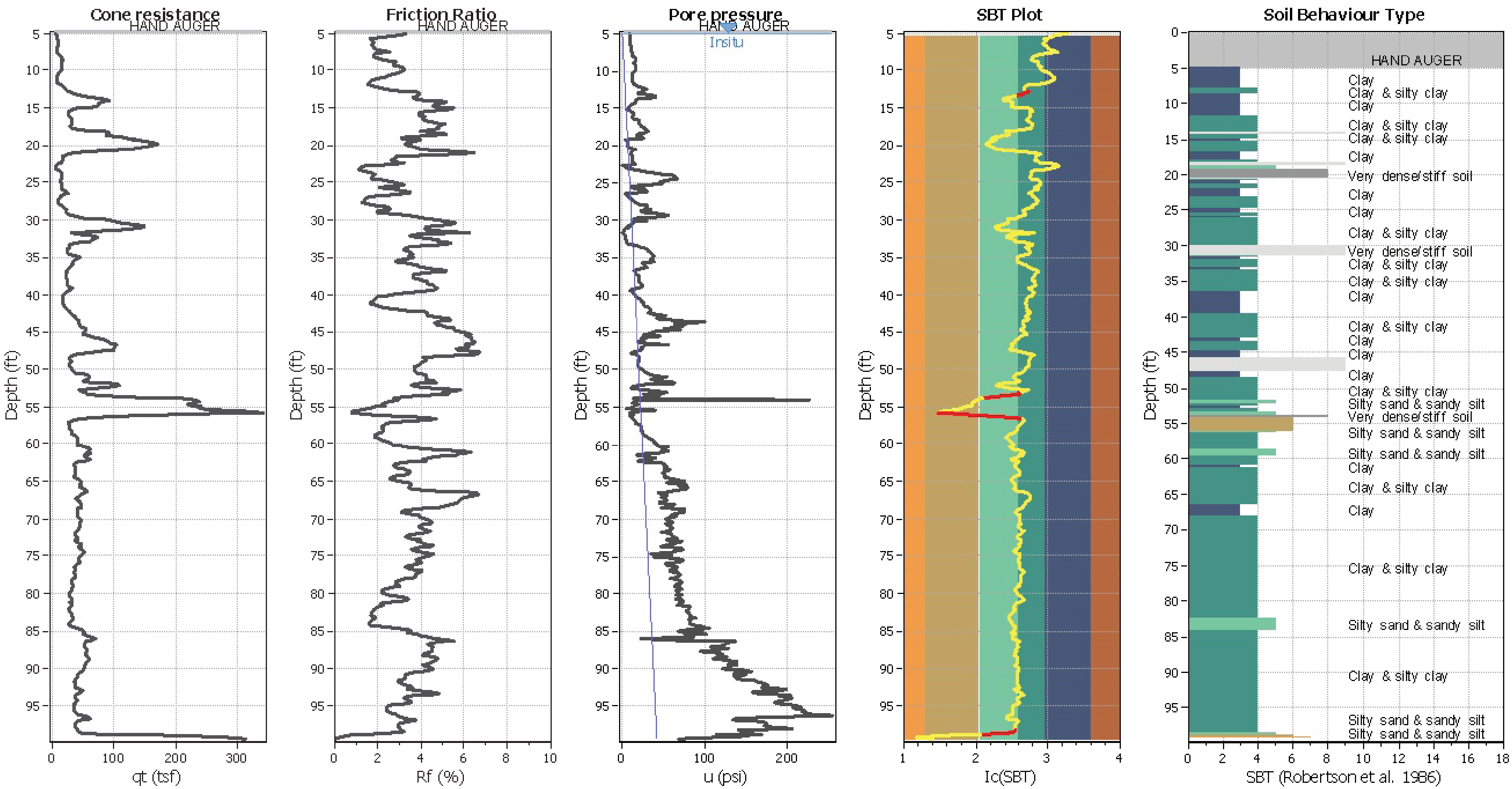
CPT basic interpretation plo



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
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Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
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SBT legend

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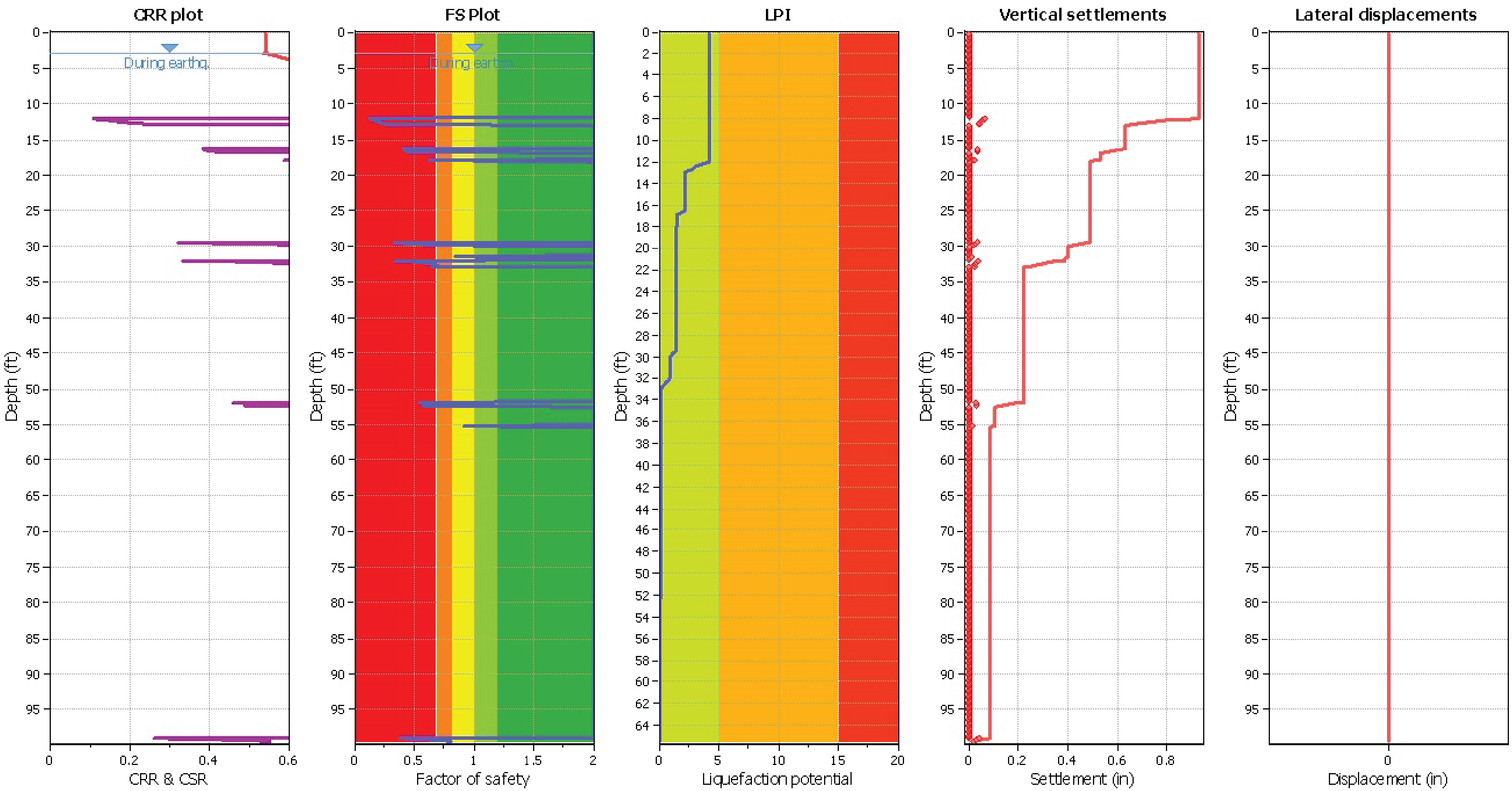
8. Very stiff sand to
9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.73	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
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LPI color scheme

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- High risk
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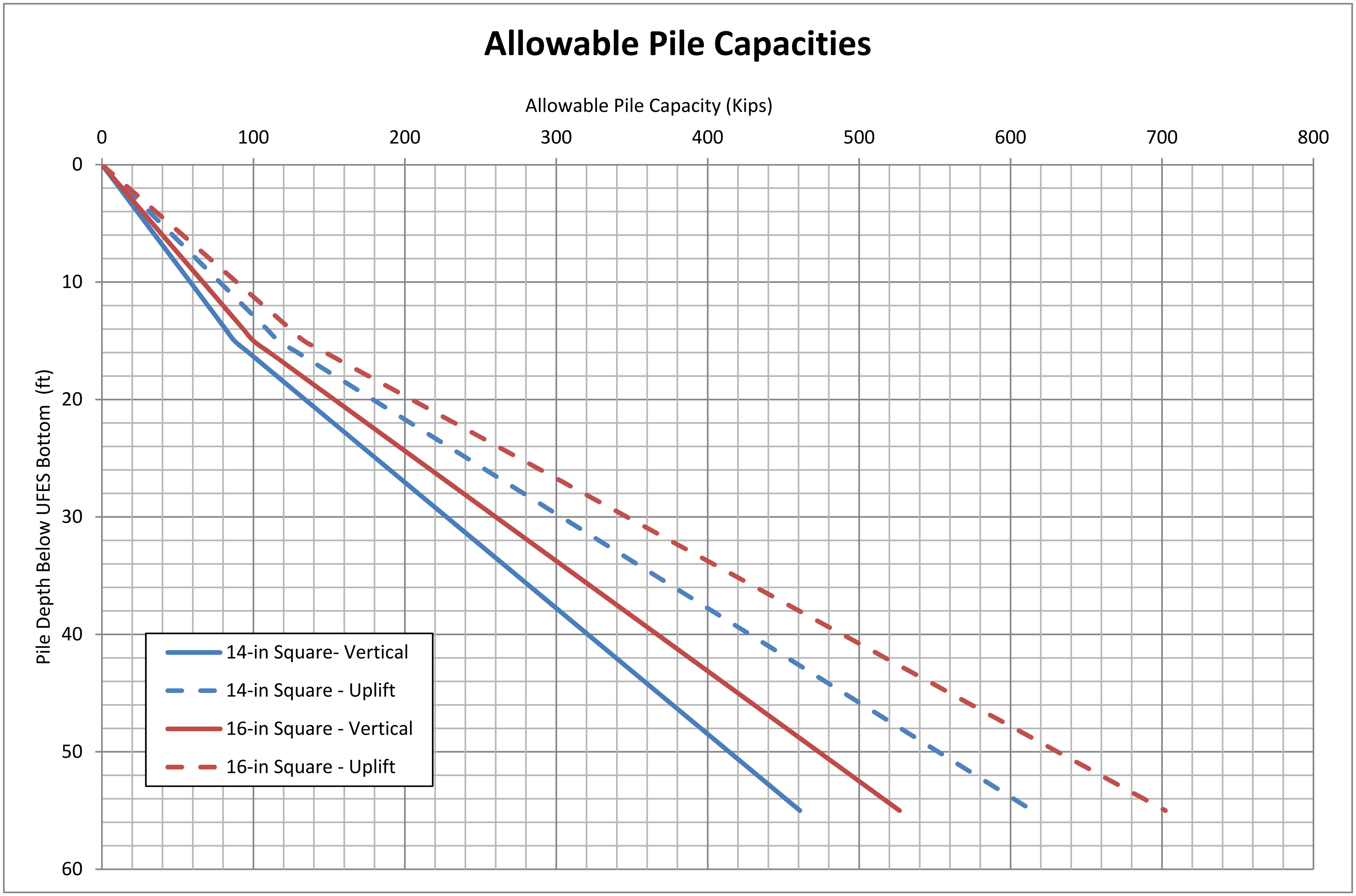
APPENDIX B

ALLOWABLE VERTICAL PILE CAPACITY CHART

Appendix B: Allowable Vertical Pile Capacitiy Chart



Note: Vertical pile capacities calculated using the alpha method, as recommended by the Federal Highway Administration (FHWA)

APPENDIX C

SUPPLEMENTAL RECOMMENDATIONS

TABLE OF CONTENTS

GENERAL INFORMATION

PREFACE

These supplemental recommendations are intended as a guide for earthwork and are in addition to any previous earthwork recommendations made by the Geotechnical Engineer. If there is a conflict between these supplemental recommendations and any previous recommendations, it should be immediately brought to the attention of ENGEO. Testing standards identified in this document shall be the most current revision (unless stated otherwise).

DEFINITIONS

BACKFILL	Soil, rock or soil-rock material used to fill excavations and trenches.
DRAWINGS	Documents approved for construction which describe the work.
THE GEOTECHNICAL ENGINEER	The project geotechnical engineering consulting firm, its employees, or its designated representatives.
ENGINEERED FILL	Fill upon which the Geotechnical Engineer has made sufficient observations and tests to confirm that the fill has been placed and compacted in accordance with geotechnical engineering recommendations.
FILL	Soil, rock, or soil-rock materials placed to raise the grades of the site or to backfill excavations.
IMPORTED MATERIAL	Soil and/or rock material which is brought to the site from offsite areas.
ONSITE MATERIAL	Soil and/or rock material which is obtained from the site.
OPTIMUM MOISTURE	Water content, percentage by dry weight, corresponding to the maximum dry density as determined by ASTM D-1557.
RELATIVE COMPACTION	The ratio, expressed as a percentage, of the in-place dry density of the fill or backfill material as compacted in the field to the maximum dry density of the same material as determined by ASTM D-1557.
SELECT MATERIAL	Onsite and/or imported material which is approved by the Geotechnical Engineer as a specific-purpose fill.

PART I - EARTHWORK

1.0 GENERAL

1.1 WORK COVERED

Supplemental recommendations for performing earthwork and grading. Activities include:

- ✓ Site Preparation and Demolition
- ✓ Excavation
- ✓ Grading
- ✓ Backfill of Excavations and Trenches
- ✓ Engineered Fill Placement, Moisture Conditioning, and Compaction

1.2 CODES AND STANDARDS

The contractor should perform their work complying with applicable occupational safety and health standards, rules, regulations, and orders. The Occupational Safety and Health Standards (OSHA) Board is the only agency authorized in the State to adopt and enforce occupational safety and health standards (Labor Code § 142 et seq.). The owner, their representative and contractor are responsible for site safety; ENGEO representatives are not responsible for site safety.

Excavating, trenching, filling, backfilling, shoring and grading work should meet the minimum requirements of the applicable Building Code, and the standards and ordinances of state and local governing authorities.

1.3 TESTING AND OBSERVATION

Site preparation, cutting and shaping, excavating, filling, and backfilling should be carried out under the testing and observation of ENGEO. ENGEO shall be retained to perform appropriate field and laboratory tests to check compliance with the recommendations. Any fill or backfill that does not meet the supplemental recommendations shall be removed and/or reworked, until the supplemental recommendations are satisfied.

Tests for compaction shall be made in accordance with test procedures outlined in ASTM D-1557, as applicable, unless other testing methods are deemed appropriate by ENGEO. These and other tests shall be performed in accordance with accepted testing procedures, subject to the engineering discretion of ENGEO.

2.0 MATERIALS

2.1 STANDARD

Materials, tools, equipment, facilities, and services as required for performing the required excavating, trenching, filling and backfilling should be furnished by the Contractor.

2.2 ENGINEERED FILL AND BACKFILL

Material to be used for engineered fill and backfill should be free from organic matter and other deleterious substances, and of such quality that it will compact thoroughly without excessive voids when watered and rolled.

Unless specified elsewhere by ENGEO, engineered fill and backfill shall be free of significant organics (soil which contains more than 3 percent organic content by weight), or any other unsatisfactory material. In addition, engineered fill and backfill shall comply with the grading requirements shown in the following table:

TABLE 2.2-1: Engineered Fill and Backfill Requirements

US STANDARD SIEVE	PERCENTAGE PASSING
3"	100
No. 4	35–100
No. 30	20–100

Earth materials to be used as engineered fill and backfill shall be cleared of debris, rubble and deleterious matter. Rocks and aggregate exceeding the maximum allowable size shall be removed from the site. Rocks of maximum dimension in excess of two-thirds of the lift thickness shall be removed from any fill material to the satisfaction of ENGEO.

ENGEO shall be immediately notified if potential hazardous materials or suspect soils exhibiting staining or odor are encountered. Work activities shall be discontinued within the area of potentially hazardous materials. ENGEO shall be notified at least 72 hours prior to the start of filling and backfilling operations. Materials to be used for filling and backfilling shall be submitted to ENGEO no less than 10 days prior to intended delivery to the site. Unless specified elsewhere by ENGEO, where conditions require the importation of low expansive fill material, the material shall be an inert, low to non-expansive soil, or soil-rock material, free of organic matter and meeting the following requirements:

TABLE 2.2-2: Imported Fill Material Requirements

	SIEVE SIZE	PERCENT PASSING
GRADATION (ASTM D-421)	2-inch	100
	#200	15 - 70
PLASTICITY (ASTM D-4318)	Plasticity Index < 12	
ORGANIC CONTENT (ASTM D-2974)	Less than 2 percent	

A sample of the proposed import material should be submitted to ENGEO no less than 10 days prior to intended delivery to the site.

2.3 SUBDRAINS

A subdrain system is an underground network of piping used to remove water from areas that collect or retain surface water or subsurface water. Subsurface water is collected by allowing water into the pipe through perforations. Subdrain systems may drain and discharge to an appropriate outlet such as storm drain, natural swales or drainage, etc.. Details for subdrain systems may vary depending on many items, including but not limited to site conditions, soil types, subdrain spacing, depth of the pipe and pervious medium, as well as pipe diameter.

2.4 PIPE

Subdrain pipe shall conform with these supplemental recommendations unless specified elsewhere by ENGEO. Perforated pipe for various depths shall be manufactured in accordance with the following requirements:

TABLE 2.4-1: Perforated Pipe Requirements

PIPE TYPE	STANDARD	TYPICAL SIZES (INCHES)	PIPE STIFFNESS (PSI)
PIPE STIFFNESS ABOVE 200 PSI (BELOW 50 FEET OF FINISHED GRADE)			
ABS SDR 15.3		4 to 6	450
PVC Schedule 80	ASTM D1785	3 to 10	530
PIPE STIFFNESS BETWEEN 100 PSI AND 150 PSI (BETWEEN 15 AND 50 FEET OF FINISHED GRADE)			
ABS SDR 23.5	ASTM D2751	4 to 6	150
PVC SDR 23.5	ASTM D3034	4 to 6	153
PVC Schedule 40	ASTM D1785	3 to 10	135
ABS Schedule 40/DWV	ASTM D1527 & D2661	3 to 10	
PIPE STIFFNESS BETWEEN 45 PSI AND 50 PSI* (BETWEEN 0 TO 15 FEET OF FINISHED GRADE)			
PVC A-2000	ASTM F949	4 to 10	50
PVC SDR 35	ASTM D3034	4 to 8	46
ABS SDR 35	ASTM D2751	4 to 8	45
Corrugated PE	AASHTO M294 Type S	4 to 10	45

***Pipe with a stiffness less than 45 psi should not be used.**

Other pipes not listed in the table above shall be submitted for review by the Geotechnical Engineer not less 72 hours before proposed use.

2.5 OUTLETS AND RISERS

Subdrain outlets and risers must be fabricated from the same material as the subdrain pipe. Outlet and riser pipe and fittings must not be perforated. Covers must be fitted and bolted into the riser pipe or elbow. Covers must seat uniformly and not be subject to rocking.

2.6 PERMEABLE MATERIAL

Permeable material shall generally conform to Caltrans Standard Specification unless specified otherwise by ENGEO. Class 2 permeable material shall comply with the gradation requirements shown in the following table.

TABLE 2.6-1: Class 2 Permeable Material Grading Requirements

SIEVE SIZES	PERCENTAGE PASSING
1"	100
3/4"	90 to 100
3/8"	40 to 100
No. 4	25 to 40
No. 8	18 to 33
No. 30	5 to 15
No. 50	0 to 7
No. 200	0 to 3

2.7 FILTER FABRIC

Filter fabric shall meet the following Minimum Average Roll Values unless specified elsewhere by ENGEO.

Grab Strength (ASTM D-4632)..180 lbs
Mass per Unit Area (ASTM D-4751) ....6 oz/yd^2
Apparent Opening Size (ASTM D-4751)........70-100 U.S. Std. Sieve
Flow Rate (ASTM D-4491)..80 gal/min/ft^2
Puncture Strength (ASTM D-4833) ....80 lbs

Areas to receive filter fabric must comply with the compaction and elevation tolerance specified for the material involved. Handle and place filter fabric under the manufacturer's instructions. Align and place filter fabric without wrinkles.

Overlap adjacent roll ends of filter fabric in accordance with manufacturer’s recommendations. The preceding roll must overlap the following roll in the direction that the permeable material is being spread. Completely replace torn or punctured sections damaged during placement or repair by placing a piece of filter fabric that is large enough to cover the damaged area and comply with the overlap specified. Cover filter fabric with the thickness of overlying material shown within 72 hours of placing the fabric.

2.8 GEOCOMPOSITE DRAINAGE

Geocomposite drainage is a prefabricated material that includes filter fabric and plastic pipe. Filter fabric must be Class A. The drain shall be of composite construction consisting of a supporting structure or drainage core material surrounded by a geotextile. The geotextile shall encapsulate the drainage core and prevent random soil intrusion into the drainage structure. The drainage core material shall consist of a three-dimensional polymeric material with a structure that permits flow along the core laterally. The core structure shall also be constructed to permit flow regardless of the water inlet surface. The drainage core shall provide support to the geotextile.

A geotextile flap shall be provided along drainage core edges. This flap shall be of sufficient width for sealing the geotextile to the adjacent drainage structure edge to prevent soil intrusion into the structure during and after installation. The geotextile shall cover the full length of the core. The geocomposite core shall be furnished with an approved method of constructing and connecting with outlet pipes. If the fabric on the geocomposite drain is torn or punctured, replace the damaged section completely. The specific drainage composite material and supplier shall be preapproved by ENGEO.

The Contractor shall submit a manufacturer's certification that the geocomposite meets the design properties and respective index criteria measured in full accordance with applicable test methods. The manufacturer's certification shall include a submittal package of documented test results that confirm the design values. In case of dispute over validity of design values, the Contractor should supply design property test data from a laboratory approved by ENGEO, to support the certified values submitted.

Geocomposite material suppliers shall provide a qualified and experienced representative onsite to assist the Contractor and ENGEO at the start of construction with directions on the use of drainage composite. If there is more than one application on a project, this criterion will apply to construction of the initial application only. The representative shall also be available on an as-needed basis, as requested by ENGEO, during construction of the remaining applications. The soil surface against which the geocomposite is to be placed shall be free of debris and inordinate irregularities that will prevent intimate contact between the soil surface and the drain.

Edge seams shall be formed by utilizing the flap of the geotextile extending from the geocomposite's edge and lapping over the top of the fabric of the adjacent course. The fabric flap shall be securely fastened to the adjacent fabric by means of plastic tape or non-water-soluble construction adhesive, as recommended by the supplier. To prevent soil intrusion, exposed edges of the geocomposite drainage core edge must be covered.

Approved backfill shall be placed immediately over the geocomposite drain. Backfill operations should be performed to not damage the geotextile surface of the drain. Also during operations, avoid excessive settlement of the backfill material. The geocomposite drain, once installed, shall not be exposed for more than 7 days prior to backfilling.

PART II - GEOGRID SOIL REINFORCEMENT

Geogrid soil reinforcement (geogrid) shall be submitted to ENGEO and should be approved before use. The geogrid shall be a regular network of integrally connected polymer tensile elements with aperture geometry sufficient to permit significant mechanical interlock with the surrounding soil or rock. The geogrid structure shall be dimensionally stable and able to retain its geometry under construction stresses and shall have high resistance to damage during construction to ultraviolet degradation and to chemical and biological degradation encountered in the soil being reinforced. The geogrids shall have an Allowable Tensile Strength (T_a) and Pullout Resistance, for the soil type(s) as specified on design plans.

The contractor shall submit a manufacturer's certification that the geogrids supplied meet plans and project specifications. The contractor shall check the geogrid upon delivery to ensure that the proper material has been received. During periods of shipment and storage, the geogrid shall be protected from temperatures greater than 140°F, mud, dirt, dust, and debris. Manufacturer's recommendations in regard to protection from direct sunlight must also be followed. At the time of installation, the geogrid will be rejected if it has defects, tears, punctures, flaws, deterioration, or damage incurred during manufacture, transportation, or storage. If approved by ENGEO, torn or punctured sections may be repaired by placing a patch over the damaged area. Any geogrid damaged during storage or installation shall be replaced by the Contractor at no additional cost to the owner.

Geogrid material suppliers shall provide a qualified and experienced representative onsite at the initiation of the project, for a minimum of three days, to assist the Contractor and ENGEO personnel at the start of construction. If there is more than one slope on a project, this criterion will apply to construction of the initial slope only. The representative shall also be available on an as-needed basis, as requested by ENGEO, during construction of the remaining slope(s). Geogrid reinforcement may be joined with mechanical connections or overlaps as recommended and approved by the manufacturer. Joints shall not be placed within 6 feet of the slope face, within 4 feet below top of slope, nor horizontally or vertically adjacent to another joint.

The geogrid reinforcement shall be installed in accordance with the manufacturer's recommendations. The geogrid reinforcement shall be placed within the layers of the compacted soil as shown on the plans or as directed. The geogrid reinforcement shall be placed in continuous longitudinal strips in the direction of main reinforcement. However, if the Contractor is unable to complete a required length with a single continuous length of geogrid, a joint may be made with the manufacturer's approval. Only one joint per length of geogrid shall be allowed. This joint shall be made for the full width of the strip by using a similar material with similar strength. Joints in geogrid reinforcement shall be pulled and held taut during fill placement.

Adjacent strips, in the case of 100 percent coverage in plan view, need not be overlapped. The minimum horizontal coverage is 50 percent, with horizontal spacing between reinforcement no greater than 40 inches. Horizontal coverage of less than 100 percent shall not be allowed unless specifically detailed in the construction drawings. Adjacent rolls of geogrid reinforcement shall be overlapped or mechanically connected where exposed in a wrap around face system, as applicable.

The Contractor may place only that amount of geogrid reinforcement required for immediately pending work to prevent undue damage. After a layer of geogrid reinforcement has been placed, the next succeeding layer of soil shall be placed and compacted as appropriate. After the specified soil layer has been placed, the next geogrid reinforcement layer shall be installed. The process shall be repeated for each subsequent layer of geogrid reinforcement and soil. Geogrid reinforcement shall be placed to lay flat and pulled tight prior to backfilling. After a layer of geogrid reinforcement has been placed, suitable means, such as pins or small piles of soil, shall be used to hold the geogrid reinforcement in position until the subsequent soil layer can be placed.

Under no circumstances shall a track-type vehicle be allowed on the geogrid reinforcement before at least 6 inches of soil have been placed. Turning of tracked vehicles should be kept to a minimum to prevent tracks from displacing the fill and the geogrid reinforcement. If approved by the Manufacturer, rubber-tired equipment may pass over the geosynthetic reinforcement at slow speeds, less than 10 mph. Sudden braking and sharp turning shall be avoided. During construction, the surface of the fill should be kept approximately horizontal. Geogrid reinforcement shall be placed directly on the compacted horizontal fill surface. Geogrid reinforcements are to be placed as shown on plans, and oriented correctly.

PART III - GEOTEXTILE SOIL REINFORCEMENT

The specific geotextile material and supplier shall be preapproved by ENGEO. The contractor shall submit a manufacturer's certification that the geotextiles supplied meet the respective index criteria set when geotextile was approved by ENGEO, measured in full accordance with specified test methods and standards.

The contractor shall check the geotextile upon delivery to ensure that the proper material has been received. During periods of shipment and storage, the geotextile shall be protected from temperatures greater than 140°F, mud, dirt, dust, and debris. Manufacturer's recommendations in regard to protection from direct sunlight must also be followed. At the time of installation, the geotextile will be rejected if it has defects, tears, punctures, flaws, deterioration, or damage incurred during manufacture, transportation, or storage. If approved by ENGEO, torn or punctured sections may be repaired by placing a patch over the damaged area. Any geotextile damaged during storage or installation shall be replaced by the Contractor at no additional cost to the owner.

Geotextile material suppliers shall provide a qualified and experienced representative onsite at the initiation of the project to assist the Contractor and ENGEO personnel at the start of construction. The geotextile reinforcement shall be installed in accordance with the manufacturer's recommendations. The geotextile reinforcement shall be placed within the layers of the compacted soil as shown on the plans or as directed, secured with staples, pins, or small piles of backfill, placed without wrinkles, and aligned with the primary strength direction perpendicular to slope contours. Cover geotextile reinforcement with backfill within the same work shift. Place at least 6 inches of backfill on the geotextile reinforcement before operating or driving equipment or vehicles over it, except those used under the conditions specified below for spreading backfill.

Adjacent strips, in the case of 100 percent coverage in plan view, need not be overlapped. The minimum horizontal coverage is 50 percent, with horizontal spacing between reinforcement no greater than 40 inches. Horizontal coverage of less than 100 percent shall not be allowed unless specifically detailed in the construction drawings. Adjacent rolls of geotextile reinforcement shall be overlapped or mechanically connected where exposed in a wraparound face system, as applicable.

The contractor may place only that amount of geotextile reinforcement required for immediately pending work to prevent undue damage. After a layer of geotextile reinforcement has been placed, the succeeding layer of soil shall be placed and compacted as appropriate. After the specified soil layer has been placed, the next geotextile reinforcement layer shall be installed. The process shall be repeated for each subsequent layer of geotextile reinforcement and soil.

Geotextile reinforcement shall be placed to lay flat and be pulled tight prior to backfilling. After a layer of geotextile reinforcement has been placed, suitable means, such as pins or small piles of soil, shall be used to hold the geotextile reinforcement in position until the subsequent soil layer can be placed. Under no circumstances shall a track-type vehicle be allowed on the geotextile reinforcement before at least six inches of soil has been placed. Turning of tracked vehicles should be kept to a minimum to prevent tracks from displacing the fill and the geotextile reinforcement. If approved by the Manufacturer, rubber-tired equipment may pass over the geotextile reinforcement as slow speeds, less than 10 mph. Sudden braking and sharp turning shall be avoided.

During construction, the surface of the fill should be kept approximately horizontal. Geotextile reinforcement shall be placed directly on the compacted horizontal fill surface. Geotextile reinforcements are to be placed within three inches of the design elevations and extend the length as shown on the elevation view unless otherwise directed by ENGEO.

Replace or repair any geotextile reinforcement damaged during construction. Grade and compact backfill to ensure the reinforcement remains taut. Geotextile soil reinforcement must be tested to the required design values using the following ASTM test methods.

TABLE III-1: Geotextile Soil Reinforcements

PROPERTY	TEST
Elongation at break, percent	ASTM D 4632
Grab breaking load, lb, 1-inch grip (min) in each direction	ASTM D 4632
Wide width tensile strength at 5 percent strain, lb/ft (min)	ASTM D 4595
Wide width tensile strength at ultimate strength, lb/ft (min)	ASTM D 4595
Tear strength, lb (min)	ASTM D 4533
Puncture strength, lb (min)	ASTM D 6241
Permittivity, sec^{-1} (min)	ASTM D 4491
Apparent opening size, inches (max)	ASTM D 4751
Ultraviolet resistance, percent (min) retained grab break load, 500 hours	ASTM D 4355

PART IV - EROSION CONTROL MAT

Work shall consist of furnishing and placing a synthetic erosion control mat and/or degradable erosion control blanket for slope face protection and lining of runoff channels. The specific erosion control material and supplier shall be pre-approved by ENGEO.

The Contractor shall submit a manufacturer's certification that the erosion mat/blanket supplied meets the criteria specified when the material was approved by ENGEO. The manufacturer's certification shall include a submittal package of documented test results that confirm the property values. Jute mesh shall consist of processed natural jute yarns woven into a matrix, and netting shall consist of coconut fiber woven into a matrix. Erosion control blankets shall be made of processed natural fibers that are mechanically, structurally, or chemically bound together to form a continuous matrix that is surrounded by two natural nets.

The Contractor shall check the erosion control material upon delivery to ensure that the proper material has been received. During periods of shipment and storage, the erosion mat shall be protected from temperatures greater than 140°F, mud, dirt, and debris. Manufacturer's recommendations in regard to protection from direct sunlight must also be followed. At the time of installation, the erosion mat/blanket shall be rejected if it has defects, tears, punctures, flaws, deterioration, or damage incurred during manufacture, transportation, or storage. If approved by ENGEO, torn or punctured sections may be removed by cutting out a section of the mat. The remaining ends should be overlapped and secured with ground anchors. Any erosion mat/blanket damaged during storage or installation shall be replaced by the Contractor at no additional cost to the Owner.

Erosion control material suppliers shall provide a qualified and experienced representative onsite, to assist the Contractor and ENGEO personnel at the start of construction. If there is more than one slope on a project, this criterion will apply to construction of the initial slope only. The representative shall be available on an as-needed basis, as requested by ENGEO, during construction of the remaining slope(s). The erosion control material shall be placed and anchored on a smooth graded, firm surface approved by the Engineer. Anchoring terminal ends of the erosion control material shall be accomplished through use of key trenches. The material in the trenches shall be anchored to the soil on maximum 1½-foot centers. Topsoil, if required by construction drawings, placed over final grade prior to installation of the erosion control material shall be limited to a depth not exceeding 3 inches.

Erosion control material shall be anchored, overlapped, and otherwise constructed to ensure performance until vegetation is well established. Anchors shall be as designated on the construction drawings, with a minimum of 12-inch length, and shall be spaced as designated on the construction drawings, with a maximum spacing of 4 feet.

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