PREPARED FOR:

ASHTON 3, LLC
5 HOYA STREET
RANCHO MISSION VIEJO, CALIFORNIA 92694

ASHTON 3

PREPARED BY:

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GEOCON

GEOCON PROJECT NO. E9226-04-01 **JANUARY 2021**

Project No. E9226-04-01
January 22, 2021

Ashton 3, LLC
5 Hoya Street
Rancho Mission Viejo, California 92694

Attention: Mr. Taylor Ashton

Subject: 88 VIVIAN STREET
SAN RAFAEL, CALIFORNIA
GEOTECHNICAL INVESTIGATION

Dear Mr. Ashton:

In accordance with your authorization, we have performed a geotechnical investigation for subject site in San Rafael, California. Our investigation was performed to observe the soil and geologic conditions that may impact site development for the proposed townhome project. The accompanying report presents the results of our investigation and geotechnical conclusions and recommendations. The findings of this study indicate the site is suitable for development as planned provided the recommendations of this report are implemented during design and construction.

If you have any questions regarding this report, or if we may be of further service, please contact the undersigned at your convenience.

Sincerely,

GEOCON CONSULTANTS, INC.

A. E. Ashour

REGISTERED PROFESSIONAL ENGINEER
ANDRE E. ASHOUR
C 88145
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Andre E. Ashour, PE
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(1/e-mail) Addressee

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for a proposed townhome development at 88 Vivian Street in San Rafael, California (see Vicinity Map, Figure 1). The purpose of this investigation was to evaluate the subsurface soil and geologic conditions in the areas of the planned development and provide conclusions and recommendations pertaining to the geotechnical aspects of project design and construction, based on the conditions encountered during our study.

The scope of this investigation included field exploration, laboratory testing, engineering analysis and the preparation of this report. Our field exploration consisted of four soil borings drilled on November 19, 2020 to maximum depths of about 25 feet and three Cone Penetrometer Tests (CPTs) advanced on November 10, 2020 to depths ranging from about 82 to about 90 feet below the existing grade. Seismic shear wave velocity measurements were collected from one of the CPTs. The locations of our explorations are depicted on the Site Plan, Figure 2. A detailed discussion of our field investigation, boring logs and CPT profiles are presented in Appendix A.

Laboratory tests were performed on selected soil samples obtained during the investigation to evaluate pertinent physical properties for engineering analyses. In addition, three soil samples were submitted to our laboratory for screening-level corrosion testing. Laboratory test results are presented in Appendix B. Figures related to our site-specific ground motion hazard analysis are presented in Appendix C. Our liquefaction analysis is included as Appendix D.

The opinions expressed herein are based on analysis of the data obtained during the investigation and our experience with similar soil and geologic conditions. References reviewed to prepare this report are provided in the *List of References* section.

If project details vary significantly from those described herein, Geocon should be contacted to determine the necessity for review and possible revision of this report.

2. SITE CONDITIONS AND PROJECT DESCRIPTION

The site is a 2.41-acre parcel (Marin Co. APN 008-092-02) on the southeastern side of Vivian Street. The site is currently occupied by the one- to two-story Country Club Bowl building with at-grade parking in the southwestern portion of the parcel. The site is relatively flat with ground surface elevations of approximately 5 feet above Mean Seal Level (MSL) per web-based mapping. Existing development in the immediate site vicinity generally consists of one- to two-story industrial, multi-family, and commercial buildings.

Based on the site development plan that you provided, we understand that all existing improvements will be razed, and the site will be redeveloped as a 66-unit multifamily residential community. The new residential structures (townhomes) will be up to three stories in height with no subterranean levels. Landscaping, at-grade asphalt parking and driveways, utilities and other improvements necessary for the site development are also expected. Grading plans were not provided; we understand that cuts and fills to attain design subgrade elevation will be minimal.

3. GEOLOGIC SETTING

San Rafael is located within the Coast Ranges Geomorphic Province of California, which is characterized by a series of northwest trending mountains and valleys along the north and central coast of California. Topography is controlled by the predominant geological structural trends within the Coast Range that generally consist of northwest trending synclines, anticlines and faulted blocks. The dominant structure is a result of both active northwest trending strike-slip faulting, associated with the San Andreas Fault system, and east-west compression within the province.

The San Andreas Fault (SAF) is a major right-lateral strike-slip fault that extends from the Gulf of California in Mexico to Cape Mendocino in northern California. The SAF forms a portion of the boundary between two tectonic plates on the surface of the earth. To the west of the SAF is the Pacific Plate, which moves north relative to the North American Plate, located east of the fault. In the San Francisco Bay Area, movement across this plate boundary is concentrated on the SAF but also distributed, to a lesser extent, across several other faults including the Hayward, Calaveras and Rodgers Creek faults, among others. Together, these faults are referred to as the SAF system.

Basement rock west of the SAF is generally granitic, while to the east it consists of a chaotic mixture of highly deformed marine sedimentary, submarine volcanic and metamorphic rocks of the Franciscan Complex. Both are typically Jurassic to Cretaceous in age (205 to 65 million years old). Overlying the basement rocks are Cretaceous (about 140 to 65 million years old) marine, as well as Tertiary (about 65 to 1.6 million years old) marine and non-marine sedimentary rocks with some continental volcanic rock. These Cretaceous and Tertiary rocks have typically been extensively folded and faulted largely because of movement along the SAF system, which has been ongoing for about the last 25 million years, and regional compression during the last about 4 million years. The inland valleys, as well as the structural depression within which San Francisco Bay is located, are filled with unconsolidated to semi-consolidated deposits of Quaternary age (about the last 1.6 million years). Continental deposits (alluvium) consist of unconsolidated to semi-consolidated sand, silt, clay and gravel, while the bay deposits typically consist of soft organic-rich silt and clay (bay mud) or sand.

Available geologic mapping by the United States Geological Survey (USGS) indicates the site vicinity is in an area of artificial fill over Quaternary-age marine and marsh deposits (Bay Mud).

4. GEOLOGIC HAZARDS

4.1 Faulting and Seismicity

Geologists and seismologists recognize the greater San Francisco Bay Area as one of the most active seismic regions in the United States. The significant earthquakes that occur in the Bay Area are associated with crustal movements along well-defined active fault zones that generally trend in a northwesterly direction.

The San Francisco Bay Area is seismically dominated by the presence of the active San Andreas Fault System. In the theory of plate tectonics, the San Andreas Fault System is a transform fault that forms the boundary between the northward moving Pacific Plate (west of the fault) and the southward moving North American Plate (east of the fault). In the Bay Ares, the movement is distributed across a complex system of strike-slip, right lateral parallel and subparallel faults, which include the San Andreas, Hayward and Calaveras Faults, among others.

The table below presents approximate distances to active faults within approximately 25 miles of the site based on web-based mapping by CGS, as previously published by Caltrans. WGS 84 site coordinates are N 37.9634°, W 122.5081°.

TABLE 4.1
REGIONAL FAULT SUMMARY

Fault Name	Distance to Site (miles)	Maximum Earthquake Magnitude, M_w
Hayward (North)	7¾	7.3
San Andreas	8¾	8.0
San Gregorio	10¼	7.4
Rodgers Creek	14½	7.3
Contra Costa Shear Zone	19	6.5
West Napa	19½	6.6
Concord	22¾	6.6
Green Valley	24	6.8

The faults tabulated above and numerous other faults in the Bay Area are sources of potential ground motion. However, earthquakes that might occur on other faults within the northern and central California area are also potential generators of significant ground motion and could subject the site to intense ground shaking. The faults and distances tabulated above are intended to acquaint the reader with the seismic setting of the site; this information is not intended to be a basis for our ground motion hazard analysis.

4.2 Surface Fault Rupture

The site is not within a currently established State of California Earthquake Fault Zone for surface fault rupture hazards. No active or potentially-active faults are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. By definition, an active fault is one with surface displacement within the last 11,000 years. A potentially-active fault has demonstrated evidence of surface displacement with the past 1.6 million years. Faults that have not moved in the last 1.6 million years are typically considered inactive.

4.3 Specific Ground Motion Hazard Analysis

A site-specific ground motion hazard analyses was performed in accordance with ASCE 7-16 Chapter 21 and Section 1613A of the 2019 CBC using online applications developed by USGS.

4.3.1 Site- Specific Shear Wave Velocity

On November 10, 2020, Middle Earth Geo Testing Inc. performed seismic CPT (SCPT) soundings in CPT1. The SCPT soundings measured the shear waves generated at the ground surface at approximately 5-foot intervals to a depth of about 82 feet below the existing ground surface. The SCPT profile is included herein as Figure C4.

Based on the results of the SCPT, the site-specific soil shear wave velocity for the upper 30 meters feet of soil (Vs30) is estimated as 110 meters/second. In accordance with Section 1613A.3.2 of the 2019 California Building Code and Table 20.3-1 of ASCE 7-16, the estimated soil shear wave velocity falls within the boundaries of a Site Class “E”.

Although there are liquefiable soils underlying the site, we assume that the proposed townhome structures will have a fundamental period of less than 0.5 seconds and therefore will not require a site-response analysis.

4.3.2 Probabilistic Seismic Hazard Analysis

The risk-targeted Maximum Considered Earthquake (MCER) probabilistic response spectrum consists of the spectral response accelerations which are expected to achieve a 1 percent probability of collapse within a 50-year period, evaluated at 5 percent damping.

The mean spectral response accelerations having a 2 percent chance of exceedance in 50 years were evaluated at 5 percent damping using the USGS Unified Hazard Tool (UHT). The Dynamic U.S. 2014 (v4.2.0) edition was used within the analysis, which is based on the UCERF-3 fault model. The soil underlying the site was modeled as a Site Class “D/E” with a corresponding average shear wave velocity (VS30) of 180 meters per second. The site class definition is based on the SCPT data, which indicates a VS30 of approximately 110 meters per second or Site Class “E”. The lowest VS30 value available in the USGS UHT is Site Class “D/E”; therefore, 180 meters per second value was used for VS30 within the probabilistic analysis.

The web application uses the ground motion prediction equations (GMPEs) from the NGA-West 2 project: Abrahamson-et al. (2014) NGA West 2, Boore et al. (2014) NGA West 2, Campbell-Bozorgnia (2014) NGA West 2, and Chiou-Youngs (2014) NGA West 2. Each GMPE was assigned an equal weight and the mean value of the four GMPEs was evaluated. The mean spectral accelerations were rotated to maximum direction using the period specific ratios from Shahi et al. (2013 & 2014).

The GMPE of Campbell and Borzorgnia requires that the depth to where the shear wave velocity reaches 2.5 kilometers per second (Z2.5) be defined. Additionally, the GMPEs of Abrahamson-et al., Boore et al. and Chiou-Youngs require that the depth to where the shear wave velocity reaches 1 kilometer per second (Z1.0) be defined. The values of Z2.5 and Z1.0 are internally calculated by the Uniform Hazard Tool.

The MCE uniform hazard response spectra was adjusted to risk-targeted spectral accelerations corresponding to a 1 percent chance of collapse in 50 years by using the USGS Risk-Targeted Ground Motion Calculator and following ASCE 7-16 Section 21.2.1.2 Method 2.

The risk-targeted Maximum Considered Earthquake (MCER) probabilistic response spectrum is provided on Figure C1.

4.3.3 Deterministic Seismic Hazard Analysis

In order to define the deterministic scenario events, deaggregation of the uniform hazard probabilistic response spectrum was performed using the USGS UHT. The inversion approach used by UCERF-3 allows for a large number of variations for each source scenario, including multi-fault ruptures. Therefore, deaggregation of UCERF-3 consists of the contributions from multi-fault ruptures rather than individual source contributions. To address this, the UHT aggregates the contributions on a per-fault-section basis, with rupture contributions only ever counted once. The UHT deaggregation contributor list shows the fault sections which contribute most to hazard at a site and report a mean earthquake magnitude for each section identified by a 'parent' fault name and section index. Based on the deaggregation, we have considered scenario events with the greatest contribution to the deterministic ground motions.

The input values used to evaluate the deterministic scenario events are provided in the following table.

TABLE 4.3.3
INPUT VALUES TO EVALUATE DETERMINISTIC SCENARIO EVENTS

Parameter	Scenario 1	Scenario 2	Reference
Parent Fault Name	San Andreas (North Coast)	Hayward (North)	
Scenario Name	N. San Andreas: SAO+SAN+SAP+SAS	Hayward: RC+HN+HS+HE	BSSC Online Scenario Catalog
Earthquake Magnitude	8.04	7.58	BSSC Online Scenario Catalog
Fault Mechanism	Strike-Slip	Strike-Slip	
Fault Dip	88.2	76.6	BSSC 2014 [1]
Fault Width	11.8 km	10.58 km	BSSC 2014 [1]
Rake	180	178.2	BSSC 2014 [1]
Z_{TOR}	1.1 km	2.04 km	BSSC 2014 [1]
Rrup	15.90 km	13.09 km	Derived from Rx and Fault Type
Rjb	15.86 km	12.93 km	Derived from Rx and Fault Type
Rx	15.86 km	12.93 km	USGS Quartenary Faults & Folds Database
Vs30	180 m/s	180 m/s	Average Site Class D/E Value
$Z_{1.0}$	0.038 km	0.038 km	Bay Area Seismic Velocity Model, Release 8.3.0
$Z_{2.5}$	0.855 km	0.855 km	Bay Area Seismic Velocity Model, Release 8.3.0

1. BSSC 2014, aka. UCERF3_EventSet_All on GitHub

The deterministic median and standard deviation (sigma) for the scenario events were evaluated using the USGS NSHMP-HAZ-WS Response Spectra online application. The deterministic analysis used the same four GMPEs, equally weighted, to generate the median and standard deviation of the ground motion which were then used to calculate the 84th percentile at 5% damping. The geometric median spectral accelerations were rotated to maximum direction using the period specific ratios from Shahi et al. (2013 & 2014).

The deterministic scenarios were compared and a combination of events controls the deterministic spectrum. The fault source resulting in the highest spectral accelerations from 0 to 0.02 seconds would be a magnitude 8.04 event on the San Andreas fault; from 0.03 to 0.4 seconds would be a magnitude 7.58 event on the Hayward fault; and from 0.5 to 10 seconds would be a magnitude 8.04 event on the San Andreas fault.

The largest spectral ordinate of the deterministic spectra was compared to 1.5Fa, with Fa determined using Table 11.4.1. Based on this comparison, a scale factor was applied uniformly across all periods of the deterministic spectrum such that the largest ordinate is not less than 1.5Fa. The scaled 84th percentile maximum rotated component deterministic response spectra is provided on Figure C2.

4.3.4 Site-Specific Response Spectrum

The lesser of the probabilistic and deterministic MCE_R response spectrums is the Site-Specific MCE_R. Two thirds of the Site-Specific MCE_R is the Design Earthquake (DE) Response Spectrum, provided the results are not less than 80 percent of the modified General Design Response Spectrum determined by ASCE 7-16 Section 11.4.6 with F_a and F_v determined as specified in Section 21.3.

Graphical representations of the analyses are presented on Figures C1 and C2. The Site-Specific Design Earthquake response spectrum at 5 percent damping is presented on graphically on Figure C2 and in tabular form on Figure C3.

4.3.5 Mapped Acceleration Parameters

The following table summarizes the mapped acceleration parameters obtained from the 2019 California Building Code (CBC; Based on the 2018 International Building Code [IBC] and ASCE 7-16), Chapter 16A Structural Design, and Section 1613A Earthquake Loads. The data was calculated using the online application *Seismic Design Maps*, provided by OSHPD. The short spectral response uses a period of 0.2 second.

TABLE 4.3.5
MAPPED SPECTRAL ACCELERATIONS

Parameter	Value	2019 CBC Reference
Site Class	E	Section 1613.2.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.5g	Figure 1613.2.1 (1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.6g	Figure 1613.2.1 (2)
Site Coefficient, F_A	1.2	Table 1613.2.3 (1)
Site Coefficient, F_V	2*	Table 1613.2.3 (2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.8g	Section 1613.2.3 (Eq. 16-36)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.2g*	Section 1613.2.3 (Eq. 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.2g	Section 1613.2.4 (Eq. 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.8g*	Section 1613.2.4 (Eq. 16-39)
T_S	0.67 sec	ASCE 7-16 Chapter 11

Note:
*Per Section 11.4.8 of ASCE/SEI 7-16, a ground motion hazard analysis shall be performed for projects for Site Class "E" sites with Ss greater than or equal to 1.0g and for Site Class "D" and "E" sites with S1 greater than 0.2g. Section 11.4.8 also provides exceptions which indicates that the ground motion hazard analysis may be waived provided the exceptions are followed. Using the code based values presented in the table above, in lieu of a performing a ground motion hazard analysis, requires the exceptions outlined in ASCE 7-16 Section 11.4.8 be followed.

4.3.6 Site-Specific Seismic Design Criteria

In accordance with the ASCE 7-16 Section 21.4, site-specific design acceleration parameters shall be derived using the results of the site-specific ground motion hazard analysis.

The parameter S_{DS} shall be taken as equal to 90 percent of the maximum spectral acceleration obtained from the site-specific analysis at any period within the range from 0.2 to 5 seconds, inclusive. The parameter S_{D1} shall be taken as the maximum value of the product of the spectral acceleration and period for periods from 1 to 5 seconds, inclusive. The values of S_{MS} and S_{M1} shall be taken as 1.5 times the site-specific values of S_{DS} and S_{D1}. The site-specific design acceleration parameters shall not be less than 80 percent of the general seismic design values determined by ASCE 7-16 Section 11.4.

The following table presents the site-specific seismic design parameters based on the site-specific ground motion hazard analysis.

TABLE 4.3.6
SITE-SPECIFIC DESIGN ACCELERATION PARAMETERS

Parameter	Value
Site Class Modified MCER Spectral Response Acceleration (short), S_{MS}	1.440g
Site Class Modified MCER Spectral Response Acceleration – (1 sec), S_{M1}	1.920g
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.96g
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	1.280g

4.3.7 Site-Specific Peak Ground Acceleration

The site-specific Maximum Considered Earthquake (MCE_G) geometric mean peak ground acceleration was evaluated in accordance with ASCE 7-16 Section 21.5.

The probabilistic geometric mean peak ground acceleration and the deterministic 84th percentile geometric mean peak ground acceleration were analyzed using the same approaches as described above. The analysis used the same Site Class and scenario earthquake.

The deterministic MCE_G shall not be less than $0.5F_{PGA}$, where F_{PGA} is determined from ASCE 7-16 Table 11.8-1 with the value of PGA taken as 0.5g. The site-specific MCE_G peak ground acceleration is taken as the lesser of the probabilistic and deterministic MCE_G, provided the value is not less than 80 percent of the value of PGAM as determined by ASCE 7-16 Equation 11.8.1.

TABLE 4.3.7
ASCE 7-16 SITE-SPECIFIC PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Site-Specific MCE_G Peak Ground Acceleration, PGA_M	0.481g	Section 21.5

4.4 Liquefaction

The site is not currently mapped by the California Geological Survey (CGS) for liquefaction hazards as such mapping has not been performed in the project area; however, web-based mapping by the USGS indicates the entire site possesses a “very high” susceptibility to liquefaction. Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary loss of shear strength due to pore pressure buildup under the cyclic shear stresses associated with intense earthquakes. Primary factors that trigger liquefaction are: moderate to strong ground shaking (seismic source), relatively clean, loose granular soils (primarily poorly graded sands and silty sands), and saturated soil conditions (shallow groundwater). Due to the increasing overburden pressure with depth, liquefaction of granular soils is generally limited to the upper 50 feet of a soil profile.

We used the computer software program *CLiq* (Version 2.2.0.35, Geologismiki) and the in-situ soil parameters measured in the CPT soundings to evaluate liquefaction potential at the site. The software utilized the 2014 methodology of Boulanger and Idriss (2014) and also considered the potential for dry sand settlements above groundwater. Our evaluation incorporated an earthquake moment magnitude (M_w) of 7.3 (overall all sources) and a groundwater depth of 4 feet. Per 2019 CBC, we used a ground motion (peak ground acceleration) of 0.481g in our analysis.

Liquefaction analyses typically evaluate the potential for liquefaction in soils to depths of 50 feet. As a conservatism, we evaluated liquefaction potential for the full depth of the Bay Mud encountered in our CPTs, based on the geologic recency of those deposits.

Consequences of liquefaction can include ground surface settlement, ground loss (sand boils) and lateral slope displacements (lateral spreading). For liquefaction-induced sand boils or fissures to occur, pore water pressure induced within liquefied strata must exert enough force to break through overlying, non-liquefiable layers. Based on methodology recommended by Youd and Garris (1995), which advanced original research by Ishihara (1985), a capping layer of non-liquefiable soil can prevent the occurrence of sand boils and fissures. Based on the presence of the non-liquefiable layer that mantles the site and the depth to liquefiable layers, the potential for ground loss due to sand boils or fissures in a seismic event is considered low.

The likely consequence of potential liquefaction at the site is settlement. Our analysis indicates that, if liquefaction and cyclic softening were to occur, total ground surface settlements on the order of ½ inch or less may result. Selected output from our liquefaction analysis is presented in Appendix D.

4.5 Landslides

There are no known landslides near the site, nor is the site in the path of any known or potential landslides. We do not consider the potential for a landslide to be a significant hazard to this project.

4.6 Tsunamis and Seiches

Based on mapping published by the California Emergency Management Agency and CGS, the site would not be inundated during an extreme tsunami.

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. No major water-retaining structures are located immediately up gradient from the project site. Flooding from a seismically induced seiche is considered unlikely.

5. SOIL AND GROUNDWATER CONDITIONS

5.1 Surface Materials

Pavement within the project limits generally consists of approximately 2 to 6 inches of asphalt concrete over 5½ to 9 inches of aggregate base materials. Based on visual observations, the existing surface pavements are in poor condition with severe cracking, potholes, and patching.

5.2 Artificial Fill

Below surface materials, artificial fill was encountered in our soil borings and was observed to be variable throughout the project limits to depths ranging from 5 ½ to 6 ½ feet below existing surface grades. The fill materials comprise stiff to hard clay (CL) with various amounts of silt, sand and gravel; loose to medium dense clayey sand (SC) with various amounts of silt and gravel; and medium dense gravel (GC/GM) with various amount of sand and clay/silt. The artificial fill materials may differ from those than encountered in our borings. The fills may contain constituents not encountered in our borings or reported herein. Based on our laboratory test results, the clayey soils within the artificial fills possess moderate plasticity and should be considered moderately expansive.

5.3 Bay Mud

Bay Mud deposits were encountered beneath the artificial fills in all our soil borings and CPTs. As encountered in our CPT soundings, the Bay Mud deposits extend to depths of approximately 42 to 62 feet below existing grade at CPT locations CPT3 and CPT1, respectively. As observed in our borings, the deposits consisted of very soft, organic rich fat clay (CH). Our laboratory testing indicates the Bay Mud deposits are highly compressible and weak, consistent with our prior experience and typical Bay Mud properties.

5.4 Older Alluvium

Based on our review of the information obtained in our CPT soundings, it appears soil conditions at the site transition from Bay Mud to an older alluvial deposit that extends to depths of approximately 82 to 90 feet below the existing grade (the maximum depths explored) at CPT1 and CPT3, respectively. The soil types within this alluvial unit were not directly observed as we used CPTs for our deeper subsurface explorations.

5.5 Groundwater

Groundwater was encountered at depths of approximately 4 to 5 feet in our borings at the time drilling. Actual groundwater levels will fluctuate with variations in rainfall, temperature and other factors and may be higher or lower than observed during our study. Additionally, it is not uncommon for perched groundwater conditions to develop where none previously existed, especially in or atop fine-grained soils that are subjected to irrigation or precipitation.

5.6 Soil Corrosion Screening

Soil samples obtained during our field exploration were subjected to laboratory testing for minimum resistivity, pH, and chloride and water-soluble sulfate. The laboratory test results and published screening levels are presented in Appendix B. Soil corrosivity should be considered in the design of buried metal pipes, underground structures, etc.

Water-soluble sulfate test results on selected samples of site soils indicate an S0 exposure classification for sulfate attack on normal portland cement concrete (PCC) as defined in Chapter 318, Table 19.3.1.1 of the ACI Building Code Requirements for Structural Concrete. ACI does not set forth requirements for S0 sulfate exposure classification. In addition, none of the two soil samples tested would be classified as corrosive to buried metal improvements based on Caltrans criteria.

Geocon does not practice in the field of corrosion engineering and mitigation. If corrosion sensitive improvements are planned, it is recommended that a corrosion engineer be retained to evaluate corrosion test results and incorporate the necessary precautions to avoid premature corrosion of buried metal pipes and concrete structures in direct contact with the soils.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 General

6.1.1 No overriding geotechnical constraints were encountered during our investigation that would preclude the project as presently proposed. Primary geotechnical considerations are the presence of highly compressible Bay Mud deposits within the upper approximately 42 to 62 feet below existing grade and the potential for strong seismic shaking. A layer of lightweight fill material will be required below the proposed townhomes to mitigate settlement from building loads.

6.1.2 Based on the subsurface conditions at the site and the anticipated structural loading, post-tension foundation systems, used in conjunction with the remedial grading described herein, can be used to support the planned townhome buildings. Post-construction settlements due to static foundation loads should in order of 1 ¾ inch or less with differential settlements of 1 inch or less across a horizontal distance of 50 feet.

6.1.3 Any changes in the design, location or elevation, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

6.1.4 The proposed project redevelops a site with past episodes of grading and construction. As such, unknown underground improvements and areas of undocumented fill materials (not discussed herein) may be present. If encountered, supplemental recommendations will be provided during site development.

6.1.5 All references to relative compaction and optimum moisture content in this report are based on the latest edition of ASTM D 1557.

6.2 Soil and Excavation Characteristics

6.2.1 Based on the soils conditions encountered in our field explorations, we anticipate the onsite soils can be excavated with moderate effort using conventional excavation equipment. We do not anticipate excavations at the site will generate oversize material (greater than 6 inches in nominal dimension). Any artificial fills encountered at the site are undocumented and may contain constituents not reported herein.

6.2.2 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable Occupation Safety and Health Administration (OSHA) rules and regulations to maintain safety and maintain the stability of adjacent existing improvements.

6.2.3 The existing soils encountered at the site should be considered "expansive" as defined by 2019 CBC. However, the recommendations of this report assume proposed building foundation systems will derive support in properly compacted non expansive fills (i.e., lightweight fill).

6.3 Materials for Fill

6.3.1 Excavated soils generated from cut operations at the site should be suitable for use as engineered fill in structural areas provided, they do not contain deleterious matter, organic material, or cementations larger than 6 inches in maximum dimension.

6.3.2 Import fill material should be primarily granular with a “low” expansion potential (Expansion Index less than 50), a Plasticity Index less than 15, be free of organic material and construction debris, and not contain rock larger than 6 inches in greatest dimension.

6.3.3 Lightweight fill (LWF), where required, should have maximum unit weight of 30 pounds per cubic foot (pcf). LWF materials include, but are not limited to, foamed concrete or expanded polystyrene (EPS), low-density cellular concrete (LDCC), and lightweight aggregate (LWA). In our experience, LDCC is the most common of the listed LWF materials.

6.3.4 Environmental characteristics and corrosion potential of import soil materials may also be considered. Proposed import materials should be sampled, tested, and approved by Geocon prior to transportation to the site.

6.4 Grading

6.4.1 All clearing operations and earthwork (including over-excavation, scarification, and recompaction) should be observed and all fills tested for recommended compaction and moisture content by representatives of Geocon.

6.4.2 Structural areas should be considered as areas extending a minimum of 5 feet horizontally from a foundation or beyond the outside dimensions of buildings, including footings and overhangs carrying structural loads, and where not restricted by property boundaries.

6.4.3 A preconstruction conference should be held at the site prior to the beginning of grading operations with the owner, contractor, civil engineer and geotechnical engineer in attendance. Special soil handling requirements can be discussed at that time.

6.4.4 Site preparation should begin with removal of any surface and subsurface structures including all foundations, flatwork, and pavement. Within landscaped area, all surface vegetation should be stripped and all the trees, root balls and shrubs should be removed such that no roots larger than approximately 1 inch in diameter remain within proposed building footprints. All active or inactive utilities within the construction area should be protected, relocated, or abandoned. Any pipelines to be abandoned that are greater than 2 inches and less than 18 inches in diameter should be removed or filled with sand-cement slurry. Utilities larger than 18 inches in diameter should be removed. Excavations or depressions resulting from site clearing operations, or other existing excavations or depressions, should be restored with engineered fill in accordance with the recommendations of this report.

6.4.5 After demolition and the removal of existing improvements, the existing subgrade within building pad areas should be over-excavated to a depth of 3 feet below the existing surface grade or two feet below proposed pad grade, whichever is deeper, and replaced with LWF meeting the requirements of Section 6.3.3. The exposed subgrade should be subjected to several passes with a small drum roller without vibratory action before placing LWF.

6.4.6 The existing soils outside of building pad areas should be over-excavated to a depth of approximately 1 foot. The exposed bottom should be scarified 8 to 12 inches, moisture conditioned to at least 2% above optimum moisture and recompacted to at least 90% relative compaction (at near optimum moisture where fill materials are predominantly sands or gravels).

6.4.7 The exposed bottom surfaces and bottom processing should be observed by our representatives on a full-time basis. Supplemental recommendations may be provided based on-site conditions during grading. Deeper over-excavations may be needed in some areas.

6.4.8 If grading commences in winter or spring, or in periods of precipitation, excavated and in-place soils may be wet. Earthwork contractors should be aware of potential compaction/workability difficulties. The most effective site preparation alternatives will depend on site conditions prior to and during grading operations; we should evaluate site conditions at those times and provide supplemental recommendations, if necessary.

6.4.9 All engineered fills should be placed in layers no thicker than will allow for adequate bonding and compaction (typically 8 inches). Fill soils should be placed, moisture conditioned to minimum 2 percent above optimum moisture content (near optimum moisture where fill materials are predominantly sands or gravels) and compacted to at least 90% relative compaction.

6.5 Temporary Excavations

6.5.1 We anticipate that the majority of the site will be classified as Cal-OSHA "Type C" soil when encountered in excavations during site development and construction. Excavation sloping, benching, the use of trench shields, and the placement of trench spoils should conform to the latest applicable Cal-OSHA standards. The contractor should have a Cal-OSHA-approved "competent person" onsite during excavation to evaluate trench conditions and make appropriate recommendations where necessary.

6.5.2 All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping and possibly shoring.

6.5.3 It is the contractor's responsibility to provide sufficient and safe excavation support as well as protecting nearby utilities, structures, and other improvements that may be damaged by earth movements.

6.5.4 Temporary excavations such as utility trench sidewalls should remain near vertical to depths of at least 3 feet below ground surface, although some sloughing and caving may occur, particularly if clean sandy or gravelly soils, poorly compacted fills or groundwater are encountered. Excavations greater than approximately 3 feet in height or those that are surcharged by adjacent traffic or structures may require sloping or shoring measures in order to provide a stable excavation.

6.5.5 Temporary excavations should be protected from rainfall and erosion. Surface runoff should be directed away from excavations or slopes.

6.6 Underground Utilities

6.6.1 Underground utility trenches should be backfilled with properly compacted material. The material excavated from the trenches should be adequate for use as backfill provided it does not contain deleterious matter, vegetation or rock larger than six inches in maximum dimension. Trench backfill should be placed in loose lifts not exceeding eight inches and compacted to at least 90% relative compaction to minimum 2 percent above optimum moisture content.

6.6.2 Bedding and pipe zone backfill typically extends from the bottom of the trench excavations to a minimum of six inches above the crown of the pipe. Pipe bedding material should consist of crushed aggregate, clean sand or similar open-graded material. Proposed bedding and pipe zone materials should be reviewed by Geocon prior to construction; materials such as ¾-inch drain rock may require wrapping with filter fabric to mitigate the potential for piping. Bedding and backfill should also conform to the requirements of the governing utility agency.

6.7 Post-Tensioned Slabs

6.7.1 Thickened post-tensioned slab foundations may be used for the new townhome buildings. Post-tensioned (PT) slab thickness and reinforcement should be designed by the project structural engineer. PT slabs should be designed to accommodate the estimated seismic and static settlements discussed in Sections 4.4 and 6.1.2, respectively.

6.7.2 Post-tensioned foundations should be designed by a structural engineer experienced in post-tensioned slab design and design criteria of the Post-Tensioning Institute (PTI), Third Edition, as required by the 2019 California Building Code. The post-tensioned design should incorporate the geotechnical parameters presented on the table below. The parameters presented are based on the guidelines presented in the PTI, Third Edition design manual.

TABLE 6.7.2
POST-TENSIONED FOUNDATION SYSTEM DESIGN PARAMETERS

Post-Tensioning Institute (PTI) Third Edition Design Parameters	Recommended Value
Equilibrium Suction	3.0
Edge Lift Moisture Variation Distance, e_M (feet)	5.1
Edge Lift, y_M (inches)	1.10
Center Lift Moisture Variation Distance, e_M (feet)	9.0
Center Lift, y_M (inches)	1.52

6.7.3 PT slab contact pressure should be generally limited to 350 psf for dead plus live loads. We recognize that isolated areas of higher contact pressure may exist at wall or column locations. When available, PT slab contact pressures should be reviewed by Geocon to confirm the settlement estimates provided herein. Supplemental recommendations may be provided after our review.

6.7.4 Post-tensioned foundations should be embedded in accordance with the recommendations of the structural engineer. If a post-tensioned mat foundation system is planned, the slab should possess a thickened edge with a minimum width of 12 inches. The thickened edge should extend below the crushed rock underlayment layer.

6.7.5 The thickness of post-tensioned foundation systems should be determined by the project structural engineer. Based on our experience with similar projects and soils conditions, we anticipate the post-tensioned slab thicknesses will be on the order of 10 to 12 inches.

6.7.6 Our experience indicates that post-tensioned slabs are susceptible to excessive edge lift, regardless of the underlying soil conditions. Placing reinforcing steel at the bottom of the perimeter footings and the interior stiffener beams may mitigate this potential. Because of the placement of the reinforcing tendons in the top of the slab, the resulting eccentricity after tensioning reduces the ability of the system to mitigate edge lift. The structural engineer should design the foundation system to reduce the potential of edge lift occurring for the proposed structures.

6.7.7 During the construction of the post-tension foundation system, the concrete should be placed monolithically. Under no circumstances should cold joints be allowed to form between the footings/grade beams and the slab during the construction of the post-tension foundation system.

6.7.8 The use of isolated footings, which are located beyond the perimeter of the building and support structural elements connected to the building, are not recommended. Where this condition cannot be avoided, the isolated footings should be connected and tied to the building foundation system with grade beams.

6.7.9 Consideration should be given to connecting patio slabs to the building foundation to reduce the potential for future separation to occur.

6.7.10 Post-tensioned slabs should be underlain by at least 3 inches of ½-inch or ¾-inch crushed rock with no more than 5 percent passing the No. 200 sieve to serve as a capillary break.

6.7.11 Where post-tensioned foundation systems are designed and constructed as recommended herein, post-construction settlement due to dead + live loads should be approximately 1 ¾ inch or less with differential settlements of less than 1 inch across a horizontal distance of 50 feet.

6.8 Shallow Foundations

6.8.1 Shallow foundations (footings) founded in engineered fill may be used for ancillary site structures such as short retaining walls, screen walls, or trash enclosures. The following recommendations are based on the assumption that the soils within 4 feet of finish grade will consist of moderate expansive materials.

6.8.2 It is recommended that conventional shallow footings have a minimum embedment depth of 18 inches below lowest adjacent pad grade. Strip footings should be at least 12 inches wide. Spread column footings should be at least 3 feet square.

6.8.3 Footings proportioned as recommended may be designed for an allowable soil bearing pressure of 2,000 pounds per square foot (psf). The allowable bearing pressure is for dead + live loads and may be increased by up to one-third for transient loads due to wind or seismic forces.

6.8.4 The allowable passive pressure used to resist lateral movement may be assumed to be equal to a fluid weighing 250 pounds per cubic foot (pcf) for footings poured neat against properly compacted fills or undisturbed natural soils. The allowable passive pressure assumes a horizontal surface extending at least 5 feet or 3 times the surface generating the passive pressure, whichever is greater. The allowable coefficient of friction to resist sliding is 0.30 for concrete against soil. Combined passive resistance and friction may be utilized for design provided that the frictional resistance is reduced by 50%. Where not protected by flatwork or pavement, the upper 1 foot of soil should be neglected when calculating passive resistance to lateral loads.

6.8.5 Minimum reinforcement for continuous footings should consist of four No. 4 steel reinforcing bars; two placed near the top of the footing and two near the bottom. Spread column footing reinforcement should be specified by the structural engineer.

6.8.6 The foundation dimensions and minimum reinforcement recommendations presented herein are based upon soil conditions only and are not intended to be used in lieu of those required for structural purposes.

6.8.7 Underground utilities running parallel to footings should not be constructed in the zone of influence of footings. The zone of influence may be taken to be the area beneath the footing and within a 1:1 (horizontal:vertical) plane extending out and down from the bottom edge of the footing.

6.8.9 The foundation subgrade should be sprinkled as necessary to maintain a moist condition without significant shrinkage cracks as would be expected in any concrete placement. Our representative should observe all footing excavations prior to placing reinforcing steel.

6.9 Retaining Wall Design

6.9.1 Lateral earth pressures may be used in the design of retaining walls and buried structures. Lateral earth pressures against these facilities may be assumed to be equal to the pressure exerted by an equivalent fluid. The unit weight of the equivalent fluid depends on the design conditions. Table 6.9 summarizes the weights of the equivalent fluid based on the different design conditions.

TABLE 6.9
RECOMMENDED LATERAL EARTH PRESSURES

Condition	Equivalent Fluid Density
Active	50 pcf
At-Rest	65 pcf

6.9.2 Unrestrained walls should be designed using the active case. Unrestrained walls are those that are allowed to rotate more than 0.01H (where H is the height of the wall). Walls restrained from movement should be designed using the at-rest case. The above soil pressures assume level backfill under drained conditions within an area bounded by the wall and a 1:1 plane extending upward from the base of the wall and no surcharges within that same area.

6.9.3 Retaining wall foundations should be designed as continuous strip footings in accordance with Section 6.8.

6.9.4 Unless hydrostatic conditions are incorporated into design, retaining walls greater than 2 feet tall (retained height) should be provided with a drainage system adequate to prevent the buildup of hydrostatic forces and should be waterproofed as required by the project architect. Positive drainage for retaining walls should consist of a vertical layer of permeable material positioned between the retaining wall and the soil backfill. The permeable material may be composed of a composite drainage geosynthetic or a natural permeable material such as crushed gravel at least 12 inches thick and capped with at least 12 inches of native soil. A geosynthetic filter fabric should be placed between the gravel and the soil backfill. Provisions for removal of collected water should be provided for either system by installing a perforated drainage pipe along the bottom of the permeable material that leads to suitable drainage facilities.

6.9.5 We recommend that all retaining wall designs be reviewed by Geocon to confirm the incorporation of the recommendations provided herein. In particular, potential surcharges from adjacent structures and other improvements should be reviewed by Geocon.

6.10 Moisture Protection Considerations

6.10.1 A vapor barrier is not required beneath post-tensioned slabs for geotechnical purposes. Further, the migration of moisture through concrete slabs or moisture otherwise released from slabs is not a geotechnical issue. However, for the convenience of the owner, we are providing the following general suggestions for consideration by the owner, architect, structural engineer, and contractor. The suggested procedures may reduce the potential for moisture-related floor covering failures on concrete slabs-on-grade, but moisture problems may still occur even if the procedures are followed. If more detailed recommendations are desired, we recommend consulting a specialist in this field. If a vapor barrier is used beneath mat slab foundations, we should review the geotechnical design parameters presented herein.

6.10.2 A vapor barrier meeting ASTM E 1745-09 Class C requirements may be placed directly below the slab, without a sand cushion. To reduce the potential for punctures, a higher quality vapor barrier (15 mil, Class A or B) should be used. The vapor barrier, if used, should extend to the edges of the slab, and should be sealed at all seams and penetrations.

6.10.3 The concrete water/cement ratio should be as low as possible. The water/cement ratio should not exceed 0.45 for concrete placed directly on the vapor barrier. Midrange plasticizers could be used to facilitate concrete placement and workability.

6.10.4 Proper finishing, curing, and moisture vapor emission testing should be performed in accordance with the latest guidelines provided by the American Concrete Institute, Portland Cement Association, and ASTM.

6.11 Pavement Recommendations

6.11.1 The upper 12 inches of pavement subgrade should be scarified, moisture conditioned to minimum 2 percent above optimum moisture content (at near optimum moisture where fill materials are predominantly sands or gravels) and compacted to at least 95% relative compaction. Prior to placing aggregate base, the finished subgrade should be proof rolled with a laden water truck (or similar equipment with high contact pressure) to verify stability.

6.11.2 Sidewalk, curb and gutter, and driveway encroachments should be designed and constructed in accordance with City of San Rafael requirements, as applicable.

6.11.3 We recommend the following asphalt concrete (AC) pavement sections for design to establish subgrade elevations in pavement areas. The project civil engineer should determine the appropriate Traffic Index (TI) based on anticipated traffic conditions. The flexible pavement sections below are based on estimated design TIs and an R-Value of 5 for the subgrade soils. We can provide additional sections based on other TIs if necessary.

TABLE 6.11.3
FLEXIBLE PAVEMENT SECTION RECOMMENDATIONS

Location	Estimated Traffic Index (TI)	AC Thickness (inches)	AB Thickness (inches)
Parking Stalls	4.5	3	8
Driveways	6.0	3 ½	12 ½
Heavy-Duty	7.0	4	15 ½

Note: The recommended flexible pavement sections are based on the following assumptions:

1. AB: Class 2 AB with a minimum R-Value of 78 and meeting the requirements of Section 26 of the latest Caltrans Standard Specifications.
2. AB is compacted to 95% or higher relative compaction at or near optimum moisture content. Prior to placing AB, the subgrade should be proof rolled with a loaded water truck to verify stability.
3. AC: Asphalt concrete conforming to local agency standards or Section 39 of the latest Caltrans Standard Specifications.

6.11.4 The AC sections in Table 6.12.3 are final, minimum thicknesses. If staged pavements are used, the construction bottom AC lift should be at least 2 inches thick. Following construction, the finish top AC lift should be at least 1.5 inches thick.

6.11.5 Unless specifically designed and evaluated by the project structural engineer, where concrete paving will be utilized for support of vehicles, we recommend the concrete be a minimum of 6 inches thick and reinforced with No. 3 steel reinforcing bars placed 24 inches on center in both horizontal directions. In addition, doweling, reinforcing steel or other load-transfer mechanism should be provided at joints if desired to reduce the potential for vertical offset. The concrete should have a minimum 28-day compressive strength of 3,500 psi.

6.11.6 We recommend that at least 6 inches of Class 2 Aggregate Base (Class 2 AB) be used below rigid concrete pavements. The aggregate base should be compacted to at least 95% relative compaction near optimum moisture content.

6.11.7 In general, we recommend that concrete pavements be designed, constructed and maintained in accordance with industry standards such as those provided by the American Concrete Pavement Association.

6.11.8 The performance of pavements is highly dependent upon providing positive surface drainage away from the edge of pavements. Ponding of water on or adjacent to the pavement will likely result in saturation of the subgrade materials and subsequent cracking, subsidence and pavement distress. If planters are planned adjacent to paving, it is recommended that the perimeter curb be extended at least 6 inches below the bottom of the aggregate base to minimize the introduction of water beneath the paving. Alternatives such as plastic moisture cut-offs or modified drop-inlets may also be considered in lieu of deepened curbs.

6.11.9 Asphalt pavement section recommendations for driveways and parking areas are based on Caltrans design procedures. It should be noted that most rational pavement design procedures are based on projected street or highway traffic conditions and, hence, may not be representative of vehicular loading that occurs in parking lots and driveways. Pavement proximity to landscape irrigation, reduced traffic speed and short turning radii increase the potential for pavement distress to occur in parking lots even though the volume of traffic is significantly less than that of an adjacent street. The Caltrans *Highway Design Manual* indicates that the resulting pavement sections for parking lots are minimized to keep initial costs down but are reasonable because additional AC surfacing can be added later, if needed, and generally without incurring traffic hazards or traffic handling problems. It is generally not economically feasible to design and construct the entire parking lot and driveways for the unique loading conditions previously described. Periodic maintenance of the pavement in these areas, therefore, should be anticipated.

6.12 Exterior Slabs

6.12.1 Exterior slabs, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. We recommend that at least 6 inches of Class 2 Aggregate Base (AB) compacted to at least 95% relative compaction be used below exterior concrete slabs. Prior to placing AB, the subgrade should be scarified 8 inches, moisture conditioned 2% above optimum and properly compacted to at least 90% relative compaction (at near optimum moisture where fill materials are predominantly sands or gravels).

6.12.2 The slab-on-grade dimensions and minimum reinforcement recommendations presented herein are based upon soil conditions only and are not intended to be used in lieu of those required for structural purposes.

6.12.3 Crack control joints for slabs-on-grade should be spaced at intervals not greater than 8 feet for 4-inch slabs and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. Construction joints should be designed by the project structural engineer.

6.12.4 The recommendations of this report are intended to reduce the potential for cracking of slabs due to soil movement. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade may exhibit some cracking due to soil movement. This is common for project areas that contain expansive soils since designing to eliminate potential soil movement is cost prohibitive. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

6.13 Surface Drainage

6.13.1 Proper surface drainage is critical to the future performance of the project. Uncontrolled infiltration of irrigation excess and storm runoff into the soils can adversely affect the performance of the planned improvements. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change to important engineering properties. Proper drainage should be maintained at all times.

6.13.2 All site drainage should be collected and transferred to the street in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundations or retaining walls. Drainage should not be allowed to flow uncontrolled over any descending slope. The proposed structures should be provided with roof gutters. Discharge from downspouts, roof drains and scuppers not permitted onto unprotected soils within five feet of the building perimeter. Planters which are located adjacent to foundations should be sealed or properly drained to prevent moisture intrusion into the materials providing foundation support. Landscape irrigation within five feet of the building perimeter footings should be kept to a minimum to just support vegetative life.

6.13.3 Positive site drainage should be provided away from structures, pavement, and the tops of slopes to swales or other controlled drainage structures. The building pad and pavement areas should be fine graded such that water is not allowed to pond. Final soil grade should slope a minimum of 2% away from structures.

6.13.4 We recommend implemented measures to reduce infiltrating surface water near buildings and slabs-on-grade. Such measures may include:

- Selecting drought-tolerant plants that require little or no irrigation, especially within 3 feet of buildings, slabs-on-grade, or pavements.
- Using drip irrigation or low-output sprinklers.
- Using automatic timers for irrigation systems.
- Appropriately spaced area drains.
- Hard-piping roof downspouts to appropriate collection facilities.

7. FURTHER GEOTECHNICAL SERVICES

7.1 Plan and Specification Review

7.1.1 We should review project plans and specifications prior to final design submittal to assess whether our recommendations have been properly implemented and evaluate if additional analysis and/or recommendations are required.

7.2 Testing and Observation Services

7.2.1 The recommendations provided in this report are based on the assumption that we will continue as Geotechnical Engineer of Record throughout the construction phase and provide compaction testing and observation services and foundation observations throughout the project. It is important to maintain continuity of geotechnical interpretation and confirm that field conditions encountered are similar to those anticipated during design. If we are not retained for these services, we cannot assume any responsibility for others interpretation of our recommendations, and therefore the future performance of the project.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon Consultants, Inc. should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the geotechnical scope of services provided by Geocon Consultants, Inc.

This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices used in the site area at this time. No warranty is provided, express or implied.

GEOCON CONSULTANTS, INC.

6671 BRISA STREET – LIVERMORE, CA 94550
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Vivian Street Townhomes

88 Vivian Street
San Rafael, California

VICINITY MAP

E9226-04-01	January 2021	Figure 1

LEGEND:

B5 Approximate Boring Location

CPT-3 Approximate CPT Location

0 80
Scale in Feet

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Vivian Street Townhomes		
88 Vivian Street San Rafael, California		
SITE PLAN		
E9226-04-01	January 2021	Figure 2

APPENDIX A

APPENDIX A
FIELD EXPLORATION

Fieldwork for our investigation included a site visit, subsurface exploration, and soil sampling. The locations of our exploratory borings and CPTs are shown on the Site Development Plan, Figure 2. Soil boring logs and CPT profiles for our exploration are presented as figures in this Appendix A. The borings and CPTs were located by pacing from existing reference points. Therefore, the exploration locations shown on Figure 2 are approximate.

Our field exploration included four exploratory soil borings to depths ranging from 15 to 25 feet below existing grade. Our borings were performed by Clear Heart Drilling under Geocon supervision on November 19, 2020 using a track-mounted DR8K drill rig equipped with 7-inch OD hollow-stem and 4-inch OD solid flash augers. Sampling in the borings was accomplished using an auto hammer 140-pound hammer (hammer efficiency of about 80%) with a 30-inch drop. Samples were obtained with a 3-inch outside-diameter (OD), split spoon (California Modified) sampler, and a 2-inch OD, Standard Penetration Test (SPT) sampler. The number of blows required to drive the sampler the last 12 inches (or fraction thereof) of the 18-inch sampling interval were recorded on the boring logs. The blow counts shown on the boring logs should not be interpreted as standard SPT "N" values; corrections have not been applied.

Our exploration also included three CPT soundings to maximum depths of approximately 90 feet below existing grade utilizing a truck-mounted CPT rig with a down-pressure capacity of approximately 20 tons. The CPTs were performed on November 10, 2020 by Middle Earth Geo Testing using an integrated electronic cone system under Geocon supervision. The cone has a tip area of 15 square centimeters, a friction sleeve area of 225 square centimeters, and a ratio of friction sleeve area to tip end area equal to 0.8. The cone bearing (Q_c) and sleeve friction (F_s) were measured and recorded during tests at approximately 2-inch depth intervals. The CPT data consisting of cone bearing, sleeve friction, friction ratio and equivalent standard penetration blow counts (N) versus penetration depth below the existing ground surface for each location has been recorded and is presented in this appendix.

Subsurface conditions encountered in the exploratory boring were visually examined, classified and logged in general accordance with the American Society for Testing and Materials (ASTM) Practice for Description and Identification of Soils (Visual-Manual Procedure D2488). This system uses the Unified Soil Classification System (USCS) for soil designations. The log depicts soil and geologic conditions encountered and depths at which samples were obtained. The log also includes our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, drill rig penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the field logs were revised based on subsequent laboratory testing. Upon completion, our soil borings and CPT boreholes were backfilled per Marin County permit requirements.

UNIFIED SOIL CLASSIFICATION

MAJOR DIVISIONS				TYPICAL NAMES
COARSE-GRAINED SOILS MORE THAN HALF IS COARSER THAN NO. 200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO.4 SIEVE SIZE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL GRADED GRAVELS WITH OR WITHOUT SAND, LITTLE OR NO FINES
			GP	POORLY GRADED GRAVELS WITH OR WITHOUT SAND, LITTLE OR NO FINES
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, SILTY GRAVELS WITH SAND
			GC	CLAYEY GRAVELS, CLAYEY GRAVELS WITH SAND
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO.4 SIEVE SIZE	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL GRADED SANDS WITH OR WITHOUT GRAVEL, LITTLE OR NO FINES
			SP	POORLY GRADED SANDS WITH OR WITHOUT GRAVEL, LITTLE OR NO FINES
		SANDS WITH OVER 12% FINES	SM	SILTY SANDS WITH OR WITHOUT GRAVEL
			SC	CLAYEY SANDS WITH OR WITHOUT GRAVEL
FINE-GRAINED SOILS MORE THAN HALF IS FINER THAN NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50% OR LESS		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTS WITH SANDS AND GRAVELS
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, CLAYS WITH SANDS AND GRAVELS, LEAN CLAYS
			OL	ORGANIC SILTS OR CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50%		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OR CLAYS OF MEDIUM TO HIGH PLASTICITY
	HIGHLY ORGANIC SOILS		PT	PEAT AND OTHER HIGHLY ORGANIC SOILS

BORING/TRENCH LOG LEGEND

- No Recovery
- Shelby Tube Sample
- Bulk Sample
- SPT Sample
- Modified California Sample
- Groundwater Level (At Completion)
- Groundwater Level (Seepage)

PENETRATION RESISTANCE

SAND AND GRAVEL			SILT AND CLAY			
RELATIVE DENSITY	BLOWS PER FOOT (SPT)*	BLOWS PER FOOT (MOD-CAL)*	CONSISTENCY	BLOWS PER FOOT (SPT)*	BLOWS PER FOOT (MOD-CAL)*	COMPRESSIVE STRENGTH (tsf)
VERY LOOSE	0 - 4	0 - 6	VERY SOFT	0 - 2	0 - 3	0 - 0.25
LOOSE	5 - 10	7 - 16	SOFT	3 - 4	4 - 6	0.25 - 0.50
MEDIUM DENSE	11 - 30	17 - 48	MEDIUM STIFF	5 - 8	7 - 13	0.50 - 1.0
DENSE	31 - 50	49 - 79	STIFF	9 - 15	14 - 24	1.0 - 2.0
VERY DENSE	OVER 50	OVER 79	VERY STIFF	16 - 30	25 - 48	2.0 - 4.0
			HARD	OVER 30	OVER 48	OVER 4.0

*NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE LAST 12 INCHES OF AN 18-INCH DRIVE

MOISTURE DESCRIPTIONS

FIELD TEST	APPROX. DEGREE OF SATURATION, S (%)	DESCRIPTION
NO INDICATION OF MOISTURE; DRY TO THE TOUCH	$S<25$	DRY
SLIGHT INDICATION OF MOISTURE	$25\le S<50$	DAMP
INDICATION OF MOISTURE; NO VISIBLE WATER	$50\le S<75$	MOIST
MINOR VISIBLE FREE WATER	$75\le S<100$	WET
VISIBLE FREE WATER	100	SATURATED

QUANTITY DESCRIPTIONS

APPROX. ESTIMATED PERCENT	DESCRIPTION
<5%	TRACE
5 - 10%	FEW
11 - 25%	LITTLE
26 - 50%	SOME
>50%	MOSTLY

GRAVEL/COBBLE/BOULDER DESCRIPTIONS

CRITERIA	DESCRIPTION
PASS THROUGH A 3-INCH SIEVE AND BE RETAINED ON A NO. 4 SIEVE (#4 TO 3")	GRAVEL
PASS A 12-INCH SQUARE OPENING AND BE RETAINED ON A 3-INCH SIEVE (3"-12")	COBBLE
WILL NOT PASS A 12-INCH SQUARE OPENING (>12")	BOULDER

BEDDING SPACING DESCRIPTIONS

THICKNESS/SPACING	DESCRIPTOR
GREATER THAN 10 FEET	MASSIVE
3 TO 10 FEET	VERY THICKLY BEDDED
1 TO 3 FEET	THICKLY BEDDED
3 5/8-INCH TO 1 FOOT	MODERATELY BEDDED
1 1/4-INCH TO 3 5/8-INCH	THINLY BEDDED
3/8-INCH TO 1 1/4-INCH	VERY THINLY BEDDED
LESS THAN 3/8-INCH	LAMINATED

STRUCTURE DESCRIPTIONS

CRITERIA	DESCRIPTION
ALTERNATING LAYERS OF VARYING MATERIAL OR COLOR WITH LAYERS AT LEAST 1/4-INCH THICK	STRATIFIED
ALTERNATING LAYERS OF VARYING MATERIAL OR COLOR WITH LAYERS LESS THAN 1/4-INCH THICK	LAMINATED
BREAKS ALONG DEFINITE PLANES OF FRACTURE WITH LITTLE RESISTANCE TO FRACTURING	FISSURED
FRACTURE PLANES APPEAR POLISHED OR GLOSSY, SOMETIMES STRIATED	SLICKENSIDED
COHESIVE SOIL THAT CAN BE BROKEN DOWN INTO SMALLER ANGULAR LUMPS WHICH RESIST FURTHER BREAKDOWN	BLOCKY
INCLUSION OF SMALL POCKETS OF DIFFERENT SOIL, SUCH AS SMALL LENSES OF SAND SCATTERED THROUGH A MASS OF CLAY	LENSED
SAME COLOR AND MATERIAL THROUGHOUT	HOMOGENOUS

CEMENTATION/INDURATION DESCRIPTIONS

FIELD TEST	DESCRIPTION
CRUMBLES OR BREAKS WITH HANDLING OR LITTLE FINGER PRESSURE	WEAKLY CEMENTED/INDURATED
CRUMBLES OR BREAKS WITH CONSIDERABLE FINGER PRESSURE	MODERATELY CEMENTED/INDURATED
WILL NOT CRUMBLE OR BREAK WITH FINGER PRESSURE	STRONGLY CEMENTED/INDURATED

IGNEOUS/METAMORPHIC ROCK STRENGTH DESCRIPTIONS

FIELD TEST	DESCRIPTION
MATERIAL CRUMBLES WITH BARE HAND	WEAK
MATERIAL CRUMBLES UNDER BLOWS FROM GEOLOGY HAMMER	MODERATELY WEAK
1/8-INCH INDENTATIONS WITH SHARP END FROM GEOLOGY HAMMER	MODERATELY STRONG
HAND-HELD SPECIMEN CAN BE BROKEN WITH **ONE** BLOW FROM GEOLOGY HAMMER	STRONG
HAND-HELD SPECIMEN CAN BE BROKEN WITH **COUPLE** BLOWS FROM GEOLOGY HAMMER	VERY STRONG
HAND-HELD SPECIMEN CAN BE BROKEN WITH **MANY** BLOWS FROM GEOLOGY HAMMER	EXTREMELY STRONG

IGNEOUS/METAMORPHIC ROCK WEATHERING DESCRIPTIONS

DEGREE OF DECOMPOSITION	FIELD RECOGNITION	ENGINEERING PROPERTIES
SOIL	DISCOLORED, CHANGED TO SOIL, FABRIC DESTROYED	EASY TO DIG
COMPLETELY WEATHERED	DISCOLORED, CHANGED TO SOIL, FABRIC MAINLY PRESERVED	EXCAVATED BY HAND OR RIPPING (Saprolite)
HIGHLY WEATHERED	DISCOLORED, HIGHLY FRACTURED, FABRIC ALTERED AROUND FRACTURES	EXCAVATED BY HAND OR RIPPING, WITH SLIGHT DIFFICULTY
MODERATELY WEATHERED	DISCOLORED, FRACTURES, INTACT ROCK-NOTICEABLY WEAKER THAN FRESH ROCK	EXCAVATED WITH DIFFICULTY WITHOUT EXPLOSIVES
SLIGHTLY WEATHERED	MAY BE DISCOLORED, SOME FRACTURES, INTACT ROCK-NOT NOTICEABLY WEAKER THAN FRESH ROCK	REQUIRES EXPLOSIVES FOR EXCAVATION, WITH PERMEABLE JOINTS AND FRACTURES
FRESH	NO DISCOLORATION, OR LOSS OF STRENGTH	REQUIRES EXPLOSIVES

IGNEOUS/METAMORPHIC ROCK JOINT/FRACTURE DESCRIPTIONS

FIELD TEST	DESCRIPTION
NO OBSERVED FRACTURES	UNFRACTURED/UNJOINTED
MAJORITY OF JOINTS/FRACTURES SPACED AT 1 TO 3 FOOT INTERVALS	SLIGHTLY FRACTURED/JOINTED
MAJORITY OF JOINTS/FRACTURES SPACED AT 4-INCH TO 1 FOOT INTERVALS	MODERATELY FRACTURED/JOINTED
MAJORITY OF JOINTS/FRACTURES SPACED AT 1-INCH TO 4-INCH INTERVALS WITH SCATTERED FRAGMENTED INTERVALS	INTENSELY FRACTURED/JOINTED
MAJORITY OF JOINTS/FRACTURES SPACED AT LESS THAN 1-INCH INTERVALS; MOSTLY RECOVERED AS CHIPS AND FRAGMENTS	VERY INTENSELY FRACTURED/JOINTED

GEOCON
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PHONE 925.961.5271 – FAX 925.371.5915

KEY TO LOGS

PROJECT NO. **E9226-04-01** PROJECT NAME: **Vivian St. Townhomes**

BORING B1

ELEV. (MSL.) ____________ DATE COMPLETED **11/19/2020**

ENG./GEO. **AA** DRILLER **Clear Heart Drilling**

EQUIPMENT **DR8k Track Rig w/ 7" HSA** HAMMER TYPE **Auto-hammer**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					Approximately 2 inches of AC			
1					Approximately 9 inches of AB			
				CL	FILL			
2	B1-1.5 B1-2				Hard, damp, black to dark brown, CLAY with (f) sand	45		5.3
3				SC	Medium dense, wet, brown, Clayey SAND with trace (f) gravel			
4			▼					
5	B1-4.5					20		12.4
6				CH	BAY MUD Very soft, gray, wet, fat CLAY			
7	B1-6.5 B1-7				-some organics at the top	3	57.1	67.4
8					-pp<¼			
9	B1-8.5-10.5						42.4	110.0
10					-pp<¼			
11								
12								
13								
14								
15								
16								
17								
18								
19						0		
20	B1-19.5				-pp<¼		54	74.3

END OF BORING AT APPROXIMATELY 20 FEET
GROUNDWATER INITIALLY ENCOUNTERED AT APPROXIMATELY 4 FEET
BACKFILLED WITH NEAT GROUT CEMENT

Figure A2, Log of Boring B1, Page 1 of 1

GEOCON

SAMPLE SYMBOLS
- ☐ ... SAMPLING UNSUCCESSFUL
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◧ ... STANDARD PENETRATION TEST
- ◪ ... CHUNK SAMPLE
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. **E9226-04-01** PROJECT NAME: **Vivian St. Townhomes**

BORING B2

ELEV. (MSL.)	
ENG./GEO.	AA
EQUIPMENT	DR8k Track Rig w/ 7" HSA
DATE COMPLETED	11/19/2020
DRILLER	Clear Heart Drilling
HAMMER TYPE	Auto-hammer

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					Approximately 3 inches of AC			
1					Approximately 9 inches of AB			
2	B2-1.5-2.5			CL	FILL Moist, dark brown, CLAY with (f) sand	21		
3	B2-2.5-3.5			GC	Medium dense, moist, olive, GRAVEL with (m) sand, clay, and silt			
4	B2-4-5			CL	Stiff, moist, brown, (f-m) Sandy CLAY	13		12.3
5			▼					
6								
7	B2-6.5 B2-7			CH	BAY MUD Soft, moist, gray and olive, fat CLAY -pp<¼	9		
8								
9								
10	B2-9.5				-very soft, gray -pp<¼	0		
11								
12								
13								
14								
15	B2-14.5				-pp<¼	0	49.0	83.1
16								
17								
18								
19								
20								
21								
22								
23								
24								
25	B2-24.5				-pp<¼	0	64.3	54.2

END OF BORING AT APPROXIMATELY 25 FEET
GROUNDWATER INITIALLY ENCOUNTERED AT APPROXIMATELY 5 FEET
BACKFILLED WITH NEAT GROUT CEMENT

Figure A3, Log of Boring B2, Page 1 of 1

GEOCON BORING LOG W/FIG# STARTING W/ A2 E9226-04-01 BORING LOGS.GPJ 12/22/20

GEOCON

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◧ ... STANDARD PENETRATION TEST
- ◪ ... CHUNK SAMPLE
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. **E9226-04-01** PROJECT NAME: **Vivian St. Townhomes**

BORING B3

ELEV. (MSL.) ____ DATE COMPLETED **11/19/2020**
ENG./GEO. **AA** DRILLER **Clear Heart Drilling**
EQUIPMENT **DR8k Track Rig w/ 4" SFA** HAMMER TYPE **Auto-hammer**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					Approximately 4 inches of AC			
1	B3-1-3			CL	Approximately 8 inches of AB			
2	B3-1.5-2.5				FILL Medium stiff, moist, brown/olive, CLAY with (f) sand -pp=1¼	12	116.9	15.1
3				SC	Loose, wet, dark brown/black, Clayey (f-m) SAND with trace (f) gravel			
4			▼			10		
5	B3-4.5-5							12.5
6				CH	BAY MUD Very soft, wet, black/olive/gray, fat CLAY with trace (f) sand -pp<¼			
7	B3-6.5-7.5					2		47.2
8								
9						0		
10	B3-9.5				-gray, no sand -pp<¼			
11								
12								
13								
14						0		
15	B3-14.5						51.1	82.4

END OF BORING AT APPROXIMATELY 15 FEET
GROUNDWATER INITIALLY ENCOUNTERED AT APPROXIMATELY 4 FEET
BACKFILLED WITH NEAT GROUT CEMENT

Figure A4, Log of Boring B3, Page 1 of 1

GEOCON BORING LOG W/FIG# STARTING W/ A2 E9226-04-01 BORING LOGS.GPJ 12/22/20

GEOCON

SAMPLE SYMBOLS

- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. **E9226-04-01** PROJECT NAME: **Vivian St. Townhomes**

BORING B4

ELEV. (MSL.) ____________ DATE COMPLETED **11/19/2020**

ENG./GEO. **AA** DRILLER **Clear Heart Drilling**

EQUIPMENT **DR8k Track Rig w/ 7" HSA** HAMMER TYPE **Auto-hammer**

DEPTH IN FEET	SAMPLE NO.	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				Approximately 6 inches of AC			
				Approximately 5½ inches of AB			
1	B4-1-3		CL	FILL Stiff, moist, brown, (f) Sandy CLAY and trace (f) gravel			
3	B4-3.5					128.4	13.4
4	B4-4		GM	Medium dense, moist, brown, Silty (f-c) GRAVEL with (f-m) sand	29		8.1
5		▼					
6			CH	BAY MUD Very soft to soft, wet, gray, fat CLAY			
7					2		
9	B4-9.5			-pp<¼	2	50.8	81.0
19	B4-19.5			-very soft	0	56.1	69.1
20				END OF BORING AT APPROXIMATELY 20 FEET GROUNDWATER INITIALLY ENCOUNTERED AT 5 FEET BACKFILLED WITH NEAT GROUT CEMENT			

Figure A5, Log of Boring B4, Page 1 of 1

GEOCON BORING LOG W/FIG# STARTING W/ A2 E9226-04-01 BORING LOGS.GPJ 12/22/20

GEOCON

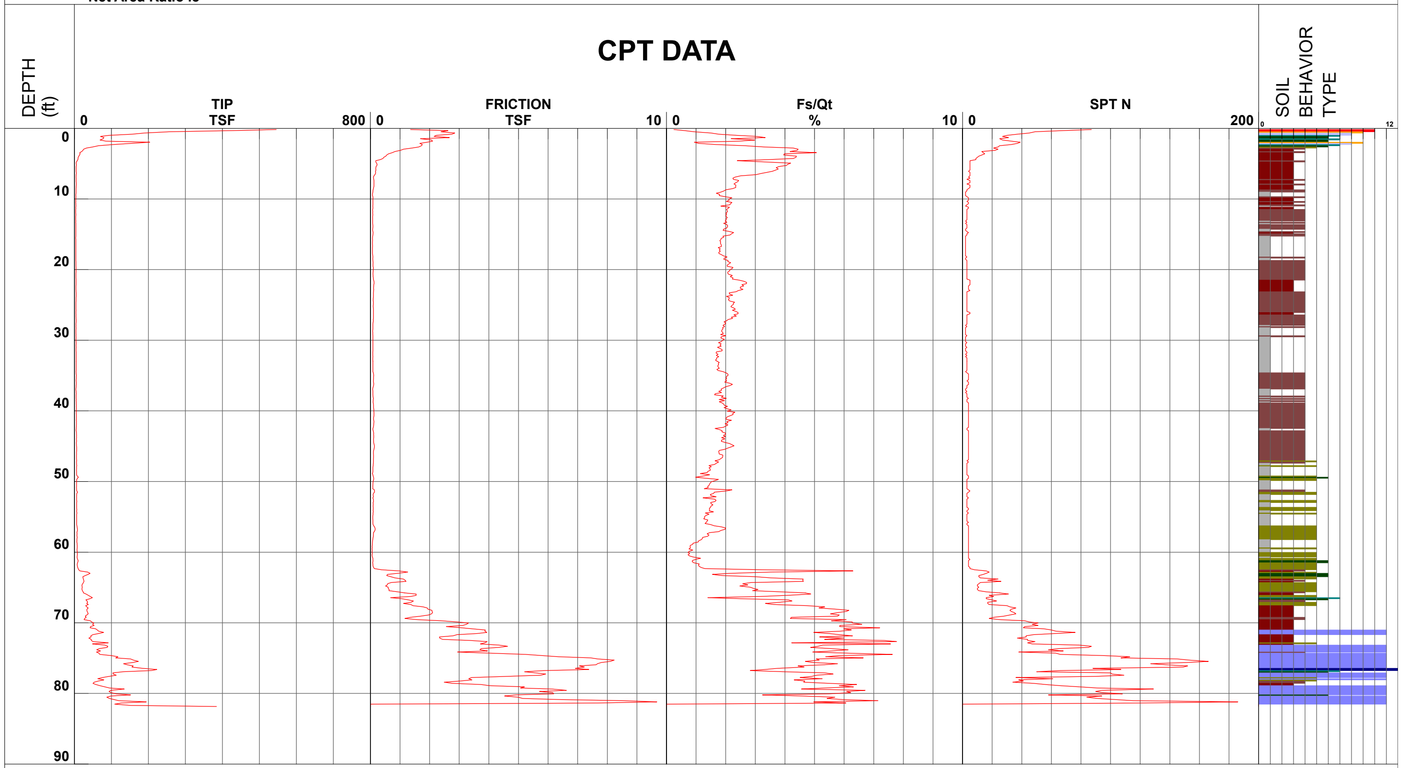
SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... STANDARD PENETRATION TEST
- ... DRIVE SAMPLE (UNDISTURBED)
- ... DISTURBED OR BAG SAMPLE
- ... CHUNK SAMPLE
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

Geocon Inc.

Project	Vivian Street Townhome	Operator	BH-AJ	Filename	SDF(364).cpt
Job Number	E9226-04-01	Cone Number	DDG1496	GPS	
Hole Number	CPT-01	Date and Time	11/10/2020 8:04:44 AM	Maximum Depth	81.86 ft
EST GW Depth During Test		5.00 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

GEOCON CONSULTANTS, INC.
6671 BRISA STREET – LIVERMORE, CA 94550
PHONE 925.371.5900 – FAX 925.371.5915

CONE PENETROMETER TEST DATA - CPT-1

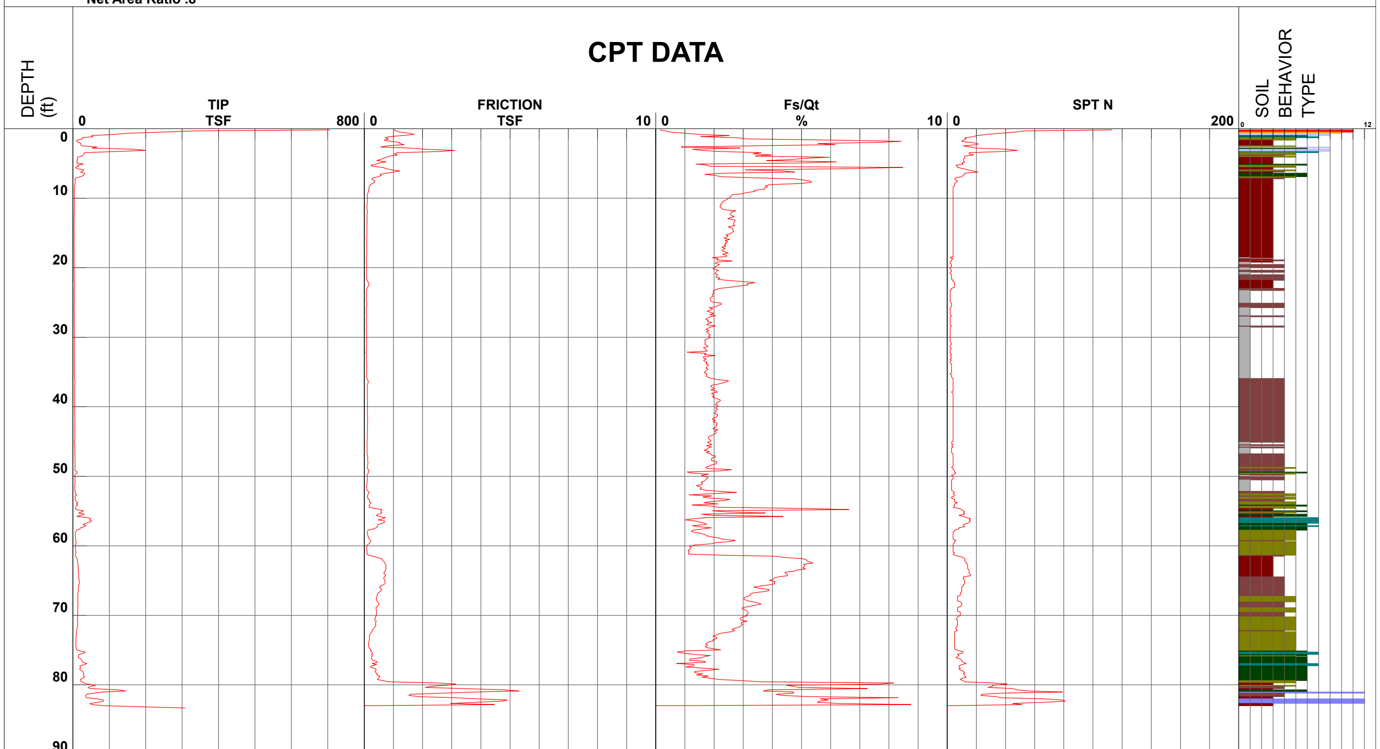
Project: Vivian Street Townhomes
Project No. E9226-04-01
Date: January 2021

FIGURE A6

Geocon Inc.

Project	Vivian Street Townhome	Operator	BH-AJ	Filename	SDF(365).cpt
Job Number	E9226-04-01	Cone Number	DDG1496	GPS	
Hole Number	CPT-02	Date and Time	11/10/2020 11:01:20 AM	Maximum Depth	83.33 ft
EST GW Depth During Test		4.00 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

GEOCON CONSULTANTS, INC.
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CONE PENETROMETER TEST DATA - CPT-2

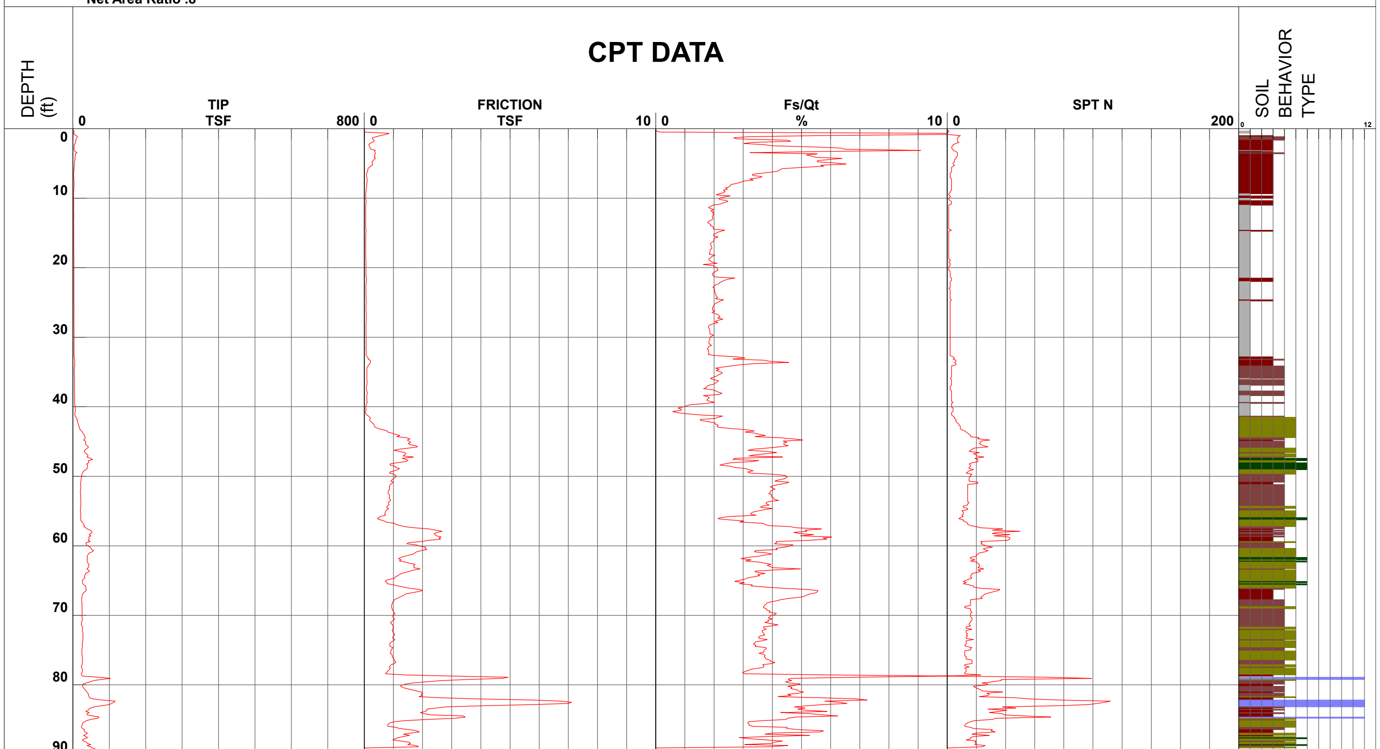
Project: Vivian Street Townhomes
Project No. E9226-04-01
Date: January 2021

FIGURE A7

Geocon Inc.

Project	Vivian Street Townhome	Operator	BH-AJ	Filename	SDF(366).cpt
Job Number	E9226-04-01	Cone Number	DDG1496	GPS	
Hole Number	CPT-03	Date and Time	11/10/2020 2:08:57 PM	Maximum Depth	89.40 ft
EST GW Depth During Test		7.00 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

GEOCON CONSULTANTS, INC.
6671 BRISA STREET - LIVERMORE, CA 94550
PHONE 925.371.5900 - FAX 925.371.5915

CONE PENETROMETER TEST DATA - CPT-3

Project: Vivian Street Townhomes
Project No. E9226-04-01
Date: January 2021

FIGURE A8

APPENDIX B

APPENDIX B
LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. Selected samples were tested for in-situ dry density and moisture content, grain size distribution, plasticity, expansion index, and screening-level corrosion parameters. The results of our testing are summarized in tabular format below and the following figures. In-situ dry density and moisture content test results are included on the boring logs in Appendix A.

TABLE B-I
SUMMARY OF LABORATORY ATTERBERG LIMITS TEST RESULTS
ASTM D 4318

Sample No.	Liquid Limit	Plastic Limit	Plasticity Index
B2/B3/B4-1-3.5	41	17	24
B2-7	93	36	57
B3-14.5	98	34	64

TABLE B-II
SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
ASTM D 4829

Sample No.	Moisture Content		Dry Density* (pcf)	Expansion Index
	Before Test (%)	After Test (%)		
B2/B3/B4-1-3.5	9.3	19.2	112.9	60

*Before saturation.

TABLE B-III
SUMMARY OF LABORATORY GRAIN SIZE ANALYSIS – NO. 200 WASH
ASTM D1140

Boring No.	Sample Depth (feet)	Fraction Passing No. 200 Sieve (%)
B1	4.5	25.6
B2	4-5	55.5
B3	4.5-5	32.3

TABLE B-IV
SUMMARY OF SOIL CORROSION PARAMETERS
(CTM 643, CTM 417, CTM 422)

Boring No. (sample depth in feet)	Soil Type (USCS Classification)	Resistivity (ohm-cm)	pH	Chloride (ppm)	Sulfate (ppm)
B1 (1-1.5)	Sandy CLAY(Fill/CL)	950	7.5	154	25
B2/B3/B4(1-3.5')	Sandy CLAY/Clayey SAND(Fill/CL/SC)	1,000	7.6	198	21

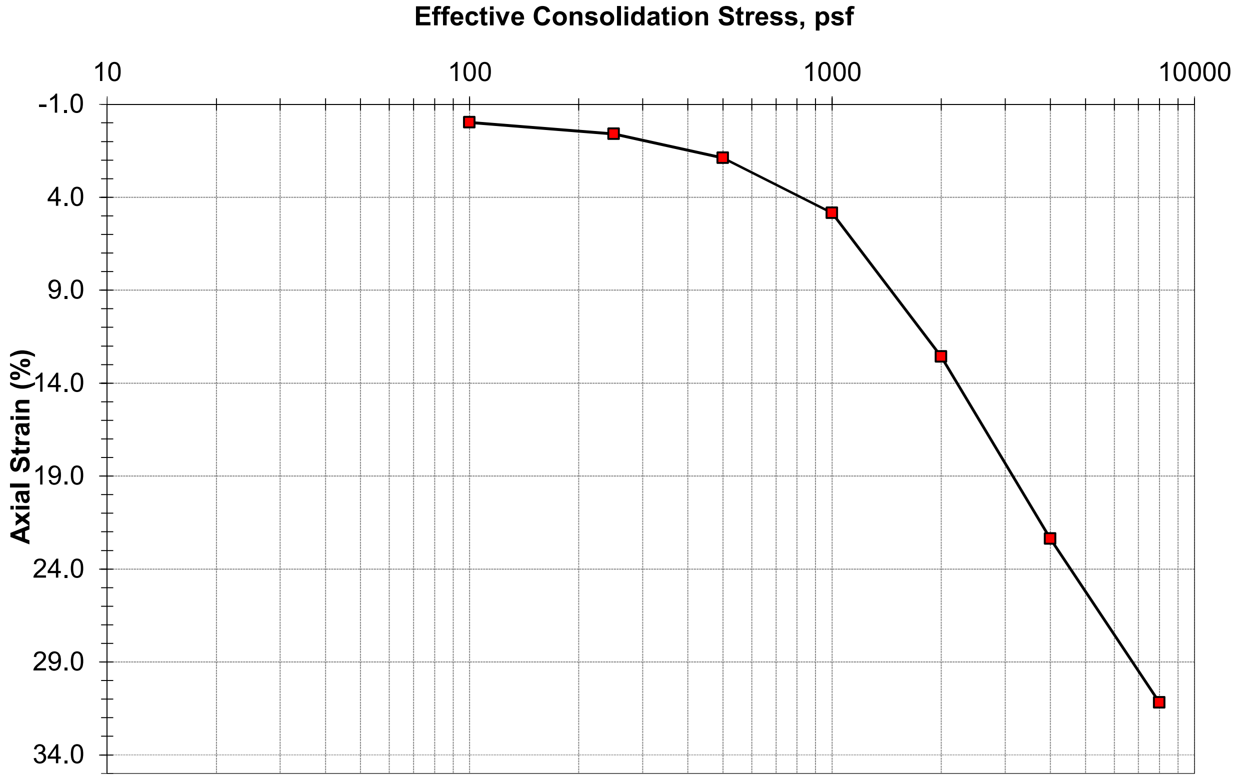
*Caltrans considers a site corrosive to foundation elements if one or more of the following conditions exist for the representative soil samples at the site:

- The pH is equal to or less than 5.5.
- Chloride concentration is equal to or greater than 500 parts per million (ppm) or 0.05%.
- Sulfate concentration is equal to or greater than 2,000 ppm (0.2%)

**According to the American Concrete Institute 318 Chapter 19, Type II cement may be used where sulfate levels are below 2,000 ppm (0.2%)

CONSOLIDATION TEST - ASTM D2435
STRESS VERSUS STRAIN

Project Name	Vivian Street Townhomes
Geocon Project Number	E9226-04-01
Boring Number	B1
Sample Number	B1-8.5-10.5
Sample Description	Very Dark Gray Fat CLAY



Axial Load, psf	Void Ratio	Axial Strain, %
initial	3.196	0.00
100	3.197	-0.03
250	3.171	0.60
500	3.117	1.88
1000	2.993	4.84
2000	2.669	12.57
4000	2.257	22.37
8000	1.888	31.19

Measurement	Initial	Final
Height (in.)	0.750	0.516
Moisture Content (%)	110.0	65.8
Dry Density (pcf)	42.4	61.6
Saturation (%)	98	99

Note:
Gs = 2.85 (assumed)

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Rancho Cordova, CA 95742
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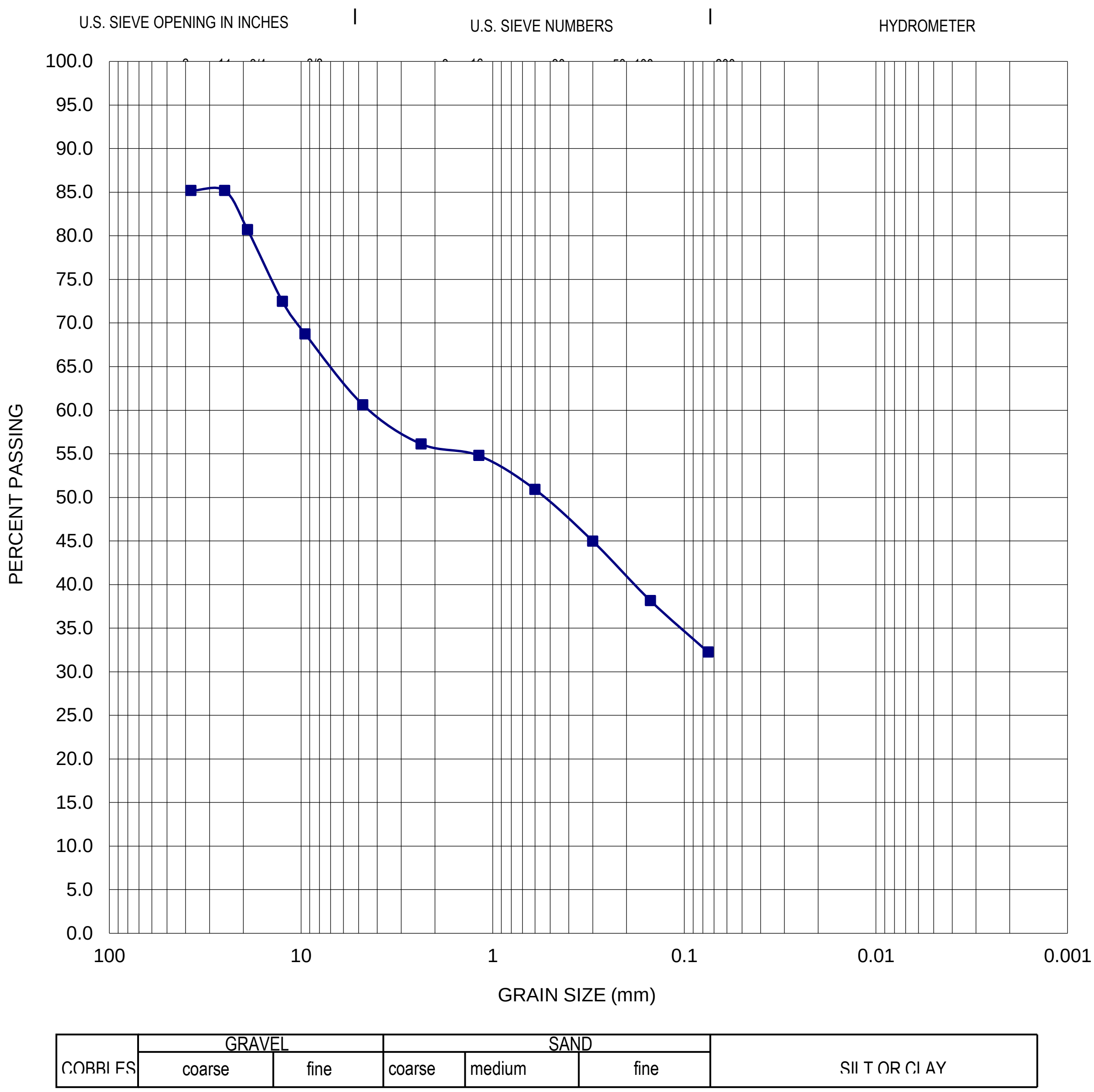
Figure B1



Boring: B4

Depth To Sample: 4'

Sieve Date: 12/7/2020

Tested and Computed by: AC

Test Data

Sieve Number	1 1/2"	1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200
% Passing	85.2	85.2	80.7	72.5	68.7	60.6	56.1	54.8	50.9	45.0	38.1	32.2

Geocon Consultants, Inc.
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Fax: (925) 371-5915

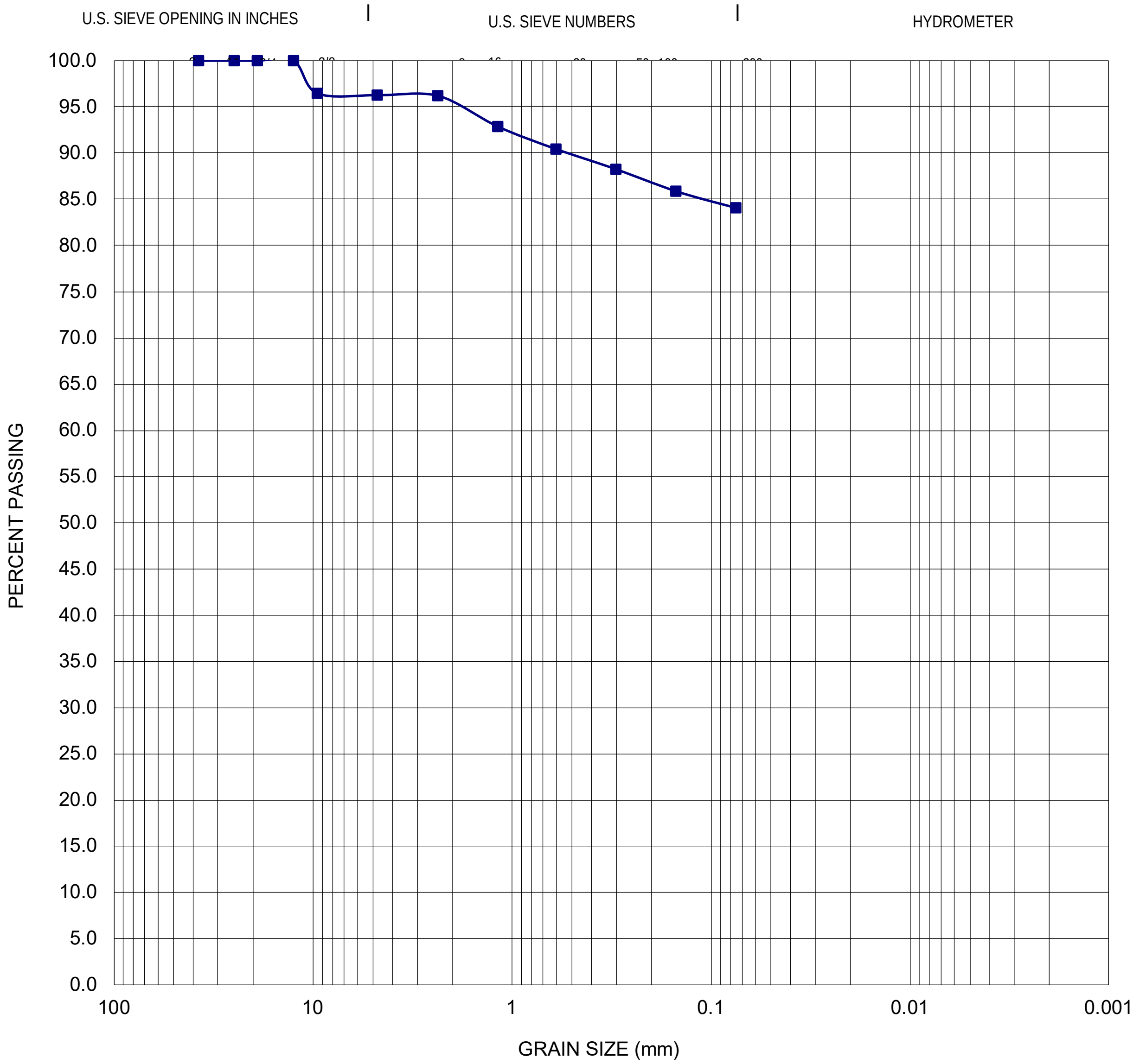
Particle Size Analysis - ASTM D422

Project: Vivian St. Townhomes

Location: San Rafael, CA

Project No.: E9226-04-01

Figure B2



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Boring: B2/B3

Depth To Sample: 1'-3.5'

Sieve Date: 12/4/2020

Tested and Computed by: TG

Test Data

Sieve Number	1 1/2"	1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200
% Passing	100.0	100.0	100.0	100.0	96.5	96.2	96.2	92.9	90.4	88.3	85.9	84.1

Geocon Consultants, Inc.
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Particle Size Analysis - ASTM D422

Project: Vivian St. Townhomes

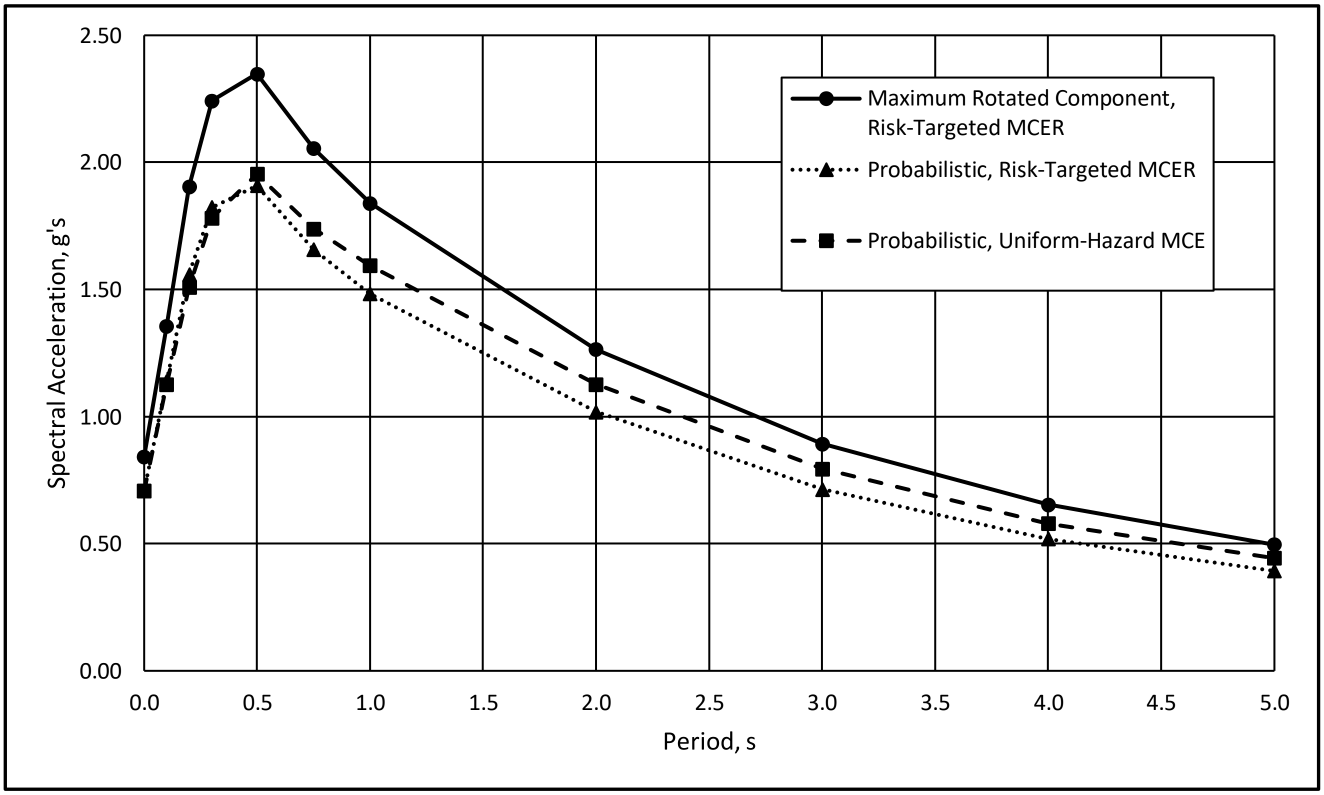
Location: San Rafael, CA

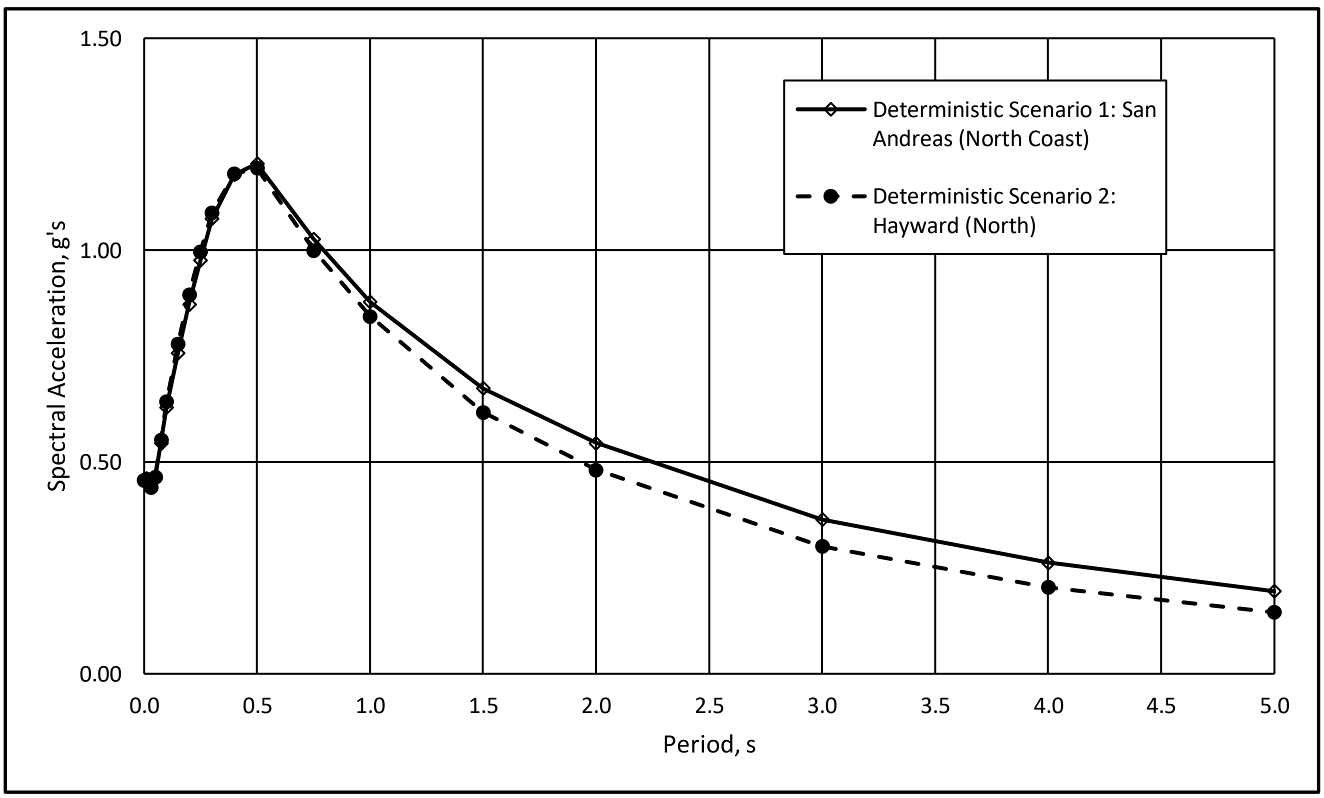
Project No.: E9226-04-01

Figure B3

APPENDIX C

APPENDIX C
GROUND MOTION HAZARD ANALYSIS





GEOCON

DESIGN RESPONSE SPECTRUM

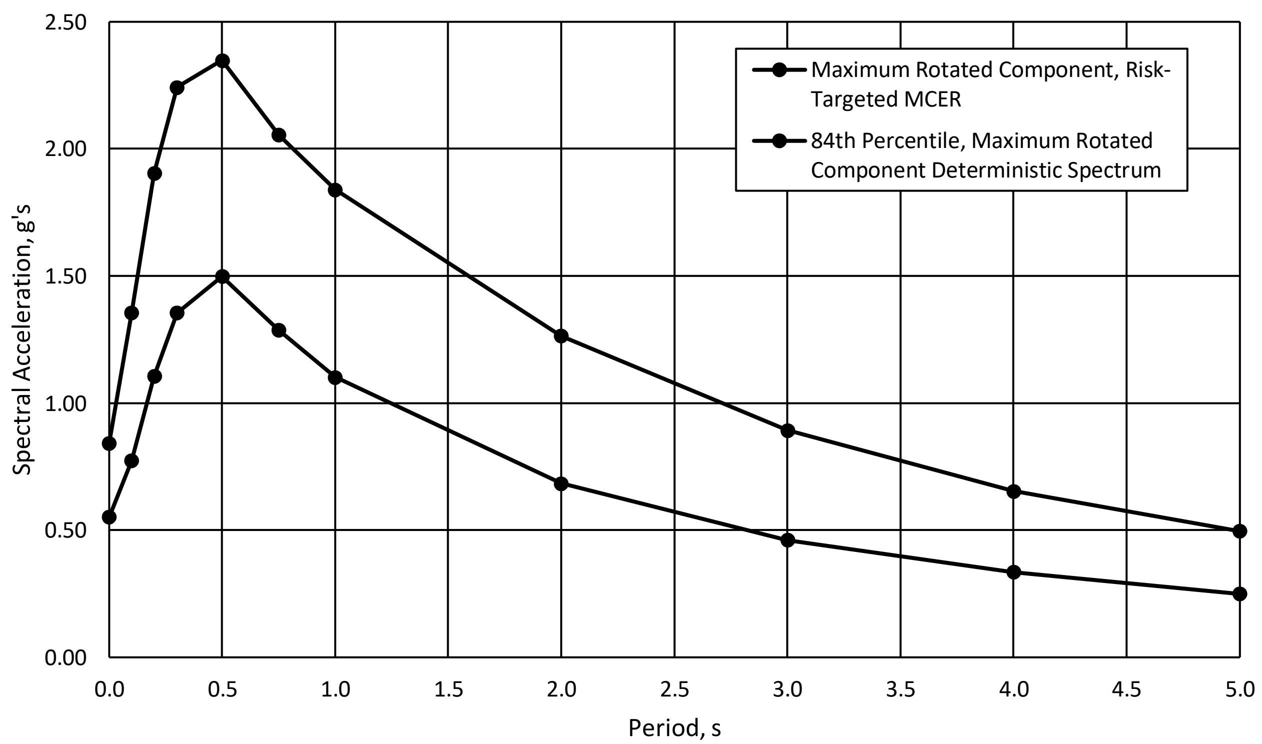
Checked by: JA

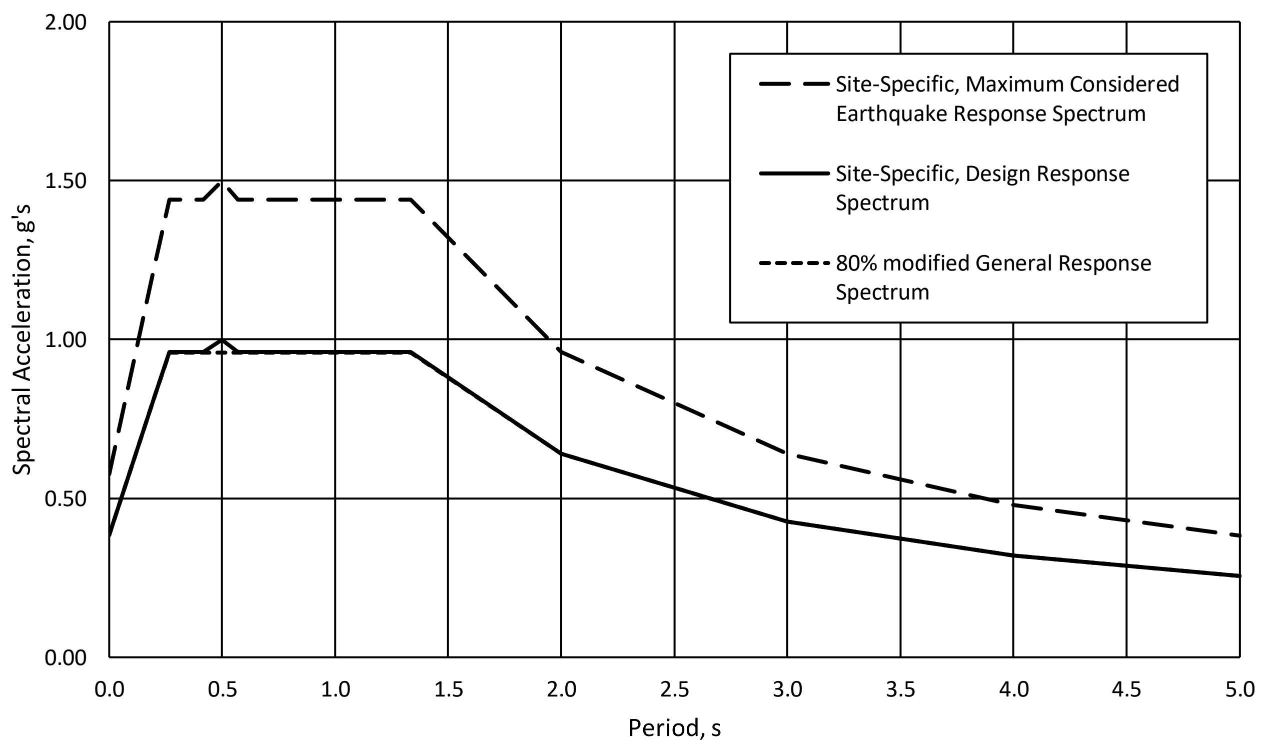
Project No.: E9226-04-01

Vivian Street Townhomes
San Rafael, CA

Dec 20

Figure C1





GEOCON	**DESIGN RESPONSE SPECTRUM**	Project No.: E9226-04-01
		Vivian Street Townhomes San Rafael, CA
	Checked by: JA	Dec 20 Figure C2

Spectral Period (seconds)	Probabilistic Uniform-Hazard	Risk-Targeted, Probabilistic	Risk Factor, Cr	Maximum-Rotated Componet Scale Factor	MRC, Risk-Targeted Probablistic	84th Percentile, Deterministic	Site-Specific Design Earthquake	80% Modifed General Response Spectrum	Site-Specific Maximum Considered Earthquake
0.00	0.708	0.708	1.000	1.190	0.843	0.552	0.384	0.384	0.576
0.10	1.126	1.139	1.011	1.190	1.355	0.774	0.600	0.600	0.900
0.20	1.509	1.560	1.034	1.220	1.903	1.106	0.816	0.816	1.224
0.27	--	--	--	--	--	--	0.960	0.960	1.440
0.30	1.782	1.823	1.023	1.230	2.242	1.355	0.960	0.960	1.440
0.42	--	--	--	--	--	--	0.960	0.960	1.440
0.50	1.954	1.909	0.977	1.230	2.348	1.500	1.000	0.960	1.500
0.57	--	--	--	--	--	--	0.960	0.960	1.440
0.75	1.739	1.657	0.953	1.240	2.055	1.287	0.960	0.960	1.440
1.00	1.594	1.484	0.931	1.240	1.840	1.102	0.960	0.960	1.440
1.33	--	--	--	--	--	--	0.960	0.960	1.440
2.00	1.127	1.019	0.905	1.240	1.264	0.684	0.640	0.640	0.960
3.00	0.795	0.714	0.898	1.250	0.893	0.461	0.427	0.427	0.640
4.00	0.580	0.519	0.894	1.260	0.654	0.335	0.320	0.320	0.480
5.00	0.444	0.394	0.887	1.260	0.497	0.249	0.256	0.256	0.384

SM_S = 1.440 g

SM_1 = 1.920 g

SD_S = 0.960 g

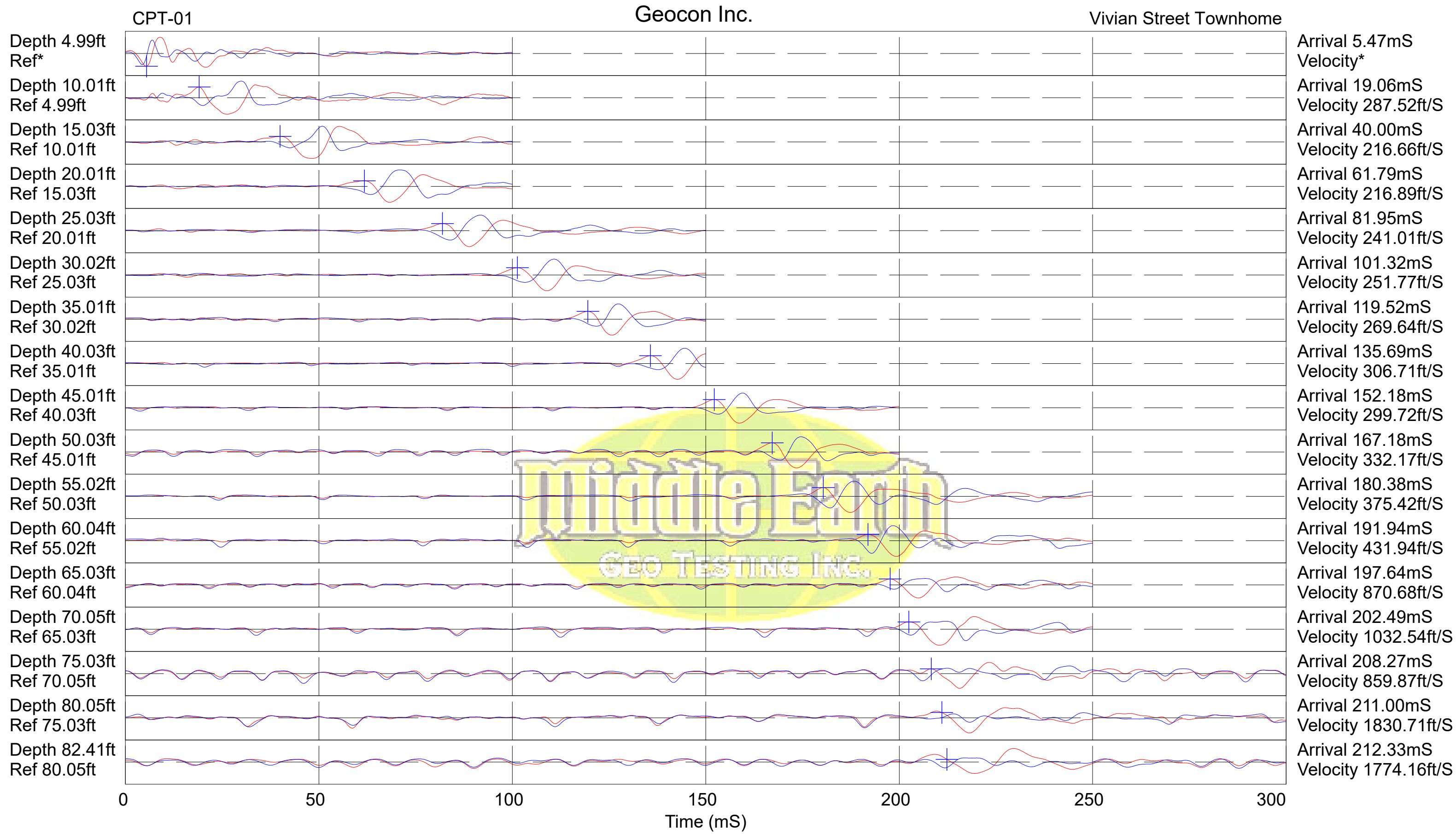
SD_1 = 1.280 g

Reference: ASCE 7-16 21.4 DESIGN ACCELERATION PARAMETERS

Where the site-specific procedure is used to determine the design ground motion in accordance with Section 21.3, the parameter S_{DS} shall be taken as 90% of the maximum spectral acceleration, S_a, obtained from the site-specific spectrum, at any period within the range from 0.2 to 5 s, inclusive. The parameter S_{D1} shall be taken as the maximum value of the product, TS_a, for periods from 1 to 2 s for sites with $v_{s,30}$ > 1,200 ft/s ($v_{s,30}$ > 365.76 m/s) and for periods from 1 to 5 s for sites with $v_{s,30}$ ≤ 1,200 ft/=s ($v_{s,30}$ ≤ 365.76 m/s). The parameters S_{MS} and S_{M1} shall be taken as 1.5 times S_{DS} and S_{D1}, respectively. The values so obtained shall not be less than 80% of the values determined in accordance with Section 11.4.3 for S_{MS} and S_{M1} and Section 11.4.5 for S_{DS} and S_{D1}.

"--" Indicates that spectral period was not used at that calculation step

GEOCON	**DESIGN RESPONSE SPECTRUM**	Project No.: E9226-04-01
		Vivian Street Townhomes San Rafael, CA
	Checked by: JA	Dec 20 Figure C3



Hammer to Rod String Distance (ft): 5.83

* = Not Determined

COMMENT:

GEOCON
CONSULTANTS, INC.
6671 BRISA STREET – LIVERMORE, CA 94550
PHONE 925.371.5900 – FAX 925.371.5915

Shear Velocity Sounding Profile(SCPT-1)

Project: Vivian Street Townhomes
Project No. E9226-04-01
Date: January 2021

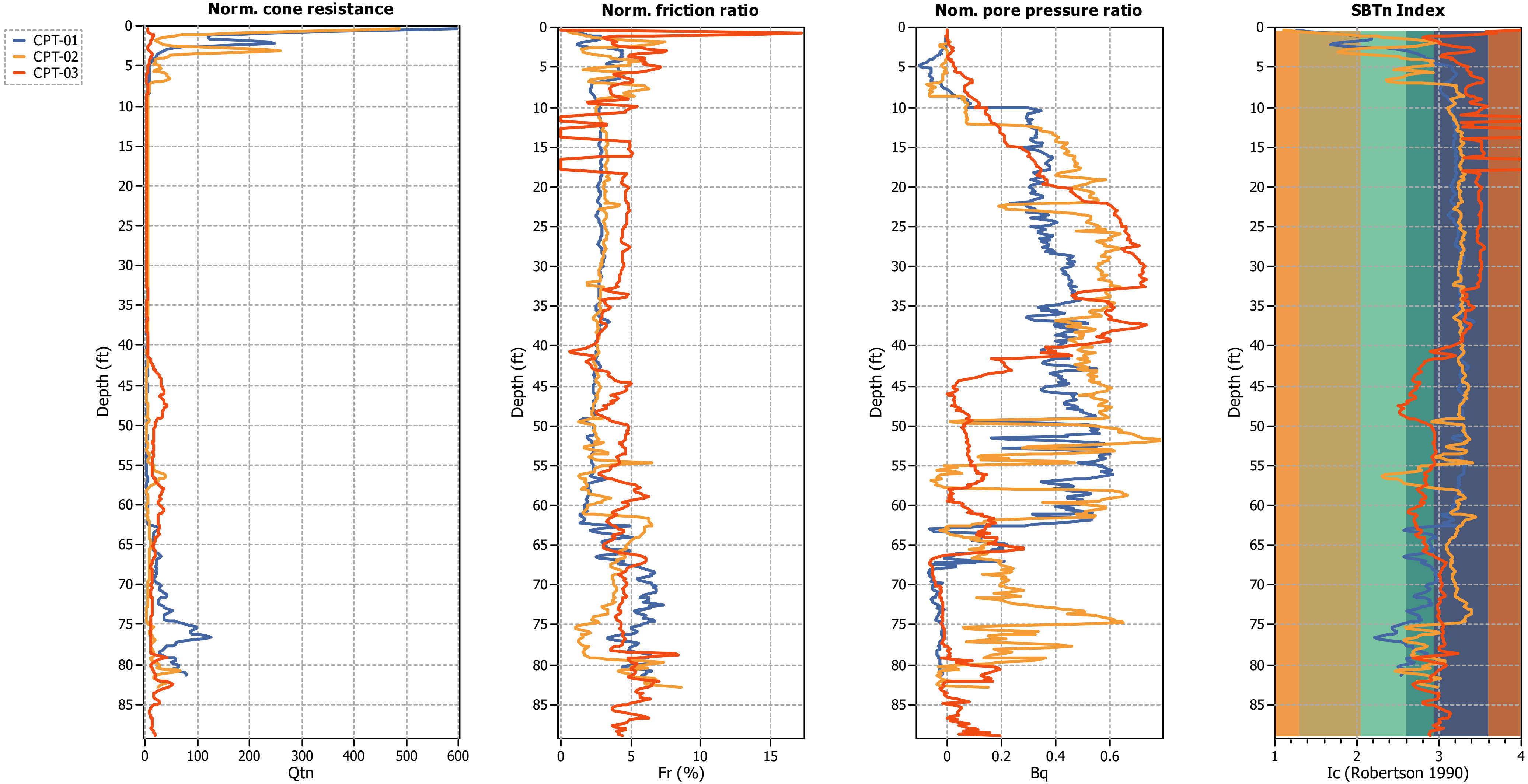
FIGURE C4

APPENDIX D

APPENDIX D
LIQUEFACTION ANALYSIS

Project: Vivian Street Townhomes

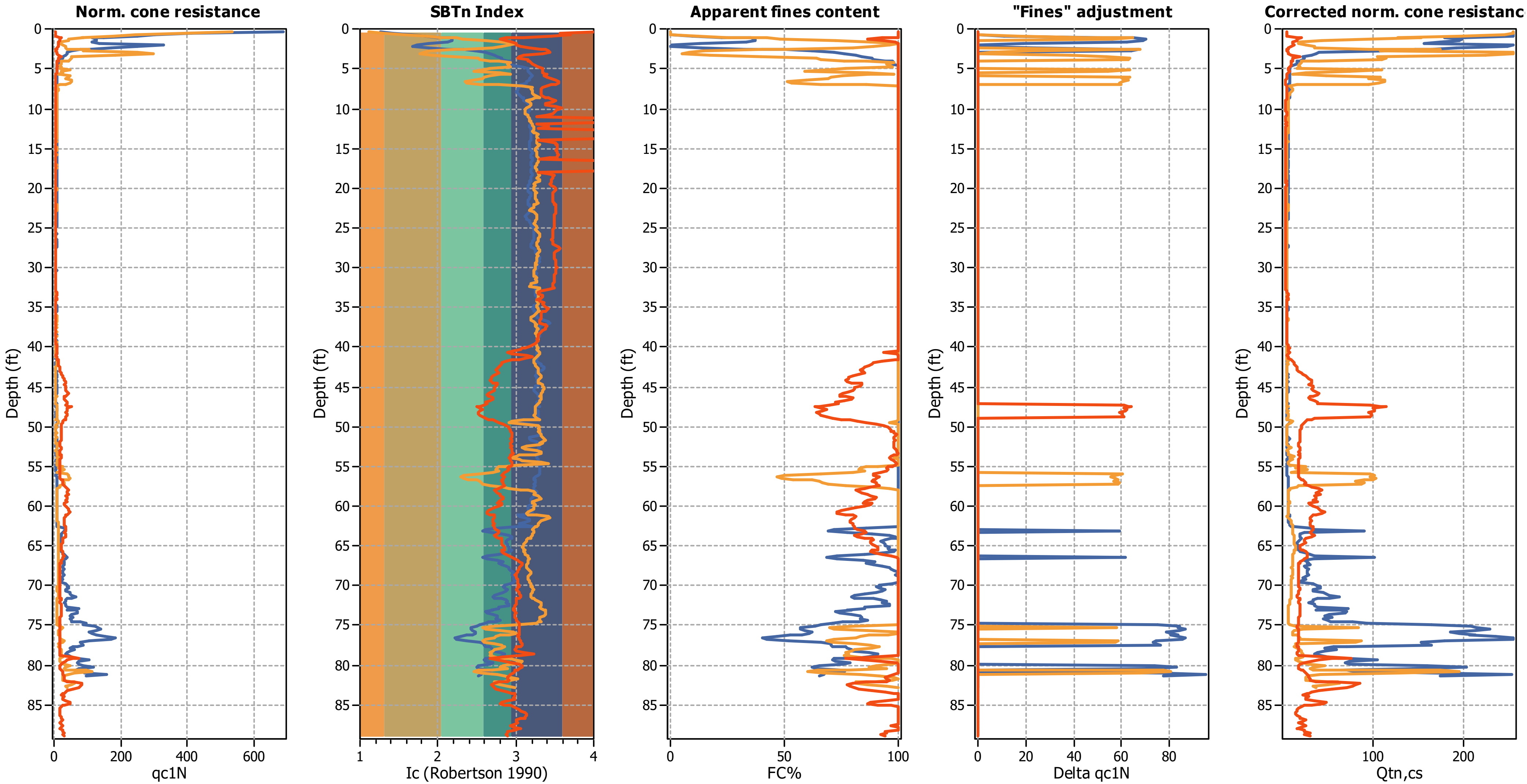
Overlay Normalized Plots



Geocon Consultants Inc.
6671 Brisa Street
Livermore, CA 94550

Project: Vivian Street Townhomes

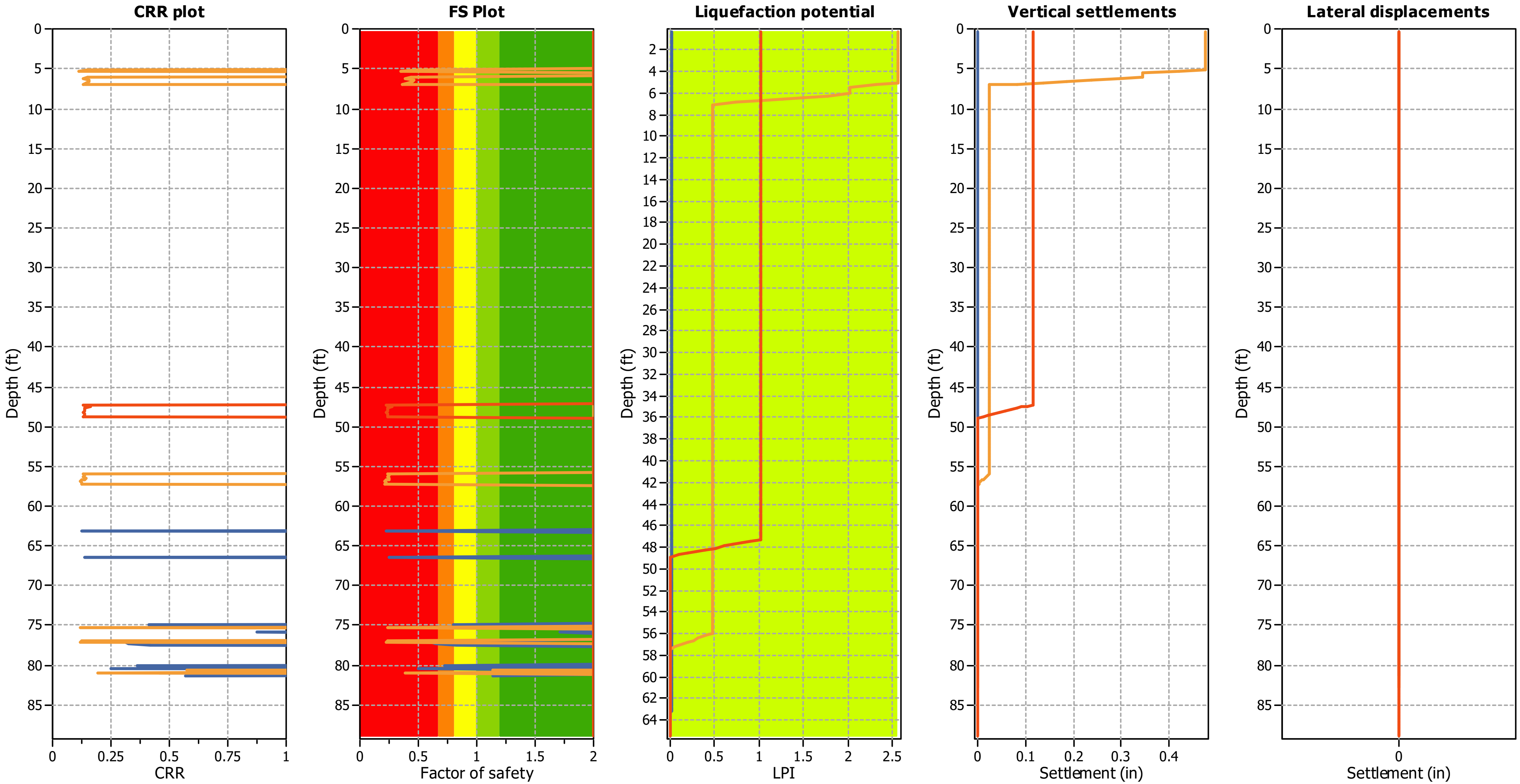
Overlay Intermediate Results



Geocon Consultants Inc.
6671 Brisa Street
Livermore, CA 94550

Project: Vivian Street Townhomes

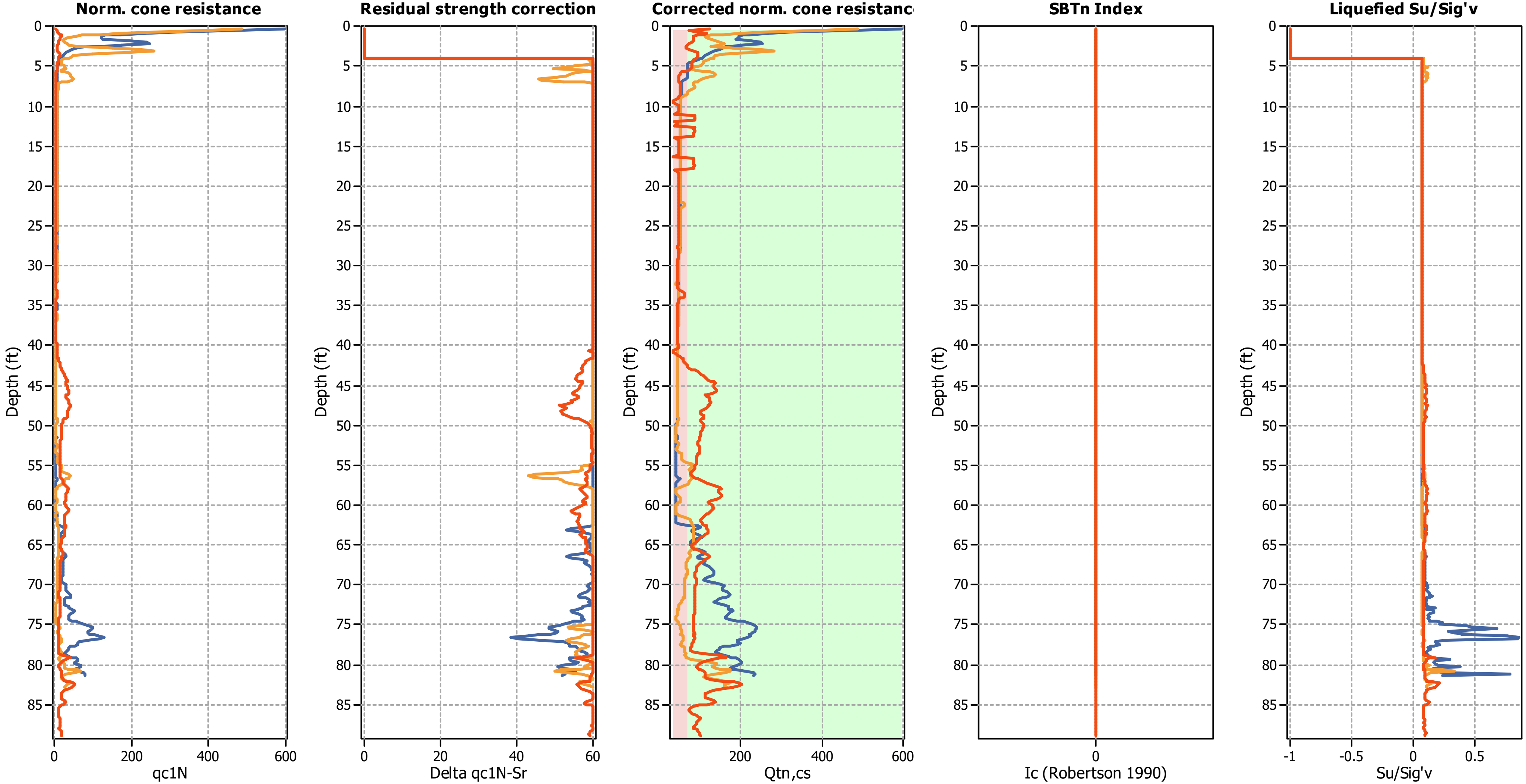
Overlay Cyclic Liquefaction Plots



GEOCON
Geocon Consultants Inc.
6671 Brisa Street
Livermore, CA 94550

Project: Vivian Street Townhomes

Overlay Strength Loss Plots



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