GEO-ENGINEERING SOLUTIONS, INC.

Geotechnical Engineering • Engineering Geology • Materials Testing

UPDATED GEOTECHNICAL ENGINEERING STUDY

1050 St. Elizabeth Drive
San Jose, CA 95126

March 30, 2022

Prepared for:

KCR Development
19620 Stevens Creek Blvd, Suite 200
Cupertino, California 95014

Prepared by:

Geo-Engineering Solutions, Inc.
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San Ramon, California 94583
Project No. 59-1266

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March 30, 2022

KCR Development
19620 Stevens Creek Blvd, Suite 200
Cupertino, California 95014

Attention: Mr. David Chan

Subject: **Updated Geotechnical Engineering Study**
1050 St Elizabeth Drive
San Jose, California 95126
Geo-Eng Project No. 59-1266

Dear Mr. Chan:

Geo-Engineering Solutions, Inc. has prepared a Geotechnical Engineering Study for the proposed redevelopment project located at 1050 St Elizabeth Drive in San Jose, California. It is our understanding that the proposed project will consist of the construction of a new building with a basement and seven-story residential units above it, with various associated improvements such as site grading, landscaping, and utilities. We previously provided an investigation for this project in September of 2020 and this report has been updated to reflect a change in the size of the building as well as the addition of a basement level.

Transmitted herewith are the results of our findings, conclusions, and recommendations for the design and construction of proposed foundation support, interior concrete slabs, site development/grading and drainage, and utility trench backfilling. In general, the proposed improvements at the site are considered to be geotechnically feasible provided the recommendations of this report are implemented in the design and construction of the project.

Should you or members of the design team have questions or need additional information, please contact the undersigned at (925) 433-0450 or by e-mail at eswenson@geo-eng.net. We greatly appreciate the opportunity to be of service to Sand Hill Property Company, and to be involved in the design of this project.

Sincerely,

GEO-ENGINEERING SOLUTIONS, INC.

REGISTERED PROFESSIONAL ENGINEER NICOLAS B. HADDAD C 78126 CIVIL STATE OF CALIFORNIA

Nicolas B. Haddad, PE
Senior Geotechnical Engineer

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1.0 INTRODUCTION

1.1 Purpose and Scope

The purpose of our work was to prepare a Geotechnical Engineering Study, evaluate the subsurface conditions at the site and prepare geotechnical recommendations for the proposed development. We have provided specific recommendations regarding suitability and geotechnical concerns relative to the proposed structural design. This updated investigation report has been prepared to reflect the new project design parameters which include a basement and larger structure.

The scope of this study included the field exploration, laboratory testing, engineering analysis of the collected samples and test results, and preparation of this report. The conclusions and recommendations presented in this report are based on the limited samples collected and analyzed during this study, and on prudent engineering judgment and experience. This study did not include an in-depth assessment of potentially toxic or hazardous materials that may be present on or beneath the site.

1.2 Site Description

The proposed improvement project is located at 1050 St. Elizabeth Drive in San Jose, California as shown on *Figure 1-Site Vicinity Map*. The project site is bordered by St. Elizabeth Drive to the west, residential development to the north, and Los Gatos Creek to the east and south. The project site is currently occupied by single and two-story senior residence buildings and an asphalt parking lot. The topography of the site is relatively flat, with approximate elevations of +137 feet above mean see level. The average geographical coordinates used in our engineering analyses are 37.3052 degrees north latitude and -121.9161 degrees west longitude.

1.3 Proposed Development

We understand that the redevelopment project will consist of the demolition of the existing buildings and asphalt parking lot on site, followed by the construction of a new building with a basement and seven stories of wood-framed residential units above it. In addition, it is assumed there will be associated improvements such as site grading, landscaping, paving and utilities.

1.4 Validity of Report

This report is valid for three years after publication. If construction begins after this time, Geo-Eng should be contacted to confirm that the site conditions have not changed significantly. If the proposed development differs considerably from that described above, Geo-Eng should be notified to determine if additional recommendations are required. Additionally, if Geo-Eng is not involved during the geotechnical aspects of construction, this report may become wholly or in part invalid; since Geo-Eng's geotechnical personnel need to verify that the subsurface conditions anticipated preparing this report are similar to the subsurface conditions revealed during construction. Geo-Eng's involvement should include foundation and grading plan review; observation of foundation excavations; grading observation and testing; testing of utility trench backfill.

2.0 PROCEDURES AND RESULTS

2.1 Literature Review

Pertinent geologic and geotechnical literature pertaining to the site area, and previous geotechnical studies performed by others for projects in the site vicinity were reviewed. These included United States Geological Survey (USGS), California Geological Survey (CGS), and other online resources, and other applicable government and private publications and maps, as included in the References section.

2.2 Field Exploration

Our field exploration program consisted of advancing 3 CPT soundings and drilling 2 soil borings at the project site as shown on *Figure 2-Site Map and Boring Locations.*

2.2.1 Cone Penetration Tests (CPT)

Three CPTs, designated as CPT-1 through CPT-3, were conducted on September 2, 2020, to depths of 100 feet below existing ground surface. Middle Earth Geo Testing, Inc. (MEGT), of Orange, California conducted the CPTs, using a specially designed, truck-mounted, 25-ton cone apparatus. The instrumented cone assembly used for this project included a cone tip with a 60-degree apex, diameter of 44.45 millimeters (mm), and a projected cross-sectional area of 15 square centimeters (cm^2), a sleeve segment with a surface area of 225 cm^2, and a pore pressure transducer near the base (shoulder) of the cone tip.

Prior to the start of the test, the truck was jacked up and leveled on four pads to provide a stable reaction for the cone thrust. During the test, the instrumented cone was hydraulically pushed into the ground at a rate of about 20 millimeters per second (about four feet per minute), and continuous readings of cone tip resistance, sleeve friction, and pore pressure were digitally recorded. As the cone advanced, additional cone rods were added. PC-based data acquisition hardware in the CPT truck received electric signals from strain gauges mounted in the cone assembly, and generated graphical logs including cone resistance, friction resistance, friction ratio, and pore pressure ratio versus depth.

CPT data was subsequently processed based on generally accepted soil behavior type correlations (e.g., Robertson et al., 1989) to interpret soil classification, and other properties such as SPT N-value and undrained shear strength were also estimated through correlations. CPT data plots and detailed tabulated logs for all the project CPTs are presented in Appendix A.

2.2.2 Soil Borings

Borings B-1 to B-2 were drilled at the site on September 3, 2020, by California Geotech Services, using a truck mounted B-24 drill rig equipped with 4-inch diameter solid flight augers, to a maximum depth of 30 feet below existing ground surface.

A Geo-Eng Staff Engineer visually classified the materials encountered in the borings according to the Unified Soil Classification System as the borings were advanced. Relatively undisturbed soil samples were recovered at selected intervals using a three-inch outside diameter Modified California split spoon sampler containing six-inch long brass liners. A two-inch outside diameter Standard Penetration Test (SPT) sampler was also used to obtain SPT blow counts and obtain disturbed soil samples. The samplers were driven by using a 140-pound safety hammer with an approximate 30-inch fall utilizing N-rods as necessary. Resistance to penetration was recorded as the number of hammer blows required to drive the sampler the final foot of an 18-inch drive. All the blow counts recorded using Modified California split spoon samplers in the field were converted to equivalent SPT blow counts using appropriate modification factors suggested by Burmister (1948), i.e., a factor of 0.65 assuming an inner diameter of 2.5 inches. Therefore, all blow counts shown on the final boring logs are either directly measured (SPT sampler) or equivalent SPT (MC sampler) blow counts. Bulk samples were obtained in the upper few feet of the borings from the auger cuttings as needed.

The boring logs with descriptions of the various materials encountered in each boring, the penetration resistance values, and the laboratory test results are presented in Appendix A. The ground surface elevations indicated on the soil boring logs were determined using Google Earth. Actual surface elevations at the boring locations may differ slightly than indicated. The locations of the borings should only be considered accurate to the degree implied by the means and methods used to define them.

2.3 Laboratory Testing

Laboratory tests were performed on selected samples to determine some of the physical and engineering properties of the subsurface soils. The results of the laboratory testing are presented on the boring logs, and included in Appendix B. The following soil tests were performed for this study:

Dry Density and Moisture Content (ASTM D2216 and ASTM 2937) – In-situ dry density and/or moisture tests were conducted on various samples to measure the in-place dry density and moisture content of the subsurface materials. These properties provide information to assist in evaluating the physical characteristics of the subsurface soils. Test results are shown on the boring logs.

Atterberg Limits (ASTM D4318 and CT204) – Liquid Limit, Plastic Limit, and Plasticity Index are useful in the classification and characterization of the engineering properties of soil, helps evaluate the expansive characteristics of the soil, and for determining the soil type according to the USCS. One test was performed from boring B-1 and the test results are presented on the boring log and in Appendix B.

Particle Size Analysis (Wet and Dry Sieve) and Fines Content (ASTM D422 and D1140) - Sieve analysis or fines content (minus No. 200 sieve) tests were conducted on several selected samples to measure the soil particle size distribution. This information is useful for the evaluation of liquefaction potential and characterizing the soil type according to USCS. Test results are presented on the boring logs or in Appendix B.

Soil Corrosivity, Redox (ASTM D1498), pH (ASTM D4972), Resistivity (ASTM G57), Chloride (ASTM D4327), and Sulfate (ASTM D4327) – Soil corrosivity testing was performed to determine the effects of constituents in the soil on buried steel and concrete. Water-soluble sulfate testing is required by the CBC and IBC. Test results are discussed in Section 4.3.

3.0 GEOLOGY AND SEISMICITY

3.1 Geologic Setting

The site is located within the central portion of the Coast Ranges geomorphic province of California. The Coast Ranges geomorphic province consists of numerous small to moderate linear mountain ranges trending north to south and northwest to southeast. The Coast Ranges lies between the Pacific Ocean to the west and the Great Valley Geomorphic Province to the east. This province is approximately 400 miles long and extends from the Klamath Mountains in the north to the Santa Ynez River within Santa Barbara County in the south. It generally consists of marine sedimentary rocks and volcanic rocks. The province is characterized by northwest-trending faults and folds, as well as erosion and deposition within the broad transform boundary between the North American and Pacific plates. Translational motion along the plate boundary occurs across a distributed zone of right-lateral shear expressed as a nearly 50-mile-wide zone of northwest-trending, near-vertical active strike-slip faults. This motion occurs primarily along the active San Andreas, Hayward, Calaveras and San Gregorio faults.

The site is located in the Los Gatos Creek of the Santa Clara Valley, located south of the San Francisco Bay and east and north of the Santa Cruz Mountains. The subject property is located in a generally flat area that overlies alluvial gravel, sand, silt, and clay of older Holocene alluvium, *Figure 3-Site Geologic Map*.

3.2 Seismic Setting

Regional transpression has caused uplift and folding of the bedrock units within the Coast Ranges. This structural deformation occurred during periods of tectonic activity that began in the Pliocene and continues today. The Bay Area of Northern California is a seismically active region dominated by four major northwest trending right lateral strike slip faults that include the San Andreas Fault, the Hayward Fault, the Calaveras Fault, and the Greenville Fault.

Major faults near the subject property include the San Andreas Fault located about 9.5 miles southwest, the Calaveras Fault located about 4 miles northeast, and the Hayward Fault located about 8 miles northeast. The State of California Earthquake Zones of Required Investigation map shows the subject property is not located in any liquefaction or active faulting hazard zone, *Figure 5-Geologic Hazard Map*.

4.0 FIELD AND LABORATORY FINDINGS

Subsurface conditions below the project site were interpreted based on the results of the test borings performed for this study, as well as the results of our laboratory testing. Detailed descriptions of the various subsurface soil units encountered during subsurface explorations are described in the following paragraphs.

4.1 Subsurface Soil Conditions

Subsurface conditions below the project site were interpreted based on the results of the test borings performed for this study and the results of our laboratory testing. Detailed descriptions of the various subsurface soil units encountered during subsurface explorations are described in the following.

During our subsurface exploration program, we investigated the subsurface soils and evaluated soil conditions to a maximum depth of 100 feet in the 2 soil borings and 3 CPTs performed for this study. From the ground surface to the maximum depth explored, the soils underlying the project site consist of primarily a layer of stiff to very stiff silty clay in the upper 12 to 22 feet, underlain by a layer of very dense silty sand to an approximate depth of 38 feet, underlain by layers of silty clay and clayey silt to an approximate depth of 50 feet, underlain by layers of sand and gravelly sand to a depth of approximately 90 feet, underlain by layers of silty clay and clayey silt to the maximum depth explored of 100 feet below existing ground surface.

Soil sample of the near surface fine grained material from Boring B-1 at 2 feet below ground surface was tested for Atterberg Limit, with measured Liquid Limit (LL) of 28, Plastic Limit (PL) of 14, and corresponding Plasticity Index (PI) of 14. Based on these test results the near surface soil would be considered to be of low plasticity and have a low expansion potential.

Additional details of the soils encountered in the exploratory borings are included in the boring log presented in Appendix A.

4.2 Groundwater

Groundwater was encountered in the advanced CPTs at approximately 62 feet below existing ground surface. The soil borings and CPTs were backfilled with a neat cement grout shortly after drilling. We note that the borings may not have been left open for a sufficient period of time to establish equilibrium groundwater conditions. Groundwater levels can vary in response to time of year, variations in seasonal rainfall, tidal influence, well pumping, irrigation, and alterations to site drainage. Published historic high groundwater was at approximately 50-feet below the surface.

4.3 Corrosion Testing

A bulk sample collected from the upper three feet of Boring B-2 was tested to measure sulfate content, chloride content, redox potential, pH, resistivity, and presence of sulfides. Test results are included in Appendix B and are summarized on the following tables.

Table 1: Summary of Corrosion Test Results

Soil Description	Sample Depth (feet)	Sulfate (mg/kg)	Chloride (mg/kg)	Redox (mV)	Resistivity (ohm-cm)	Sulfide	pH
Dark Yellowish-Brown Sandy CLAY with organics	2	152	9	492	3,034	Negative	7.7

Water-soluble sulfate can affect the concrete mix design for concrete in contact with the ground, such as shallow foundations, piles, piers, and concrete slabs. Section 4.3 in American Concrete Institute (ACI) 318, as referenced by the CBC, provides the following evaluation criteria:

Table 2: Sulfate Evaluation Criteria

Sulfate Exposure	Water-Soluble Sulfate in Soil, Percentage by Weight or (mg/kg)	Sulfate in Water, ppm	Cement Type	Max. Water Cementitious Ratio by Weight	Min. Unconfined Compressive Strength, psi
Negligible	0.00-0.10 (0-1,000)	0-150	NA	NA	NA
Moderate	0.10-0.20 (1,000-2,000)	150-1,500	II, IP (MS), IS (MS)	0.50	4,000
Severe	0.20-2.00 (2,000-20,000)	1,500-10,000	V	0.45	4,500
Very Severe	Over 2.00 (20,000)	Over 10,000	V plus pozzolan	0.45	4,500

The water-soluble sulfate content was measured to be about 152 mg/kg (ppm) or 0.0152% by dry weight in the soil sample, suggesting the site soil should have moderate impact on buried concrete structures at the site. However, it should be pointed out that the water-soluble sulfate concentrations can vary due to the addition of fertilizer, irrigation, and other possible development activities.

Table 4.4.1 in ACI 318 suggests use of mitigation measures to protect reinforcing steel from corrosion where chloride ion contents are above 0.06% by dry weight. The chloride content was measured to be 9 mg/kg (ppm) or 0.0009% by dry weight in the soil sample. Therefore, the test result for chloride content does not suggest a corrosion hazard for mortar-coated steel and reinforced concrete structures due to high concentration of chloride.

In addition to sulfate and chloride contents described above, pH, oxidation reduction potential (Redox), and resistivity values were measured in the soil sample. For cast and ductile iron pipes, an evaluation was based on the 10-Point scaling method developed by the Cast Iron Pipe Research Association (CIPRA) and as detailed in Appendix A of the American Water Works Association (AWWA) publication C-105 and shown on Table 3.

Table 3: Soil Test Evaluation Criteria (AWWA C-105)

Soil Characteristics	Points		Soil Characteristics	Points
Resistivity, ohm-cm, based on single probe or water-saturated soil box.			**Redox Potential, mV**	
<700	10		>+100	0
700-1,000	8		+50 to +100	3.5
1,000-1,200	5		0 to 50	4
1,200-1,500	2		Negative	5
1,500-2,000	1		**Sulfides**	
>2,000	0		Positive	3.5
PH			Trace	2
0-2	5		Negative	0
2-4	3		**Moisture**	
4-6.5	0		Poor drainage, continuously wet	2
6.5-7.5	0		Fair drainage, generally moist	1
7.5-8.5	0		Good drainage, generally dry	0
>8.5	5			

Assuming fair site drainage, the tested soil sample had a total score of 1 point, indicating a low corrosive rating. When total points on the AWWA corrosivity scale are at least 10, the soil is classified as corrosive to cast and ductile iron pipe and use of cathodic corrosion protection is often recommended.

These results are preliminary and provide information only on the specific soil sampled and tested. Other soil at the site may be more or less corrosive. Providing a complete assessment of the corrosion potential of the site soils are not within our scope of work. For specific long-term corrosion control design recommendations, we recommend that a California-registered professional corrosion engineer evaluate the corrosion potential of the soil environment on buried concrete structures, steel pipe coated with cement-mortar, and ferrous metals.

5.0 GEOLOGIC HAZARDS

5.1 Seismic Induced Hazards

Seismic hazards resulting from the effects of an earthquake generally include ground shaking, liquefaction and dynamic settlement (densification), lateral spreading, fault ground rupture and fault creep, and tsunamis and seiches. The site is not necessarily impacted by these potential seismic hazards. Applicable potential seismic hazards are discussed and evaluated in the following sections in relation to the planned construction.

5.1.1 Ground Shaking

The site will likely experience severe ground shaking from a major earthquake originating from many significant faults in the San Francisco Bay Area, including the Calaveras and San Andreas faults. Earthquake intensities vary throughout the Bay Area depending upon the magnitude of the earthquake, the distance of the site from the causative fault, the type of materials underlying the site and other factors.

In addition to shaking of the structure, strong ground shaking can induce other related phenomena that may influence structures, such as liquefaction or dynamic densification settlement; adjacent seismic slope failure, lurching or lateral spreading, or seismically induced waves (tsunamis and seiches).

5.1.2 Liquefaction Induced Phenomena

The site is not mapped within either a state or county identified geologic hazard zones requiring liquefaction investigation.

Research and historical data indicate that soil liquefaction generally occurs in saturated, loose granular soil (primarily fine to medium-grained, clean, poorly graded sand deposits) during or after strong seismic ground shaking and is typified by a loss of shear strength in the affected soil layer, thereby causing the soil to flow as a liquid. Typically, liquefaction potential increases with increased duration and magnitude of cyclic loading. However, because of the higher intergranular pressure of the soil at greater depths, the potential for liquefaction is generally limited to the upper 40 feet of the soil. Potential hazards associated with soil liquefaction below or near a structure include loss of foundation support, lateral spreading, sand boils, and areal and differential settlement.

Lateral spreading is lateral ground movement, with some vertical component, as a result of liquefaction. The soil literally rides on top of the liquefied layer. Lateral spreading can occur on relatively flat sites with slopes less than two percent under certain circumstances, generally when the liquefied layer is in relatively close proximity to an open, free slope face such as the bank of a creek channel. Lateral spreading can cause surficial ground tension cracking (i.e., lurch cracking) and settlement.

The soils encountered in the subsurface investigation included layers of stiff to very stiff silty clay and very dense silty sand. These soils are expected to be generally less susceptible to liquefaction due to their fine-grained content and relatively high density. Additionally, groundwater was encountered at approximately 62 feet below existing ground surface at the site. Therefore, the potential for liquefaction of the site subsurface soils is judged to be low.

5.1.3 Dynamic Densification (Settlement)

Dynamic compaction is a phenomenon where loose, relatively clean, near-surface sandy soil located above the water table is densified from vibratory loading, typically from strong seismic shaking or vibratory equipment. The site soils generally consist of stiff to very stiff silty clay and very dense silty sand. Therefore, in our opinion, dynamic settlement and/or any potential effect of dynamic settlement on the proposed construction is not expected to be significant.

5.1.4 Fault Ground Rupture and Fault Creep

The State of California adopted the Alquist-Priolo Earthquake Fault Zone Act of 1972 (Chapter 7.5, Division 2, Sections 2621 – 2630, California Public Resources Code), which regulates development near active faults for the purpose of preventing surface fault rupture hazards to structures for human occupancy. In accordance with the Alquist-Priolo (A-P) Act, the California Geological Survey established boundary zones or *Earthquake Fault Zones* surrounding faults or fault segments judged to be sufficiently active, well-defined and mapped for some distance. In addition to the State of California zones, Santa Clara County has identified additional fault rupture hazard zones. Structures for human occupancy within designated Earthquake Fault Zone boundaries are not permitted unless surface fault rupture and fault creep hazards are adequately addressed in a site-specific evaluation of the development site.

The site is not currently within a designated Earthquake Fault Zone as defined by the State (Hart and Bryant, 1997) or any local zone. Based on our evaluation, the potential for fault ground rupture or creep at the site is very low to nil.

5.2 Expansive Soils

The near surface soils observed and/or sampled during the exploration program generally consisted of low plasticity fine grained material. We did not encounter any potentially highly expansive soil. Therefore, special measures to mitigate the potential effects of expansive soils are not expected to be required for the project.

6.0 CONCLUSIONS AND ENGINEERING RECOMMENDATIONS

The following conclusions and engineering recommendations are based upon the analysis of the information gathered during the course of this study and our understanding of the proposed improvements.

The site is considered suitable from a geotechnical and geologic perspective for the proposed improvements provided the recommendations of this report are incorporated into the design and implemented during construction. The predominant geotechnical and geological issues affecting design or construction that will need to be addressed at this site are summarized below and addressed in the following sections.

Seismic Considerations - The site is located within a seismically active region and the structures should be designed to account for earthquake ground motions, using the applicable building codes, as described in Section 6.1 of this report.

Undocumented Fill Soils – No surficial undocumented fill soils and debris were encountered in our borings during our subsurface investigation. However, due to the presence of existing buildings at the site of the proposed new building, undocumented fills associated with the demolition of the building and removal of associated foundations and utilities may be present. Undocumented onsite fill soils if encountered in the new building pad and loose or debris laden soils if encountered in other areas, should be completely removed and replaced by engineered compacted fill. The portion of over-excavated material not consisting of debris or organic topsoil may be reused as fill material upon approval of the geotechnical engineer.

Winter Construction - If grading occurs in the winter rainy season, appropriate erosion control measures may be required, and weatherproofing of the building pad and/or hardscape areas may need to be considered. Winter rains may also impact foundation excavations and underground utilities.

6.1 Seismic Coefficients

The subject site is located within a seismically active region and should be designed to account for earthquake ground motions as described in this report. Based on the subsurface conditions encountered and our evaluation of the geology of the site, Site Class “D”, representative of stiff soil averaged over the uppermost 100 feet of the subsurface profile would be appropriate for this site.

For seismic analysis of the proposed site in accordance with the seismic provisions of the 2019 California Building Code (CBC), we recommend the following seismic ground motion values be used for design shown in Table 4, which are based on procedures outlined in ASCE 7-16 Section 11.4 and Table 11.4-2 of Supplement 1. ASCE 7-16 Section 11.4.8 states that a site-specific ground motion hazard analysis should be performed for all structures on Site Class D soils with S_1 greater than or equal to 0.2, unless the exceptions outlined in Section 11.4.8 are followed and the seismic response coefficient is properly modified during design. A site-specific ground motion hazard analysis was not performed for this site and is outside the scope of this report. If a site-specific ground motion hazard analysis is required for this project or if the project is designed under a different building code than CBC 2019, we should be notified so that we may provide the appropriate seismic design parameters.

Table 4: Seismic Parameters Based on 2019 CBC (per ASCE 7-16)

Item	Value	2019 CBC Source[R1]	ASCE 7-16 Table/Figure[R2]
Site Class	D	Table 1613A.3.2.	Table 20.3-1
Mapped Spectral Response Accelerations			
Short Period, S_s	1.5		Figure 22-1
1-second Period, S_1	0.6		Figure 22-2
Site Coefficient, F_a	1.2	Table 1613A.3.3(1)	Table 11.4-1
Site Coefficient, F_{v*}	1.7	Table 1613A.3.3(2)	Table 11.4-2
MCE (S_{MS})	1.8	Equation 16A-37	Equation 11.4-1
MCE (S_{M1})	1.02	Equation 16A-38	Equation 11.4-2
Design Spectral Response Acceleration			
Short Period, S_{DS}	1.2	Equation 16A-39	Equation 11.4-3
1-second Period, S_{D1**}	0.68	Equation 16A-40	Equation 11.4-4

R1: California Building Standards Commission (CBSC), "California Building Code," 2019 Edition.
R2: U.S. Seismic "Design Maps" Web Application, https://seismicmaps.org/
*F_v are based off Table 11.4-2 from the ASCE 7-16 Supplement 1
**The above design spectral response acceleration parameters may only be used provided that the exception outlined in section 11.4.8 of ASCE 7-16 is met.

6.2 Site Grading

6.2.1 General Grading and Material Requirements

Site grading is generally anticipated to consist of finish grading to establish site grades, or additional mass grading for improved foundation bearing capacities if desired; utility trench excavation and backfills, preparation of supporting subgrades for site pavements and hardscape; and placement of aggregate base (baserock) sections for hardscape and pavements.

On-site soils having an organic content of less than three percent by weight and Plasticity Index of less than 15 can be reused as fill as approved by the Geotechnical Engineer. Imported soil should be non-expansive, having a Plasticity Index of 15 or less, an R-Value greater than 40, and contain sufficient fines so the soil can bind together. Imported materials should be free of environmental contaminants, organic materials and debris, and should not contain rocks or lumps greater than three inches in maximum size. Import fill materials should be approved by the Geotechnical Engineer prior to use on site.

6.2.2 Project Compaction Recommendations

Table 5 provides the recommended compaction requirements for this project. Some items listed below may not apply to this project. Specific moisture conditioning and relative compaction recommendations will be discussed individually within applicable sections of this report.

Table 5: Project Compaction Recommendations

Description	Percent Relative Compaction	Minimum Percent Above Optimum Moisture Content
Building Pad, Onsite Soil	90	2
Building Pad, Subgrade Soil	90	2
Building Pad, Imported Select Fill	90	2
Building Pad, Treated Soil	90	2
AC or Concrete Pavement, Subgrade, Upper 6”	95	2
AC or Concrete Pavement, Onsite Soil or Fill	90	2
AC or Concrete Pavement, Class 2 Baserock	95	2
AC or Concrete Pavement, Treated Soil, Subgrade	93	2
Concrete Flatwork, Class 2 Baserock	90	2
Concrete Flatwork, Subgrade Soil	90	2
Underground Utility Trench Backfill	90	2
Underground Utility Trench Backfill - Landscape Areas (not including areas below flatwork)	85	2
Underground Utility Trench Backfill, Clean Sand	95	4
Underground Utility Trench Backfill, Upper 3’ Feet below Existing Pavement Sections or 6” below New Pavement Sections	95	2

Fill materials should be properly moisture conditioned in accordance with Table 5 as determined using ASTM D-1557 and placed in uniform loose lifts not to exceed eight inches. Smaller lifts may be necessary to achieve the minimum required compaction using lighter weight compaction equipment. It should be noted that the use of on-site soils for fill will require moisture conditioning (drying or wetting). Moisture conditioning may be difficult to achieve during cold, wet periods of the year, or during extreme temperatures and after precipitation events.

6.2.3 Site Preparation and Demolition

Site grading should be performed in accordance with these recommendations. A pre-construction conference should be held at the jobsite with representatives from the owner, general contractor, grading contractor, and Geo-Eng prior to starting the stripping and demolition operations at the site.

The site should be cleared of existing pavements (if any), vegetation, organic topsoil, debris, existing undocumented loose or soft fill, and other deleterious materials within the proposed development area. Removed fill soil may be evaluated by the Geotechnical Engineer for possible reuse and placement as engineered fill. The grading contractor should be aware of the possibility of buried objects and underground utilities at the site which are to be removed or abandoned appropriately. Holes resulting from the removal of underground obstructions extending below the proposed finish grade should be cleared and backfilled with properly compacted engineered fill or other material approved by the Geotechnical Engineer. We recommend backfilling operations for any excavations to remove deleterious material be carried out under the observation of the Geotechnical Engineer.

It is possible that existing underground utilities exist and if so, may impact the project construction. If encountered, the utilities will need to be properly abandoned and/or entirely removed from proposed building area. In general, utility pipelines less than four inches in diameter to be abandoned may be left in place provided they will not be in close proximity to new foundation elements or interfere with new utilities. Such pipes should be plugged at the ends with concrete or sand-cement slurry. Larger utility pipelines or pipelines that underlie new foundations should be removed and replaced with engineered fill or left in place and completely grouted with flowable sand-cement slurry or other approved Controlled Density Fill (CDF; also, known as Controlled Low Strength Material, or CLSM).

6.2.4 Building Subgrade Preparation

Imported soil should be non-expansive, having a Plasticity Index of 15 or less, an R-Value greater than 40, and contain sufficient fines so the soil can bind together. Imported materials should be free of organic materials and debris and should not contain rocks or lumps greater than three inches in maximum size. Import fill materials should be approved by the Geotechnical Engineer prior to use onsite.

Following excavation to the required grades, subgrades in areas to receive engineered fill, slabs-on-grade or hardscape should be scarified to a depth of at least six inches; moisture conditioned and compacted to the requirements for engineered fill presented in Table 5. The compacted surface should be firm and unyielding and should be protected from damage caused by traffic or weather. Soil subgrades should be kept moist during construction. To achieve satisfactory compaction of the subgrade and fill materials, it may be necessary to adjust the water content at the time of construction. This may require that water be added to soils that are too dry, or that scarification and aeration be performed in any soils that are too wet. Fill material should be evenly spread and compacted in lifts not exceeding eight inches in pre-compacted thickness.

Newly exposed near-surface soils under existing site pavement once removed are typically saturated to near-saturated. Therefore, it is anticipated that after the underlying soils are over-excavated to construct the non-expansive fill layer, unstable subgrade conditions unworkable for compaction by construction equipment are locally possible, and compaction of the exposed soil subgrade to engineered fill requirements immediately after exposure may not be feasible. Possible options for subgrade stabilization include ripping, air-drying and re-compacting exposed subgrade material; admixtures such as cement; or use of reinforcing stabilization geotextile or geogrid, as discussed below. More detailed recommendations can be provided during construction should unstable subgrades be encountered by the contractor.

Unstable subgrades in smaller, isolated areas can be stabilized by over excavating to a minimum of 18-inch depth below finished subgrade elevation where competent, stable soils are not encountered. The bottom of the excavation should then be completely covered with a ground stabilization geotextile fabric such as Mirafi 500X or equivalent, and typically backfilled with Class 2 aggregate base. Alternatively, with the approval of the Geotechnical Engineer, such areas can be stabilized by over-excavating at least one foot, placing Tensar TriAx TX-140 or equivalent geogrid on the soil, and then placing 12 inches of Class 2 baserock on the geogrid. The upper six inches of the baserock in either case should be compacted to at least 90 percent relative compaction.

Larger unstable areas if encountered may be remedied using soil admixtures, such as cement. A four percent mixture of cement based on a dry soil unit weight of 110 pcf may be assumed if needed. Treatment should vary between 12 to 18 inches, depending on the anticipated construction equipment loads. More detailed and final recommendations can be provided during construction.

Final grading should be designed to provide positive drainage away from the building. We suggest exposed soil/landscape areas, if any, within 10 feet of the proposed building be sloped at a minimum of three percent away from the building. Roof leaders and downspouts should discharge onto paved surfaces sloping away from the building or into a closed pipe system channeled away from the building to an approved collector or outfall.

6.2.5 Flatwork Areas

The existing soil in flatwork areas should be scarified to a depth of at least six inches, moisture conditioned and compacted. Once the compacted subgrade has been reached, it is recommended that baserock in paved areas be placed immediately after grading to protect the subgrade soil from drying. Alternatively, the subgrade should be kept moist by watering until the baserock is placed. Rubber-tired heavy equipment, such as a full water truck, should be used to proof roll exposed pavement subgrade areas where pumping is suspected. Proof rolling will determine if the subgrade soil is capable of supporting construction paving equipment without excessive pumping or rutting.

6.2.6 Site Winterization and Unstable Subgrade Conditions

If grading occurs in the winter rainy season, unstable and unworkable subgrade conditions may be present, and compaction of on-site soils may not be feasible. These conditions may be remedied using appropriate soil admixtures, such as lime or other admixtures. More detailed recommendations can be provided during construction. Stabilizing subgrade in small, isolated areas can be accomplished with the approval of the Geotechnical Engineer by over-excavating one foot, placing Tensar BX1100 or TriAx TX-140 geogrid or equivalent geogrid on the soil, and then placing 12 inches of Class 2 baserock on the geogrid. The upper six inches of the baserock should be compacted to at least 90 percent relative compaction. Alternatively, a non-woven stabilization geotextile such as Mirafi 500X overlain by a minimum 18 inches of baserock may be substituted for geogrid and baserock.

6.3 Utility Trench Construction

6.3.1 Trench Backfilling

Utility trenches may be backfilled with onsite soil or import soil pre-approved by the Geotechnical Engineer above the utility bedding and shading materials. If cobbles, rocks or concrete larger than four inches in maximum size are encountered, they should be removed from the fill material prior to placement in the utility trenches.

Pipeline trenches should be backfilled with fill placed in lifts of approximately eight inches in pre-compacted thickness and compacted to the requirements presented in Section 6.2.2. However, thicker lifts can be used, provided the method of compaction is approved by the Geotechnical Engineer, and the required minimum degree of compaction is achieved.

6.3.2 Utility Penetrations at Building Perimeter

Flexible connections at building perimeters should be considered for utility lines going through perimeter foundations. This would provide flexibility during a seismic event. This could be provided by special flexible connections, pipe sleeving with appropriate waterproofing, or other methods.

6.4 Temporary Excavation Slopes

Below-grade construction, if any is ultimately proposed for the project, may require temporary excavation slopes if more than a few feet below existing grade. The Contractor should incorporate all appropriate requirements of OSHA/ Cal OSHA into the design of the temporary construction slopes and shoring system, whichever is used. Excavation safety regulations are provided in the OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, Subpart P, and apply to excavations greater than five feet in depth.

The Contractor, or his specialty subcontractor, should design temporary construction slopes to conform to the OSHA regulations and should determine actual temporary slope inclinations based on the subsurface conditions exposed at the time of construction. For pre-construction planning purposes, the on-site near-surface materials may be assumed to be granular or weak cohesive materials and categorized as OSHA Type B with temporary slope inclination of no steeper than 1:1 (horizontal: vertical) for excavations less than 20 feet deep.

If temporary slopes are left open for extended periods of time, exposure to weather and rain could have detrimental effects such as sloughing and erosion on surficial soils exposed in the excavations. We recommend that all vehicles and other surcharge loads be kept at least 10 feet away from the top of temporary slopes, and that such temporary slopes are protected from excessive drying or saturation during construction. In addition, adequate provisions should be made to prevent water from ponding on top of the slope and from flowing over the slope face. Desiccation or excessive moisture in the excavation could reduce stability and require shoring or laying back side slopes.

6.5 Foundations

We recommend that the proposed structures be found on a conventional shallow foundation. Recommendations for conventional shallow foundations are provided below.

6.5.1 Spread Footing Foundations

The proposed building can be supported on conventional continuous and/or isolated spread footings bearing on undisturbed onsite native soil. Where over excavations below design footing depth is required, the over excavated portion of footing excavation should be backfilled with structural or lean concrete or a Controlled Low Strength Material (CLSM). Footings should be founded a minimum of 24 inches below lowest adjacent finished grade (typically the top of exterior grade) for exterior, perimeter footings, and a minimum of 24 inches below building pad subgrade for interior footings. Continuous footings should have a minimum width of at least 18 inches, and isolated column footings should have a minimum width of at least 24 inches. In addition, footings located adjacent to other footings or utility trenches should bear below an imaginary 1.5:1 (horizontal to vertical) plane projected upward from the bottom edge of the adjacent footings or utility trench. Footing reinforcement should be determined by the project Structural Engineer.

For the design of the footings bearing within tested and approved new fill or on stiff/very stiff native soil, we recommend the allowable bearing pressures presented in Table 6. The allowable pressures provided are net values, as the weight of the footing itself has already been accounted for and can be neglected as a load for design purposes.

Table 6: Allowable Bearing Pressures for Spread Footings

Load Condition	Allowable Bearing Pressure (psf)
Dead Load	3,500
Dead plus Live Loads	4,500
Total Loads (including wind or seismic)	6,000

We estimate that total elastic settlement will be on the order of 1 inch and differential settlement of about ½-inch. We should be consulted during foundation design to further evaluate and refine these estimates based on actual design loads. Geo-Eng should perform a final review the foundation design plans and calculations prior to submission of the plans for approval and construction.

6.5.2 Lateral Resistance

For resistance to lateral loads, an allowable coefficient of friction of 0.35 between the base of the foundation elements and underlying material is recommended. In addition, an ultimate passive resistance equal to an equivalent fluid weighing 350 pounds per cubic foot (pcf) acting against the foundation may be used to resist lateral forces. The top 12 inches of passive resistance at foundations not adjacent to and confined by pavement, interior floor slab, or hardscape should be neglected. In order to fully mobilize this passive resistance, a lateral footing deflection on the order of one to two percent of the embedment of the footing is required. If it is desired to limit the amount of lateral deflection to mobilize the passive resistance, a proportional safety factor should be applied.

6.5.3 Construction Considerations

Geo-Eng personnel should be retained to observe and confirm that footing excavations prior to formwork and reinforcing steel placement bear in soils suitable for the recommended maximum design bearing pressure. If unsuitable soil is present, the excavation should be deepened until suitable supporting material is encountered. The over excavation should be backfilled using structural or lean concrete up to the bottom of the footing concrete.

Footing excavations should have firm bottoms and be free from excessive slough prior to concrete or reinforcing steel placement. Care should also be taken to prevent excessive wetting or drying of the bearing materials during construction. Extremely wet or dry or any loose or disturbed material in the bottom of the footing excavations should be removed prior to placing concrete. If construction occurs during the winter months, a thin layer of concrete (sometimes referred to as a rat slab) could be placed at the bottom of the footing excavations. This will protect the bearing soil and facilitate removal of water and slough if rainwater fills the excavations.

6.6 Concrete Slabs-on-Grade

6.6.1 General Recommendations

Non-structural concrete interior slab-on-grade floors should be a minimum of five inches in thickness. As a minimum, slab reinforcing should consist of No. 4 steel reinforcement spaced at 18-inch centers each way, and in any case, be sufficient to satisfy the anticipated use and loading of the slab. Slab-on-grade subgrade surfaces should be proof-rolled to provide a smooth, unyielding surface for slab support.

Care should be taken to maintain the minimum recommended moisture content in the subgrade until floor slabs and/or engineered fills are constructed. Positive drainage should also be developed away from the building to prevent water from ponding along the perimeter and affecting future floor slab performance. We recommend a positive cutoff in utility trenches at the structure/building lines to reduce the potential for water migrating through the utility trench backfill to areas under the building.

Slab-on-grade concrete floors with moisture sensitive floor coverings should be underlain by a moisture retarder system constructed between the slab and subgrade. Such a system could consist of four inches of free-draining gravel, such as 3/4-inch, clean, crushed, uniformly graded gravel with less than three percent passing No. 200 sieve, or equivalent, overlain by a relatively impermeable vapor retarder placed between the subgrade soil and the slab. The vapor retarder should be at least 10-mil thick and should conform to the requirements for ASTM E 1745 Class A, B, or C Underslab Vapor Retarders (e.g., Griffolyn Type 65, Griffolyn Vapor Guard, Moistop Ultra C, or equivalent). If additional protection is desired by the owner, a higher quality vapor barrier conforming to the requirements of ASTM E 1745 Class A, with a water vapor transmission rate less than or equal to 0.006 gr/ft^2/hr (i.e., 0.012 perms) per ASTM E 96 (e.g., 15-mil thick “Stego Wrap Class A”) may be used in place of the retarder.

The vapor retarder or barrier should be placed directly under the slab. A capillary rock layer or rock cushion is not required if Class A barriers has been used beneath the floor slab and a sand layer is not required over the vapor retarder from a geotechnical standpoint. If sand on top of the vapor retarder is required by the design structural engineer, we suggest the thickness be minimized to less than one inch. If construction occurs in the winter months, water may pond within the sand layer since the vapor retarder may prevent the vertical percolation of rainwater.

ASTM E1643 should be utilized as a guideline for the installation of the vapor retarder. During construction, all penetrations (e.g., pipes and conduits,) overlap seams, and punctures should be completely sealed using a waterproof tape or mastic applied in accordance with the vapor retarder manufacturer's specifications. The vapor retarder or barrier should extend to the perimeter cutoff beam or footing.

6.6.2 Exterior Concrete Flatwork

Exterior concrete flatwork with pedestrian traffic should be at least four inches thick and should be underlain by at least six inches of aggregate baserock. The subgrade beneath the flatwork should be moisture conditioned and compacted as specified in the grading section of this report.

Control joints should be constructed in accordance with ACI 224 "Control of Cracking in Concrete Structures". In general, for typical flatwork, joints would be required every 24 to 36 times the concrete thickness.

6.7 Retaining/Basement Walls

6.7.1 Lateral Earth Pressures

The following recommended lateral earth design pressures are based on the assumption that on-site soils will be used as wall backfill. For a level backfill condition, unrestrained walls (i.e., walls that are free to deflect or rotate) should be designed to resist an equivalent fluid pressure of 40 pounds per cubic foot. Restrained walls for a level backfill condition should be designed to resist an equivalent fluid pressure of 40 pounds per cubic foot, plus an additional uniform lateral pressure of 5H pounds per square foot, where H = height of backfill above the top of the wall footing, in feet. For seismic design of walls greater than six feet in retained height, unrestrained and restrained walls with level backfill should be designed to resist an additional uniform load equal to 15H psf, added to the *unrestrained* condition in either case. A seismic increment is not required for site walls retaining less than six feet.

Walls with inclined backfill should be designed for an additional equivalent fluid pressure of one pound per cubic foot for every two degrees of slope inclination from horizontal. Walls subjected to surcharge loads should be designed for an additional uniform lateral pressure equal to 0.33 times the anticipated surcharge load for unrestrained walls, and 0.50 times the anticipated surcharge load for restrained walls.

For resistance to lateral loads, an allowable coefficient of friction of 0.35 between the base of the foundation elements and underlying material is recommended. In addition, an *ultimate* passive resistance equal to an equivalent fluid weighing 350 pounds per cubic foot (pcf) acting against the foundation may be used for lateral load resistance against the sides of the footing perpendicular to the direction of loading where the footing is poured neat against undisturbed material (i.e., native soils or engineered fills). The top foot of passive resistance at foundations not adjacent to and confined by pavement, interior floor slab, or hardscape should be neglected. In order to fully mobilize this passive resistance, a lateral footing deflection on the order of one to two percent of the embedment of the footing is required. If it is desired to limit the amount of lateral deflection to mobilize the passive resistance, a proportional safety factor should be applied.

The lateral earth pressures herein do not include any factor-of-safety and are not applicable for submerged soils/hydrostatic loading. Additional recommendations may be necessary if submerged conditions are to be included in the design.

6.7.2 Retaining Wall Foundations

Retaining and below-grade walls may be founded on spread footing foundations following the recommendations outlined in section 6.5. Assuming a minimum 24-inch footing embedment below lowest adjacent grade, retaining wall footings may be designed using an allowable bearing capacity based off Table 6, in section 6.5.1.

6.7.3 Retaining Wall Drainage

The aforementioned recommended lateral pressures assume that walls are fully back drained to prevent the build-up of hydrostatic pressures. To reduce the potential for hydrostatic loading on retaining and below-grade walls due to possible seasonal subsurface groundwater seepage, a subsurface drain system may be considered for construction behind below-grade walls. Alternatively, below-grade walls can be designed to accommodate an additional hydrostatic pressure increment.

The drain system should consist of free-draining granular soils containing less than five percent fines passing a No. 200 sieve, placed adjacent to the wall. The free-draining granular material should be graded to prevent the intrusion of fines, or else should be encapsulated in a suitable filter fabric. A drainage system consisting of perforated drain lines (minimum 4" diameter placed near the base of the wall) should be used to intercept and discharge water which would tend to saturate the backfill. Sub drains constructed to protect interior spaces should have the invert elevation of the sub drain a minimum of six inches below the interior finished floor elevation. Where used, drain lines should be embedded in a uniformly graded filter material and provided with adequate clean-outs for periodic maintenance. An impervious soil should be used in the upper one-foot layer of backfill to reduce the potential for water infiltration. As an alternative, a prefabricated drainage structure, such as geo-composite, may be used as a substitute for the granular backfill adjacent to the wall.

The retaining wall drainage system should be sloped to outfall to the storm drain system or other appropriate facility. The foundation of the retaining wall should be protected and prevented from any erosion of the surroundings.

6.7.4 Retaining Wall Backfill Compaction

Retaining wall backfill less than five feet deep should be compacted to at least 90 percent relative compaction using light compaction equipment. Backfill greater than a depth of five feet should be compacted to at least 95 percent relative compaction. If heavy compaction equipment is used, the walls should be appropriately designed to withstand loads exerted by the heavy equipment, and/or temporarily braced. Over compaction or surcharge from heavy equipment too close to the wall may cause excessive lateral earth pressures which could result in excessive outward wall movement.

6.8 Observation and Testing During Construction

We recommend that Geo-Eng be retained to provide observation and testing services during site preparation, site grading, pavement section preparation, utility construction, foundation excavation, and to observe final site drainage. This is to observe compliance with the design concepts, specifications and recommendations, and to allow for possible changes if subsurface conditions differ from those anticipated prior to the start of construction.

7.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

The recommendations of this report are based upon the soil and conditions encountered in the field explorations (i.e., borings). If variations or undesirable conditions are encountered during construction, Geo-Eng should be contacted so that supplemental recommendations may be provided.

This report is issued with the understanding that it is the responsibility of the owner or his representatives to see that the information and recommendations contained herein are called to the attention of the other members of the design team and incorporated into the plans and specifications, and that the necessary steps are taken to see that the recommendations are implemented during construction.

The findings and recommendations presented in this report are valid as of the present time for the development as currently proposed. However, changes in the conditions of the property or adjacent properties may occur with the passage of time, whether by natural processes or the acts of other persons. In addition, changes in applicable or appropriate standards may occur through legislation or the broadening of knowledge. Accordingly, the findings and recommendations presented in this report may be invalidated, wholly or in part, by changes outside our control. Therefore, this report is subject to review by Geo-Eng after a period of three (3) years has elapsed from the date of issuance of this report. In addition, if the currently proposed design scheme as noted in this report is altered, Geo-Eng should be provided the opportunity to review the changed design and provide supplemental recommendations as needed.

Recommendations are presented in this report which specifically request that Geo-Eng be provided the opportunity to review the project plans prior to construction and that we be retained to provide observation and testing services during construction. The validity of the recommendations of this report assumes that Geo-Eng will be retained to provide these services.

This report was prepared upon your request for our services, and in accordance with currently accepted geotechnical engineering practice. No warranty based on the contents of this report is intended, and none shall be inferred from the statements or opinions expressed herein. The scope of our services for this report did not include an environmental assessment or investigation for the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater or air, on, below or around this site. Any statements within this report or on the attached figures, logs or records regarding odors noted or other items or conditions observed are for the information of our client only.

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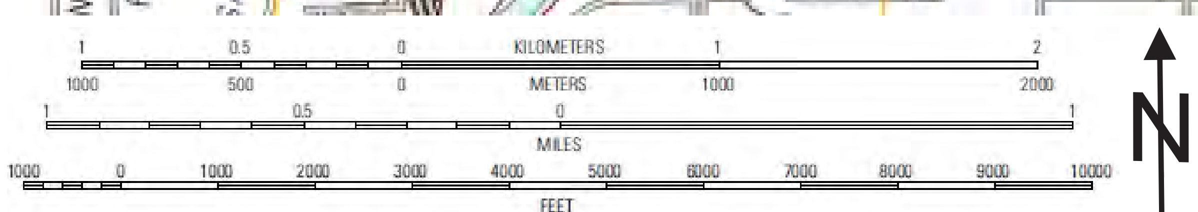
FIGURES

Figure 1 – Site Vicinity Map
Figure 2 – Site Map and Boring Locations
Figure 3 – Site Geologic Map
Figure 4 – Regional Fault Map
Figure 5 – Geologic Hazard Map



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		Site Vicinity Map	Figure 1

Approximate Boring Location   Approximate CPT Location

N

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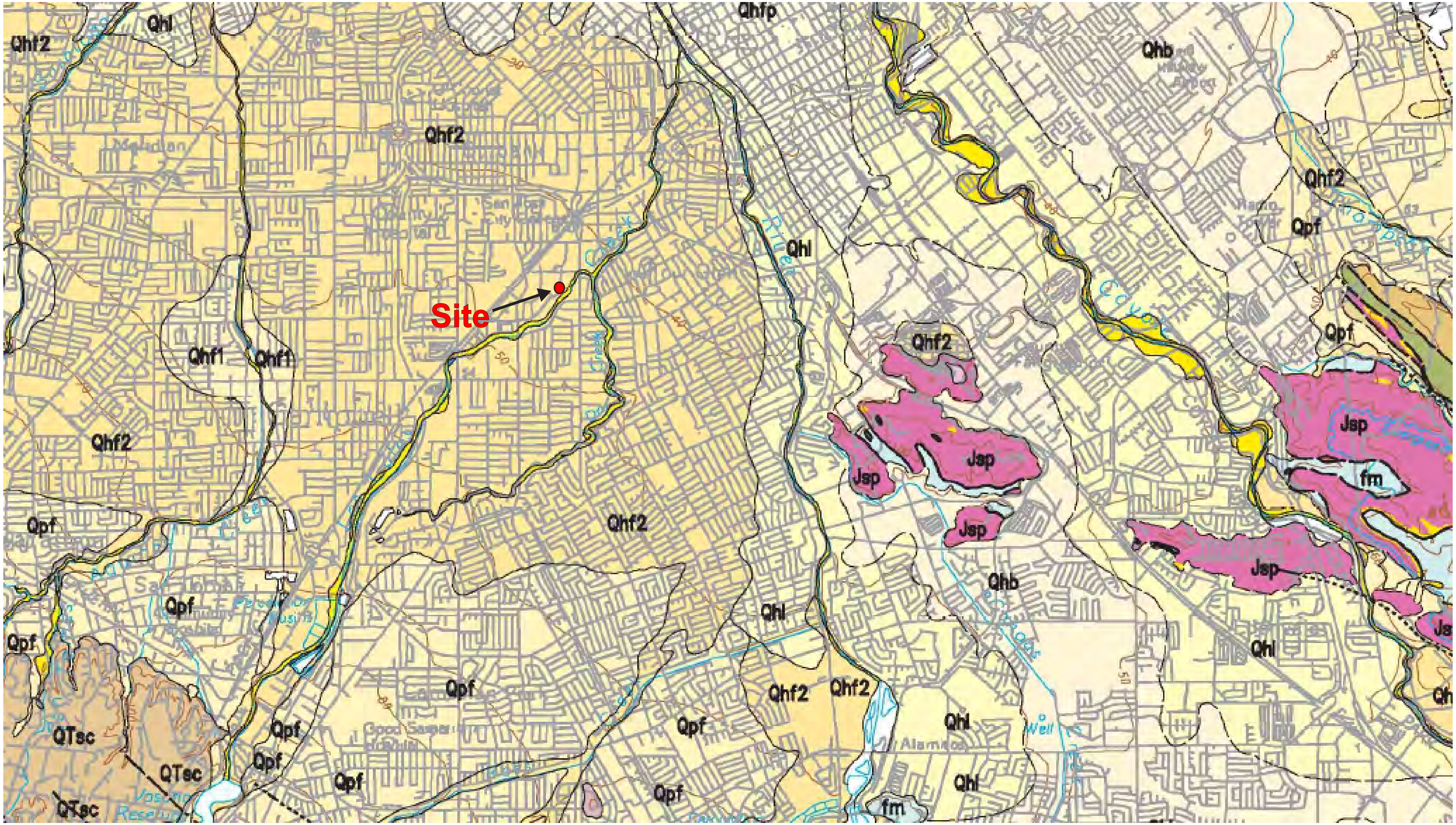
	Site Map and Boring Locations	Figure 2



Qhl	Levee deposits (HOLOCENE)	
Qhf1	Younger	(HOLOCENE)
Qhf2	Older	(HOLOCENE)

N

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	Site Geologic Map	Figure 3

N

Unspecified age, well constrained location

Unspecified age, moderately constrained location

Unspecified age, inferred location

Undifferentiated Quaternary (< 130,000 years), well constrained location

Undifferentiated Quaternary (< 130,000 years), moderately constrained location

Undifferentiated Quaternary (< 130,000 years), inferred location

Middle and late Quaternary (< 1.6 million years), well constrained location

Middle and late Quaternary (< 1.6 million years), moderately constrained location

Middle and late Quaternary (< 1.6 million years), inferred location

Latest Quaternary (<15,000 years), well constrained location

Latest Quaternary (<15,000 years), moderately constrained location

Latest Quaternary (<15,000 years), inferred location

Late Quaternary (< 130,000 years), well constrained location

Late Quaternary (< 130,000 years), moderately constrained location

Late Quaternary (< 130,000 years), inferred location

Historical (< 150 years), well constrained location

Historical (< 150 years), moderately constrained location

Historical (< 150 years), inferred location

Class B (various age), well constrained location

Class B (various age), moderately constrained location

Class B (various age), inferred location

Base Map Source: USGS Quaternary Fault Report

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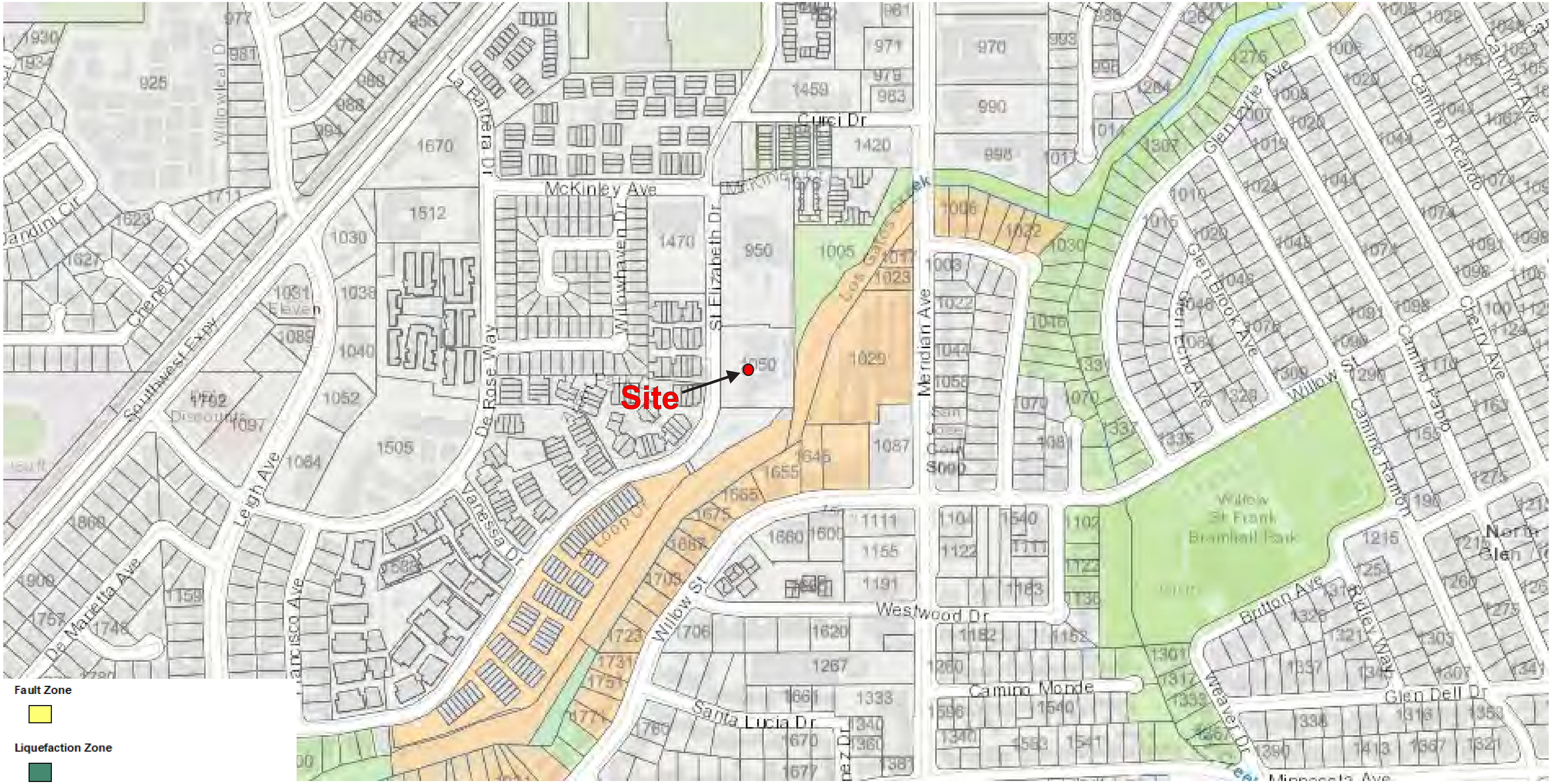
	Regional Fault Map	Figure 4



Fault Zone

Liquefaction Zone

Landslide Zone

Liquefaction Landslide Overlap Zone

Area Not Evaluated for Liquefaction or Landslides

Source: https://maps.conservation.ca.gov/cgs/EQZApp/app/

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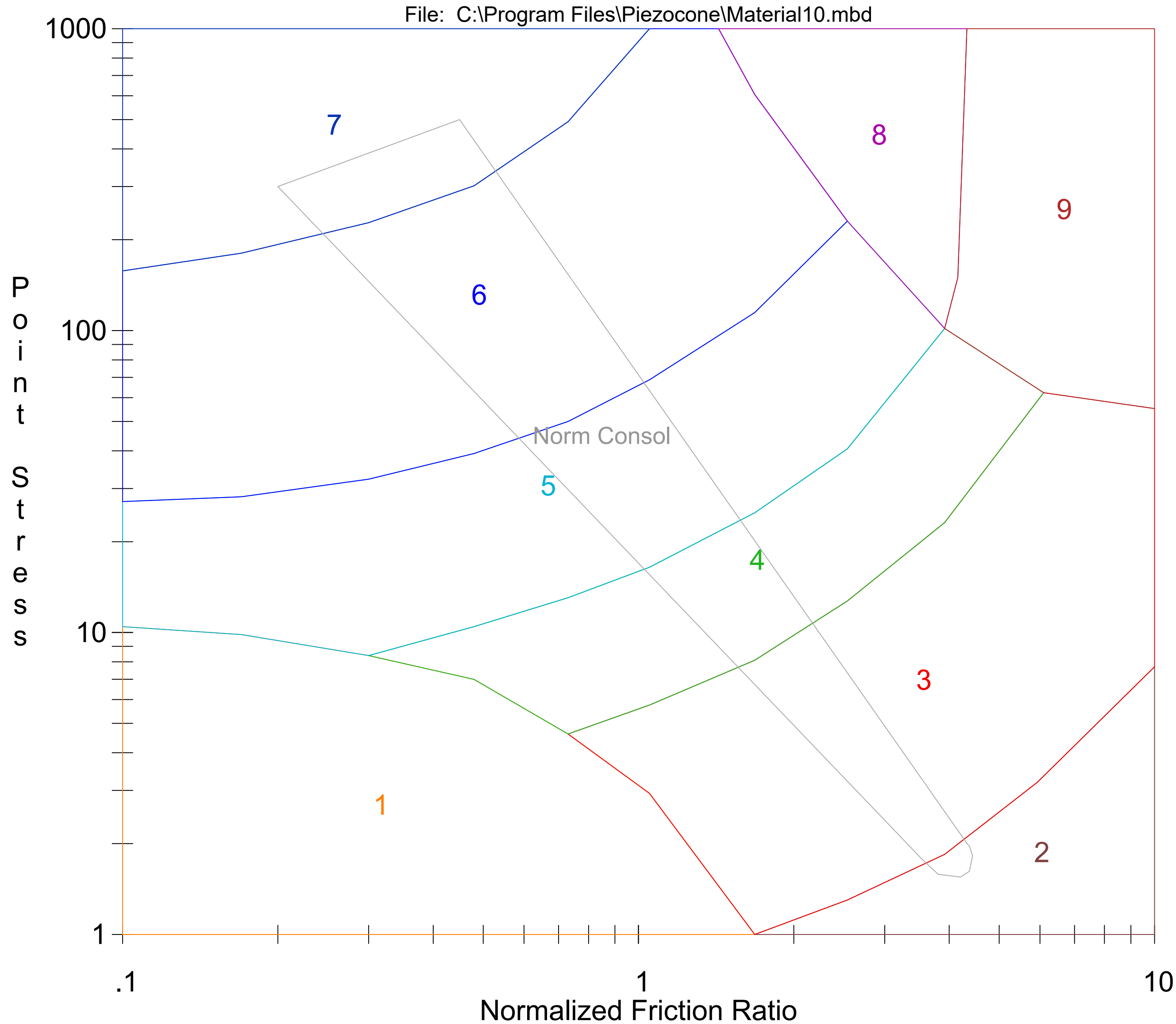
	Geologic Hazard Map	**Figure 5**

APPENDIX A

FIELD EXPLORATION
CPT Data
Key to Exploratory Boring Logs
Boring Logs

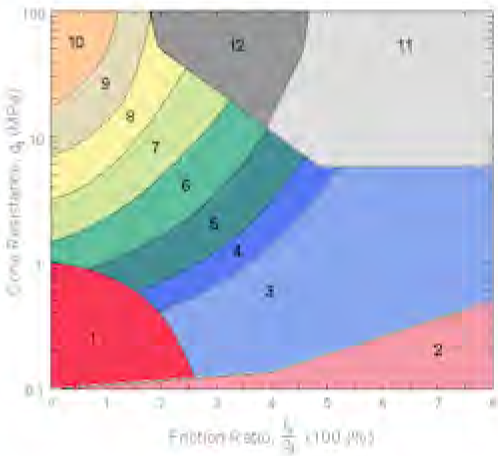
Material Behavior Type Zones

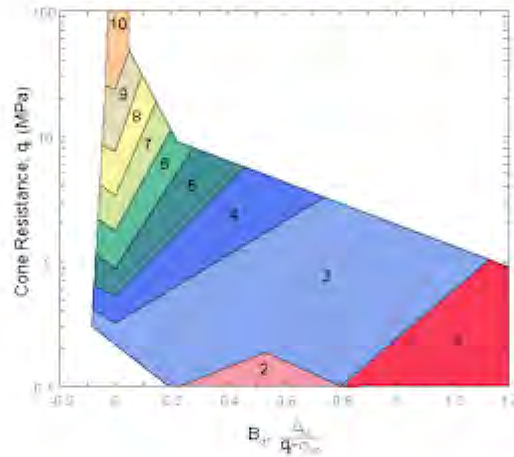


1. sensitive fine SOIL
2. Organic SOILS - Peats
3. silty CLAY to CLAY
4. clayy SILT to silty CLAY
5. silty SAND to sandy SILT
6. clean SAND to silty SAND
7. grvly SAND to dense SAND
8. stiff SAND to clayy SAND
9. very stiff fine SOIL

CPT Soil Behavior Type Legend

Robertson et al. 1986

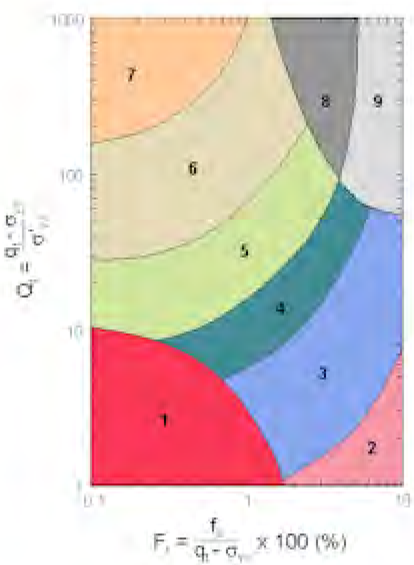


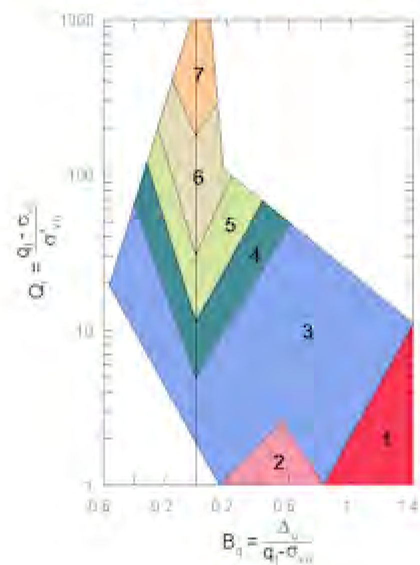


Zone	Soil Behavior Type
1	Sensitive, Fine Grained
2	Organic Material
3	Clay
4	Silty Clay to Clay
5	Clayey Silt to Silty Clay
6	Sandy Silt to Clayey Silt
7	Silty Sand to Sandy Silt
8	Sand to Silty Sand
9	Sand
10	Gravelly Sand to Sand
11	Very Stiff Fine Grained*
12	Sand to Clayey Sand*

*Overconsolidated or Cemented

Robertson et al. 1990





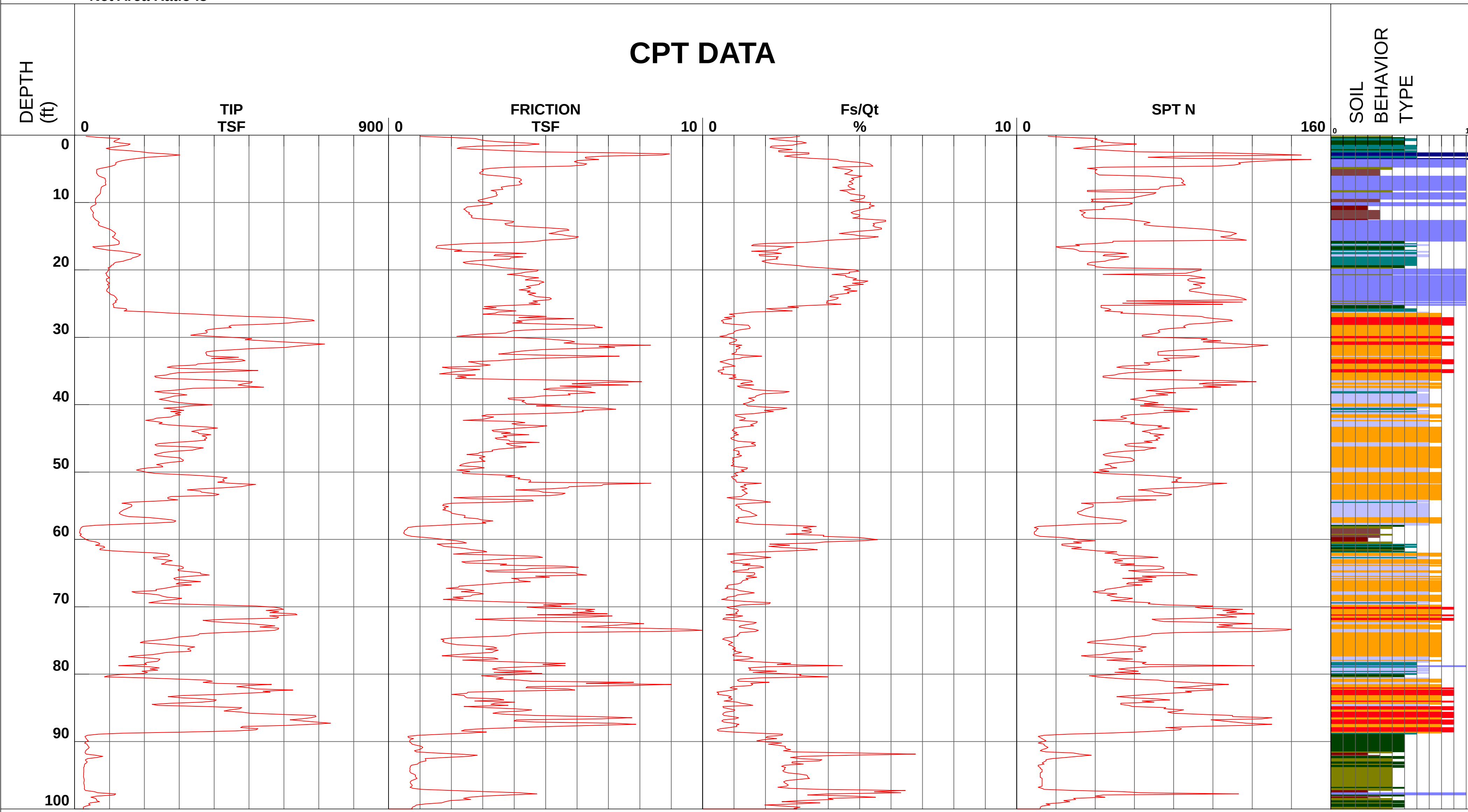
Zone	Soil Behavior Type
1	Sensitive, Fine Grained
2	Organic Soils-Peats
3	Clays: Clay to Silty Clay
4	Silt Mixtures: Clayey Silt to Silty Clay
5	Sand Mixtures: Silty Sand to Sandy Silt
6	Sands: Clean Sands to Silty Sands
7	Gravelly Sand to Sand
8	Very Stiff Sand to Clayey Sand*
9	Very Stiff Fine Grained*

*Overconsolidated or Cemented

GeoEngineering Solutions

Project	St Elizabeth	Operator	JM-AJ	Filename	SDF(079).cpt
Job Number	59-1266	Cone Number	DDG1530	GPS	
Hole Number	CPT-01	Date and Time	9/2/2020 10:33:27 AM	Maximum Depth	100.88 ft
EST GW Depth During Test		61.80 ft			

Net Area Ratio .8



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

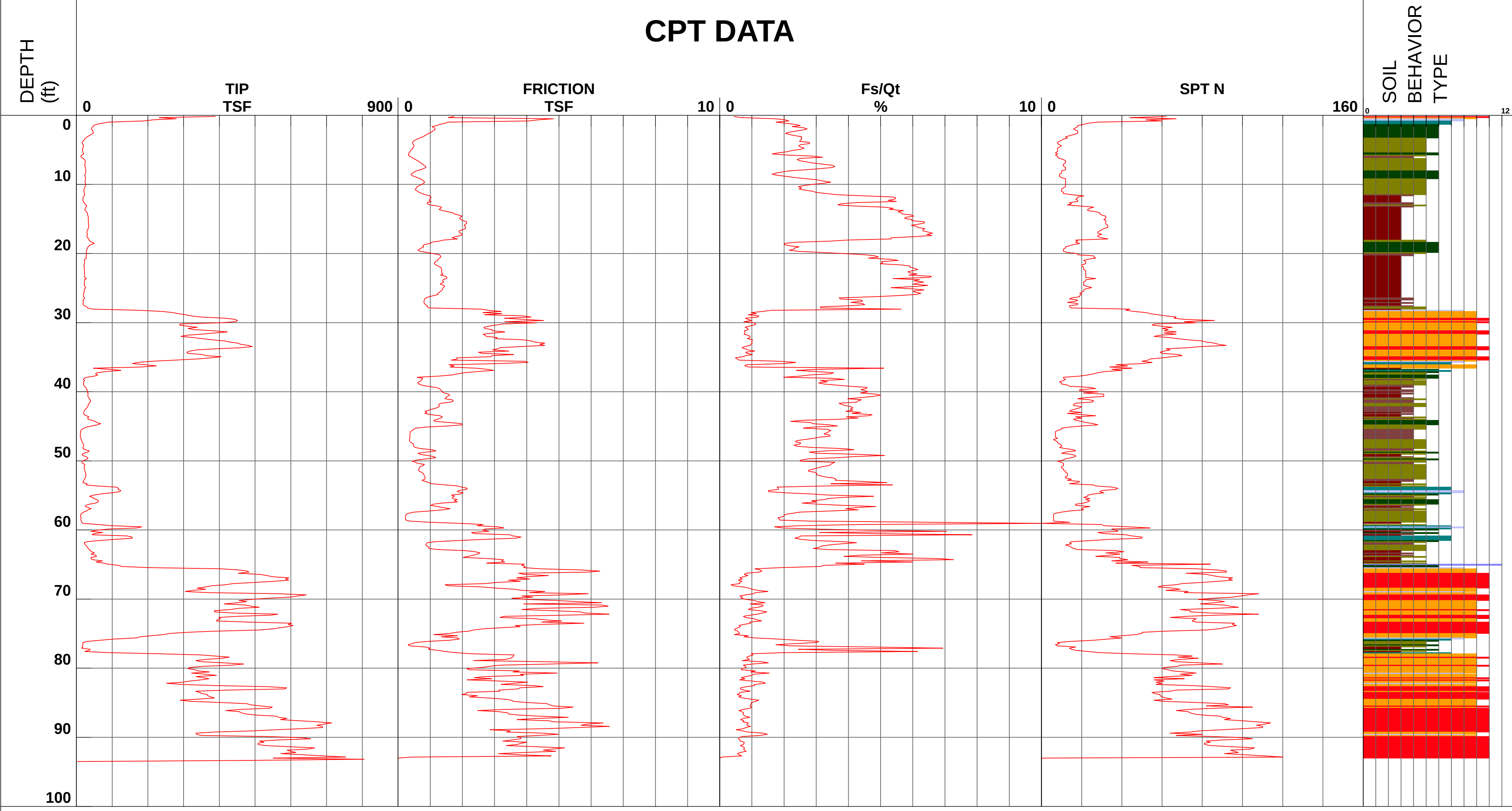
Cone Size 15cm squared

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

GeoEngineering Solutions

Project	St Elizabeth	Operator	JM-AJ	Filename	SDF(080).cpt
Job Number	59-1266	Cone Number	DDG1530	GPS	
Hole Number	CPT-02	Date and Time	9/2/2020 12:36:57 PM	Maximum Depth	93.50 ft
EST GW Depth During Test			62.00 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

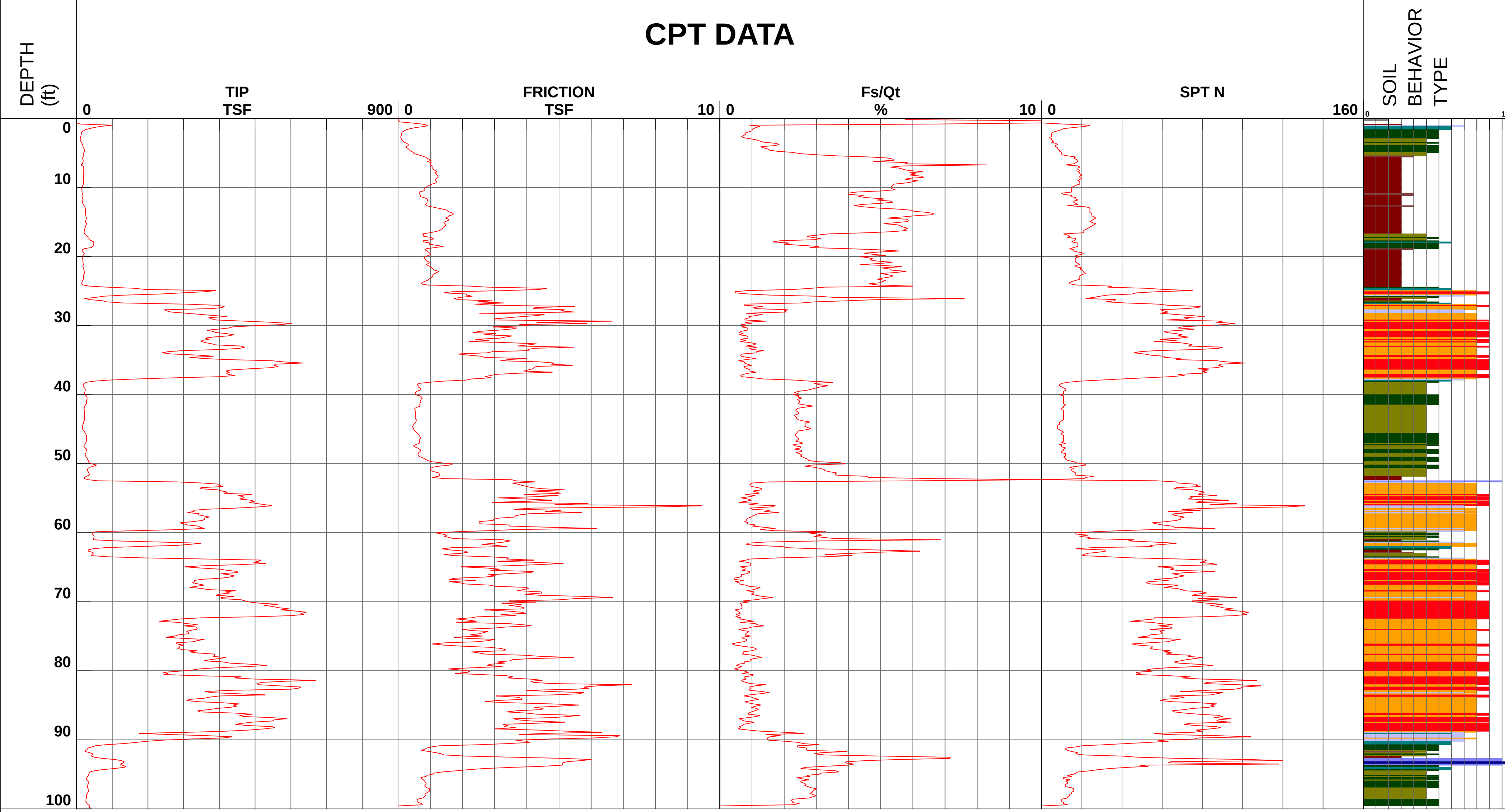
GeoEngineering Solutions

Project	St Elizabeth	Operator	JM-AJ
Job Number	59-1266	Cone Number	DDG1530
Hole Number	CPT-03	Date and Time	9/2/2020 9:02:24 AM
EST GW Depth During Test			63.90 ft

Filename	SDF(078).cpt
GPS	
Maximum Depth	99.90 ft

Net Area Ratio .8

CPT DATA



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

Cone Size 15cm squared

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

Unified Soil Classification (USC) System (from ASTM D 2487)

Major Divisions				Typical Names
Course-Grained Soils More than 50% retained on the 0.075 mm (No. 200) sieve	**Gravels** 50% or more of course fraction retained on the 4.75 mm (No. 4) sieve	Clean Gravels	GW	Well-graded gravels and gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
		Gravels with Fines	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sands 50% or more of course fraction passes the 4.75 (No. 4) sieve	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines
			SP	Poorly graded sands and gravelly sands, little or no fines
		Sands with Fines	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
Fine-Grained Soils More than 50% passes the 0.075 mm (No. 200) sieve	**Silts and Clays** Liquid Limit 50% or less		ML	Inorganic silts, very fine sands, rock four, silty or clayey fine sands
			CL	Inorganic clays of low to medium plasticity, gravelly/sandy/silty/lean clays
			OL	Organic silts and organic silty clays of low plasticity
	Silts and Clays Liquid Limit greater than 50%		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
			CH	Inorganic clays or high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
Highly Organic Soils			PT	Peat, muck, and other highly organic soils

PENETRATION RESISTANCE (RECORDED AS BLOWS/0.5 FEET)				
SAND AND GRAVEL		SILT AND CLAY		
RELATIVE DENSITY	N-VALUE (BLOWS/FOOT)*	CONSISTENCY	N-VALUE (BLOWS/FOOT)*	COMPRESSIVE STRENGTH
Very Loose	0 - 3	Very Soft	0 - 1	0 - 0.25
Loose	4 -10	Soft	2 - 4	0.25 - 0.50
Medium Dense	11 - 29	Medium Stiff	5 - 7	0.50 - 1.0
Dense	30 - 49	Stiff	8 - 14	1.0 - 2.0
Very Dense	50 +	Very Stiff	15 - 29	2.0 - 4.0
		Hard	30 +	Over 4.0

Particle Sizes		
Components		*Size or Sieve Number*
Boulders		Over 12 inches
Cobbles		3 to 12 inches
Gravels	Coarse	3/4 to 3 inches
	Fine	Number 4 to 3/4 inch
Sand	Coarse	Number 10 to Number 4
	Medium	Number 40 to Number 10
	Fine	Number 200 to Number 40
Fines (Silt and Clay)		Below Number 200

Soil Moisture	
Descriptor	*Description*
Dry	Dry of Standard Proctor Optimum
Damp	Sand Dry
Moist	Near Standard Proctor Optimum
Wet	Wet of Standard Proctor Optimum
Saturated	Free Water in Sample

Bulk Sample

Standard Penetration Test

2.5 Inch Modified California Sampler

Shelby Tube

First Water Level Reading

Final Water Level Reading

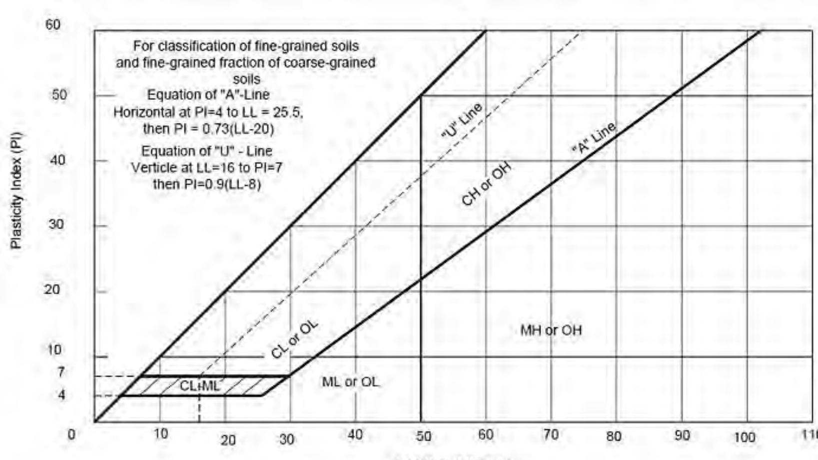
Blow Count
The number of blows of the sampling hammer required to drive the sampler through each of three 6-inch increments. Less than three increments may be reported if more than 50 blows are counted for any increment. The notation 50/5” indicates 50 blows recorded for 5 inches of penetration. Note all of the field blow counts recorded using a Modified California sampler were converted to equivalent SPT blow counts.

N-Value
Number of blows 140 LB hammer falling 30 inches to drive a 2 inch outside diameter (1-3/8 innch I.D.) split barrel sampler the last 12 inches of an 18 inch drive (ASTM-1586 Standard Penetration Test).

General Notes:
1. The boring locations were determined by pacing, sighting and/or measuring from site features. Locations are approximate. Elevations of borings (if included) were determined by interpolation between plan contours or from another source identified in the report. The location and elevation of borings should be considered accurate only to the degree implied by the method.

2.The stratification lines represent the approximate boundary between soil types. The transition may be gradual.

3. Water level readings in the drill holes were recorded at the time and under the conditions stated on the boring logs. It should be noted that flucuations in the level of groundwater may occur due to variations in rainfall, tides and other factors at the time measurements were made



GEO-ENGINEERING SOLUTIONS, INC.
A PEI COMPANY
Geotechnical Engineering • Engineering Geology • Materials Testing

Key to Exploratory Boring Logs

BORING NUMBER B1

CLIENT: Sand Hill Company
PROJECT NAME: Saint Elizabeth
PROJECT NUMBER: 59-1266
PROJECT LOCATION: 1050 St. Elizabeth Dr
DATE STARTED: 9/3/20 COMPLETED: 9/3/20
GROUND ELEVATION: 137 ft HOLE SIZE: 4
DRILLING CONTRACTOR: California Geotech Services, LLC
GROUND WATER LEVELS:
DRILLING METHOD: Solid Flight
AT TIME OF DRILLING: ---
LOGGED BY: CK CHECKED BY: NH
AT END OF DRILLING: ---
NOTES:
AFTER DRILLING: ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	SPT BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT	ATTERBERG LIMITS PLASTIC LIMIT	ATTERBERG LIMITS PLASTICITY INDEX	FINES CONTENT (%)
0.0		(CL) **Silty CLAY** : Brown, Dry to Moist, Low Plastic, Stiff to Very Stiff										
2.5			MC 1-1	100	9-9-8 (17)		92	10	28	14	14	
			MC 1-2	100	5-7-7 (14)		78	10				
5.0												
7.5												
			MC 1-3	100	6-7-8 (15)		85	10				
10.0												
12.5												
			MC 1-4	100	7-11-13 (24)		95	10				
15.0												

(Continued Next Page)

BORING NUMBER B1

GEO-ENGINEERING SOLUTIONS, INC.

CLIENT Sand Hill Company **PROJECT NAME** Saint Elizabeth

PROJECT NUMBER 59-1266 **PROJECT LOCATION** 1050 St. Elizabeth Dr

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	SPT BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
15.0		(CL) **Silty CLAY** : Brown, Dry to Moist, Low Plastic, Stiff to Very Stiff *(continued)*										
17.5												
			MC 1-5	100	12-15-16 (31)		103	9				
20.0												
22.5		(SM) **Silty SAND**: Brown, Moist, Sub Angular Gravel, Very Dense										
			MC 1-6	100	10-9-10 (19)			10				
25.0												
27.5												
			MC 1-7	100	15-40-40 (80)			5				
30.0												

Bottom of borehole at 30.0 feet.

Geo-Engineering Solutions, Inc.

CLIENT Sand Hill Company
PROJECT NAME Saint Elizabeth
PROJECT NUMBER 59-1266
PROJECT LOCATION 1050 St. Elizabeth Dr
DATE STARTED 9/3/20 **COMPLETED** 9/3/20
GROUND ELEVATION 139 ft **HOLE SIZE** 4
DRILLING CONTRACTOR California Geotech Services, LLC
GROUND WATER LEVELS:
DRILLING METHOD Solid Flight
AT TIME OF DRILLING ---
LOGGED BY CK **CHECKED BY** NH
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	SPT BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0.0		(CL) **Silty CLAY** : Brown, Dry to Moist, Low Plastic, Stiff to Very Stiff										
2.5			MC 2-1	100	4-5-9 (14)							
			MC 2-2	100	6-7-8 (15)		90	4				
5.0		Gravels Encountered										
7.5												
			MC 2-3	100	6-7-7 (14)		88	15				
10.0												
12.5		(CL) **Sandy CLAY**: Moist, Brown, Fine Sands, Low Plastic, Very Stiff										
			MC 2-4	100	6-12-12 (24)		107	15				72
15.0												

(Continued Next Page)

BORING NUMBER B2

Geo-Engineering Solutions, Inc.

CLIENT Sand Hill Company
PROJECT NAME Saint Elizabeth
PROJECT NUMBER 59-1266
PROJECT LOCATION 1050 St. Elizabeth Dr

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	SPT BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
15.0		(CL) **Sandy CLAY**: Moist, Brown, Fine Sands, Low Plastic, Very Stiff *(continued)*										
		(CL) **Silty CLAY**: Moist, Brown, Low Plastic, Stiff										
17.5												
			MC 2-5	100	4-6-8 (14)		103	17				
20.0												
		(SM) **Silty SAND** : Moist, Brown, Sub Angular Gravel, Low Plastic, Very Dense										
22.5												
			MC 2-6	100	12-22-28 (50)		106	5				46
25.0												
27.5												
			MC 2-7	100	6-25-31 (56)		118	3				
30.0												

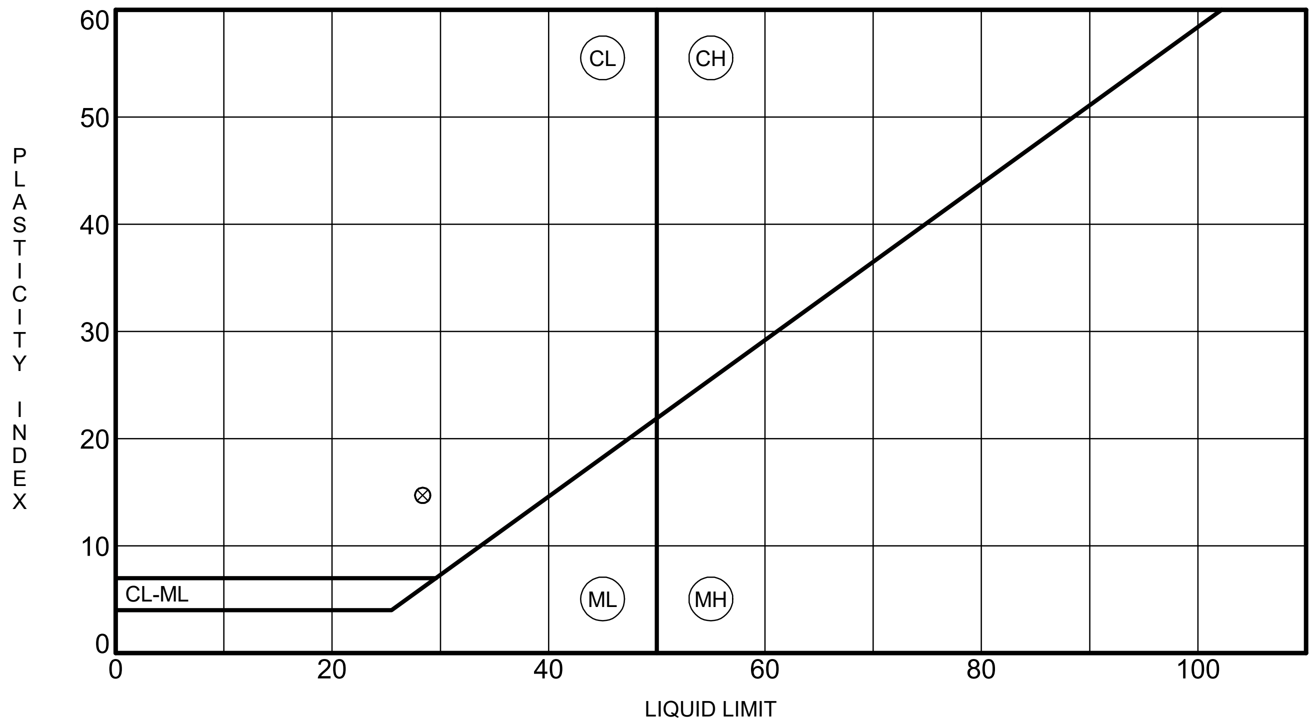
Bottom of borehole at 30.0 feet.

APPENDIX B

LABORATORY TEST RESULTS
Atterberg Limits
Grain Size Distribution Test Results
Corrosivity Test Results

ATTERBERG LIMITS' RESULTS

CLIENT Sand Hill Company
PROJECT NAME Saint Elizabeth
PROJECT NUMBER 59-1266
PROJECT LOCATION 1050 St. Elizabeth Dr



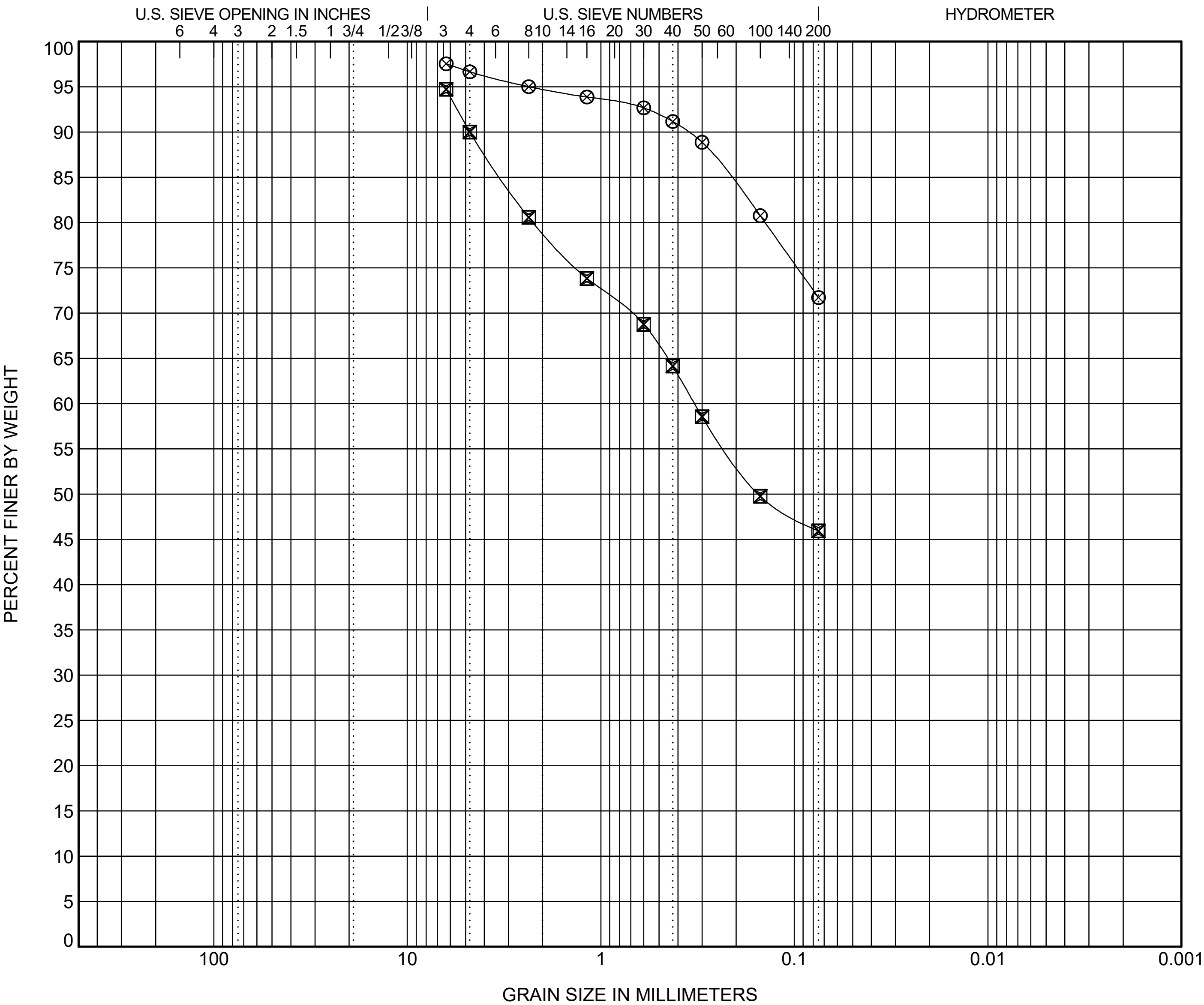
	Specimen Identification		LL	PL	PI	Fines	Classification
⊗	**B1**	**Depth:2.0'**	**28**	**14**	**14**		

GRAIN SIZE DISTRIBUTION

Geo-Engineering Solutions, Inc.

CLIENT Sand Hill Company **PROJECT NAME** Saint Elizabeth

PROJECT NUMBER 59-1266 **PROJECT LOCATION** 1050 St. Elizabeth Dr



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification		Classification	LL	PL	PI	Cc	Cu
⊗ **B2**	**Depth: 14.0'**						
⊠ **B2**	**Depth: 24.0'**						

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
⊗ **B2**	**Depth: 14.0'**	**6.3**				**0.9**	**24.9**	**71.7**	
⊠ **B2**	**Depth: 24.0'**	**6.3**	**0.328**			**4.7**	**44.1**	**45.9**	

COOPER TESTING LABORATORY

Corrosivity Tests Summary

CTL # 1003-113 **Date:** 9/8/2020 **Tested By:** JC **Checked:** PJ
Client: Geo-Engineering Solutions **Project:** Saint Elizabeth **Proj. No:** 59-1266
Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP		Sulfide	Moisture	Soil Visual Description
			As Rec.	Min	Sat.	mg/kg	mg/kg	%		(Redox)		Qualitative	At Test	
						Dry Wt.	Dry Wt.	Dry Wt.		E_H (mv)	At Test	by Lead	%	
Boring	Sample, No.	Depth, ft.	ASTM G57	Cal 643	ASTM G57	ASTM D4327	ASTM D4327	ASTM D4327	ASTM G51	ASTM G200	Temp °C	Acetate Paper	ASTM D2216	
B2	1	2	-	-	3,034	9	152	0.0152	7.7	492	22	Negative	6.9	Dark Yellowish Brown Sandy CLAY w/ organics