WAYNE TING & ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

Project No. 6018
14 February 2022

Mr. Yi Zhang
VeGuard USA LLC
960 N California Ave,
Palo Alto, CA 94303

Subject: **GEOTECHNICAL INVESTIGATION**
Proposed 24 Townhouses
3378-3386 El Camino Real
Santa Clara, California

References:
1. Guidelines for Evaluating and Mitigating Seismic Hazards in California Special Publication 117A, Division of Mines and Geology, 2008.
2. Recommendation Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California, By ASCE Los Angeles Section Geotechnical Group Dated 1999.
3. Seismic Hazard Zone Report for the San Jose West 7.5 Minute Quadrangle, Santa Clara County, California, 2002.

Dear Mr. Zhang:

In accordance with your authorization, **Wayne Ting & Associates, Inc. (WTAI)** has completed a geotechnical investigation for the proposed 24 townhouses at the subject site. The purpose of this study was to investigate the subsurface conditions and obtain geotechnical data for use in the design and construction of the proposed 24 townhouses. The scope of this investigation included the following:

a. A site and area reconnaissance by the Project Engineer.
b. An excavation, logging, and sampling of two exploratory CPT borings.
c. Laboratory testing of selected soil samples.
d. An engineering analysis of the data and information obtained.
e. Preparation and writing of this report which presents our findings, conclusions, and recommendations.

• 42329 Osgood Road, Unit A, Fremont, CA 94539 • Tel: (510) 623-7768 • Email: wayne@wayneting.net

SITE LOCATION AND DESCRIPTION

The subject site is located at 3378-3386 El Camino Real, Santa Clara, California. The property is located to the south of El Camino Real. The property is adjacent to commercial facilities and single-family homes. The ground surface is relatively flat throughout the property. An existing commercial structure are present on the property at the time of our investigation.

PROPOSED PROJECT

The proposed project consists of constructing 24 townhouses. It is our understanding that the existing structure will be demolished. We anticipate that the proposed structure will utilize wood-framed construction. Light to moderate building loads are typically associated with this type of construction.

FIELD INVESTIGATION

WTAI conducted the field investigation on January 13, 2021. The field investigation consisted of a site reconnaissance by the Project Engineer and an excavation of two CPT borings. The approximate location of the borings are shown on the Site Plan, Figure 1. The results of the CPT borings can be found in Appendix A.

LABORATORY TESTING

ATTERBERG LIMITS

The Atterberg Limits Test was determined for the selected soil sample to classify, as well as to obtain an indication of the expansion and shrinkage potential with respect to moisture content variations. The liquid limit and plasticity index of the soil were found to be:

Sample	*Liquid Limit*	*Plasticity Index*
Gray silty clay	41%	23

The Atterberg Limits tests indicate that a representative sample of the soil is of moderate plasticity. The expansion potential for these soils is thus moderate.

SUBSURFACE SOIL CONDITIONS

The following soil descriptions were derived from our site reconnaissance and information obtained from our exploratory CPT borings. Detailed description of the materials is presented in the Appendix A.

Boring 1 soils encountered at the site consisted of 26.0 feet of clayey silt to silty clay, stiff to very stiff, followed by followed by silty sand, medium dense, to the depth of 33.0 feet below ground surface, followed by clayey silt to silty clay, very stiff, to the maximum depth explored of 45.0 feet below the ground surface.

Boring 2 soils encountered at the site consisted of 25.0 feet of silty clay, stiff to very stiff, followed by followed by silty sand, medium dense, to the depth of 32.0 feet below ground surface, followed by clayey silt to silty clay, very stiff, to the depth of 35.0 feet below ground surface, followed by silty sand, medium dense, to the depth of 43.0 feet below ground surface, followed by silty clay, very stiff to the maximum depth explored of 45.0 feet below the ground surface.

Quaternary geologic map, Plate 1.1 of Reference 1 shows soils under this site are Holocene alluvial fan deposits (Qhf).

Groundwater was encountered at 24.0 feet in the exploratory CPT borings at the time of our field study. Fluctuations in the groundwater table are anticipated to vary with respect to seasonal rainfall.

SEISMIC CONSIDERATIONS

According to the published maps by the International Conference of Building Officials (I.C.B.O.), in February 1998, the distances from active faults to the subject site are listed in the following table.

Fault Name	Distance (kilometers)	Direction From Site
Monte Vista	8.2	Southwest
San Andreas	14.1	Southwest

Damage resulting from earthquakes is not necessarily related directly to the distance from the fault. More important than distance, are the foundation materials upon which structures are to be built. If structures are not located across the trace of the fault, are located on structurally competent materials, and are designed with state-of-the-art seismic considerations, the probability of continued usefulness after an earthquake is relatively good.

CALIFORNIA BUILDING CODE SITE CHARACTERIZATION

The following design values are base on the geologic information, longitude and latitude of the site, and the USGS computer program. Furthermore, in accordance with California Building Code 2019 (ASCE 7-16), the site seismic design values are provided as follow:

CBC Category/Coefficient ASCE 7-16	Design Value
Short-Period MCE at 0.2s, Ss	1.500
1.0s Period MCE, S1	0.600
Soil Profile Type, Site Class	D
Site Coefficient, Fa:	1.0
Site Coefficient, Fv:	null or See section 11.4.8 or 1.7
S_{MS} = Spectral Response Accelerations	1.500
S_{M1} = Spectral Response Accelerations	null or See section 11.4.8
S_{DS} = Design Spectral Response Accelerations	1.000
S_{D1} = Design Spectral Response Accelerations	null or See section 11.4.8
** Latitude: 37.3519809 Longitude: -122.9889912	

It is noted that final values should be determined by the project structural engineer according to site class, risk categories of the proposed structures, and ASCE 7-16 Table 11.4-1 and 11.4-2.

QUANTITATIVE LIQUEFACTION ANALYSIS USING CPT

Soil liquefaction is a phenomenon in which saturated (submerged) cohesionless soils can be subjected to a temporary loss of strength due to the buildup pore water pressures, especially as a result of cyclic loadings such as induced by earthquakes. In the process, the soil acquires a mobility sufficient to permit both horizontal and vertical deformations, if not confined. Soils that are most susceptible to liquefaction are clean, loose, saturated, uniformly graded, fine sands.

Our liquefaction analysis followed the methods presented by the 1998 NCEER Workshops (Youd et al., 2001) in accordance with guidelines set forth in the CGS Special Publication 117A (2008). The NCEER methods for CPT analysis update simplified procedures presented by Seed and Idriss (1971). These methods are used to calculate a factor of safety against liquefaction triggering by comparing the resistance of the soil to cyclic shaking to the seismic demand that can be caused during seismic events.

The resistance to cyclic shaking is quantified by the Cyclic Resistance Ratio (CRR), which is a function of soil density, layer depth, ground water depth, earthquake magnitude, and soil behavior. Our CPT tip pressures were corrected for the overburden and fines content. The CPT method utilizes the soil behavior type index (I_C) and the exponential factor, n, applied to the Normalized Cone Resistance, Q, to evaluate how plastic the soil behaves. The Cyclic Stress Ratio (CSR) is used to quantify the stresses that are anticipated to develop during cyclic shaking. The formula for CSR is shown below:

$$CSR = 0.65\ (a_{max}/g)(s_{vo}/s_{vo})r_d$$

Where a_{max} is the peak horizontal acceleration at the ground surface generated by an earthquake, g, is the acceleration of gravity, s_{vo} and s_{vo} are total and effective overburden stresses, respectively, and r_d is a stress reduction coefficient. We evaluated the liquefaction potential of the sand strata encountered with no ground water. In addition, a peak ground acceleration (PGAm) of 0.623g (PSHA, 10% exceedance in 50 years) and magnitude of 7.9 was obtained from Reference 3 for analysis.

According to the ground water data presented on Plate 1.2 of Reference 3, the historic ground water may be approximately 15.0 feet below the ground surface. In addition, according to the CPT test, groundwater was at approximately between 24.0 feet below the ground surface. It is noted that fluctuations in the groundwater table are anticipated to vary with respect to seasonal rainfall.

We analyzed the site liquefaction potential utilizing a computer program call GeoSuite by GeoAdvanced; this program is based on the most recent publications of NCEER Workshop and procedure outline in SP117A Implementation.

Based on our analysis using Boulanger & Idriss (2010-16), the settlement results of the liquefaction analysis are presented in following Table 1 and in Appendix A.

TABLE 1

CPT Boring Nos.	*Historical Ground Water Depth (feet)*	*Dry Settlement (inches)*	*Saturated Settlement (inches)*	*Total Settlement (inches)*	*Differential Settlement (inches)*
1	15	0.56	0.55	1.11	0.74
2	15	0	0.34	0.34	0.23

Total Settlement: Saturated settlement plus dry settlement.

Estimates of volumetric change for dry settlement were made by UCLA (2008-2014). Estimates of volumetric change for saturated settlement were made by Idriss & Boulanger (2008). As discussed in the Southern California Earthquake Center report (SCEC, 1999), differential movement for level ground, deep soil sites, will be on the order of 2/3 the total estimated settlement.

LATERAL SPREADING

Lateral spreading typically occurs as a form of horizontal displacement of relatively flat-lying alluvium material toward an open face such as an open body of water, channel, or excavation. In soils this

movement is generally due to failure along a weak plane, and may often be associated with liquefaction. As cracks develop within the weakened material, blocks of soils displace laterally toward the open face. Cracking and lateral movement may gradually propagate away from the face as blocks continue to break free. Generally, failure in this mode is analytically unpredictable, since it is difficult to determine where the first tension crack will occur.

Nearest open face to the project site is Calabazas Creek which is located at a lateral distance of 0.25km. The depth of creek is approximately 15.0 feet. Based on the liquefaction analysis, it is noted that the lateral displacement occurring at the site is low.

DISCUSSIONS, CONCLUSIONS AND RECOMMENDATIONS

1. Based on the results of our investigation, WTAI concludes that the subject site is geotechnically suitable for the proposed 24 townhouses provided the recommendations presented in this report are incorporated into the project plans and specifications.

2. WTAI should review the grading and foundation plans and specifications so that comments can be made regarding the interpretation and implementation of our geotechnical recommendations in the design and specifications.

3. It is recommended that WTAI be retained for observation during foundation construction phases to help determine that the design requirements are fulfilled. Our firm should be notified at least two working days prior to grading and/or foundation operations on the property.

4. Any work related to the grading and foundation operations performed without the direct observation of WTAI will invalidate the recommendations of this report.

5. The recommendations given in this report are applicable only for the design of the previously described 24 townhouses and only at the location indicated on the site plan. They should not be used for any other purpose.

SITE PREPARATION AND GRADING

6. Prior to grading, the proposed structure area should be cleared of all obstructions and deleterious materials. After clearing, these areas should be stripped of all organic topsoil. The predominantly organic materials generated from the stripping should be removed from the site.

7. After the organic topsoil has been stripped, the proposed pad area can be excavated. The top 10 inches of the subgrade soil should be scarified, watered or aerated as necessary to bring the soil to about 5 percent above the optimum moisture content. The subgrade should then be uniformly recompacted to at least 90 percent relative compaction. Relative compaction is based on the maximum dry density as determined by ASTM D1557 Latest Version Laboratory Test Procedure.

FOUNDATION

8. Due to the potential liquefaction settlement due to an earthquake and moderately expansive clay, the proposed 24 townhouses can be satisfactorily supported on a mat slab foundation.

9. The proposed townhouses can each be satisfactorily supported on a mat slab foundation. A modulus of subgrade reaction of 50 k.c.f. may be used in the mat slab foundation design. The slabs should be designed based on the allowable bearing capacity of 2,100 p.s.f. due to dead loads plus design live loads, and 2,800 p.s.f. due to all loads which include wind or seismic forces. The edges of the slab should extend a minimum of 12 inches below the bottom of the slab.

10. The available resistance to lateral loads when utilizing a mat slab is limited to the sliding resistance along the base of the slab. Sliding resistance between the bottom of the slab and the underlying soil should be based on a friction value of 0.30. Passive earth pressure may be computed as an equivalent fluid weighing of 200 p.c.f. below the first 12 inches of the bottom of the mat slab.

11. Settlements under the anticipated building loads are expected to be within tolerable limits for the proposed townhouses. We estimate that the total settlement will be less than 2.0-inches, and post-construction differential settlements across townhouses should not exceed approximately 1.5-inches during the life of the each structure following construction. ***The proposed townhouses should be designed so that each structure can withstand 1.50-inches of differential movement across each perimeter length of the townhouses.***

CONCRETE SLABS ON GRADE

12. To reduce the potential cracking of the concrete slabs, the following recommendations are made:

a. Any concrete slab-on-grade should be underlain by at least 4 inches of clean crushed, 3/4-inch size rock, to act as a cushion and capillary break between the subgrade soil and the slab. In addition, an additional 18 inches of soil under the crushed rock should be overexcavated and replaced with class II baserock and compacted to a minimum relative compaction of 95 percent.

b. In areas where moisture transmission through slabs is undesirable, a membrane such as, Bituthene, Paraseal or equal may be installed according to the instruction of the manufacturer. Design of waterproofing for any slab-on-grade is not within the scope of work of WTAI. Waterproofing should be designed by a professional waterproofing designer.

PAVEMENTS

13. Prior to the construction of the proposed street, the upper 10 inches of subgrade soil should be scarified and recompacted to 95% of the maximum dry density at 2% above the optimum moisture value.

14. After compaction of the subgrade, aggregate base material should be placed and also compacted to a minimum relative compaction of 95% at 2% above the optimum moisture content. The Class II aggregate base should conform to the requirements of Standard Specifications of Caltrans.

15. Pavement Sections: The recommended pavement sections are 3.0 inches asphaltic concrete over 18 inches Class II baserock. As an alternative, a pervious paver system may be utilized. The pervious paver system should be designed by the Project Civil Engineer.

TRENCH BACKFILL

16. Backfilling and compaction of utility trenches must meet the requirements published by the City of Santa Clara, Department of Public Works. All trench backfill under pavement areas must be backfilled with native or imported soil and compacted to at least 90% relative compaction as determined by ASTM D1557 Latest Version Laboratory Test Procedure. The top 12 inches of the subgrade should be compacted to 95%.

17. The backfill of utility trenches extending under the building and landscaping area should be properly compacted to ensure against water migration underneath the structure.

18. Specific excavation considerations are beyond the scope of this report. However, stable excavations over 5 feet deep for utility construction will require a temporary stable cut slope and/or proper shoring. Proper shoring and stable cut slope construction should be in accordance with the Occupational Safety and Health Administration (OSHA) requirements as well as other applicable building code requirements.

GENERAL CONSTRUCTION REQUIREMENTS

19. All finished grading must be adjusted to provide positive drainage away from the structure to prevent ponding of water toward the building.

20. All roof drains should be collected by a system of gutters and downspouts and discharged to a solid pipe system to carry storm water away from the building structure.

21. Flowerbeds and planting are not recommended along the building perimeter. Only drip systems can be installed where they may cause saturation of the foundation soils. Landscape mounds or concrete flatwork should not block or obstruct the surface drainage measures.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

22. Our client should recognize that this report is prepared for the exclusive use of the proposed 24 townhouses. Our professional services, findings, and recommendations were prepared in accordance with generally accepted engineering principles and practices. No other warranty, expressed or implied, is made.

23. The conclusions and recommendations contained in this report will not be considered valid after a period of two years unless the changes are reviewed, and the conclusions of this report are modified or verified in writing.

24. This report is issued with the understanding that it is the responsibility of the owner, or his representative, to ensure the information and recommendations contained in this report are brought to the attention of the Architect, Engineer, and Contractor. In all cases, the contractor shall retain responsibility for the quality of the work and for repairing defects regardless of when they are found. It is also the responsibility of the contractor for conforming to the project plans and specifications.

Should you have any questions relating to the contents of this report, please contact our office at your convenience.

Very truly yours,

WAYNE TING & ASSOCIATES, INC.

Tyler Brown, C.E.
Project Engineer

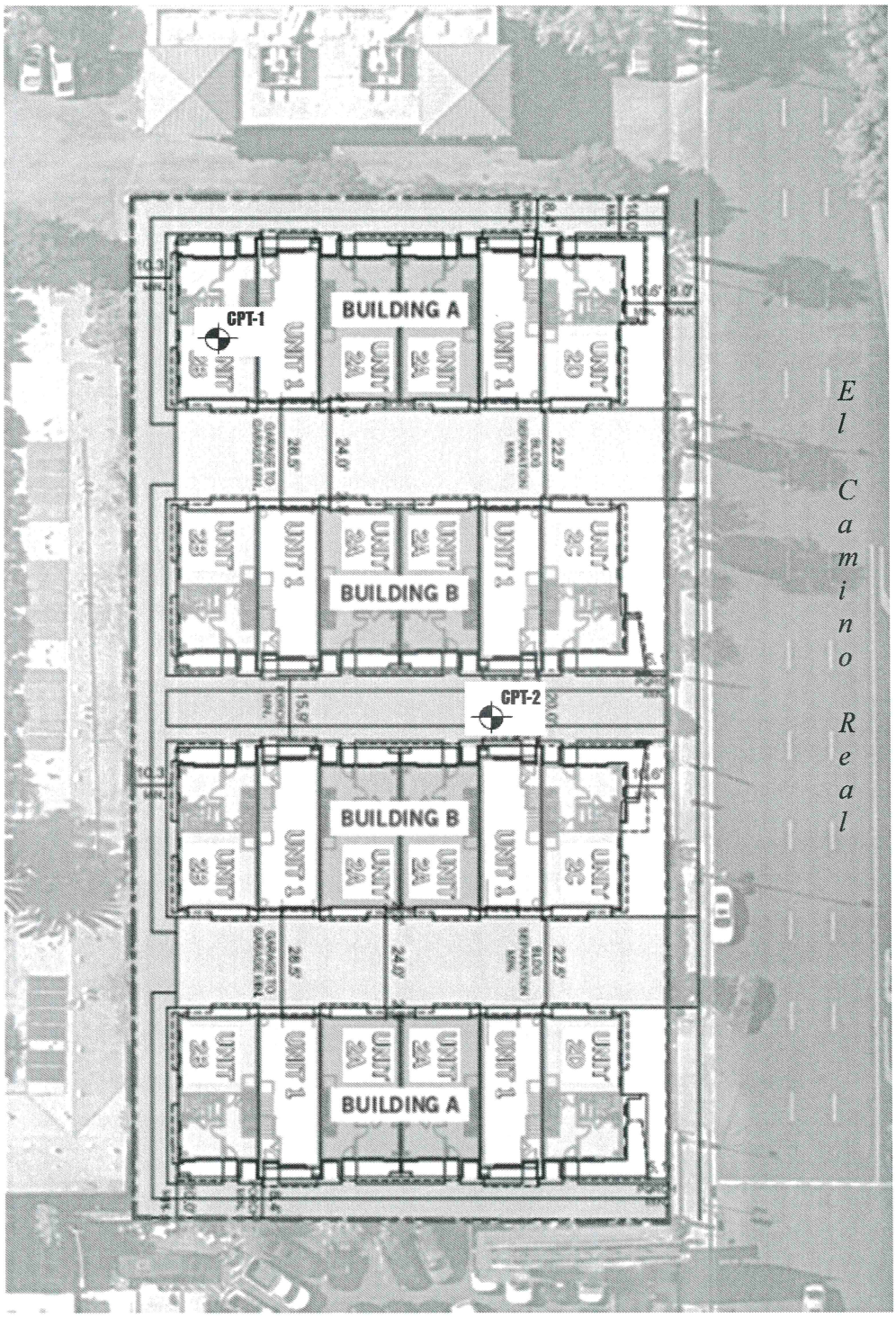
Copy: 1 to Mr. Zhang

APPENDIX A

Site Plan, Figure 1

CPT Boring Log, Figures 2 and 3

Liquefaction Analysis



Approximate Boring Location

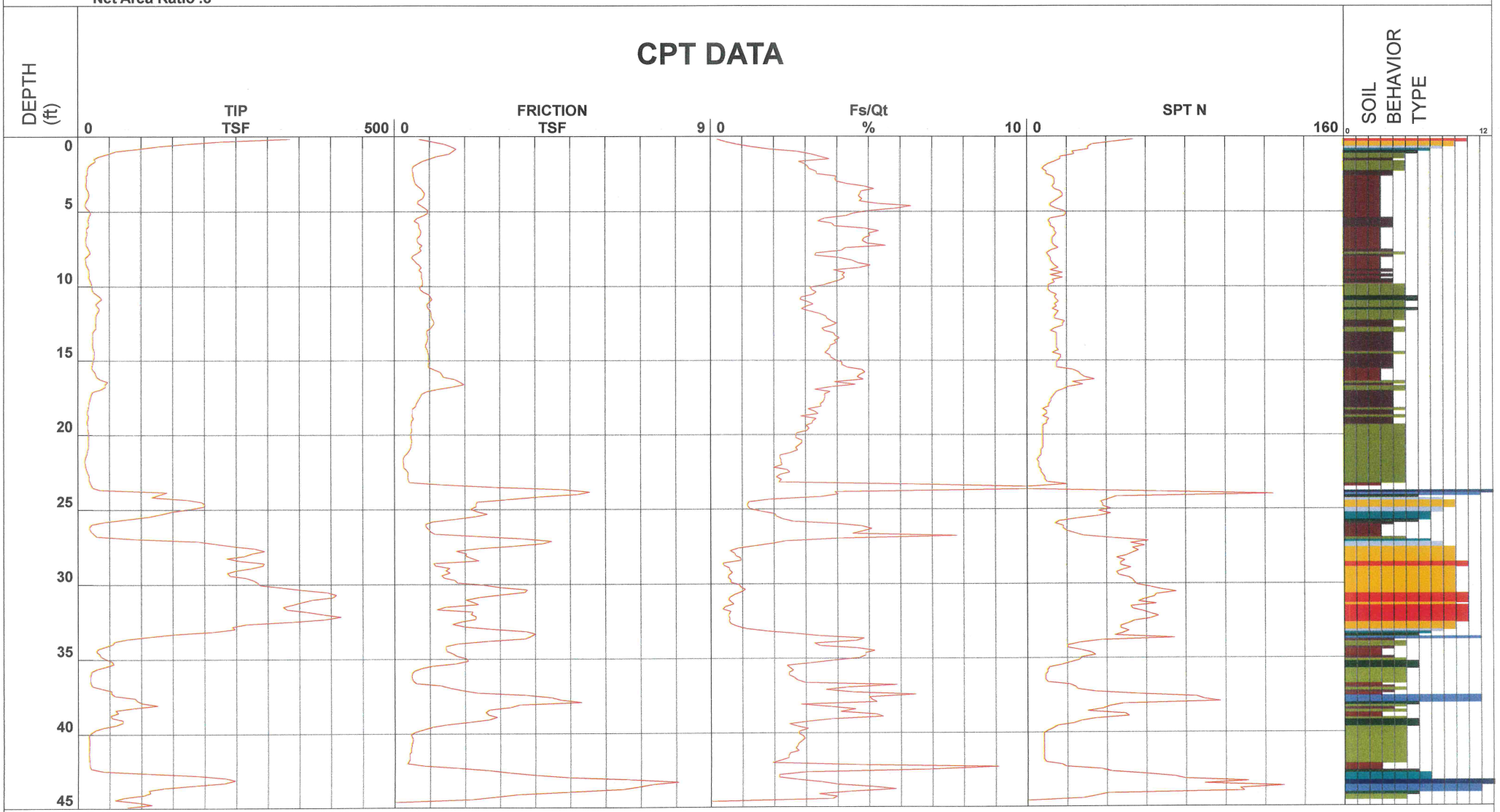
WAYNE TING & ASSOCIATES, INC.	***SITE PLAN***	*Figure No. 1*
GEOTECHNICAL CONSULTANTS	Scale : N/A	*Page No. 11*

Wayne Ting & Associates

Project	El Camino Real	Operator	AJ-OO	Filename	SDF(350).cpt
Job Number	6018	Cone Number	DDG1587	GPS	
Hole Number	CPT-01	Date and Time	1/13/2022 1:30:11 PM	Maximum Depth	44.95 ft
EST GW Depth During Test		24.00 ft			

Net Area Ratio .8

CPT DATA



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

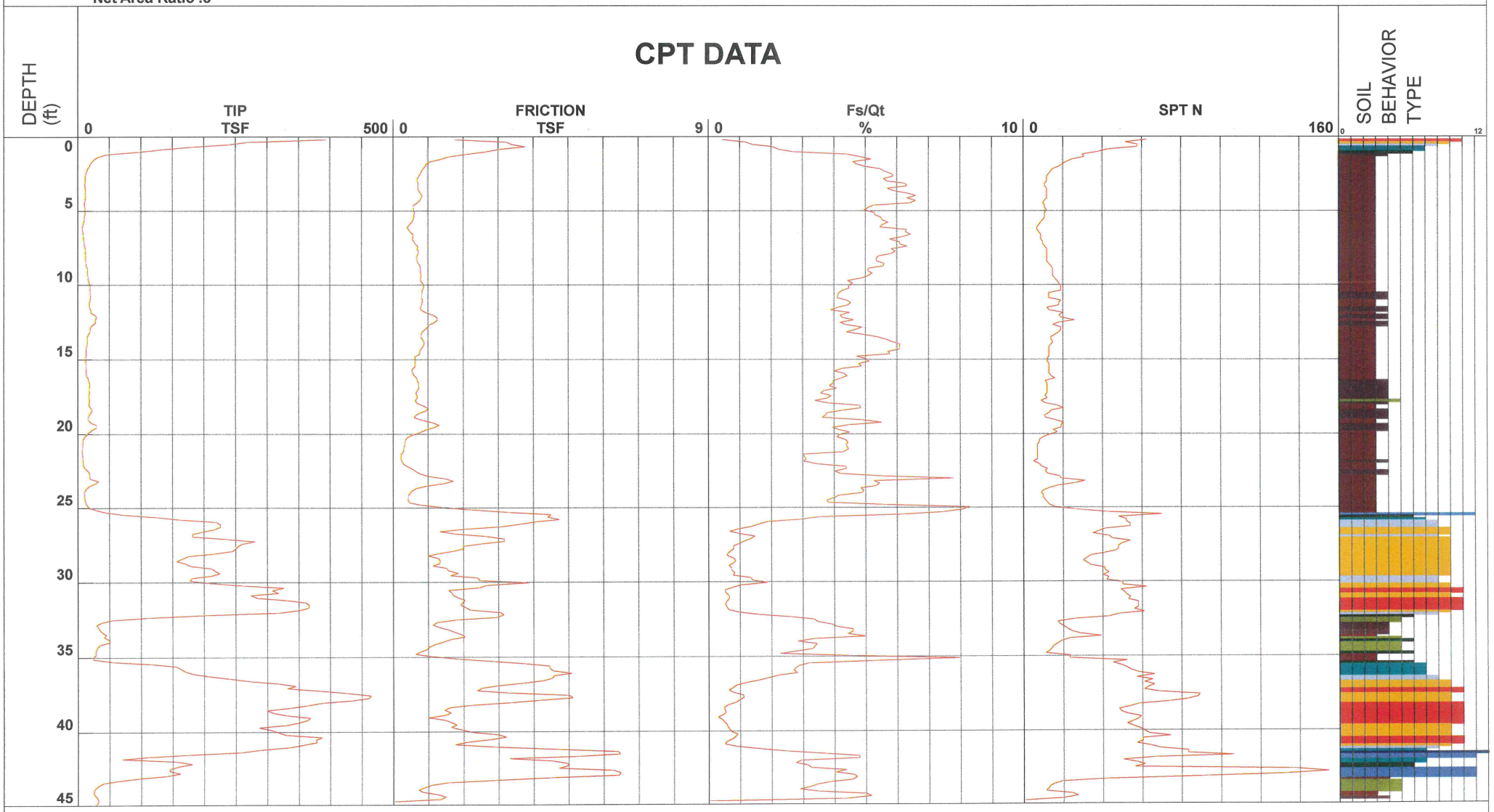
Cone Size 15cm squared

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

Wayne Ting & Associates

Project	El Camino Real	Operator	AJ-OO	Filename	SDF(351).cpt
Job Number	6018	Cone Number	DDG1587	GPS	
Hole Number	CPT-02	Date and Time	1/13/2022 2:12:27 PM	Maximum Depth	45.11 ft
EST GW Depth During Test		24.00 ft			

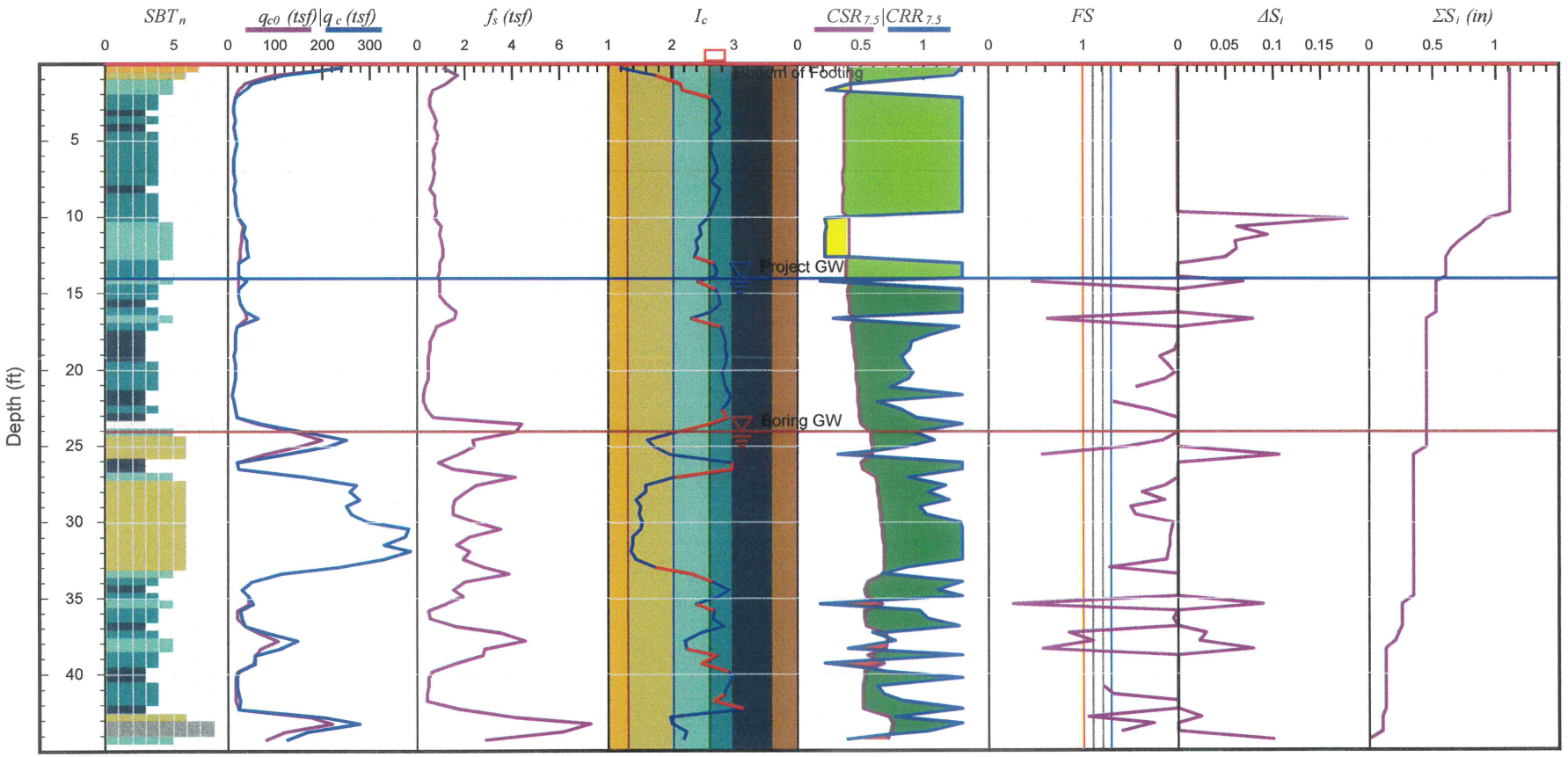
Net Area Ratio .8



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

Cone Size 15cm squared

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



- Sensitive fine grained
- Organic soils - peats
- Clay to silty clay
- Silty clay to clayey silt
- Sandy silt to silty sand
- Silty sand to clean sand
- Dense sand to gravelly sand
- Clayey sand to very stiff sand
- Very stiff fine grained *

* Overconsolidated or cemented

Earthquake & Groundwater Information:
Magnitude = 7.9
Max. Acceleration = 0.623 g
Project GW = 14 ft
Maximum Settlement = 1.11 in
Settl. at Bottom of Footing = 1.11 in

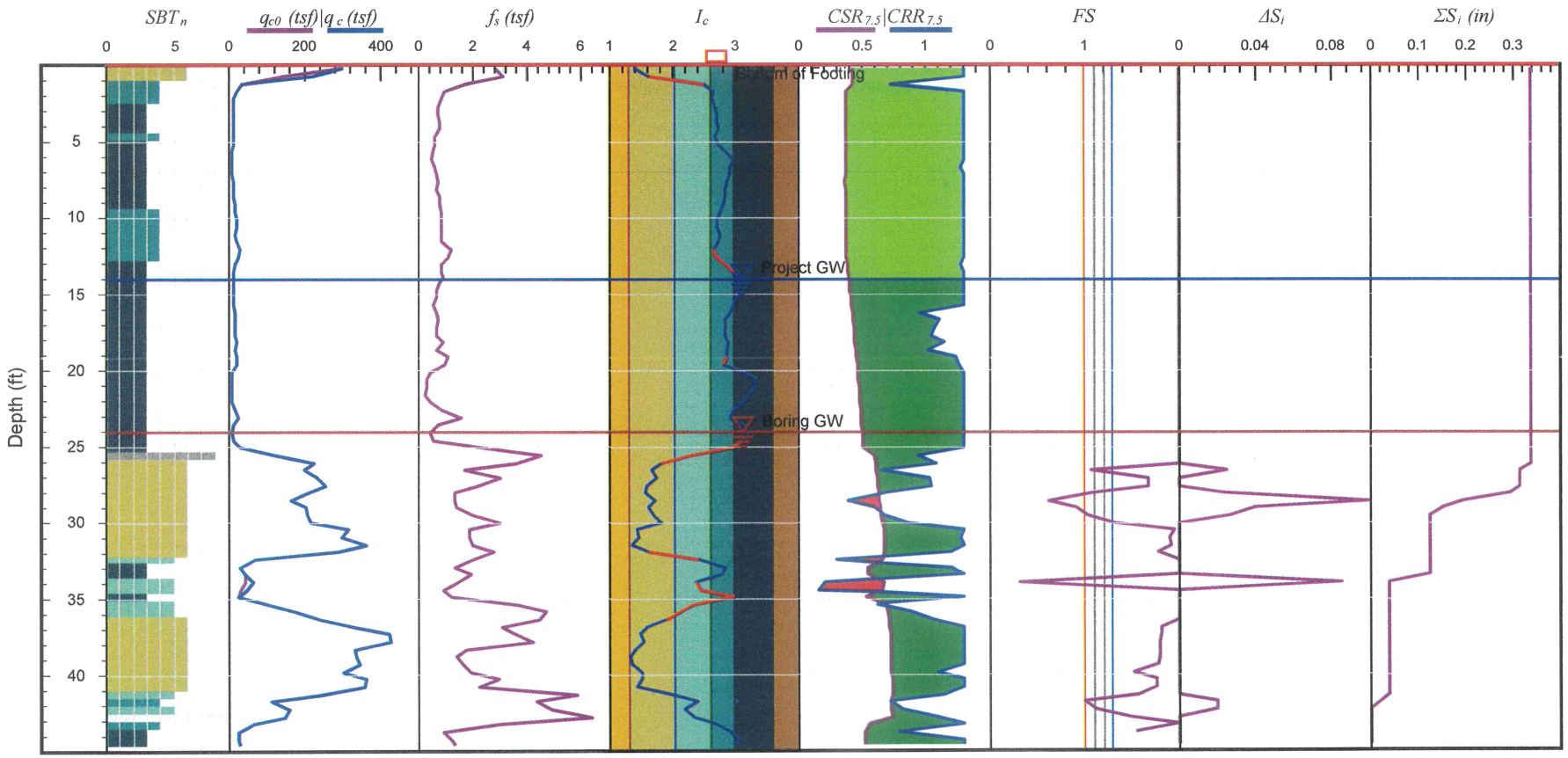
Liquefaction: Robertson & Wride (1998)
Settl.: [dry] Yi (2010); [sat] Zhang et al (2002)
Lateral spreading: Zhang et al (2004)
M correction: [Sand] Idriss(1995); [Clay] Boulanger & Idriss(2004)
σv correction: Idriss & Boulanger (2008)
Stress reduction: Laio & Whiteman (1986)

GeoSuite©

Liquefaction Potential - CPT Data

Project:	El Camino Real				
Location:	3378-3386 El Camino Real, Santa Clara				
Job Number:	6018	CPT No.:	1	Enclosure:	-1

Z:\Liquefaction\El Camino Real, Santa Clara 6018\GeoLiqu©_6018_1.csv



Earthquake & Groundwater Information:
Magnitude = 7.9
Max. Acceleration = 0.623 g
Project GW = 14 ft
Maximum Settlement = 0.34 in
Settl. at Bottom of Footing = 0.34 in

Liquefaction: Robertson & Wride (1998)
Settl.: [dry] Yi (2010); [sat] Zhang et al (2002)
Lateral spreading: Zhang et al (2004)
M correction: [Sand] Idriss(1995); [Clay] Boulanger & Idriss(2004)
σv correction: Idriss & Boulanger (2008)
Stress reduction: Laio & Whiteman (1986)

GeoSuite©

Liquefaction Potential - CPT Data

Project:	El Camino Real				
Location:	3378-3386 El Camino Real, Santa Clara				
Job Number:	6018	CPT No.:	2	Enclosure:	0

Z:\Liquefaction\El Camino Real, Santa Clara 6018\Boring 2\GeoLiq\q©_6018_2.csv

Prepared at 1/31/2022 11:58:12 AM