Appendix D:

Geology and Soils Supporting Information

THIS PAGE INTENTIONALLY LEFT BLANK

CORNERSTONE EARTH GROUP

TYPE OF SERVICES	Geotechnical Investigation
PROJECT NAME	Montague Pointe Industrial
LOCATION	Montague Expressway and Zanker Road San Jose, California
CLIENT	McCarthy Development
PROJECT NUMBER	1543-2-1
DATE	May 2, 2025



CORNERSTONE EARTH GROUP

Type of Services	**Geotechnical Investigation**
Project Name	**Montague Pointe Industrial**
Location	**Montague Expressway and Zanker Road** **San Jose, California**
Client	**McCarthy Development**
Client Address	**210 Almendra Avenue** **Los Gatos, California**
Project Number	**1543-2-1**
Date	**May 2, 2025**

Prepared by

Jesse D. Serpa, P.E.
Project Engineer
Geotechnical Project Manager



Maura F. Ruffatto, P.E.
Principal Engineer
Quality Assurance Reviewer



1259 Oakmead Parkway | Sunnyvale, CA 94085
T 408 245 4600

3697 Mt. Diablo Boulevard, Suite 310 | Lafayette, CA 94549
T 925 988 9500

www.cornerstoneearth.com

CORNERSTONE EARTH GROUP

TABLE OF CONTENTS

SECTION 1: INTRODUCTION 1
1.1 Project Description 1
1.2 Scope of Services 1
1.3 Exploration Program 2
1.4 Laboratory Testing Program 2
1.5 Corrosion Evaluation 2
1.6 Environmental Services 2
SECTION 2: REGIONAL SETTING 2
2.1 Geological Setting 2
2.2 Regional Seismicity 3
Table 1: Approximate Fault Distances 3
SECTION 3: SITE CONDITIONS 3
3.1 Site Background 3
3.2 Surface Description 4
3.3 Subsurface Conditions 4
3.3.1 Plasticity/Expansion Potential 5
3.3.2 In-Situ Moisture Contents 5
3.4 Groundwater 5
3.5 In-Situ Infiltration Testing Program 6
Table 2: In-Situ Field Guelph Permeameter Test Results 6
3.5.1 Subsurface Conditions 6
3.5.2 Reliability of Field Test Data 6
3.5.3 Findings and Recommendations 7
3.5.4 General Comments and Design Considerations 7
SECTION 4: GEOLOGIC HAZARDS 7
4.1 Fault Surface Rupture 7
4.2 Estimated Ground Shaking 7
4.3 Liquefaction Potential 7
4.3.1 Background 8
4.3.2 Analysis 8
4.3.3 Summary 9
4.3.4 Ground Deformation and Surficial Cracking Potential 9
4.4 Lateral Spreading 9
4.5 Seismic Settlement/Unsaturated Sand Shaking 10
4.6 Tsunami/seiche 10
4.7 Flooding 11
SECTION 5: CONCLUSIONS 11
5.1 Summary 11
5.1.1 Potential for Liquefaction-Induced Settlements 11
5.1.2 Potential for Static Settlements 12

5.1.3 Redevelopment Considerations 12
5.1.4 Shallow Groundwater 12
5.1.5 Presence of Moderately to Highly Expansive Soils 12
5.1.6 Undocumented Fill 13
5.1.7 Soil Corrosion Potential 13
5.2 Plans and Specifications Review ---- 14
5.3 Construction Observation and Testing ---- 14
SECTION 6: EARTHWORK 14
6.1 Site Demolition ---- 14
6.1.1 Demolition of Existing Slabs, Foundations and Pavements 14
6.1.2 Abandonment of Existing Utilities 15
6.2 Site Clearing and Preparation ---- 15
6.2.1 Site Stripping 15
6.2.2 Tree and Shrub Removal 16
6.3 Mitigation of Undocumented Fills ---- 16
6.4 Temporary Cut and Fill Slopes ---- 16
6.5 Subgrade Preparation ---- 17
6.6 Wet Soil Stabilization Guidelines ---- 17
6.6.1 Scarification and Drying 17
6.6.2 Removal and Replacement 17
6.6.3 Chemical Treatment 17
6.7 Material for Fill ---- 18
6.7.1 Re-Use of On-site Soils 18
6.7.2 Re-Use of On-Site Site Improvements 18
6.7.3 Potential Import Sources 18
6.7.4 Non-Expansive Fill Using Lime Treatment 19
6.8 Compaction Requirements ---- 19
Table 3: Compaction Requirements 20
6.8.1 Construction Moisture Conditioning 20
6.9 Trench Backfill ---- 20
6.10 Site Drainage ---- 21
6.10.1 Surface Drainage 21
6.11 Low-Impact Development (LID) Improvements ---- 22
6.11.1 Storm Water Treatment Design Considerations 22
6.12 Landscape Considerations ---- 24
SECTION 7: 2022 CBC SEISMIC DESIGN CRITERIA 24
7.1 Seismic Design Criteria ---- 24
7.1.1 Site Location and Provided Data For 2022 CBC Seismic Design 24
7.2 2022 CBC Seismic Design Criteria ---- 25
7.2.1 2022 CBC Seismic Design 25
SECTION 8: FOUNDATIONS 25
8.1 Summary of Recommendations ---- 25
8.2 Shallow Foundations ---- 25
8.2.1 Conventional Shallow Footings 25

8.2.2 Footing Settlement 26
Table 4: Assumed Structural Loading 26
8.2.3 Lateral Loading 26
8.2.4 Conventional Shallow Footing Construction Considerations 27
SECTION 9: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS 27
9.1 Interior Office Slabs-on-Grade ---- 27
9.2 Warehouse Slabs-on-Grade ---- 27
9.3 Interior Slabs Moisture Protection Considerations ---- 28
9.4 Exterior Flatwork ---- 29
SECTION 10: VEHICULAR PAVEMENTS 30
10.1 Asphalt Concrete ---- 30
Table 5: Asphalt Concrete Pavement Recommendations, Untreated Subgrade 31
Table 6: Asphalt Concrete Pavement Recommendations, Chemical-Treated Subgrade 32
10.2 Portland Cement Concrete ---- 32
Table 7: PCC Pavement Recommendations, Untreated Subgrade 33
Table 8: PCC Pavement Recommendations, Chemical-Treated Subgrade 34
10.3 Trash Enclosures ---- 34
10.4 Pavement Cutoff ---- 34
SECTION 11: RETAINING WALLS 34
11.1 Static Lateral Earth Pressures ---- 34
Table 9: Recommended Lateral Earth Pressures 35
11.2 Seismic Lateral Earth Pressures ---- 35
11.3 Backfill ---- 36
11.4 Foundations ---- 36
SECTION 12: LIMITATIONS 36
SECTION 13: REFERENCES 37

FIGURE 1: VICINITY MAP
FIGURE 2: SITE PLAN
FIGURE 3: REGIONAL FAULT MAP
FIGURE 4A TO 4K: LIQUEFACTION ANALYSIS SUMMARY – CPT-1 TO CPT-11
FIGURE 5: MCE_R DESIGN RESPONSE SPECTRA
FIGURE 6: DESIGN RESPONSE SPECTRA

APPENDIX A: FIELD INVESTIGATION
APPENDIX B: LABORATORY TEST PROGRAM
APPENDIX C: SITE CORROSIVITY EVALUATION

CORNERSTONE EARTH GROUP

Type of Services	**Geotechnical Investigation**
Project Name	**Montague Pointe Industrial**
Location	**Montague Expressway and Zanker Road** **San Jose, California**

SECTION 1: INTRODUCTION

This geotechnical report was prepared for the sole use of McCarthy Development for the Montague Pointe Industrial project in San Jose, California. The location of the site is shown on the Vicinity Map, Figure 1. For our use, we were provided with the following documents:

- A conceptual site plan titled "Montague Pointe | Site Plan Analysis," prepared by Lowney Architects, dated January 19, 2024.

1.1 PROJECT DESCRIPTION

The project will consist of redeveloping the approximately 23¾-acre site for three new buildings ranging from about 114,000 to 178,000 square feet. We understand that the final building layout may change and the current Building C may be split into two buildings. The new buildings will be of concrete tilt-up construction, and be single story with typical column spacings of 50 to 60 feet. Dock doors will be generally located toward the south of the buildings. Office and mezzanine areas are also planned. At-grade auto and trailer parking, and drive aisles will cover the remainder of the site. Appurtenant utilities, landscaping, storm water management areas, and other improvements necessary for overall site development will also be constructed. Existing buildings and site improvements will be demolished for new construction.

Structural loads are not available; however, structural loads are expected to be typical for similar structures with interior column dead plus live loads on the order of 50 to 100 kips and perimeter wall loads on the order of 7 to 9 kips per linear foot. Cuts and fills up to about 2 to 5 feet are expected for site grading.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated November 6, 2024, and consisted of field and laboratory programs to evaluate physical and engineering properties of the subsurface soils, engineering analysis to prepare recommendations for site work and grading, building

foundations, flatwork, retaining walls, and pavements, and preparation of this report. Brief descriptions of our exploration and laboratory programs are presented below.

1.3 EXPLORATION PROGRAM

Field exploration consisted of eight borings drilled on March 21 and 25, 2025 with truck-mounted, hollow-stem auger drilling equipment, and eleven Cone Penetration Tests (CPTs) advanced on March 13 and 20, 2025. The borings were drilled to depths of 25 to 45 feet; the CPTs were advanced to depths of approximately 50 to 110 feet. Seismic shear wave velocity measurements were collected from CPT-7 and CPT-11. Borings EB-1, EB-2, EB-3, EB-4, EB-5, EB-6, EB-7, and EB-8 were advanced adjacent to CPT-1, CPT-2, CPT-11, CPT-4, CPT-8, CPT-6, CPT-7, and CPT-10, respectively, for direct evaluation of physical samples to correlated soil behavior. The borings and CPTs were backfilled with cement grout in accordance with local requirements; exploration permits were obtained as required by local jurisdictions.

The approximate locations of our exploratory borings and CPTs are shown on the Site Plan, Figure 2. Details regarding our field program are included in Appendix A.

1.4 LABORATORY TESTING PROGRAM

In addition to visual classification of samples, the laboratory program focused on obtaining data for foundation design and seismic ground deformation estimates. Testing included moisture contents, dry densities, washed sieve analyses, and Plasticity Index tests. Details regarding our laboratory program are included in Appendix B.

1.5 CORROSION EVALUATION

Six samples from our borings from depths of 2 to 6 feet were tested for saturated resistivity, pH, and soluble sulfates and chlorides. JDH Corrosion Consultants prepared a brief corrosion evaluation based on the laboratory data, which is attached to this report in Appendix C. In general, the on-site soils can be characterized as corrosive to buried metal, and non-corrosive to buried concrete.

1.6 ENVIRONMENTAL SERVICES

Environmental services were not requested for this project. If environmental concerns are determined to be present during future evaluations, the project environmental consultant should review our geotechnical recommendations for compatibility with the environmental concerns.

SECTION 2: REGIONAL SETTING

2.1 GEOLOGICAL SETTING

The site is located within the Santa Clara Valley, which is a broad alluvial plane between the Santa Cruz Mountains to the southwest and west, and the Diablo Range to the northeast. The San Andreas Fault system, including the Monte Vista-Shannon Fault, exists within the Santa

Cruz Mountains and the Hayward and Calaveras Fault systems exist within the Diablo Range. Alluvial soil thicknesses in the general project vicinity are mapped as greater than 500 feet (Rogers & Williams, 1974).

2.2 REGIONAL SEISMICITY

While seismologists cannot predict earthquake events, geologists from the U.S. Geological Survey have recently updated (in 2015) earlier estimates from their 2014 Uniform California Earthquake Rupture Forecast (Version 3; UCERF3) publication. The estimated probability of one or more magnitude 6.7 earthquakes (the size of the destructive 1994 Northridge earthquake) expected to occur somewhere in the San Francisco Bay Area has been revised (increased) to 72 percent for the period 2014 to 2043 (Aagaard et al., 2016). The faults in the region with the highest estimated probability of generating damaging earthquakes between 2014 and 2043 are the Hayward (33%), Calaveras (26%), and San Andreas Faults (22%). In this 30-year period, the probability of an earthquake of magnitude 6.7 or larger occurring is 22 percent along the San Andreas Fault and 33 percent for the Hayward Fault.

The faults considered capable of generating significant earthquakes are generally associated with the well-defined areas of crustal movement, which trend northwesterly. The table below presents the State-considered active faults within 25 kilometers of the site.

Table 1: Approximate Fault Distances

Fault Name	Distance	
	(miles)	(kilometers)
Hayward (Southeast Extension)	4.2	6.8
Hayward (Total Length)	6.8	10.8
Calaveras	7.7	12.4
Monte Vista-Shannon	10.9	17.5
San Andreas (1906)	13.4	21.6

A regional fault map is presented as Figure 3, illustrating the relative distances of the site to significant fault zones.

SECTION 3: SITE CONDITIONS

3.1 SITE BACKGROUND

We reviewed historic aerial imagery provided online by Historic Aerials (historicaerials.com). A summary of pertinent changes at and near the project site is as follows:

- 1948: The project site is occupied by multiple agricultural fields and dirt/gravel roads.

- 1980: The existing alignments for Montague Expressway, Junction Avenue, Zanker Road, and Research Place have been constructed. The project site is undeveloped at this time.

- 1982: The existing alignment of East Plumeria Drive has been constructed. The project site remains undeveloped.

- 1987: The existing buildings and surrounding asphalt concrete parking lots at 3050 Zanker Road, 2860 Junction Avenue, and 411 East Plumeria Drive have been constructed. The commercial/industrial developments to the northwest, west, south, and southeast of the project site have been constructed or are currently undergoing construction

- 1993: The existing buildings and surrounding asphalt concrete parking lots at 2840 and 2880 Junction Avenue have been constructed. The existing residential development to the northeast and east of the site have also been constructed.

- 2002: The existing commercial building to the south of the project site has been constructed. No pertinent surface changes were observed since this image.

3.2 SURFACE DESCRIPTION

The project site is occupied by five commercial buildings (3050 Zanker Road, 2840 Junction Avenue, 2860 Junction Avenue, 2880 Junction Avenue, and 411 East Plumeria Drive) surrounded by asphalt concrete parking lots with numerous landscaping islands throughout the parking lots. Mature trees generally line the outside boundary of the site. Pedestrian flatwork and landscaping features are generally present adjacent to each of the existing buildings. The site is generally level and graded to drain to storm drainage facilities, with elevations ranging from 15 to 19 feet based on elevations obtained from Google Earth. The project site is bounded by Montague Expressway to the north and east, Junction Avenue and East Plumeria Drive to the south and west, and Zanker Road to the west.

Surface pavements generally consisted of 2 to 3 inches of asphalt concrete over 3 to 7 inches of aggregate base. Based on visual observations, the existing pavements are generally in fair to good condition with localized areas of minor cracking.

3.3 SUBSURFACE CONDITIONS

Below the surface pavements in the northwest half of the project site, Borings EB-1, EB-3, and EB-4 generally encountered undocumented fills to depths ranging from approximately 2 to 2½ feet below existing grades. The undocumented fills generally consisted of sandy lean clays with variable amounts of sand and gravel. Boring EB-2 is also located in the northwest half of the site and did not encounter undocumented fills below the surface pavements. Below the undocumented fills or where no fills where encountered, Borings EB-1 through EB-4 generally encountered native stiff to hard lean clays with variable amounts of sand to depths ranging from

23 to 25 feet, underlain by loose to dense sands with variable amounts of silt and clay down to a depth of 40 feet, or terminal boring depths.

Below the terminal boring depth of 40 feet, CPT-1, CPT-2, CPT-4, CPT-5, CPT-5, and CPT-11 located within the northwest half of the site, generally encountered stiff to hard clays to a depth of 58 feet, underlain by dense to very dense layers of sand with gravel down to a depth of 95 feet followed by very stiff to hard clays to a depth of 109 feet, the maximum depth explored in the northwest half of the site.

Below the surface pavements in the southeast half of the project site, Borings EB-5, EB-6, EB-7, and EB-8 generally encountered very stiff to hard fat clay to depths ranging from 4½ to 7 feet below existing grades underlain by lean clays with variable amounts of sand to depths ranging from 26 to 28½ feet below existing grades and medium dense to very dense sands with variable amounts of silt and clay to a depth of 45 feet, or terminal boring depths.

Below the terminal boring depth of 45 feet, CPT-6, CPT-7, CPT-8, CPT-9, and CPT-10, located within the southeast half of the site, generally encountered interbedded layers of dense to very dense layers of sand with gravel and stiff to hard clays to a depth of 95 feet, underlain by stiff to hard clays to a depth of 110 feet, the maximum depth explored in the southeast half of the site.

3.3.1 Plasticity/Expansion Potential

We performed two Plasticity Index (PI) tests on representative samples. Test results were used to evaluate expansion potential of surficial soils. The results of the surficial PI tests indicated PIs ranging from 22 in the northeast half to 32 in the southeast half, indicating moderate to high expansion potential to wetting and drying cycles.

3.3.2 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents within the upper 10 feet range from about 6 to 15 percent over the estimated laboratory optimum moisture.

3.4 GROUNDWATER

Groundwater was encountered in our Borings at depths ranging from 10 to 19 feet below current grades. Groundwater was inferred based on CPT pore pressure dissipation measurements at depths ranging from about 9 to 11 feet below current grades. All measurements were taken at the time of drilling and may not represent the stabilized levels that can be higher than the initial levels encountered.

Historic high groundwater is mapped within the project vicinity at a depth of approximately 7 feet (CGS, Milpitas 7.5-Minute Quadrangle, 2001). We also reviewed groundwater data available online from the website GeoTracker, https://geotracker.waterboards.ca.gov. Nearby data from monitoring wells located approximately 250 feet north of the project site at 260 Innovation Drive indicates that groundwater has been measured at depths of approximately 7½ to 13 feet between 2010 to 2012. Monitoring wells located approximately 2,200 feet west of the project

site at the northwest corner of North First Street and Montague Expressway indicate that groundwater has been measured at depths of approximately 9½ to 14½ feet in November 2006.

Based on the above, we recommend a design groundwater depth of 7 feet. Fluctuations in groundwater levels occur due to many factors including seasonal fluctuation, underground drainage patterns, regional fluctuations, and other factors.

3.5 IN-SITU INFILTRATION TESTING PROGRAM

To estimate the infiltration rate of the near-surface soil, we performed two in-situ field infiltration tests on March 6, 2025. The infiltration tests were performed using a Guelph permeameter by Soil Moisture Equipment Corp., Model #2800, in general accordance with ASTM D5126. Generally, the Guelph permeameter is a constant head device which uses two water-filled chambers to measure infiltration rate in a shallow borehole. A constant head level is established in the borehole and the rate of water outflow into the surrounding soil is documented. The rate of flow when it reaches a steady state, or constant rate, is used to determine an approximate infiltration rate for that location and depth. Results from our tests are shown below in Table 2.

Table 2: In-Situ Field Guelph Permeameter Test Results

Boring / Infiltration Test	Soil Type	Infiltration Depth (feet)	Infiltration Rate (in/hour)
I-1	Lean Clay (CL)	5	0.04
I-2	Fat Clay (CH)	3	0.02

3.5.1 Subsurface Conditions

In general, the soils encountered at our infiltration tests I-1 consisted of undocumented fills consisting of lean clay with sand and gravel to a depth of 2½ feet, underlain by native lean clay with sand to a depth of 5 feet, the maximum depth of the infiltration test. Infiltration test I-2 encountered native fat clay to a depth of 3 feet, the maximum depth of the infiltration test. Copies of the infiltration test boring logs are included in Appendix A.

3.5.2 Reliability of Field Test Data

Test results may not be truly indicative of the long-term, in-situ infiltration. Other factors including stratifications, heterogeneous deposits, overburden stress, disturbance, organic content, depth to groundwater, and other factors can influence test results. In addition, for stratified soils such as those encountered at the site, the average horizontal infiltration is typically greater than the average vertical infiltration.

3.5.3 Findings and Recommendations

Based on our findings, the soils at the locations tested and at depths of 3 to 5 feet below existing grades have infiltration rates ranging from approximately 0.02 to 0.04 inches per hour. Based on our test results, the in-situ field tests indicated a low infiltration rate at the depths and locations tested.

We recommend the project Civil Engineer review the above information and provide additional recommendations as deemed necessary.

3.5.4 General Comments and Design Considerations

As discussed, the tests were performed at discrete locations and depths. In addition, some disturbance in preparing the tests may have occurred. Therefore, the above results can vary significantly and may not be representative over the entire site. Localized areas/depths with higher or lower permeable materials can increase or decrease the actual infiltration rates. Therefore, we recommend the potential for variations be considered when evaluating the soil infiltration capacity or performance.

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT SURFACE RUPTURE

As discussed above several significant faults are located within 25 kilometers of the site. The site is not located within a State-designated Alquist Priolo Earthquake Fault Zone, or a Santa Clara County Fault Hazard Zone, or a City of San Jose Potential Hazard Zone. As shown in Figure 3, no known surface expression of fault traces is thought to cross the site; therefore, fault surface rupture hazard is not a significant geologic hazard at the site.

4.2 ESTIMATED GROUND SHAKING

Moderate to severe (design-level) earthquakes can cause strong ground shaking, which is the case for most sites within the Bay Area. A peak ground acceleration (PGA_M) was estimated following the ground motion hazard analysis procedure presented in Chapter 16 and 18 and Appendix J of the 2022 California Building Code (CBC) and Chapter 21, Section 21.2 of ASCE 7-16 and Supplement No. 1. For our analysis we used a PGA_M of 0.76g which was determined in accordance with Section 21.5 of ASCE 7-16.

4.3 LIQUEFACTION POTENTIAL

The site is within a State-designated Liquefaction Hazard Zone (CGS, Milpitas 7.5-Minute Quadrangle, 2001) as well as a Santa Clara County Liquefaction Hazard Zone (Santa Clara County, 2003). Our field and laboratory programs addressed this issue by testing and sampling potentially liquefiable layers to depths of at least 50 feet, performing visual classification on sampled materials, evaluating CPT data, and performing various tests to further classify soil properties.

4.3.1 Background

During strong seismic shaking, cyclically induced stresses can cause increased pore pressures within the soil matrix that can result in liquefaction triggering, soil softening due to shear stress loss, potentially significant ground deformation due to settlement within sandy liquefiable layers as pore pressures dissipate, and/or flow failures in sloping ground or where open faces are present (lateral spreading) (NCEER 1998). Limited field and laboratory data is available regarding ground deformation due to settlement; however, in clean sand layers settlement on the order of 2 to 4 percent of the liquefied layer thickness can occur. Soils most susceptible to liquefaction are loose, non-cohesive soils that are saturated and are bedded with poor drainage, such as sand and silt layers bedded with a cohesive cap.

4.3.2 Analysis

As discussed in the "Subsurface" section above, several sand layers were encountered below the design groundwater depth of 7 feet. Following the liquefaction analysis framework in the 2008 monograph, *Soil Liquefaction During Earthquakes* (Idriss and Boulanger, 2008), incorporating updates in *CPT and SPT Based Liquefaction Triggering Procedures* (Boulanger and Idriss, 2014), and in accordance with CDMG Special Publication 117A guidelines (CDMG, 2008) for quantitative analysis, these layers were analyzed for liquefaction triggering and potential post-liquefaction settlement. These methods compare the ratio of the estimated cyclic shaking (Cyclic Stress Ratio - CSR) to the soil's estimated resistance to cyclic shaking (Cyclic Resistance Ratio - CRR), providing a factor of safety against liquefaction triggering. Factors of safety less than or equal to 1.3 are considered to be potentially liquefiable and capable of post-liquefaction re-consolidation (i.e. settlement).

The CSR for each layer quantifies the stresses anticipated to be generated due to a design-level seismic event, is based on the peak horizontal acceleration generated at the ground surface discussed in the "Estimated Ground Shaking" section above, and is corrected for overburden and stress reduction factors as discussed in the procedure developed by Seed and Idriss (1971) and updated in the 2008 Idriss and Boulanger monograph.

In estimating post-liquefaction settlement at the site, we have implemented a depth weighting factor proposed by Cetin (2009). Following evaluation of 49 high-quality, cyclically induced, ground settlement case histories from seven different earthquakes, Cetin proposed the use of a weighting factor based on the depth of layers. The weighting procedure was used to tune the surface observations at liquefaction sites to produce a better model fit with measured data. Aside from the better model fit it produced, the rationale behind the use of a depth weighting factor is based on the following: 1) upward seepage, triggering void ratio redistribution, and resulting in unfavorably higher void ratios for the shallower sublayers of soil layers; 2) reduced induced shear stresses and number of shear stress cycles transmitted to deeper soil layers due to initial liquefaction of surficial layers; and 3) possible arching effects due to nonliquefied soil layers. All these may significantly reduce the contribution of volumetric settlement of deeper soil layers to the overall ground surface settlement (Cetin, 2009).

The soil's CRR is estimated from the in-situ measurements from CPTs and laboratory testing on samples retrieved from our borings. SPT "N" values obtained from hollow-stem auger borings were not used in our analyses, as the "N" values obtained are less reliable in sands below groundwater. The tip pressures are corrected for effective overburden stresses, taking into consideration both the groundwater level at the time of exploration and the design groundwater level, and stress reduction versus depth factors. The CPT method utilizes the soil behavior type index (I_C) to estimate the plasticity of the layers.

The results of our CPT analyses (CPT-1 through CPT-11) are presented on Figures 4A through 4K of this report.

4.3.3 Summary

Our analyses indicate that several layers could potentially experience liquefaction triggering that could result in post-liquefaction total settlement at the ground surface ranging up to about 1 inch based on the Yoshimine (2006) method. As discussed in SP 117A, differential movement for level ground sites over deep soil sites will be up to about two-thirds of the total settlement between independent foundation elements. In our opinion, differential settlements are anticipated to be on the order of ¾-inch or less over a horizontal distance of 50 to 60 feet.

4.3.4 Ground Deformation and Surficial Cracking Potential

The methods used to estimate liquefaction settlements assume that there is a sufficient cap of non-liquefiable material to prevent ground deformation or sand boils. For ground deformation to occur, the pore water pressure within the liquefiable soil layer will need to be great enough to break through the overlying non-liquefiable layer, which could cause significant ground deformation and settlement. The work of Youd and Garris (1995) indicates that the 17- to 29-foot-thick layer of non-liquefiable cap is sufficient to prevent ground deformation and significant surficial cracking; therefore, the above total settlement estimates are reasonable.

4.4 LATERAL SPREADING

Lateral spreading is horizontal/lateral ground movement of relatively flat-lying soil deposits towards a free face such as an excavation, channel, or open body of water; typically, lateral spreading is associated with liquefaction of one or more subsurface layers near the bottom of the exposed slope. As failure tends to propagate as block failures, it is difficult to analyze and estimate where the first tension crack will form.

The current Guadalupe River runs approximately 2,770 feet west of the project site, and the current Coyote Creek runs approximately 2,650 feet east of the project site. The Guadalupe River channel bottom is at approximately Elevation -1 to 2 feet based on elevations obtained from Google Earth. Therefore, the Guadalupe River channel bottom is approximately 16 to 20 feet deep relative to existing site grades. The Coyote Creek channel bottom is at approximately Elevation 14 to 17 feet based on elevations obtained from Google Earth. Therefore, the Coyote Creek channel bottom is approximately 1 to 5 feet deep relative to existing site grades. As part of our liquefaction analyses, we calculated the Lateral Displacement Index (LDI) for potentially

liquefiable layers based on methods presented in the 2008 monograph, *Soil Liquefaction During Earthquakes* (Idriss and Boulanger, 2008). LDI is a summation of the maximum shear strains versus depth, which is a measurement of the potential maximum displacement at that exploration location. Summations of the LDI values to a depth equal to twice the open face height were included. Estimated displacements in the vicinity of CPT-1 through CPT-11 based on the LDI calculations are considered negligible up to a few inches.

We also reviewed the historical ground failure map presented in the Seismic Hazard Zone Report for the vicinity (CGS, Milpitas 7.5-Minute Quadrangle, 2001), which indicated no occurrences of lateral spread within the general vicinity of the site. Therefore, in our opinion based on the above, the potential for lateral spreading to impact the project is relatively low.

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose unsaturated sandy soils can settle during strong seismic shaking. We evaluated the potential for seismic compaction of the loose to medium dense sands based on the work by Robertson and Shao (2010). Based on our analyses, the potential for significant seismic settlement affecting the proposed improvements is low.

4.6 TSUNAMI/SEICHE

The terms tsunami or seiche are described as ocean waves or similar waves usually created by undersea fault movement or by a coastal or submerged landslide. Tsunamis may be generated at great distance from shore (far field events) or nearby (near field events). Waves are formed as the displaced water moves to regain equilibrium, and radiates across the open water similar to ripples from a rock being thrown into a pond. When the waveform reaches the coastline, it quickly raises the water level, with water velocities as high as 15 to 20 knots. The water mass, as well as vessels, vehicles, or other objects in its path create tremendous forces as they impact coastal structures.

Tsunamis have affected the coastline along the Pacific Northwest during historic times. The Fort Point tide gauge in San Francisco recorded approximately 21 tsunamis between 1854 and 1964. The 1964 Alaska earthquake generated a recorded wave height of 7.4 feet and drowned eleven people in Crescent City, California. For the case of a far-field event, the Bay area would have hours of warning; for a near field event, there may be only a few minutes of warning, if any.

A tsunami or seiche originating in the Pacific Ocean would lose much of its energy passing through San Francisco Bay. Based on the mapping of tsunami inundation potential for the San Francisco Bay Area by CGS (conservation.ca.gov/cgs/tsunami/maps), areas most likely to be inundated are marshlands, tidal flats, and former bay margin lands that are now artificially filled, but are still at or below sea level, and are generally within 1½ miles of the shoreline. The site is approximately 7 miles inland from the San Francisco Bay shoreline and is approximately 15 to 19 feet above mean sea level based on elevations obtained from Google Earth. In addition, the site is mapped by the State of California as being outside a tsunami hazard area (CGS, 2021). Therefore, the potential for inundation due to tsunami or seiche is considered low.

4.7 FLOODING

Based on our internet search of the Federal Emergency Management Agency (FEMA) flood map public database, the site is located within Zone X, defined as, “0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile.” We recommend the project civil engineer be retained to confirm this information and verify the base flood elevation, if appropriate.

The Department of Water Resources (DWR), Division of Safety of Dams (DSOD) compiled a database of Dam Failure Inundation Hazard Maps (DSOD, 2015). The generalized hazard maps were prepared by dam owners as required by the State Office of Emergency Services; they are intended for planning purposes only. Based on our review of these maps, the site is located within a dam failure inundation area for the Coyote, James J. Lenihan, and Leroy Anderson Reservoirs.

SECTION 5: CONCLUSIONS

5.1 SUMMARY

From a geotechnical viewpoint, the project is feasible provided the concerns listed below are addressed in the project design. Descriptions of each concern with brief outlines of our recommendations follow the listed concerns.

- Potential for liquefaction-induced settlements
- Potential for static settlements
- Redevelopment considerations
- Shallow groundwater
- Presence of moderately to highly expansive soils
- Undocumented fill
- Soil corrosion potential

5.1.1 Potential for Liquefaction-Induced Settlements

As discussed, our liquefaction analysis indicates that there is a potential for liquefaction of localized sand layers during a significant seismic event. Although the potential for liquefied sands to vent to the ground surface through cracks in the surficial soils is low, our analysis indicates that liquefaction-induced settlement on the order of up to about 1 inch could occur, resulting in differential settlement up to ¾-inch. Foundations should be designed to tolerate the anticipated total and differential settlements. Detailed foundation recommendations are presented in the “Foundations” section.

5.1.2 Potential for Static Settlements

Based on estimated dead plus live loading (50 to 100 kips for interior columns and 7 to 9 kips per liner foot for exterior walls), we estimate that the new building loads would result in total settlement on the order of ½-inch for interior columns and exterior perimeter wall footings. However, for deepened footings at the planned truck loading docks, we estimated total static settlements ranging up to 1-inch. The above estimates are based on structural loads summarized above and should be further reviewed once additional grading and foundation loading is available. The structural engineer and design team should review the estimated settlements and determine if the structure can tolerate the estimated settlements. Detailed foundation recommendations are presented in the "Foundations" section.

5.1.3 Redevelopment Considerations

As discussed, the site is currently occupied by existing buildings and appurtenant flatwork, site fixtures, and landscaping. We understand that all of the existing improvements will be demolished for the construction of the building additions. Potential issues that are often associated with redeveloping sites include demolition of existing improvements, abandonment of existing utilities, and undocumented fills. Please refer to the "Earthwork" section below for further recommendations.

5.1.4 Shallow Groundwater

Groundwater was measured at depths ranging from about 9 to 19 feet below the existing ground surface in our explorations. However, as discussed, we anticipate that high groundwater may range up to 7 feet below the existing ground surface. Our experience with similar sites in the vicinity indicates that shallow groundwater could significantly impact grading and underground construction. These impacts typically consist of potentially wet and unstable pavement subgrade, difficulty achieving compaction, and difficult underground utility installation. Dewatering and shoring of utility trenches may be required in some isolated areas of the site. Detailed recommendations addressing this concern are presented in the "Earthwork" section of this report.

Additionally, we anticipate that cuts up to about 5 feet will be required to establish finished grade for loading dock areas of the buildings. As such, the subgrade soils for the pavements may be wet and be susceptible to softening, especially during periods of higher groundwater. Depending on the final depth and elevation of the loading docks, additional mitigation and subgrade stabilization or a sub-drainage system may be considered if water is present. We should be consulted to provide additional recommendation, if needed, based on the final elevation of the proposed loading docks.

5.1.5 Presence of Moderately to Highly Expansive Soils

Moderately to highly expansive surficial soils generally blanket the site. Expansive soils can undergo significant volume change with changes in moisture content. They shrink and harden when dried and expand and soften when wetted. To reduce the potential for damage to the

planned structures, slabs-on-grade should have sufficient reinforcement and be supported on a layer of non-expansive fill; footings should extend below the zone of seasonal moisture fluctuation. In addition, it is important to limit moisture changes in the surficial soils by using positive drainage away from buildings as well as limiting landscaping watering. Evaluation of potential import sources for the site should consider the acceptable range of plasticity. We recommend that a plug of low-permeability clay soil, sand-cement slurry, or lean concrete be placed within trenches just outside where the trenches pass into building and pavement areas. Detailed grading and foundation recommendations addressing this concern are presented in the following sections.

5.1.6 Undocumented Fill

As discussed above, Borings EB-1, EB-3, and EB-4 generally encountered undocumented fills to depths ranging from approximately 2 to 2½ feet below existing grades. While fills were not identified in all our explorations, undocumented fill may be present as a result of prior development grading. Therefore, any fills encountered during site grading should be completely removed from within building areas and to a lateral distance of at least 5 feet beyond the building footprint or to a lateral distance equal to fill depth below the perimeter footing, whichever is greater. Provided the fills meet the "Material for Fill" requirements below, the fills may be reused when backfilling the excavations. Based on review of the samples collected from our borings, it appears that the fill may be reused. If materials are encountered that do not meet the requirements, such as debris, wood, trash, those materials should be screened out of the remaining material and be removed from the site. Backfill of excavations should be placed in lifts and compacted in accordance with the "Compaction" section below.

If observation and testing records are available for the existing fill materials, they should be provided for our review. Provided undocumented fills are mitigated by removal and replacement as engineered fill, the potential impact due to undocumented fill should be low.

5.1.7 Soil Corrosion Potential

A preliminary soil corrosion screening was performed by JDH Corrosion Consultants based on the results of analytical tests on samples of the near-surface soil. In general, the JDH report concludes that the corrosion potential for buried concrete does not warrant the use of sulfate resistant concrete. However, the corrosion potential for buried metallic structures, such as metal pipes, is considered corrosive. JDH recommends that special requirements for corrosion control be made to protect metal pipes. A more detailed discussion of the site corrosion evaluation is presented in Appendix C. As the preliminary soil corrosion screening was based on the results of limited sampling, consideration may be given to collecting and testing additional samples from the upper 5 feet for sulfates and pH to confirm the classification of corrosive to mortar coated steel and concrete.

5.2 PLANS AND SPECIFICATIONS REVIEW

We recommend that we be retained to review the geotechnical aspects of the project structural, civil, and landscape plans and specifications, allowing sufficient time to provide the design team with any comments prior to issuing the plans for construction.

5.3 CONSTRUCTION OBSERVATION AND TESTING

As site conditions may vary significantly between the small-diameter borings performed during this investigation, we also recommend that a Cornerstone representative be present to provide geotechnical observation and testing during earthwork and foundation construction. This will allow us to form an opinion and prepare a letter at the end of construction regarding contractor compliance with project plans and specifications, and with the recommendations in our report. We will also be allowed to evaluate any conditions differing from those encountered during our investigation and provide supplemental recommendations as necessary. For these reasons, the recommendations in this report are contingent of Cornerstone providing observation and testing during construction. Contractors should provide at least a 48-hour notice when scheduling our field personnel.

SECTION 6: EARTHWORK

6.1 SITE DEMOLITION

All existing improvements not to be reused for the current development, including all foundations, flatwork, pavements, utilities, and other improvements should be demolished and removed from the site. Recommendations in this section apply to the removal of these improvements, which are currently present on the site, prior to the start of mass grading or the construction of new improvements for the project.

Cornerstone should be notified prior to the start of demolition and should be present on at least a part-time basis during all backfill and mass grading as a result of demolition. Occasionally, other types of buried structures (wells, cisterns, debris pits, etc.) can be found on sites with prior development. If encountered, Cornerstone should be contacted to address these types of structures on a case-by-case basis.

6.1.1 Demolition of Existing Slabs, Foundations and Pavements

All slabs, foundations, and pavements should be completely removed from within planned building areas.

As an owner value-engineered option, existing slabs, foundations, and pavements that extend into planned flatwork, pavement, or landscape areas may be left in place provided there is at least 3 feet of engineered fill overlying the remaining materials, they are shown not to conflict with new utilities, and that asphalt and concrete more than 10 feet square is broken up to allow subsurface drainage. Future distress and/or higher maintenance may result from leaving these

prior improvements in place. A discussion of recycling existing improvements is provided later in this report.

Special care should be taken during the demolition and removal of existing floor slabs, foundations, utilities and pavements to minimize disturbance of the subgrade. Excessive disturbance of the subgrade, which includes either native or previously placed engineered fill, resulting from demolition activities can have serious detrimental effects on planned foundation and paving elements.

Existing foundations are typically mat-slabs, shallow footings, or piers/piles. If slab or shallow footings are encountered, they should be completely removed. If drilled piers are encountered, they should be cut off at an elevation at least 48-inches below proposed footings or the final subgrade elevation, whichever is deeper. The remainder of the drilled pier could remain in place. Foundation elements to remain in place should be surveyed and superimposed on the proposed development plans to determine the potential for conflicts or detrimental impacts to the planned construction. Following review, additional mitigation or planned foundation elements may need to be modified.

6.1.2 Abandonment of Existing Utilities

All utilities should be completely removed from within planned building areas. For any utility line to be considered acceptable to remain within building areas, the utility line must be completely backfilled with grout or sand-cement slurry (sand slurry is not acceptable), the ends outside the building area capped with concrete, and the trench fills either removed and replaced as engineered fill with the trench side slopes flattened to at least 1:1, or the trench fills are determined not to be a risk to the structure. The assessment of the level of risk posed by the particular utility line will determine whether the utility may be abandoned in place or needs to be completely removed. The contractor should assume that all utilities will be removed from within building areas unless provided with written confirmation from both the owner and the geotechnical engineer.

Utilities extending beyond the building area may be abandoned in place provided the ends are plugged with concrete, they do not conflict with planned improvements, and that the trench fills do not pose significant risk to the planned surface improvements.

The risk for owners associated with abandoning utilities in place includes the potential for future differential settlement of existing trench fills, and/or partial collapse and potential ground loss into utility lines that are not completely filled with grout.

6.2 SITE CLEARING AND PREPARATION

6.2.1 Site Stripping

The site should be stripped of all surface vegetation, and surface and subsurface improvements to be removed within the proposed development area. Demolition of existing improvements is

discussed in the prior paragraphs. Surface vegetation and topsoil should be stripped to a sufficient depth to remove all material greater than 3 percent organic content by weight.

6.2.2 Tree and Shrub Removal

Trees and shrubs designated for removal should have the root balls and any roots greater than ½-inch diameter removed completely. Mature trees are estimated to have root balls extending to depths of 2 to 4 feet, depending on the tree size. Significant root zones are anticipated to extend to the diameter of the tree canopy. Grade depressions resulting from root ball removal should be cleaned of loose material and backfilled in accordance with the recommendations in the "Compaction" section of this report.

6.3 MITIGATION OF UNDOCUMENTED FILLS

As discussed above, Borings EB-1, EB-3, and EB-4 generally encountered undocumented fills to depths ranging from approximately 2 to 2½ feet below existing grades. While fills were not identified in all our explorations, undocumented fill may be present as a result of prior development grading. All undocumented fills should be completely removed from within building areas and to a lateral distance of at least 5 feet beyond the building footprint or to a lateral distance equal to fill depth below the perimeter footing, whichever is greater. Provided the fills meet the "Material for Fill" requirements below, the fills may be reused when backfilling the excavations. Based on review of the samples collected from our borings, it appears that the fill may be reused. If materials are encountered that do not meet the requirements, such as debris, wood, trash, those materials should be screened out of the remaining material and be removed from the site. Backfill of excavations should be placed in lifts and compacted in accordance with the "Compaction" section below.

Fills extending into planned pavement and flatwork areas may be left in place provided they are determined to be a low risk for future differential settlement and that the upper 12 to 18 inches of fill below pavement subgrade is re-worked and compacted as discussed in the "Compaction" section below.

6.4 TEMPORARY CUT AND FILL SLOPES

The contractor is responsible for maintaining all temporary slopes and providing temporary shoring where required. Temporary shoring, bracing, and cuts/fills should be performed in accordance with the strictest government safety standards. On a preliminary basis, the upper 10 feet at the site may be classified as OSHA Soil Type C materials.

Excavations performed during site demolition and fill removal should be sloped at 2:1 (horizontal:vertical) within the upper 5 feet below building subgrade. Actual excavation inclinations should be reviewed in the field during construction, as needed. Excavations below building subgrade and excavations in pavement and flatwork areas should be sloped in accordance with OSHA soil classification requirements.

6.5 SUBGRADE PREPARATION

After site clearing and demolition is complete, and prior to backfilling any excavations resulting from fill removal or demolition, the excavation subgrade and subgrade within areas to receive additional site fills, slabs-on-grade and/or pavements should be scarified to a depth of 6 inches, moisture conditioned, and compacted in accordance with the "Compaction" section below.

6.6 WET SOIL STABILIZATION GUIDELINES

Native soil and fill materials, especially soils with high fines contents such as clays and silty soils, can become unstable due to high moisture content, whether from high in-situ moisture contents or from winter rains. As the moisture content increases over the laboratory optimum, it becomes more likely the materials will be subject to softening and yielding (pumping) from construction loading or become unworkable during placement and compaction.

As discussed in the "Subsurface" section in this report, the in-situ moisture contents are about 6 to 15 percent over the estimated laboratory optimum in the upper 10 feet of the soil profile. The contractor should anticipate drying overlay saturated soils prior to reusing them as fill. In addition, repetitive rubber-tire loading will likely de-stabilize overly saturated soils.

There are several methods to address potential unstable soil conditions and facilitate fill placement and trench backfill. Some of the methods are briefly discussed below. Implementation of the appropriate stabilization measures should be evaluated on a case-by-case basis according to the project construction goals and the site conditions.

6.6.1 Scarification and Drying

The subgrade may be scarified to a depth of 6 to 12 inches and allowed to dry to near optimum conditions, if sufficient dry weather is anticipated to allow sufficient drying. More than one round of scarification may be needed to break up the soil clods.

6.6.2 Removal and Replacement

As an alternative to scarification, the contractor may choose to over-excavate the unstable soils and replace them with dry on-site or import materials. A Cornerstone representative should be present to provide recommendations regarding the appropriate depth of over-excavation, whether a geosynthetic (stabilization fabric or geogrid) is recommended, and what materials are recommended for backfill.

6.6.3 Chemical Treatment

Where the unstable area exceeds about 5,000 to 10,000 square feet and/or site winterization is desired, chemical treatment with quicklime (CaO), kiln-dust, or cement may be more cost-effective than removal and replacement. Recommended chemical treatment depths will typically range from 12 to 18 inches depending on the magnitude of the instability.

6.7 MATERIAL FOR FILL

6.7.1 Re-Use of On-site Soils

On-site soils with an organic content less than 3 percent by weight may be reused as general fill. General fill should not have lumps, clods, or cobble pieces larger than 6 inches in diameter; 85 percent of the fill should be smaller than 2½ inches in diameter. Minor amounts of oversize material (smaller than 12 inches in diameter) may be allowed provided the oversized pieces are not allowed to nest together and the compaction method will allow for loosely placed lifts not exceeding 12 inches.

6.7.2 Re-Use of On-Site Site Improvements

We anticipate that significant quantities of asphalt concrete (AC) grindings and aggregate base (AB) and Portland Cement Concrete (PCC) will be generated during site demolition. If the AC grindings are mixed with the underlying AB to meet Class 2 AB specifications, they may be reused within the new pavement and flatwork structural sections, including within below-grade parking garage slab-on-grade areas (provided crushed rock is not required due to the proximity to groundwater). AC grindings may not be reused within the habitable building areas. Laboratory testing will be required to confirm the grindings meet project specifications.

If the site area allows for on-site pulverization of PCC and provided the PCC is pulverized to meet the “Material for Fill” requirements of this report, it may be used as select fill within the building areas, excluding the capillary break layer; as typically pulverized PCC comes close to or meets Class 2 AB specifications, the recycled PCC may likely be used within the pavement structural sections. PCC grindings also make good winter construction access roads, similar to a cement-treated base (CTB) section.

6.7.3 Potential Import Sources

Non-expansive material should be inorganic with a Plasticity Index (PI) of 15 or less, and not contain recycled asphalt concrete where it will be used within the habitable building areas. Imported soil for use as general fill material should be inorganic with a Plasticity Index (PI) of 20 or less, and not contain recycled asphalt concrete where it will be used within the habitable building areas. To prevent significant caving during trenching or foundation construction, imported material should have sufficient fines. Samples of potential import sources should be delivered to our office at least 10 days prior to the desired import start date. Information regarding the import source should be provided, such as any site geotechnical reports. If the material will be derived from an excavation rather than a stockpile, potholes will likely be required to collect samples from throughout the depth of the planned cut that will be imported. At a minimum, laboratory testing will include PI tests. Material data sheets for select fill materials (Class 2 aggregate base, ¾-inch crushed rock, quarry fines, etc.) listing current laboratory testing data (not older than 6 months from the import date) may be provided for our review without providing a sample. If current data is not available, specification testing will need to be completed prior to approval.

Environmental and soil corrosion characterization should also be considered by the project team prior to acceptance. Suitable environmental laboratory data to the planned import quantity should be provided to the project environmental consultant; additional laboratory testing may be required based on the project environmental consultant's review. The potential import source should also not be more corrosive than the on-site soils, based on pH, saturated resistivity, and soluble sulfate and chloride testing.

6.7.4 Non-Expansive Fill Using Lime Treatment

As discussed above, non-expansive fill should have a Plasticity Index (PI) of 15 or less. Due to the high clay content and PI of the on-site soil materials, it is not likely that sufficient quantities of non-expansive fill would be generated from cut materials. As an alternative to importing non-expansive fill, chemical treatment can be considered to create non-expansive fill. It has been our experience that for high PI clayey soil and bedrock materials will likely need to be mixed with at least 3 to 4 percent quicklime (CaO) or approved equivalent to adequately reduce the PI of the on-site soils to 15 or less. If this option is considered, additional laboratory tests should be performed during initial site grading to further evaluate the optimum percentage of quicklime required.

6.8 COMPACTION REQUIREMENTS

All fills, and subgrade areas where fill, slabs-on-grade, and pavements are planned, should be placed in loose lifts 8 inches thick or less and compacted in accordance with ASTM D1557 (latest version) requirements as shown in the table below. In general, clayey soils should be compacted with sheepsfoot equipment and sandy/gravelly soils with vibratory equipment; open-graded materials such as crushed rock should be placed in lifts no thicker than 18 inches and consolidated in place with vibratory equipment. Each lift of fill and all subgrade should be firm and unyielding under construction equipment loading in addition to meeting the compaction requirements to be approved. The contractor (with input from a Cornerstone representative) should evaluate the in-situ moisture conditions, as the use of vibratory equipment on soils with high moistures can cause unstable conditions. General recommendations for soil stabilization are provided in the "Wet Soil Stabilization Guidelines" section of this report. Where the soil's PI is 20 or greater, the expansive soil criteria should be used.

Table 3: Compaction Requirements

Description	Material Description	Minimum Relative[1] Compaction (percent)	Moisture[2] Content (percent)
General Fill (within upper 5 feet)	On-Site Expansive Soils	87 – 92	>3
	Low Expansion Soils	90	>1
General Fill (below a depth of 5 feet)	On-Site Expansive Soils	92	>3
	Low Expansion Soils	95	>1
Retaining Wall Backfill	Without Surface Improvements	90	>1
	With Surface Improvements	95[4]	>1
Trench Backfill	On-Site Expansive Soils	87 – 92	>3
	Low Expansion Soils	90	>1
Trench Backfill (upper 6 inches of subgrade)	On-Site Low Expansion Soils	95	>1
Crushed Rock Fill	¾-inch Clean Crushed Rock	Consolidate In-Place	NA
Non-Expansive Fill	Imported Non-Expansive Fill	90	Optimum
Flatwork Subgrade	On-Site Expansive Soils	87 - 92	>3
	Low Expansion Soils	90	>1
Flatwork Aggregate Base	Class 2 Aggregate Base[3]	90	Optimum
Pavement Subgrade	On-Site Expansive Soils	87 - 92	>3
	Low Expansion Soils	95	>1
Pavement Aggregate Base	Class 2 Aggregate Base[3]	95	Optimum
Asphalt Concrete	Asphalt Concrete	95 (Marshall)	NA

1 – Relative compaction based on maximum density determined by ASTM D1557 (latest version)
2 – Moisture content based on optimum moisture content determined by ASTM D1557 (latest version)
3 – Class 2 aggregate base shall conform to Caltrans Standard Specifications, latest edition, except that the relative compaction should be determined by ASTM D1557 (latest version)
4 – Using light-weight compaction or walls should be braced

6.8.1 Construction Moisture Conditioning

Expansive soils can undergo significant volume change when dried then wetted. The contractor should keep all exposed expansive soil subgrade (and also trench excavation side walls) moist until protected by overlying improvements (or trenches are backfilled). If expansive soils are allowed to dry out significantly, re-moisture conditioning may require several days of re-wetting (flooding is not recommended), or deep scarification, moisture conditioning, and re-compaction.

6.9 TRENCH BACKFILL

Utility lines constructed within public right-of-way should be trenched, bedded and shaded, and backfilled in accordance with the local or governing jurisdictional requirements. Utility lines in

private improvement areas should be constructed in accordance with the following requirements unless superseded by other governing requirements.

All utility lines should be bedded and shaded to at least 6 inches over the top of the lines with crushed rock (⅜-inch-diameter or greater) or well-graded sand and gravel materials conforming to the pipe manufacturer's requirements. Open-graded shading materials should be consolidated in place with vibratory equipment and well-graded materials should be compacted to at least 90 percent relative compaction with vibratory equipment prior to placing subsequent backfill materials.

General backfill over shading materials may consist of on-site native materials provided they meet the requirements in the "Material for Fill" section, and are moisture conditioned and compacted in accordance with the requirements in the "Compaction" section.

Where utility lines will cross perpendicular to strip footings, the footing should be deepened to encase the utility line, providing sleeves or flexible cushions to protect the pipes from anticipated foundation settlement, or the utility lines should be backfilled to the bottom of footing with sand-cement slurry or lean concrete. Where utility lines will parallel footings and will extend below the "foundation plane of influence," an imaginary 1:1 plane projected down from the bottom edge of the footing, either the footing will need to be deepened so that the pipe is above the foundation plane of influence, or the utility trench will need to be backfilled with sand-cement slurry or lean concrete within the influence zone. Sand-cement slurry used within foundation influence zones should have a minimum compressive strength of 75 psi.

On expansive soils sites it is desirable to reduce the potential for water migration into building and pavement areas through the granular shading materials. We recommend that a plug of low-permeability clay soil, sand-cement slurry, or lean concrete be placed within trenches just outside where the trenches pass into building and pavement areas.

6.10 SITE DRAINAGE

6.10.1 Surface Drainage

Ponding should not be allowed adjacent to building foundations, slabs-on-grade, or pavements. Hardscape surfaces should slope at least 2 percent towards suitable discharge facilities; landscape areas should slope at least 3 percent towards suitable discharge facilities. Roof runoff should be directed away from building areas in closed conduits, to approved infiltration facilities, or on to hardscaped surfaces that drain to suitable facilities. Retention, detention, or infiltration facilities should be spaced at least 10 feet from buildings, and preferably at least 5 feet from slabs-on-grade or pavements. However, if retention, detention, or infiltration facilities are located within these zones, we recommend that these treatment facilities meet the requirements in the Storm Water Treatment Design Considerations section of this report.

6.11 LOW-IMPACT DEVELOPMENT (LID) IMPROVEMENTS

The Municipal Regional Permit (MRP) requires regulated projects to treat 100 percent of the amount of runoff identified in Provision C.3.d from a regulated project's drainage area with low impact development (LID) treatment measures onsite or at a joint stormwater treatment facility. LID treatment measures are defined as rainwater harvesting and use, infiltration, evapotranspiration, or biotreatment. A biotreatment system may only be used if it is infeasible to implement harvesting and use, infiltration, or evapotranspiration at a project site.

Technical infeasibility of infiltration may result from site conditions that restrict the operability of infiltration measures and devices. Various factors affecting the feasibility of infiltration treatment may create an environmental risk, structural stability risk, or physically restrict infiltration. The presence of any of these limiting factors may render infiltration technically infeasible for a proposed project. To aid in determining if infiltration may be feasible at the site, we provide the following site information regarding factors that may aid in determining the feasibility of infiltration facilities at the site.

- The near-surface soils at the site are generally clayey and may significantly limit the infiltration of stormwater. As discussed in the "In-Situ Infiltration Testing" section of this report, our infiltration tests resulted in infiltration rates of 0.02 to 0.04 inches per hour. Additionally recommendations are provided in the "In-Situ Infiltration Testing" section.

- Locally, seasonal high groundwater is mapped at a depth of 7 feet, and therefore is expected to be within 10 feet of the base of the infiltration measure.

- In our opinion, infiltration locations within 10 feet of the buildings would create a geotechnical hazard.

6.11.1 Storm Water Treatment Design Considerations

If storm water treatment improvements, such as shallow bio-retention swales, basins or pervious pavements, are required as part of the site improvements to satisfy Storm Water Quality (C.3) requirements, we recommend the following items be considered for design and construction.

6.11.1.1 General Bioswale Design Guidelines

- If possible, avoid placing bioswales or basins within 10 feet of the building perimeter or within 5 feet of exterior flatwork or pavements. If bioswales must be constructed within these setbacks, the side(s) and bottom of the trench excavation should be lined with 10-mil visqueen to reduce water infiltration into the surrounding expansive clay.

- Bioswales constructed within 3 feet of proposed buildings may be within the foundation zone of influence for perimeter wall loads. Therefore, where bioswales will parallel foundations and will extend below the "foundation plane of influence," an imaginary 1:1 plane projected down from the bottom edge of the foundation, the foundation will need to

be deepened so that the bottom edge of the bioswale filter material is above the foundation plane of influence.

- The bottom of bioswale or detention areas should include a perforated drain placed at a low point, such as a shallow trench or sloped bottom, to reduce water infiltration into the surrounding soils near structural improvements, and to address the low infiltration capacity of the on-site clay soils.

6.11.1.2 Bioswale Infiltration Material

- Gradation specifications for bioswale filter material, if required, should be specified on the grading and improvement plans.

- Compaction requirements for bioswale filter material in non-landscaped areas or in pervious pavement areas, if any, should be indicated on the plans and specifications to satisfy the anticipated use of the infiltration area.

- If bioswales are to be vegetated, the landscape architect should select planting materials that do not reduce or inhibit the water infiltration rate, such as covering the bioswale with grass sod containing a clayey soil base.

- Due to the relatively loose consistency and/or high organic content of many bioswale filter materials, long-term settlement of the bioswale medium should be anticipated. To reduce initial volume loss, bioswale filter material should be wetted in 12-inch lifts during placement to pre-consolidate the material. Mechanical compaction should not be allowed, unless specified on the grading and improvement plans, since this could significantly decrease the infiltration rate of the bioswale materials.

- It should be noted that the volume of bioswale filter material may decrease over time depending on the organic content of the material. Additional filter material may need to be added to bioswales after the initial exposure to winter rains and periodically over the life of the bioswale areas, as needed.

6.11.1.3 Bioswale Construction Adjacent to Pavements

If bio-infiltration swales or basins are considered adjacent to proposed parking lots or exterior flatwork, we recommend that mitigative measures be considered in the design and construction of these facilities to reduce potential impacts to flatwork or pavements. Exterior flatwork, concrete curbs, and pavements located directly adjacent to bio-swales may be susceptible to settlement or lateral movement, depending on the configuration of the bioswale and the setback between the improvements and edge of the swale. To reduce the potential for distress to these improvements due to vertical or lateral movement, the following options should be considered by the project civil engineer:

- Improvements should be setback from the vertical edge of a bioswale such that there is at least 1 foot of horizontal distance between the edge of improvements and the top edge of the bioswale excavation for every 1 foot of vertical bioswale depth, or

- Concrete curbs for pavements, or lateral restraint for exterior flatwork, located directly adjacent to a vertical bioswale cut should be designed to resist lateral earth pressures in accordance with the recommendations in the "Retaining Walls" section of this report, or concrete curbs or edge restraint should be adequately keyed into the native soil or engineered to reduce the potential for rotation or lateral movement of the curbs.

6.12 LANDSCAPE CONSIDERATIONS

Since the near-surface soils are moderately to highly expansive, we recommend greatly reducing the amount of surface water infiltrating these soils near foundations and exterior slabs-on-grade. This can typically be achieved by:

- Using drip irrigation

- Avoiding open planting within 3 feet of the building perimeter or near the top of existing slopes

- Regulating the amount of water distributed to lawns or planter areas by using irrigation timers

- Selecting landscaping that requires little or no watering, especially near foundations.

We recommend that the landscape architect consider these items when developing landscaping plans.

SECTION 7: 2022 CBC SEISMIC DESIGN CRITERIA

7.1 SEISMIC DESIGN CRITERIA

We developed site-specific seismic design parameters in accordance with Chapter 16, Chapter 18 and Appendix J of the 2022 California Building Code (CBC) and Chapters 11, 12, 20, and 21 and Supplement No. 1 of ASCE 7-16.

7.1.1 Site Location and Provided Data For 2022 CBC Seismic Design

The project is located at latitude 37.398655° and longitude -121.927685°, which is based on Google Earth (WGS84) coordinates at the approximate center of the site in San Jose, California. We have assumed that a Seismic Importance Factor (I_e) of 1.00 has been assigned to the structure in accordance with Table 1.5-2 of ASCE 7-16 for structures classified as Risk Category II. The building period has not been provided by the project structural engineer.

7.2 2022 CBC SEISMIC DESIGN CRITERIA

As discussed in the "Subsurface" of our report, our CPT and exploratory borings generally encountered medium dense to very dense sands and medium stiff to hard clay deposits to a depth of 110 feet, the maximum depth explored. Shear wave velocity (V_S) measurements were performed while advancing CPT-7 and CPT-11, resulting in a time-averaged shear wave velocity for the top 30 meters (V_{S30}) of 240 meters per second (788 feet per second) and 246 meters per second (806 feet per second), respectively, for the upper 100 feet.

7.2.1 2022 CBC Seismic Design

As our borings encountered deep alluvial soils with shear wave velocities for the upper 30 meters between 600 and 1200 feet per second, per section 20.3.2 of ASCE 7-16, we have classified the site as Soil Classification D, which is described as a "stiff soil" profile. Because we used site specific data from our explorations and laboratory testing, the site class should be considered as "determined" for the purposes of estimating the seismic design parameters from the code. Our site-specific ground motion hazard analysis considered a V_{S30} of 240 m/s (788 ft/s).

In accordance with Section 11.4.8 of ASCE 7-16, we performed a ground motion hazard analysis following Chapter 21, Section 21.2 of ASCE 7-16. We evaluated both Probabilistic MCE_R Ground Motions in accordance with Method 1 and Deterministic MCE_R Ground Motions to generate our recommended design response spectrum for the project, see Figure 5. The recommended design spectral accelerations and associated periods are provided in graphically on Figure 6.

SECTION 8: FOUNDATIONS

8.1 SUMMARY OF RECOMMENDATIONS

In our opinion, the proposed structures may be supported on shallow foundations provided the recommendations in the "Earthwork" section and the sections below are followed.

8.2 SHALLOW FOUNDATIONS

8.2.1 Conventional Shallow Footings

Provided the structure can tolerate the estimated static and seismic total and differential settlements, conventional shallow footings can be considered for building support. Conventional shallow footings should bear on natural, undisturbed soil or engineered fill, be at least 15 inches wide, and extend at least 24 inches below the lowest adjacent grade. Lowest adjacent grade is defined as the deeper of the following: 1) bottom of the adjacent interior slab-on-grade, or 2) finished exterior grade, excluding landscaping topsoil. The deeper footing embedment is due to the presence of moderately to highly expansive soils and is intended to embed the footing below the zone of significant seasonal moisture fluctuation, reducing the potential for differential movement.

Footings constructed to the above dimensions and in accordance with the "Earthwork" recommendations of this report are capable of supporting maximum allowable bearing pressures of 2,000 psf for dead loads, 3,000 psf for combined dead plus live loads, and 4,000 psf for all loads including wind and seismic. These pressures are based on factors of safety of 3.0, 2.0, and 1.5 applied to the ultimate bearing pressure for dead, dead plus live, and all loads, respectively. These pressures are net values; the weight of the footing may be neglected for the portion of the footing extending below grade (typically, the full footing depth). Top and bottom mats of reinforcing steel should be included in continuous footings to help span irregularities and differential settlement.

8.2.2 Footing Settlement

Structural loads were not provided to us at the time this report was prepared; therefore, we assumed the typical loading in the following table.

Table 4: Assumed Structural Loading

Foundation Area	Range of Assumed Loads
Interior Isolated Column Footing	50 to 100 kips
Perimeter Strip Footing	7 to 9 kips per lineal foot

Based on the above estimated loading, the allowable bearing pressures presented above, and the assumed bottom of footings (interior column footings and perimeter wall footings at about 2 feet below finished floor, and deepened wall footings at loading docks at about 6 to 7 feet below finished floor), we estimate that the total static footing settlement will be on the order of ½-inch for interior column footings and perimeter wall footings, and ¾- to 1-inch for deepened wall footings at the planned loading docks. Post-construction differential static settlement is estimated to be on the order of about ½ inch or less between independent foundation elements, or over an estimated horizontal distance of 50 to 60 feet.

In addition to the estimated static settlements, we estimate differential seismic movement between independent foundation elements will be on the order of about ¾-inch or less. As a result, we estimate total combined static and seismic differential footing settlement will be on the order of less than about 1¼ inches between independent foundation elements, or over a horizontal distance of about 50 to 60 feet. As our footing loads were estimated and grading information is preliminary, we recommend we be retained to review the final footing layout and loading and verify the settlement estimates above.

8.2.3 Lateral Loading

Lateral loads may be resisted by friction between the bottom of footing and the supporting subgrade, and also by passive pressures generated against footing sidewalls. An ultimate frictional resistance of 0.35 applied to the footing dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 450 pcf may be used in design. The structural

engineer should apply an appropriate factor of safety (such as 1.5) to the ultimate values above. Where footings are adjacent to landscape areas without hardscape, the upper 12 inches of soil should be neglected when determining passive pressure capacity.

8.2.4 Conventional Shallow Footing Construction Considerations

Where utility lines will cross perpendicular to strip footings, the footing should be deepened to encase the utility line, providing sleeves or flexible cushions to protect the pipes from anticipated foundation settlement, or the utility lines should be backfilled to the bottom of footing with sand-cement slurry or lean concrete. Where utility lines will parallel footings and will extend below the "foundation plane of influence," an imaginary 1:1 plane projected down from the bottom edge of the footing, either the footing will need to be deepened so that the pipe is above the foundation plane of influence or the utility trench will need to be backfilled with sand-cement slurry or lean concrete within the influence zone. Sand-cement slurry used within foundation influence zones should have a minimum compressive strength of 75 psi.

Footing excavations should be filled as soon as possible or be kept moist until concrete placement by regular sprinkling to prevent desiccation. A Cornerstone representative should observe all footing excavations prior to placing reinforcing steel and concrete. If there is a significant schedule delay between our initial observation and concrete placement, we may need to re-observe the excavations.

SECTION 9: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

9.1 INTERIOR OFFICE SLABS-ON-GRADE

Due to the moderate to highly expansion potential of the surficial soils, the proposed office slabs-on-grade should be at least 5 inches thick and be supported on at least 18 inches of non-expansive fill (NEF) to reduce the potential for slab damage due to soil heave. The NEF layer should be constructed over subgrade prepared in accordance with the recommendations in the "Earthwork" section of this report. If moisture-sensitive floor coverings are planned, the recommendations in the "Interior Slabs Moisture Protection Considerations" section below may be incorporated in the project design if desired. If significant time elapses between initial subgrade preparation and NEF construction, the subgrade should be proof-rolled to confirm subgrade stability, and if the soil has been allowed to dry out, the subgrade should be re-moisture conditioned to at least 3 percent over the optimum moisture content.

The structural engineer should determine the appropriate slab reinforcement for the loading requirements and consider the expansion potential of the underlying soils. For unreinforced concrete slabs, ACI 302.1R recommends limiting control joint spacing to 24 to 36 times the slab thickness in each direction, or a maximum of 18 feet.

9.2 WAREHOUSE SLABS-ON-GRADE

Warehouse slabs-on-grade should be at least 6 inches thick, should have a minimum compressive strength of 3,500 psi, and should be designed for the specific warehouse loading

(i.e. forklifts, rack loads, etc.). At this time, rack loading information, etc. was not available. The slab should also be designed to accommodate potential slab settlement beneath heavily loaded slab areas.

Warehouse slabs-on-grade should also be supported on at least 6 inches of non-expansive, crushed granular base having an R-value of at least 50 and no more than 10 percent passing the No. 200 sieve, such as Class 2 aggregate base. Due to the moderate to high plasticity of the surficial soils, an additional 12 inches of non-expansive fill (NEF) should underlie the upper granular base NEF, for a total slab NEF section thickness of 18 inches.

To summarize, the warehouse slab-on-grade should be underlain by the materials and alternatives outlined below. Additional details follow these points.

- Zone A Fill Material – Upper 6 inches of pad subgrade should consist of crushed granular base having an R-value of at least 50 and no more than 10 percent passing the No. 200 sieve, such as Class 2 aggregate base
- Zone B Fill Material – from 6 to 18 inches below pad subgrade (or below the Zone A noted above), the soil/material should also consist of a non-expansive fill (NEF) with a Plasticity Index (PI) of 15, or less
- Zone B Fill Material Alternative 1 – from 6 to 18 inches below pad subgrade (or below the Zone A noted above), the non-expansive fill material also can consist of the same material used for Zone A
- Zone B Fill Material Alternative 2 – from 6 to 18 inches below pad subgrade (or below the Zone A noted above), the onsite material also can be chemical-treated to meet non-expansive requirement with a PI of 15, or less

All base and subbase materials should be placed and compacted in accordance with the "Compaction" section of this report. If there will be areas within the warehouse that are moisture sensitive, such as equipment and elevator rooms, a vapor barrier may be placed over the upper granular base prior to slab construction. Please refer to the recommendations in the "Interior Slabs Moisture Protection Considerations" section for vapor barrier construction. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness.

9.3 INTERIOR SLABS MOISTURE PROTECTION CONSIDERATIONS

The following general guidelines for concrete slab-on-grade construction where floor coverings are planned are presented for consideration by the developer, design team, and contractor. These guidelines are based on information obtained from a variety of sources, including the American Concrete Institute (ACI) and are intended to reduce the potential for moisture-related problems causing floor covering failures, and may be supplemented as necessary based on

project-specific requirements. The application of these guidelines or not will not affect the geotechnical aspects of the slab-on-grade performance.

- Place a minimum 15-mil vapor retarder conforming to ASTM E 1745, Class C requirements or better directly below the concrete slab; the vapor retarder should extend to the slab edges and be sealed at all seams and penetrations in accordance with manufacturer's recommendations and ASTM E 1643 requirements. A 4-inch-thick capillary break, consisting of crushed rock should be placed below the vapor retarder and consolidated in place with vibratory equipment. The mineral aggregate shall be of such size that the percentage composition by dry weight as determined by laboratory sieves will conform to the following gradation:

Sieve Size	Percentage Passing Sieve
1"	100
¾"	90 – 100
No. 4	0 – 10
No. 200	0 – 5

- The capillary break rock may be considered as the upper 4 inches of the non-expansive fill previously recommended.

- The concrete water:cement ratio should be 0.45 or less. Mid-range plasticizers may be used to increase concrete workability and facilitate pumping and placement.

- Water should not be added after initial batching unless the slump is less than specified and/or the resulting water:cement ratio will not exceed 0.45.

- Polishing the concrete surface with metal trowels is not recommended.

- Where floor coverings are planned, all concrete surfaces should be properly cured.

- Water vapor emission levels and concrete pH should be determined in accordance with ASTM F1869-98 and F710-98 requirements and evaluated against the floor covering manufacturer's requirements prior to installation.

9.4 EXTERIOR FLATWORK

Exterior flatwork, such as pedestrian walkways, patios, driveways, and sidewalks, may experience seasonal movement due to the native expansive soils; therefore, some cracking or vertical movement of conventional slabs should be anticipated where imported fill is not planned in flatwork areas. There are several alternatives for mitigating the impacts of expansive soils beneath concrete flatwork. We are providing recommendations to reduce distress to concrete flatwork that includes moisture conditioning the subgrade soils, using non-expansive fill, and providing adequate construction and control joints to control cracks that do occur. It should be noted that minor slab movement or localized cracking and/or distress could still occur.

- The minimum recommendation for concrete flatwork constructed on moderately to highly expansive soils is to properly prepare the clayey soils prior to placing concrete. This is typically achieved by scarifying, moisture conditioning, and re-compacting the subgrade soil. Subgrade soil should be moisture conditioned to at least 3 percent over the laboratory optimum and compacted using moderate compaction effort to a relative compaction of 87 to 92 percent (ASTM Test Method D1557). Since the near surface soils may have been previously compacted and tested, the subgrade soils could possibly be moisture conditioned by gradually wetting the soil, depending on the time of year slab construction occurs. This should not include flooding or excessively watering the soil, which would likely result in a soft, unstable subgrade condition, and possible delays in the construction while waiting for the soil to dry out. In general, the subgrade should be relatively firm and non-yielding prior to construction.

- Concrete flatwork, excluding pavements that would be subject to wheel loads, should be at least 4 inches thick and underlain by at least 9 inches of non-expansive fill. Non-expansive fill may include aggregate base, crushed rock, or imported soil with a PI of 15 or less. Non-expansive fill should be compacted to at least 90 percent relative compaction. Flatwork that will be subject to heavier or frequent vehicular loading should be designed in accordance with the recommendations in the "Vehicular Pavements" section below.

- We recommend a maximum control joint spacing of about 2 feet in each direction for each inch of concrete thickness and a construction joint spacing of 10 to 12 feet. Construction joints that abut the foundations or garage slabs should include a felt strip, or approved equivalent, that extends the full depth of the exterior slab. This will help to reduce the potential for permanent vertical offset between the slabs due to friction between the concrete edges. We recommend that exterior slabs be isolated from adjacent foundations.

At the owner's option, if desired to reduce the potential for vertical offset or widening of concrete cracks, consideration should be given to using reinforcing steel, such as No. 3 rebar spaced at 18 inches on center each direction.

SECTION 10: VEHICULAR PAVEMENTS

10.1 ASPHALT CONCRETE

The following asphalt concrete pavement recommendations tabulated below are based on Procedure 608 of the Caltrans Highway Design Manual, estimated traffic indices for various pavement-loading conditions, and on a design R-value of 5. The design R-value was chosen based on engineering judgement considering the variable and expansive soil conditions. Additionally, due to the presence of moderate to highly expansive soils, we have also included an option for lime-treated subgrade soils using an estimated design R-value of 50.

Table 5: Asphalt Concrete Pavement Recommendations, Untreated Subgrade

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base[1] (inches)	Total Pavement Section Thickness (inches)
4.0	2.5	7.5	10.0
4.5	2.5	9.5	12.0
5.0	3.0	10.0	13.0
5.5	3.0	12.0	15.0
6.0	3.5	13.0	16.5
6.5	4.0	13.5	17.5
7.0	4.0	15.5	19.5
7.5	4.5	16.5	21.0
8.0	5.0	17.5	22.5
8.5	5.0	19.5	24.5
9.0	5.5	20.5	26.0
9.5	6.0	21.5	27.5
10.0	6.5	23.0	29.5
10.5	6.5	24.5	31.0
11.0	7.0	25.5	32.5

1 – Caltrans Class 2 aggregate base; minimum R-value of 78; subgrade R-value of 5

Table 6: Asphalt Concrete Pavement Recommendations, Chemical-Treated Subgrade

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base[1] (inches)	Total Pavement Section Thickness (inches)
4.0/4.5	2.5	4.0	6.5
5.0/5.5	3.0	4.0	7.0
6.0	3.5	4.0	7.5
6.5	4.0	4.0	8.0
7.0	4.0	4.5	8.5
7.5	4.5	5.0	9.5
8.0	5.0	5.0	10.0
8.5	5.0	6.0	11.0
9.0	5.5	6.5	12.0
9.5	6.0	7.0	13.0
10.0	6.5	7.0	13.5
10.5	6.5	8.0	14.5
11.0	7.0	8.0	15.0

1 – Caltrans Class 2 aggregate base; minimum R-value of 78; minimum chemical-treated subgrade R-value of 50

Frequently, the full asphalt concrete section is not constructed prior to construction traffic loading. This can result in significant loss of asphalt concrete layer life, rutting, or other pavement failures. To improve the pavement life and reduce the potential for pavement distress through construction, we recommend the full design asphalt concrete section be constructed prior to construction traffic loading. Alternatively, a higher traffic index may be chosen for the areas where construction traffic will use the pavements.

Asphalt concrete pavements constructed on expansive subgrade where the adjacent areas will not be irrigated for several months after the pavements are constructed may experience longitudinal cracking parallel to the pavement edge. These cracks typically form within a few feet of the pavement edge and are due to seasonal wetting and drying of the adjacent soil. The cracking may also occur during construction where the adjacent grade is allowed to significantly dry during the summer, pulling moisture out of the pavement subgrade. Any cracks that form should be sealed with bituminous sealant prior to the start of winter rains. One alternative to reduce the potential for this type of cracking is to install a moisture barrier at least 24 inches deep behind the pavement curb.

10.2 PORTLAND CEMENT CONCRETE

The exterior Portland Cement Concrete (PCC) pavement recommendations tabulated below are based on methods presented in “Concrete Pavement for Trucking Facilities” by the American Concrete Pavement Association (ACPA, 1995), and the Portland Cement Association (PCA)

design manual (PCA, 1984). In our opinion, the truck loading areas where trucks turn, brake, or stop should be constructed of reinforced Portland Cement Concrete (PCC) pavement. We have provided a few pavement alternatives below as the anticipated number of trucks and number of load repetitions per day on a given location of the pavement has not been provided at this time. An alternative should be chosen that is greater than what is expected for the development. At this time, we have assumed trucks will consist of tractor trailers. When more specific truck loading information is available, additional information can be provided.

Table 7: PCC Pavement Recommendations, Untreated Subgrade

Traffic Category	Minimum PCC Thickness[1] (inches)	Class 2 Aggregate Base (inches)
Trucks per day per lane = 20	7.0	6.0
Trucks per day per lane = 80	7.5	6.0
Trucks per day per lane = 400	8.0	6.0

1 – Subgrade design R-Value = 5

The PCC thicknesses recommended above are based on a concrete flexural strength of at least 550 psi, minimum 28-day compressive strength of 4,000 psi, and supporting the PCC on at least 6 inches of Class 2 aggregate base compacted to a minimum relative compaction of 95 percent (ASTM D1557) as recommended in the “Earthwork” section of our report, and laterally restraining the PCC with curbs or concrete shoulders. The aggregate base should be constructed over compacted subgrade prepared in accordance with “Compaction” section of this report. The PCC should have a low water-cement ratio (industry standards), and in no case exceed a water-cement plus pozzolan ratio of 0.53.

Adequate expansion and control joints should be included. Joint spacing should not exceed about 24 times the pavement thickness, in approximately square panels for unreinforced concrete. Joint spacing can be increased somewhat for reinforced concrete where light reinforcing steel is provided to help hold together intermediate cracks that may form in panels. Construction and expansion joints, and dowels, should be designed in accordance with industry standards; joint sealing should be included due to the expansive soils at the site.

If the subgrade soils are to be chemically treated (lime/cement), we have provided some pavement alternatives below. For chemically treated subgrade, we recommend the upper 12 inches of subgrade soil be treated and the treated subgrade should obtain a minimum R-value of 50. Additional testing will need to be performed to determine the appropriate lime/cement percentage to be mixed with the subgrade soil to achieve an estimated R-value of 50. The remaining design and construction details should be consistent with the above recommendations for PCC pavements over untreated subgrade.

Table 8: PCC Pavement Recommendations, Chemical-Treated Subgrade

Traffic Category	Minimum PCC Thickness[1] (inches)
Trucks per day per lane = 20	6.5
Trucks per day per lane = 80	7.0
Trucks per day per lane = 400	7.5

1 – Subgrade design R-Value = 50, assuming 12-inch-thick subgrade chemical treatment

If chemical treatment of the subgrade is performed, the underlying Class 2 aggregate base section may be reduced from 6 inches to 4 inches, provided it is compacted to a minimum relative compaction of 95 percent (ASTM D1557) as recommended in the "Earthwork" section of our report.

10.3 TRASH ENCLOSURES

Pads where trash containers will be stored, and where garbage trucks will park while emptying trash containers, should be constructed on Portland Cement Concrete. We recommend that the trash enclosure pads and stress (landing) pads where garbage trucks will store, pick up, and empty trash be increased to a minimum PCC thickness of 8 inches. The compressive strength, underlayment, and construction details should be consistent with the above recommendations for PCC pavements.

10.4 PAVEMENT CUTOFF

Surface water penetration into the pavement section can significantly reduce the pavement life, due to the native expansive clays. While quantifying the life reduction is difficult, a normal 20-year pavement design could be reduced to less than 10 years; therefore, increased long-term maintenance may be required.

It would be beneficial to include a pavement cut-off, such as deepened curbs, redwood-headers, or "Deep-Root Moisture Barriers" that are keyed at least 4 inches into the pavement subgrade. This will help limit the additional long-term maintenance.

SECTION 11: RETAINING WALLS

11.1 STATIC LATERAL EARTH PRESSURES

The structural design of any site retaining wall should include resistance to lateral earth pressures that develop from the soil behind the wall, any undrained water pressure, and surcharge loads acting behind the wall. Provided a drainage system is constructed behind the

wall to prevent the build-up of hydrostatic pressures as discussed in the section below, we recommend that the walls with level backfill be designed for the following pressures:

Table 9: Recommended Lateral Earth Pressures

Wall Condition	Lateral Earth Pressure*	Additional Surcharge Loads
Unrestrained – Cantilever Wall	45 pcf	⅓ of vertical loads at top of wall
Restrained – Braced Wall	45 pcf + 8H** psf	½ of vertical loads at top of wall

* Lateral earth pressures are based on an equivalent fluid pressure for level backfill conditions
** H is the distance in feet between the bottom of footing and top of retained soil

If adequate drainage cannot be provided behind the wall, an additional equivalent fluid pressure of 40 pcf should be added to the values above for both restrained and unrestrained walls for the portion of the wall that will not have drainage. Damp proofing or waterproofing of the walls may be considered where moisture penetration and/or efflorescence are not desired.

11.2 SEISMIC LATERAL EARTH PRESSURES

The 2022 CBC states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. At this time, we are not aware of any retaining walls 6 feet or greater in height and have not provided seismic earth pressures for the project. If retaining walls greater than 6 feet in height are proposed, we should be retained to provide seismic earth pressures, if warranted. In our opinion, seismic earth pressures are not warranted for design of minor retaining walls (i.e. walls 6 feet or less in height).

Adequate drainage should be provided by a subdrain system behind all walls. This system should consist of a 4-inch minimum diameter perforated pipe placed near the base of the wall (perforations placed downward). The pipe should be bedded and backfilled with Class 2 Permeable Material per Caltrans Standard Specifications, latest edition. The permeable backfill should extend at least 12 inches out from the wall and to within 2 feet of outside finished grade. Alternatively, ½-inch to ¾-inch crushed rock may be used in place of the Class 2 Permeable Material provided the crushed rock and pipe are enclosed in filter fabric, such as Mirafi 140N or approved equivalent. The upper 2 feet of wall backfill should consist of compacted on-site soil. The subdrain outlet should be connected to a free-draining outlet or sump.

Miradrain, Geotech Drainage Panels, or equivalent drainage matting can be used for wall drainage as an alternative to the Class 2 Permeable Material or drain rock backfill. Horizontal strip drains connecting to the vertical drainage matting may be used in lieu of the perforated pipe and crushed rock section. The vertical drainage panel should be connected to the perforated pipe or horizontal drainage strip at the base of the wall, or to some other closed or through-wall system such as the TotalDrain system from AmerDrain. Sections of horizontal drainage strips should be connected with either the manufacturer's connector pieces or by pulling back the filter fabric, overlapping the panel dimples, and replacing the filter fabric over the connection. At corners, a corner guard, corner connection insert, or a section of crushed rock covered with filter fabric must be used to maintain the drainage path.

Drainage panels should terminate 18 to 24 inches from final exterior grade. The Miradrain panel filter fabric should be extended over the top of and behind the panel to protect it from intrusion of the adjacent soil.

11.3 BACKFILL

Where surface improvements will be located over the retaining wall backfill, backfill placed behind the walls with a PI less than 20 should be compacted to at least 95 percent relative compaction using light compaction equipment. If the soil's PI is 20 or greater, expansive soil criteria should be used as discussed in the "Compaction" section of this report. Where no surface improvements are planned, backfill should be compacted to at least 90 percent for soils with a PI less than 20. Expansive soil criteria should be followed for soils with a PI of 20 or greater. If heavy compaction equipment is used, the walls should be temporarily braced.

11.4 FOUNDATIONS

Retaining walls may be supported on a continuous and or spread footing designed in accordance with the recommendations presented in the "Foundations" section of this report.

SECTION 12: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of McCarthy Development specifically to support the design of the Montague Pointe Industrial project in San Jose, California. The opinions, conclusions, and recommendations presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in Northern California at the time this report was prepared. No warranty, expressed or implied, is made or should be inferred.

Recommendations in this report are based upon the soil and groundwater conditions encountered during our subsurface exploration. If variations or unsuitable conditions are encountered during construction, Cornerstone must be contacted to provide supplemental recommendations, as needed.

McCarthy Development may have provided Cornerstone with plans, reports and other documents prepared by others. McCarthy Development understands that Cornerstone reviewed and relied on the information presented in these documents and cannot be responsible for their accuracy.

Cornerstone prepared this report with the understanding that it is the responsibility of the owner or their representatives to see that the recommendations contained in this report are presented to other members of the design team and incorporated into the project plans and specifications, and that appropriate actions are taken to implement the geotechnical recommendations during construction.

Conclusions and recommendations presented in this report are valid as of the present time for the development as currently planned. Changes in the condition 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. Therefore, the conclusions and recommendations presented in this report may be invalidated, wholly or in part, by changes beyond Cornerstone's control. This report should be reviewed by Cornerstone after a period of three (3) years has elapsed from the date of this report. In addition, if the current project design is changed, then Cornerstone must review the proposed changes and provide supplemental recommendations, as needed.

An electronic transmission of this report may also have been issued. While Cornerstone has taken precautions to produce a complete and secure electronic transmission, please check the electronic transmission against the hard copy version for conformity.

Recommendations provided in this report are based on the assumption that Cornerstone will be retained to provide observation and testing services during construction to confirm that conditions are similar to that assumed for design, and to form an opinion as to whether the work has been performed in accordance with the project plans and specifications. If we are not retained for these services, Cornerstone cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of Cornerstone's report by others. Furthermore, Cornerstone will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services.

SECTION 13: REFERENCES

Aagaard, B.T., Blair, J.L., Boatwright, J., Garcia, S.H., Harris, R.A., Michael, A.J., Schwartz, D.P., and DiLeo, J.S., 2016, Earthquake outlook for the San Francisco Bay region 2014–2043 (ver. 1.1, August 2016): U.S. Geological Survey Fact Sheet 2016–3020, 6 p., http://dx.doi.org/10.3133/fs20163020.

American Concrete Pavement Association (ACPA, 2006). *Design of Concrete Pavement for Streets and Roads.*

ASCE 7-16. American Society of Civil Engineers. (2016). *Minimum Design Loads and Associated Criteria for Buildings and Other Structures.*

Boulanger, R.W. and Idriss, I.M., 2004, Evaluating the Potential for Liquefaction or Cyclic Failure of Silts and Clays, Department of Civil & Environmental Engineering, College of Engineering, University of California at Davis.

Boulanger, R.W. and Idriss, I.M., 2014, CPT and SPT Based Liquefaction Triggering Procedures, Department of Civil & Environmental Engineering, College of Engineering, University of California at Davis, Report No. UCD/GCM-14/01, April 2014

California Building Code, 2022, Structural Engineering Design Provisions, Vol. 2.

California Department of Conservation Division of Mines and Geology, 1998, Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada, International Conference of Building Officials, February, 1998.

California Division of Mines and Geology (2008), “Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117A, September.

California Geological Survey, 2001, State of California Seismic Hazard Zones, Milpitas 7.5-Minute Quadrangle, California: Seismic Hazard Zone Report 051.

CGS. (2021). *Earthquake Zones of Required Investigation.* CGS Homepage. https://maps.conservation.ca.gov/cgs/EQZApp/app/

Cetin, K.O., Bilge, H.T., Wu, J., Kammerer, A.M., and Seed, R.B., Probablilistic Model for the Assessment of Cyclically Induced Reconsolidation (Volumetric) Settlements, ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vo. 135, No. 3, March 1, 2009.

Federal Emergency Management Administration (FEMA), 2009, FIRM City of San Jose, California, Community Panel #06085C0068J.

GitHub, *usgs/shakemap-scenarios*, 2020, https://github.com/usgs/shakemap-scenarios/tree/master/rupture_sets/BSSC2014

Idriss, I.M., and Boulanger, R.W., 2008, Soil Liquefaction During Earthquakes, Earthquake Engineering Research Institute, Oakland, CA, 237 p.

Ishihara, K., 1985, Stability of Natural Deposits During Earthquakes: Proceedings Eleventh International Conference on Soil Mechanics and Foundation Engineering, San Francisco.

Ishihara, K. and Yoshimine, M., 1992, Evaluation of Settlements in Sand Deposits Following Liquefaction During Earthquakes, Soils and Foundations, 32 (1): 173-188.

OSHPD, Seismic Design Maps, https://www.seismicmaps.org

Ritter, J.R., and Dupre, W.R., 1972, Map Showing Areas of Potential Inundation by Tsunamis in the San Francisco Bay Region, California: San Francisco Bay Region Environment and Resources Planning Study, USGS Basic Data Contribution 52, Misc. Field Studies Map MF-480.

Robertson, P.K., Shao, Lisheng, 2010, Estimation of Seismic Compression in Dry Soils Using the CPT, 5th International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, Paper No. 4.05a, May 24-29, 2010.

Rogers, T.H., and J.W. Williams, 1974 Potential Seismic Hazards in Santa Clara County, California, Special Report No. 107: California Division of Mines and Geology.

Seed, H.B. and I.M. Idriss, 1971, A Simplified Procedure for Evaluation soil Liquefaction Potential: JSMFC, ASCE, Vol. 97, No. SM 9, pp. 1249 – 1274.

Seed, Raymond B., Cetin, K.O., Moss, R.E.S., Kammerer, Ann Marie, Wu, J., Pestana, J.M., Riemer, M.F., Sancio, R.B., Bray, Jonathan D., Kayen, Robert E., and Faris, A., 2003, Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework., University of California, Earthquake Engineering Research Center Report 2003-06.

State of California Department of Transportation, Highway Design Manual, Latest Edition.

U.S. Geological Survey, *Unified Hazard Tool*, 2020, https://earthquake.usgs.gov/hazards/interactive/

U.S. Geological Survey, *Building Seismic Safety Council 2014 Event Set*, ArcGIS Web Application, 2020, https://usgs.maps.arcgis.com/apps/webappviewer/index.html?id=14d2f75c7c4f4619936dac0d14e1e468

U.S. Geological Survey, *USGS Earthquake Hazards Program*, 2020, https://earthquake.usgs.gov/nshmp-haz-ws/apps/spectra-plot.html

Working Group on California Earthquake Probabilities, 2015, The Third Uniform California Earthquake Rupture Forecast, Version 3 (UCERF), U.S. Geological Survey Open File Report 2013-1165 (CGS Special Report 228). *KMZ files available at:* www.scec.org/ucerf/images/ucerf3_timedep_30yr_probs.kmz

Yoshimine, M., Nishizaki, H., Amano, KI, and Hosono, Y., 2006, Flow Deformation of Liquefied Sand Under Constant Shear Load and Its Application to Analysis of Flow Slide in Infinite Slope, Soil Dynamics and Earthquake Eng. 26, 253-264.

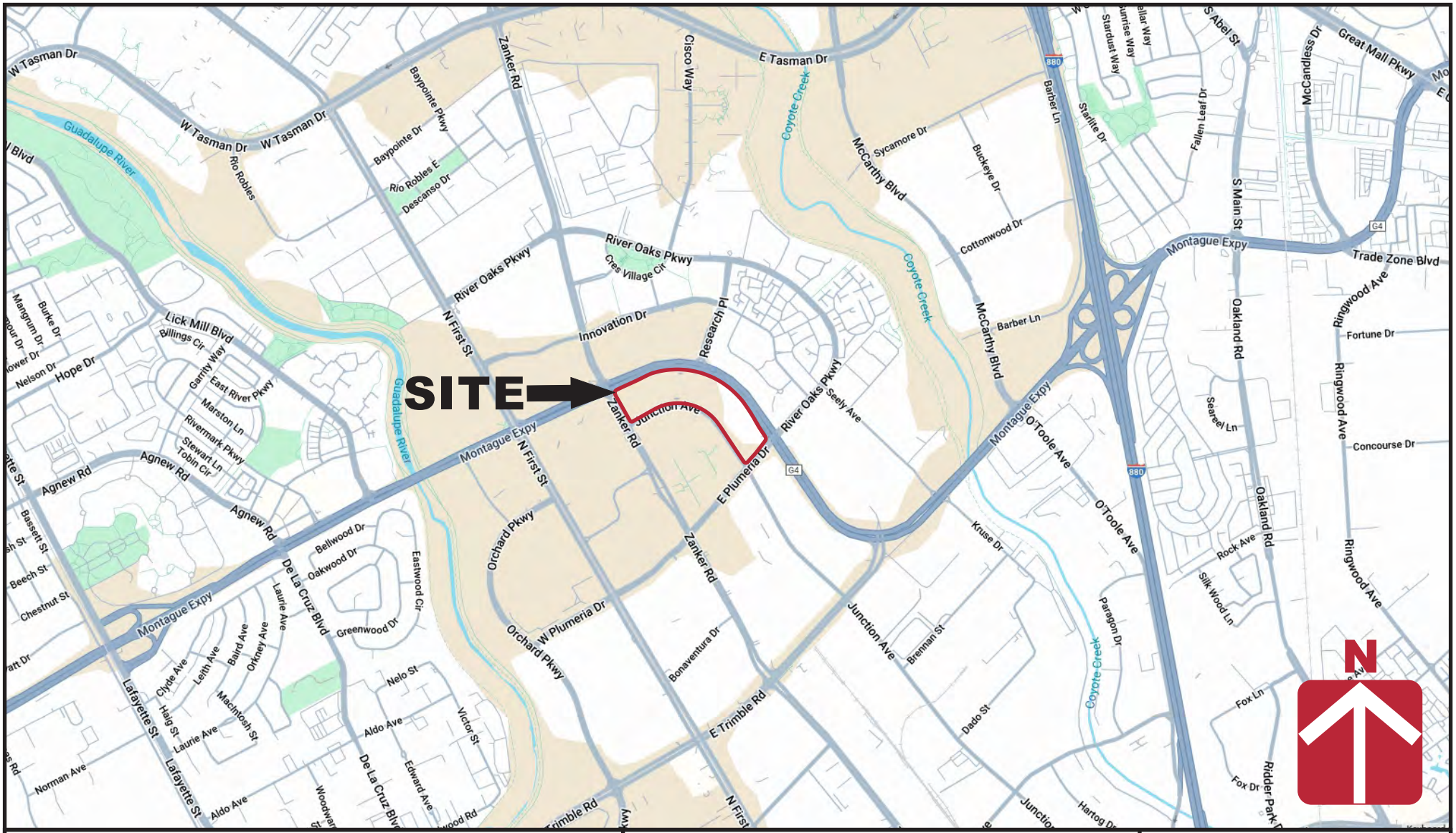
Youd, T.L. and C.T. Garris, 1995, Liquefaction-Induced Ground-Surface Disruption: Journal of Geotechnical Engineering, Vol. 121, No. 11, pp. 805 - 809.

Youd, T.L. and Idriss, I.M., et al, 1997, Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils: National Center for Earthquake Engineering Research, Technical Report NCEER - 97-0022, January 5, 6, 1996.

Youd et al., 2001, “Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils,” ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vo. 127, No. 10, October, 2001.

Youd, T. Leslie, Hansen, Corbett M., and Bartlett, Steven F., 2002, Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement: ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 128, December 2002, p 1007-1017.

Youd, T.L. and Hoose, S.N., 1978, Historic Ground Failures in Northern California Triggered by Earthquakes, United States Geologic Survey Professional Paper 993.

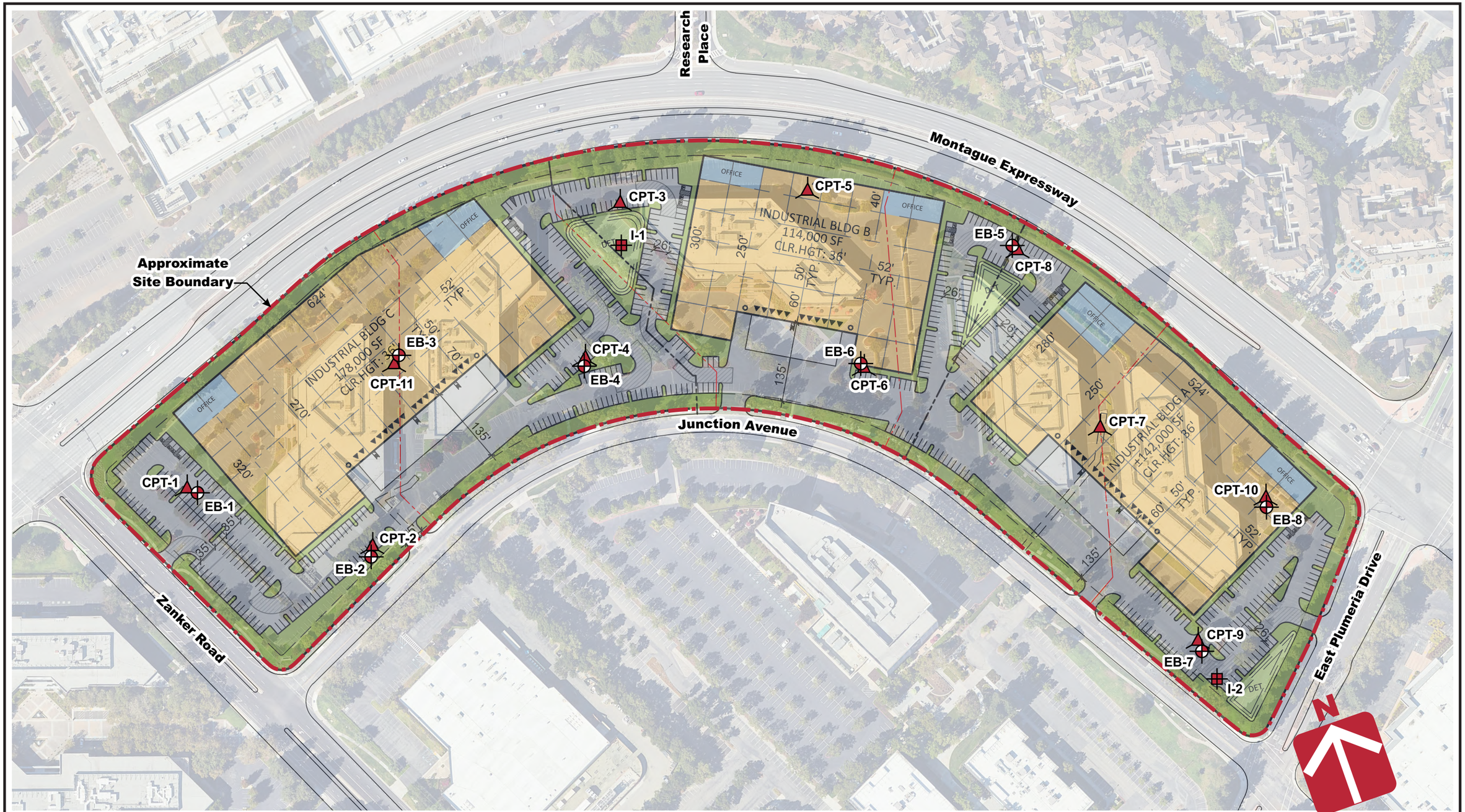


CORNERSTONE EARTH GROUP

Vicinity Map

Montague Pointe Industrial
San Jose, CA

Project Number	1543-2-1
Figure Number	Figure 1
Date	May 2025
Drawn By	RRN



Legend

- Approximate location of exploratory boring (EB)
- Approximate location of cone penetration test (CPT)
- Approximate location of infiltration test (I)

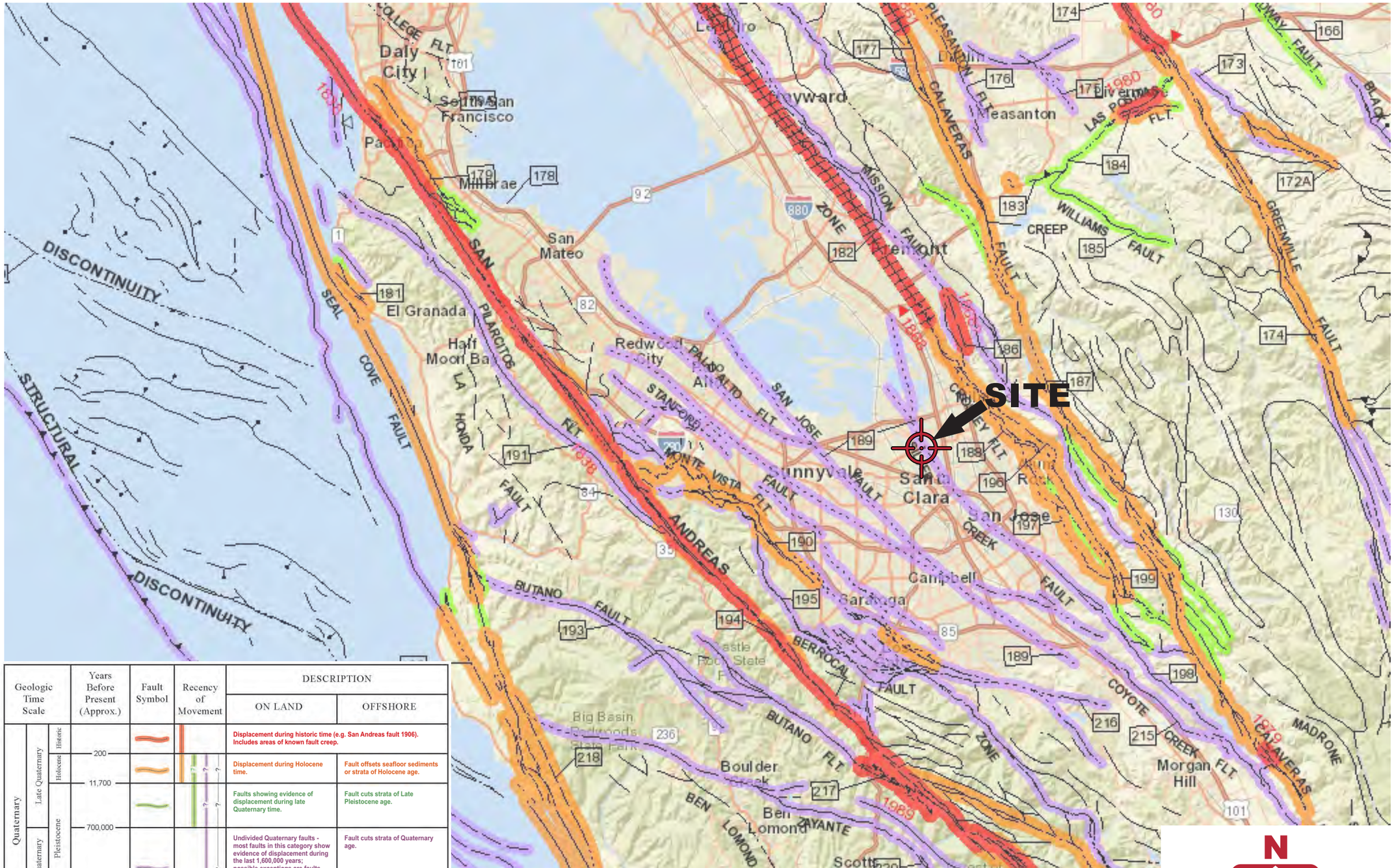
Base by Google Earth, dated 08/30/2023
Overlay by Lowney Architecture, Site Plan Analysis - Scheme 03, dated 01/19/2024

CORNERSTONE EARTH GROUP

Site Plan

Montague Pointe Industrial
San Jose, CA

Project Number	1543-2-1
Figure Number	Figure 2
Date	May 2025
Drawn By	RRN



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION: ON LAND	DESCRIPTION: OFFSHORE
Quaternary	Late Quaternary	Historic	200			Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene				Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	11,700			Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		700,000			Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Pilo-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary			1,600,000'			Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.
			4.5 billion (Age of Earth)				

Base by California Geological Survey - 2010 Fault Activity Map of California (Jennings and Bryant, 2010)

Project Number: 1543-2-1

Figure Number: Figure 3

Date: May 2025

Drawn By: RRN

Regional Fault Map

Montague Pointe Industrial
San Jose, CA

CORNERSTONE EARTH GROUP

CORNERSTONE EARTH GROUP

FIGURE 4A

CPT NO. 1

© 2014 Cornerstone Earth Group, Inc.

PROJECT/CPT DATA

Project Title	**Montague Pointe Industrial**
Project No.	**1543-2-1**
Project Manager	**JDS**

SEISMIC PARAMETERS

Controlling Fault	**Hayward**	
Earthquake Magnitude (Mw)	**7.08**	
PGA (Amax)	**0.76**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**10.4**
Design Water Depth (feet)	**7**
Ave. Unit Weight Above GW (pcf)	**120**
Ave. Unit Weight Below GW (pcf)	**125**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 7 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.35 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.3 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.57 **L/H** 170.0

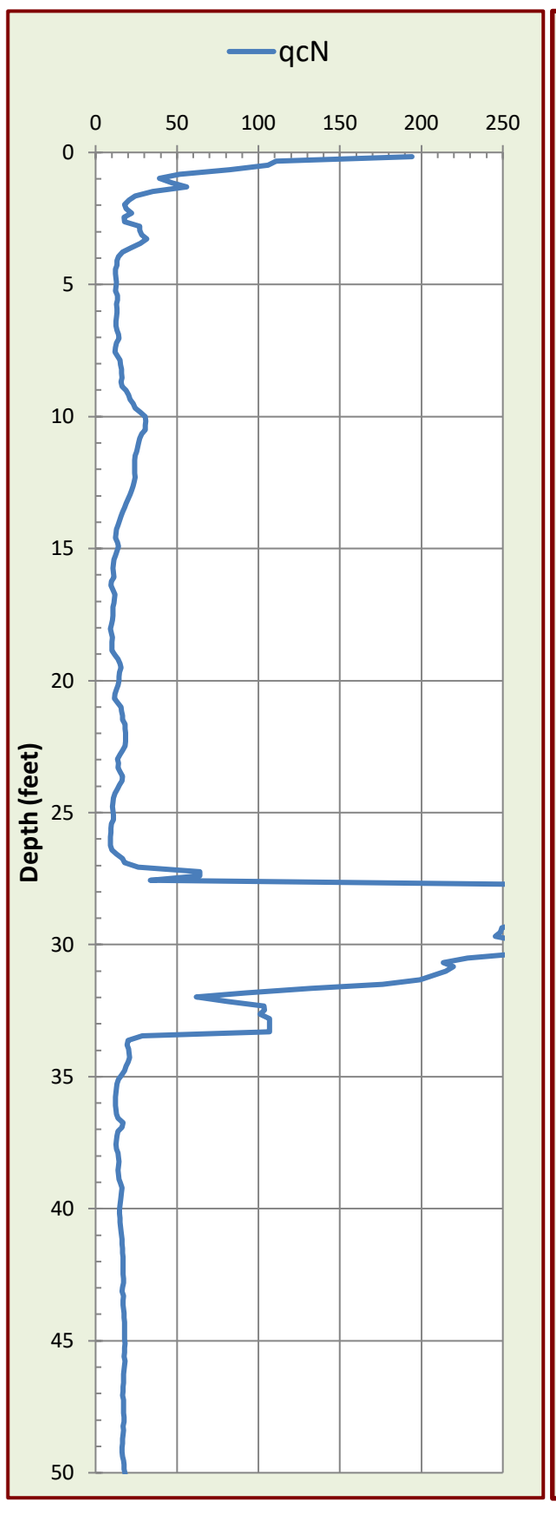
LDI[1] **Corrected for Distance** 0.06 (4 < L/H < 40)

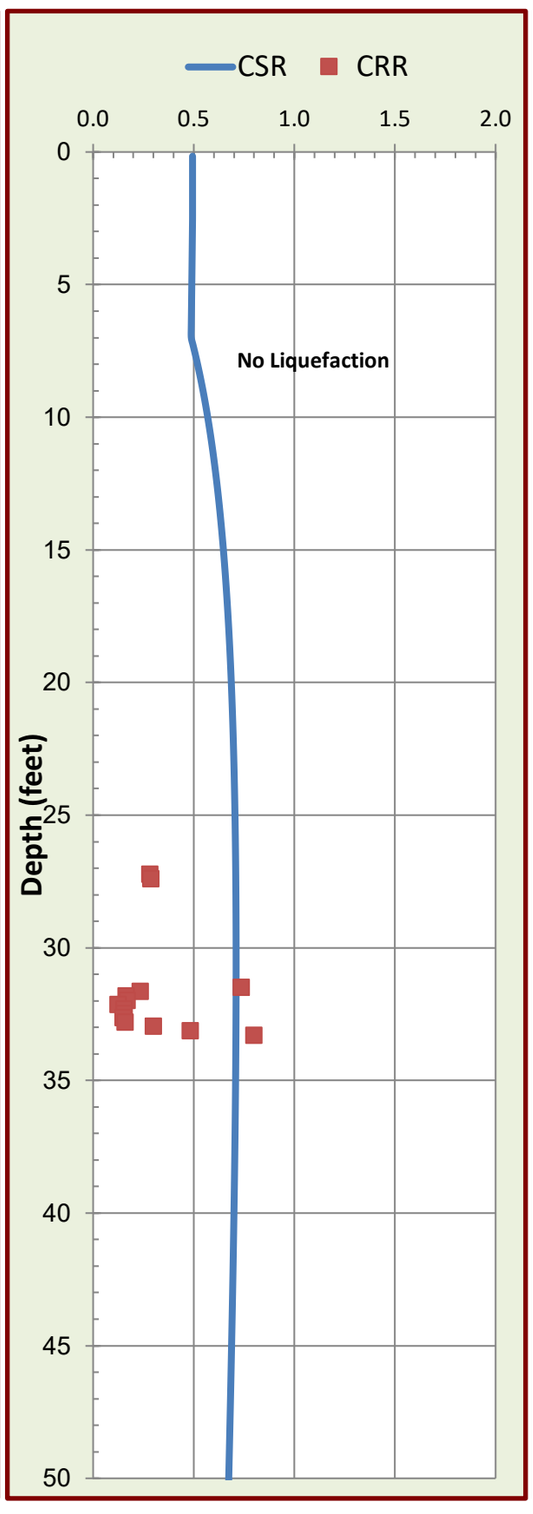
EXPECTED RANGE OF DISPLACEMENT

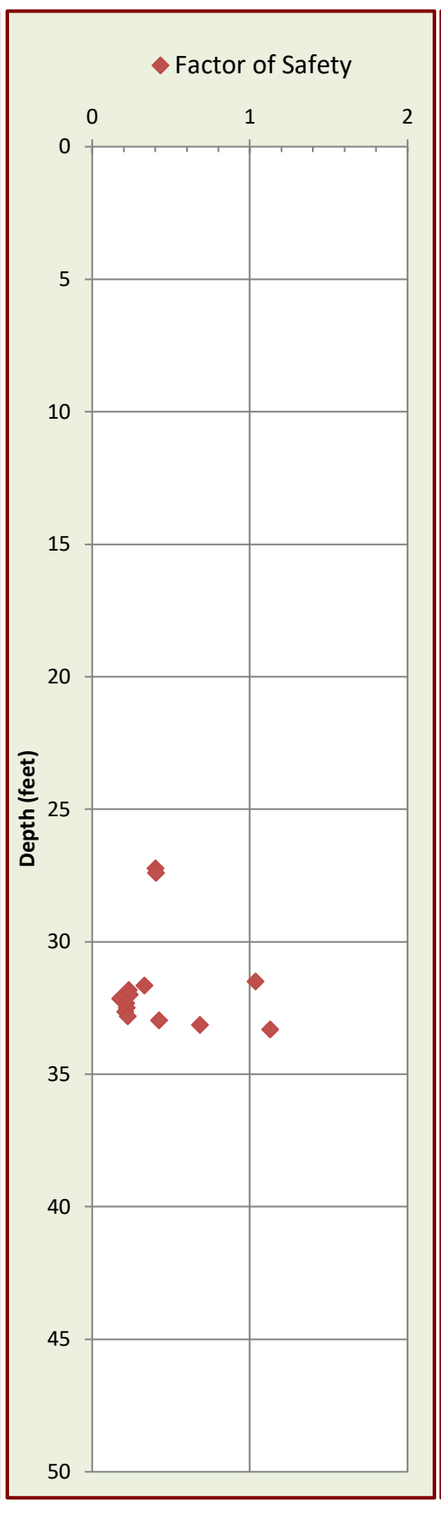
0.0 **to** **0.1** **feet**

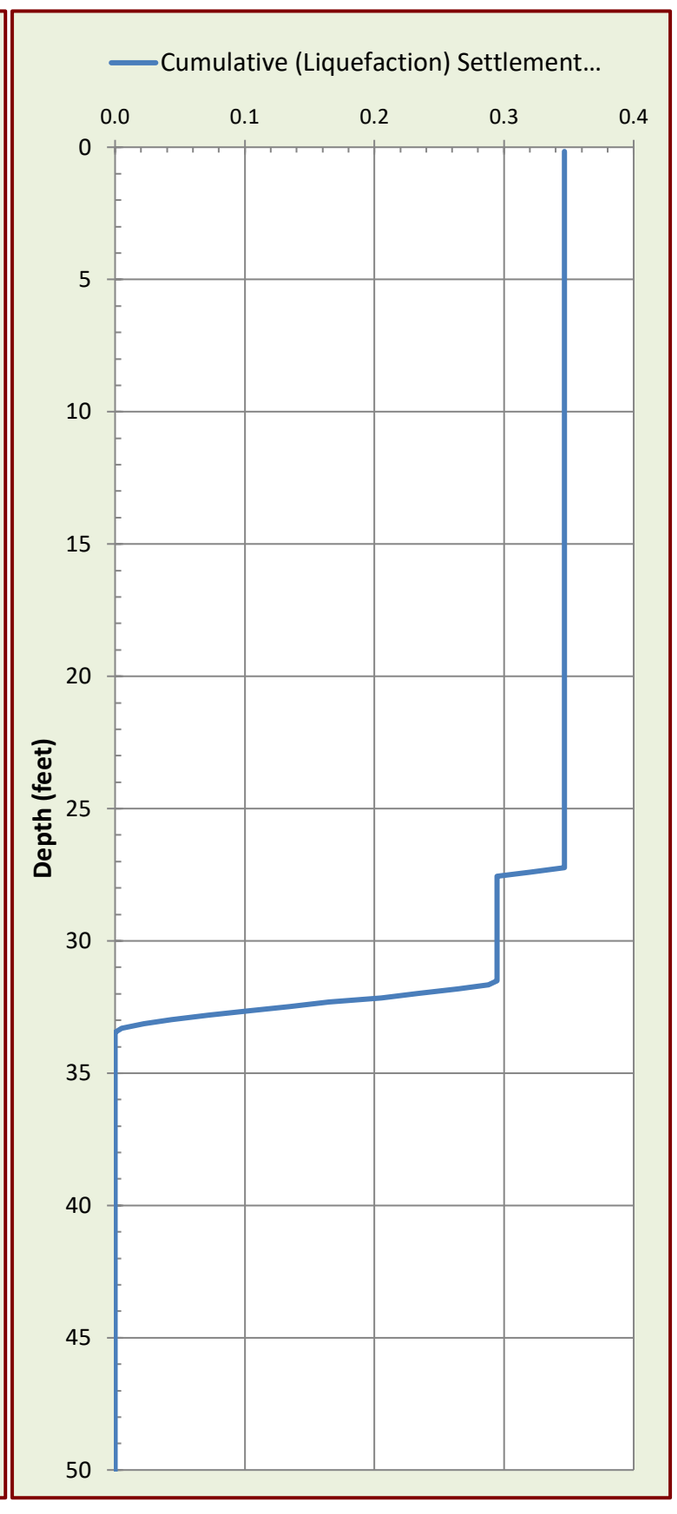
[1]Not Valid for L/H Values < 4 and > 40.

[2]LDI Values Only Summed to 2H Below Grade.









CORNERSTONE EARTH GROUP

FIGURE 4B

CPT NO. 2

© 2014 Cornerstone Earth Group, Inc.

PROJECT/CPT DATA

Project Title	**Montague Pointe Industrial**
Project No.	**1543-2-1**
Project Manager	**JDS**

SEISMIC PARAMETERS

Controlling Fault	**Hayward**	
Earthquake Magnitude (Mw)	**7.08**	
PGA (Amax)	**0.76**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**9.1**
Design Water Depth (feet)	**7**
Ave. Unit Weight Above GW (pcf)	**120**
Ave. Unit Weight Below GW (pcf)	**125**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 7 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.52 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.5 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.64 **L/H** 177.6

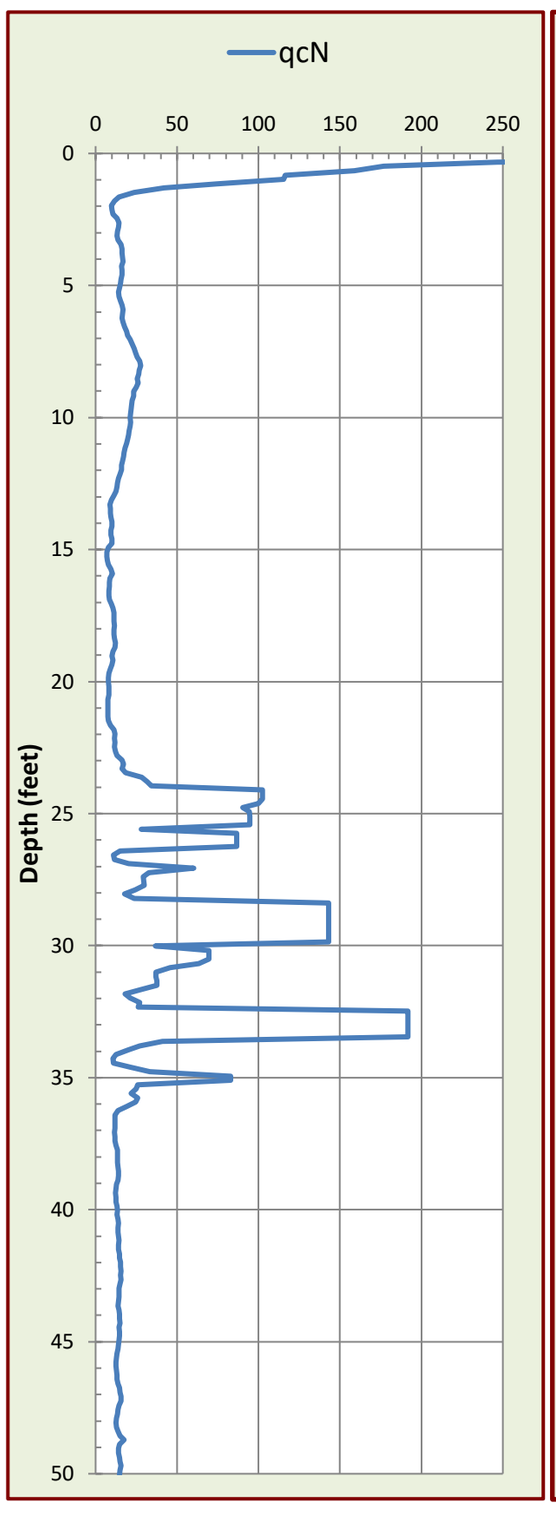
LDI[1]**Corrected for Distance** 0.06 (4 < L/H < 40)

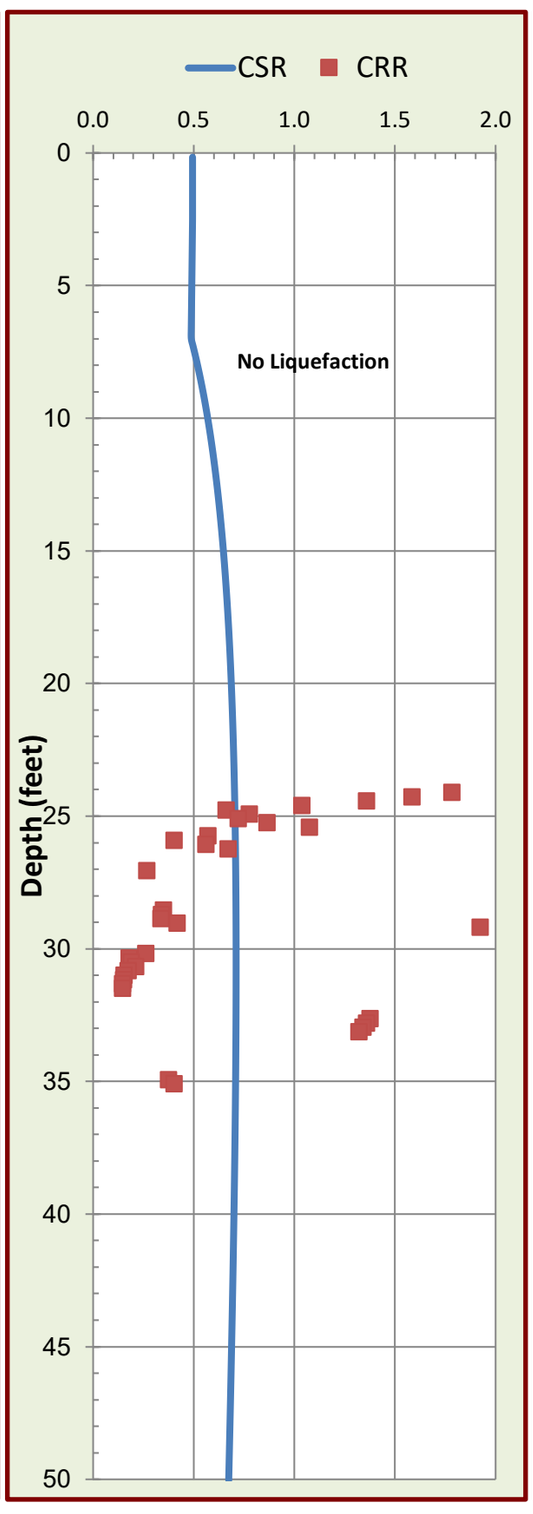
EXPECTED RANGE OF DISPLACEMENT

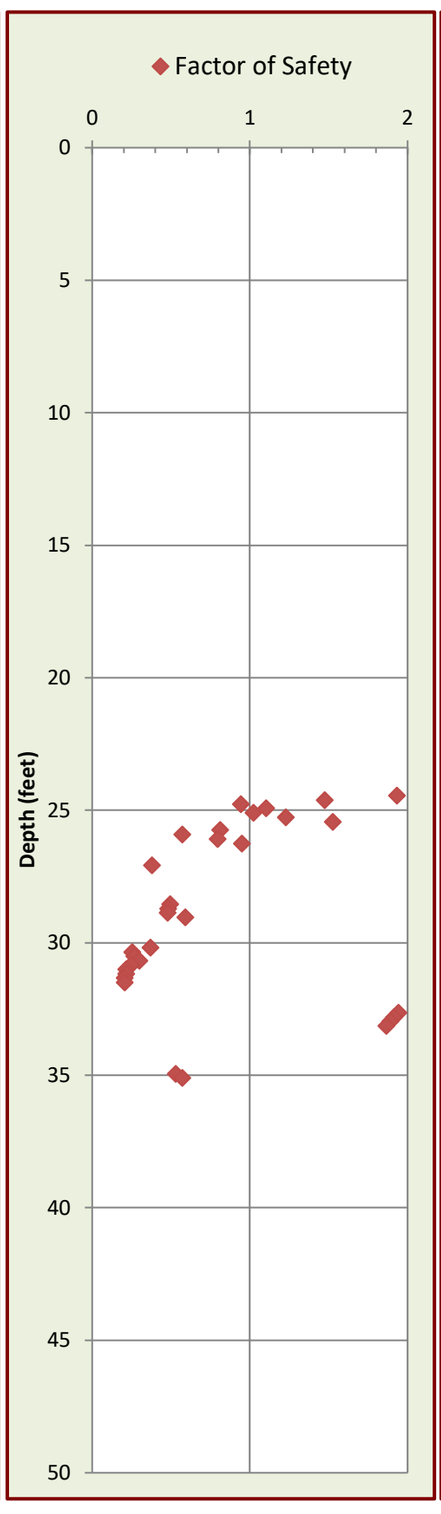
0.0 **to** **0.1** **feet**

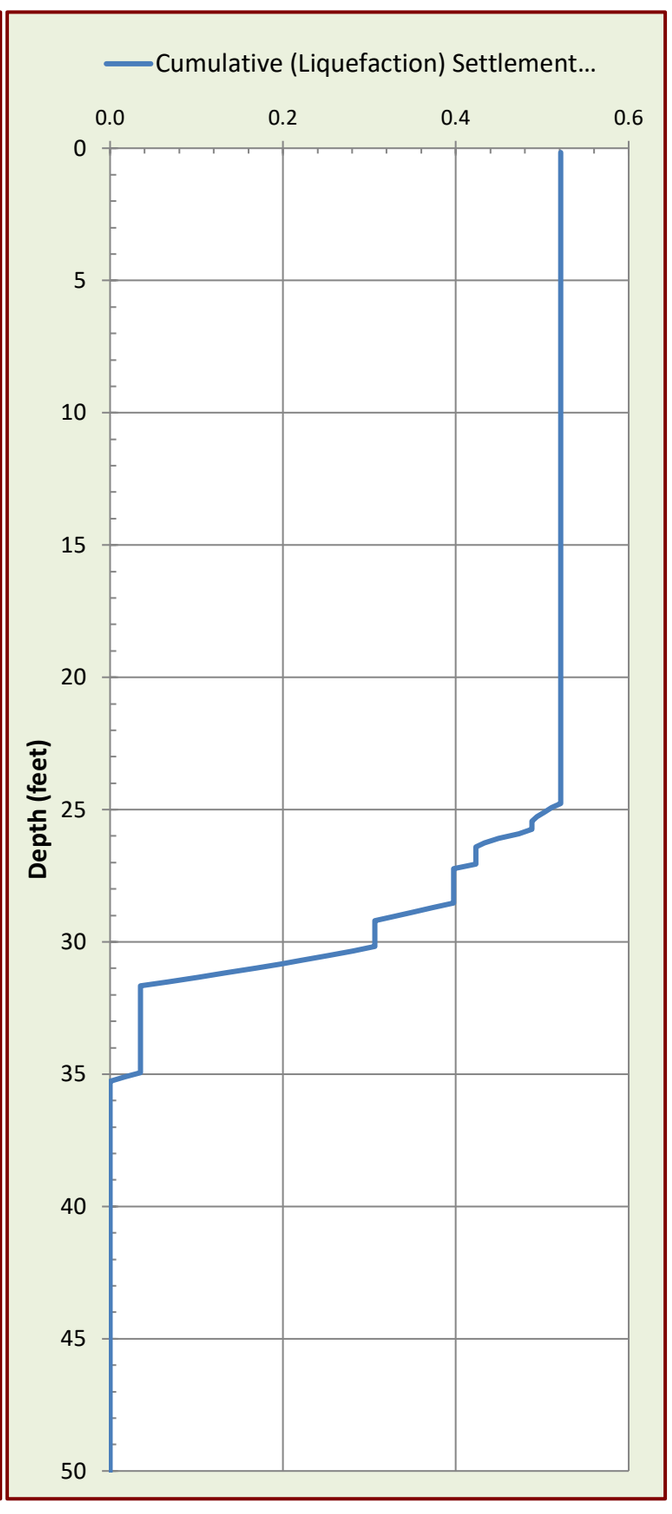
[1]Not Valid for L/H Values < 4 and > 40.

[2]LDI Values Only Summed to 2H Below Grade.









CORNERSTONE EARTH GROUP

FIGURE 4C

CPT NO. 3

© 2014 Cornerstone Earth Group, Inc.

PROJECT/CPT DATA

Project Title	**Montague Pointe Industrial**
Project No.	**1543-2-1**
Project Manager	**JDS**

SEISMIC PARAMETERS

Controlling Fault	**Hayward**	
Earthquake Magnitude (Mw)	**7.08**	
PGA (Amax)	**0.76**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**10**
Design Water Depth (feet)	**7**
Ave. Unit Weight Above GW (pcf)	**120**
Ave. Unit Weight Below GW (pcf)	**125**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 7 **FEET**

0.03 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.08 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.1 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.00 **L/H** 526.0

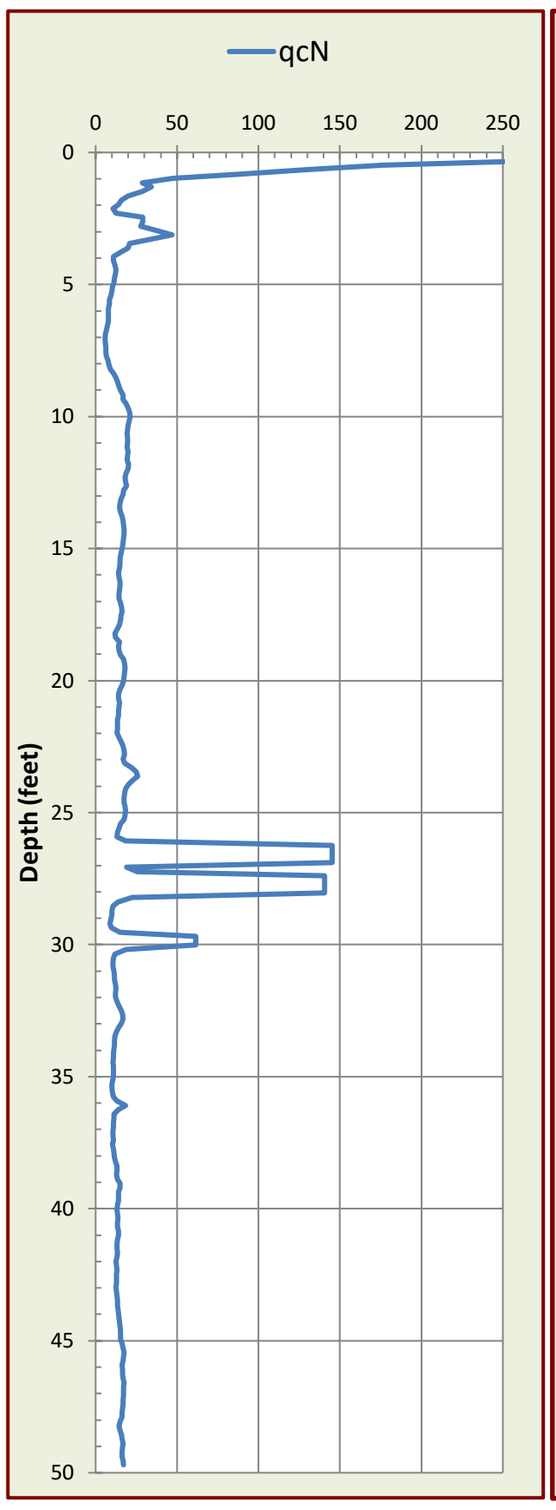
LDI[1] Corrected for Distance 0.00 (4 < L/H < 40)

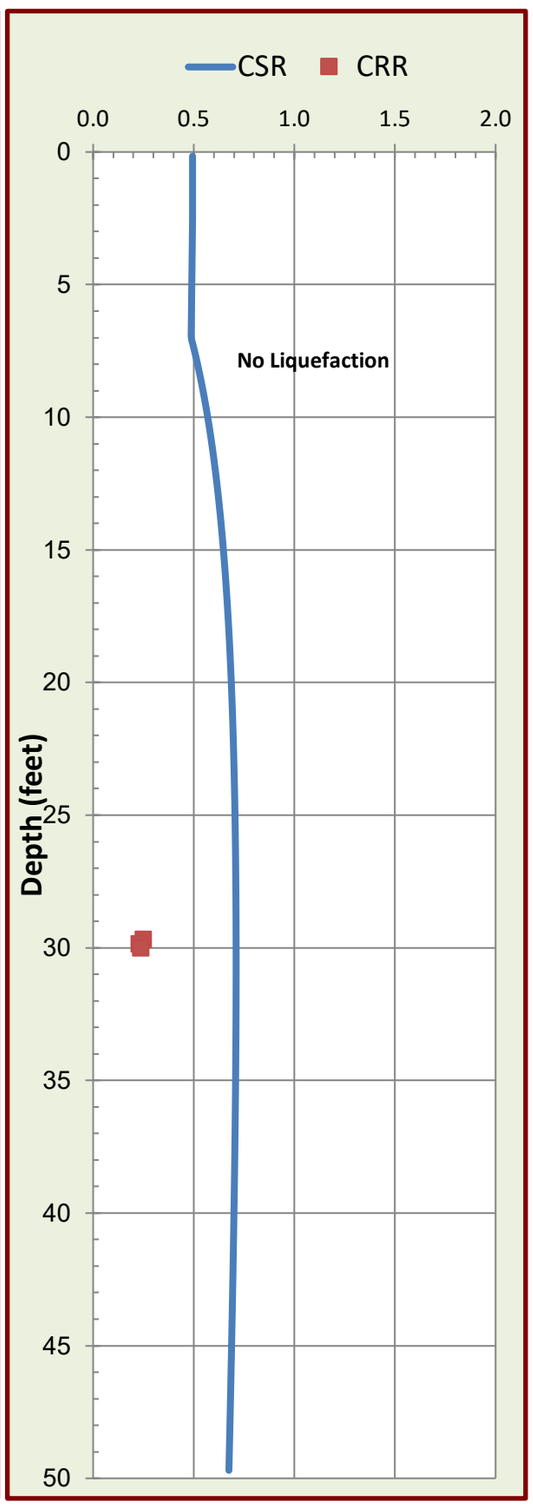
EXPECTED RANGE OF DISPLACEMENT

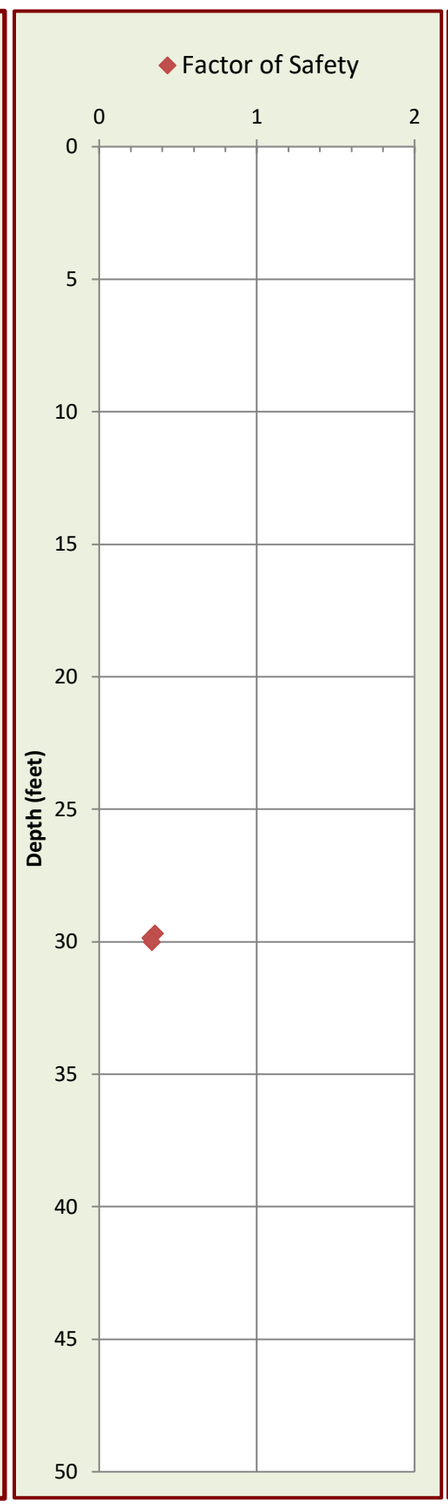
0.0 **to** **0.0** **feet**

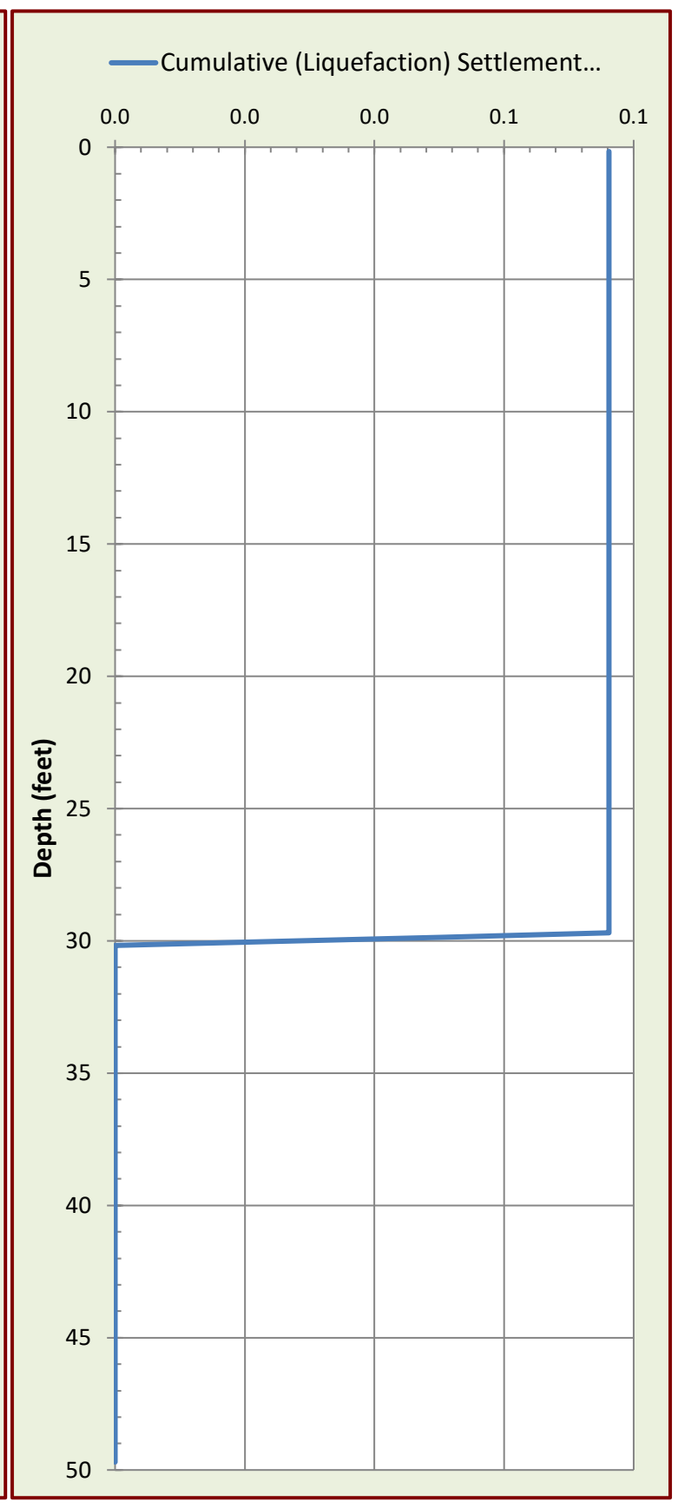
[1]Not Valid for L/H Values < 4 and > 40.

[2]LDI Values Only Summed to 2H Below Grade.









CORNERSTONE EARTH GROUP

FIGURE 4D

CPT NO. 4

© 2014 Cornerstone Earth Group, Inc.

PROJECT/CPT DATA

Project Title	**Montague Pointe Industrial**
Project No.	**1543-2-1**
Project Manager	**JDS**

SEISMIC PARAMETERS

Controlling Fault	**Hayward**	
Earthquake Magnitude (Mw)	**7.08**	
PGA (Amax)	**0.76**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**10**
Design Water Depth (feet)	**7**
Ave. Unit Weight Above GW (pcf)	**120**
Ave. Unit Weight Below GW (pcf)	**125**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 7 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.52 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.5 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.17 **L/H** 211.3

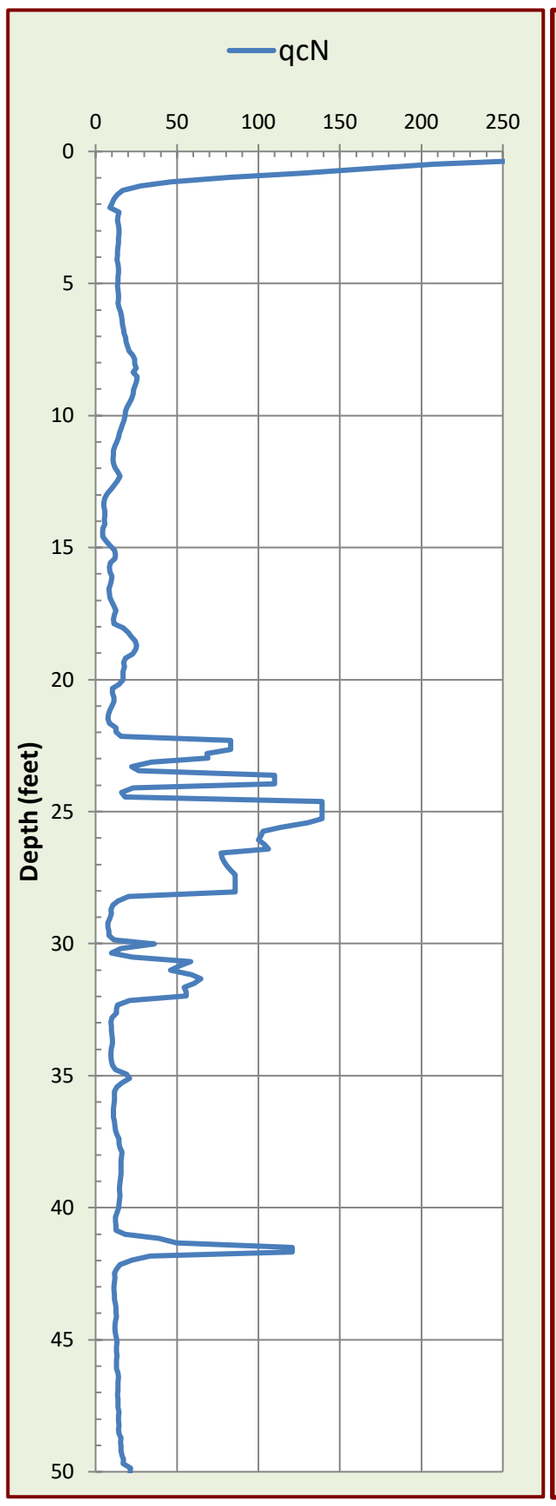
LDI[1]**Corrected for Distance** 0.01 (4 < L/H < 40)

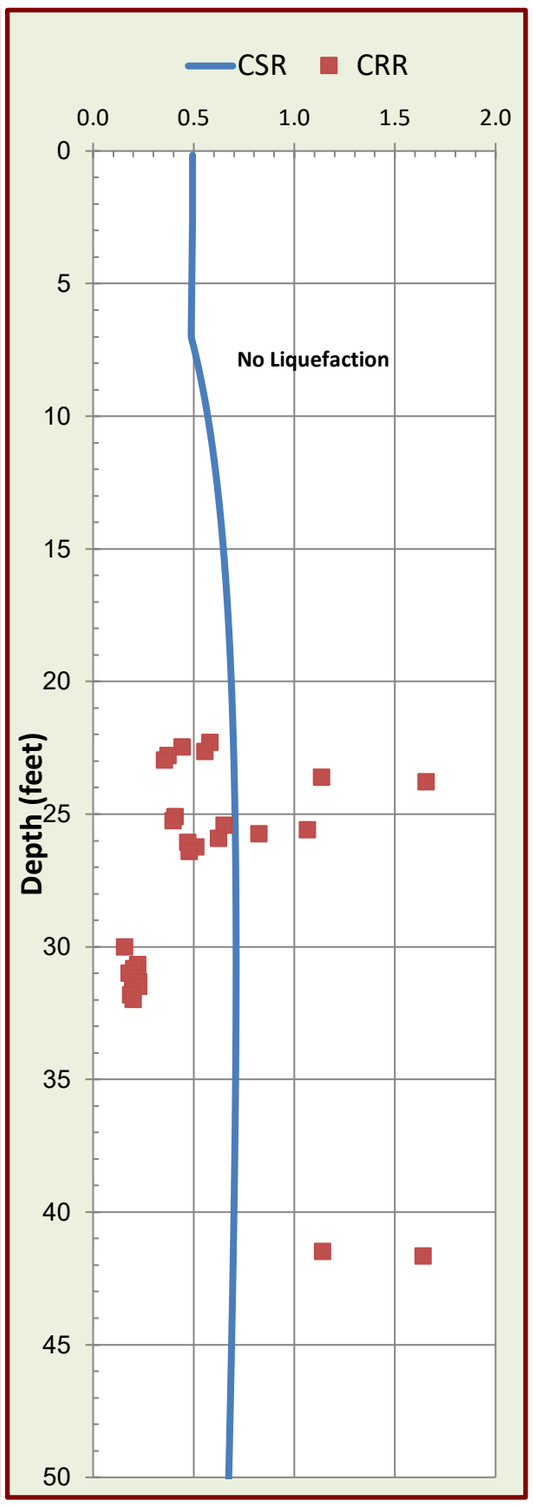
EXPECTED RANGE OF DISPLACEMENT

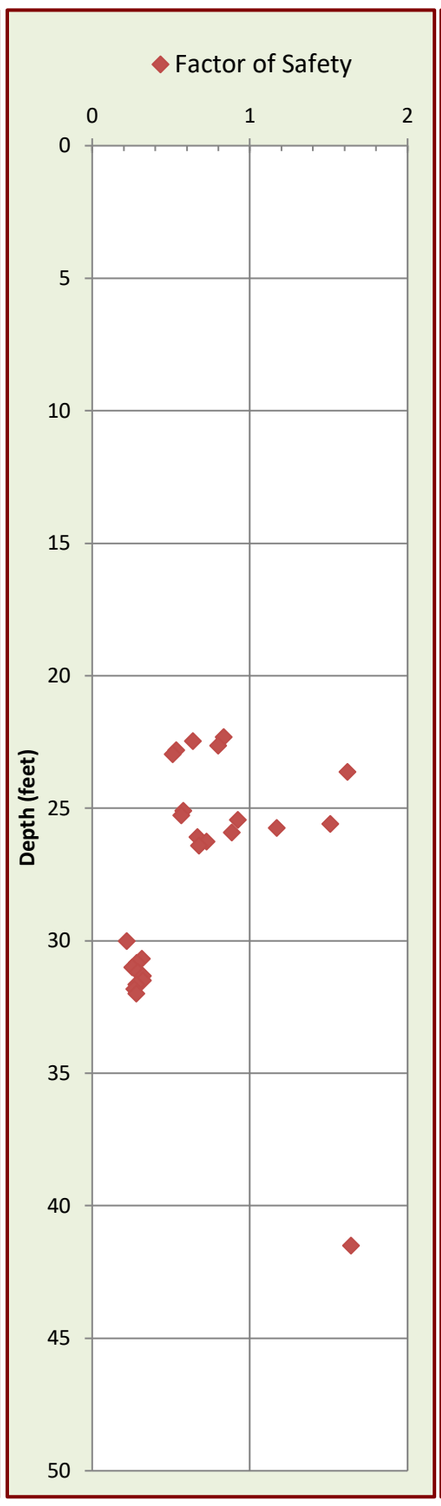
0.0 **to** **0.0** **feet**

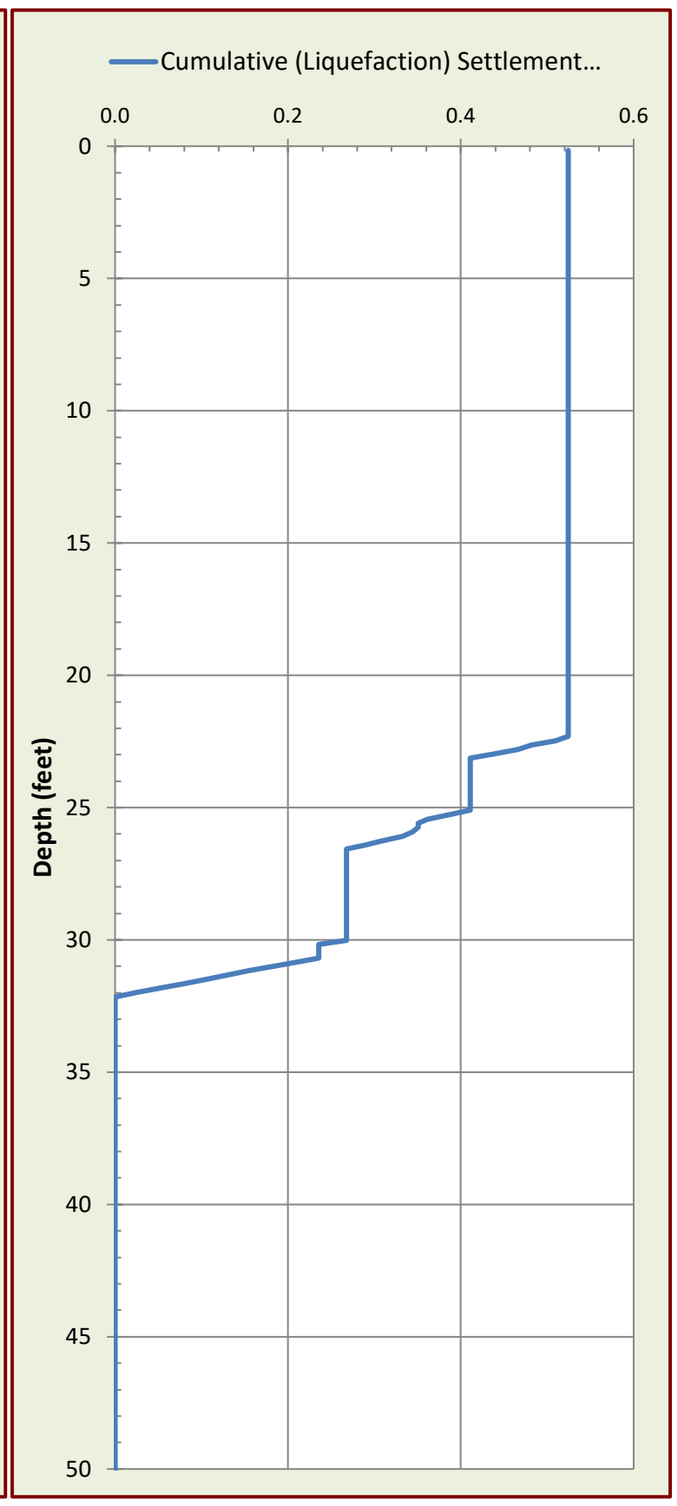
[1]Not Valid for L/H Values < 4 and > 40.

[2]LDI Values Only Summed to 2H Below Grade.









CORNERSTONE EARTH GROUP

FIGURE 4E

CPT NO. 5

© 2014 Cornerstone Earth Group, Inc.

PROJECT/CPT DATA

Project Title	**Montague Pointe Industrial**
Project No.	**1543-2-1**
Project Manager	**JDS**

SEISMIC PARAMETERS

Controlling Fault	**Hayward**	
Earthquake Magnitude (Mw)	**7.08**	
PGA (Amax)	**0.76**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**10**
Design Water Depth (feet)	**7**
Ave. Unit Weight Above GW (pcf)	**120**
Ave. Unit Weight Below GW (pcf)	**125**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 7 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.37 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.4 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.00 **L/H** 540.0

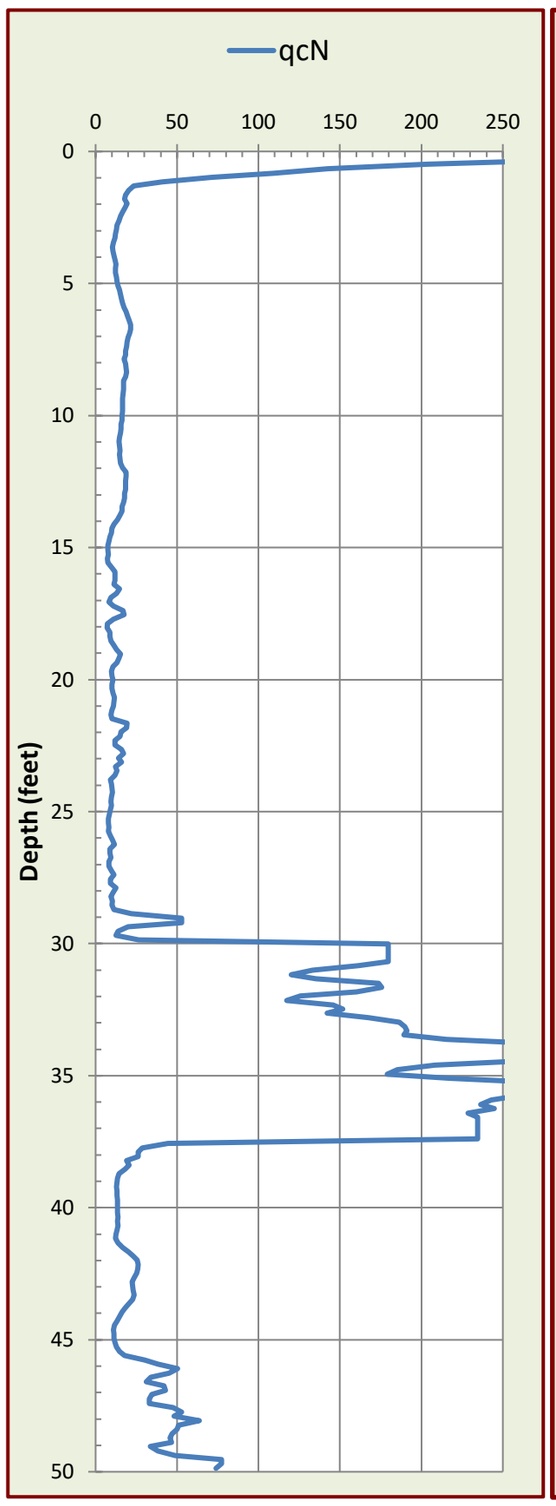
LDI[1] Corrected for Distance 0.00 (4 < L/H < 40)

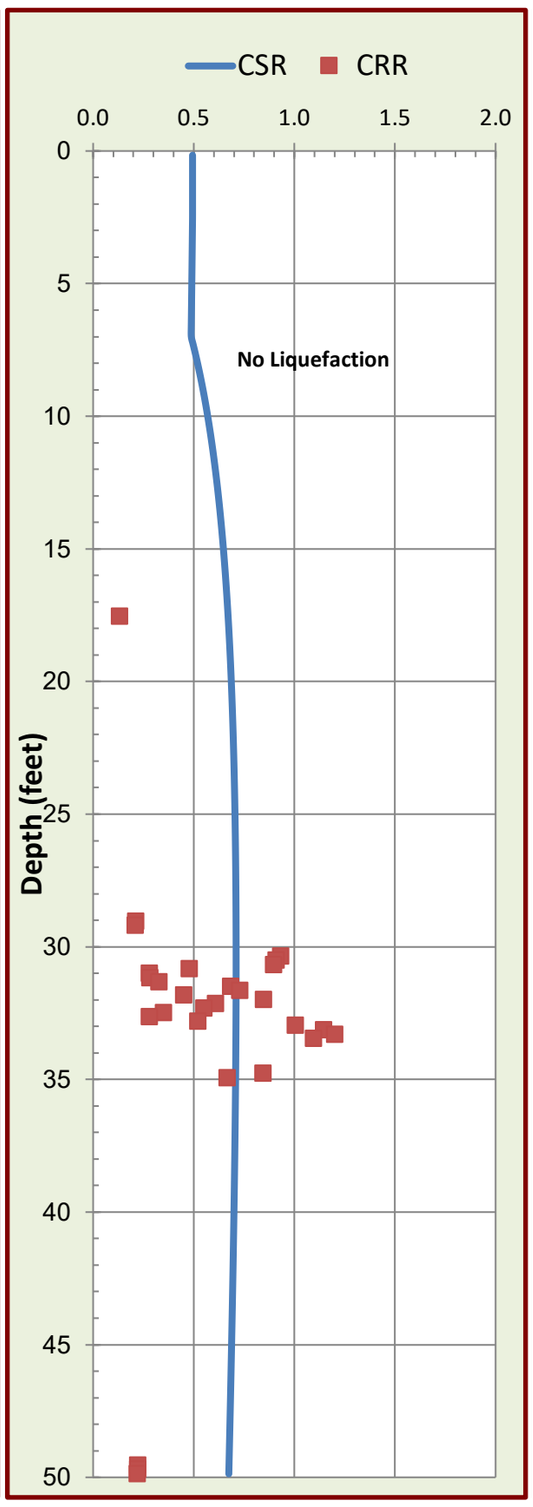
EXPECTED RANGE OF DISPLACEMENT

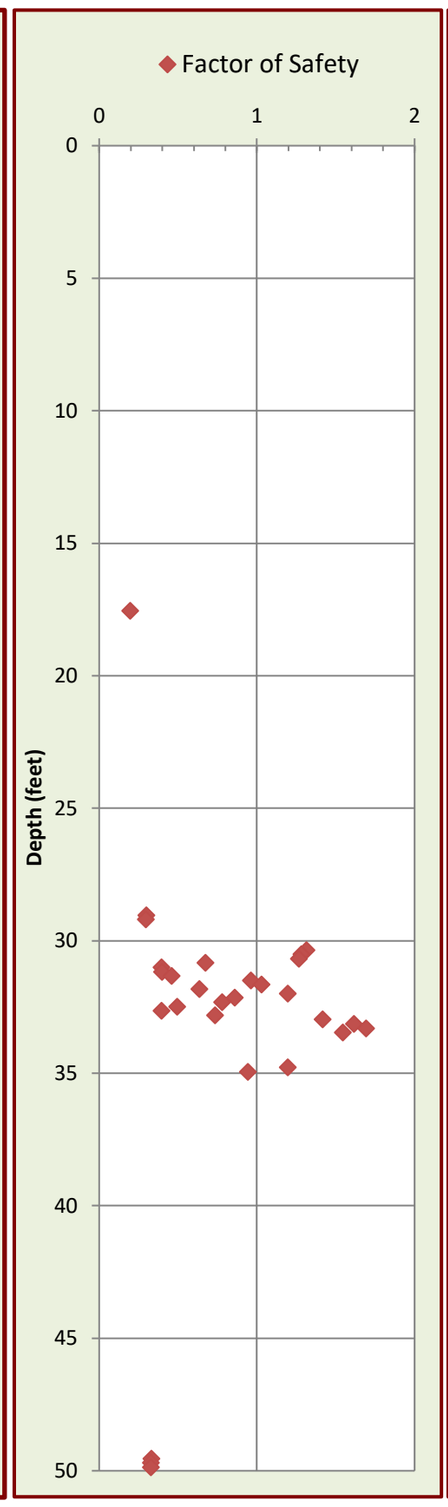
0.0 **to** **0.0** **feet**

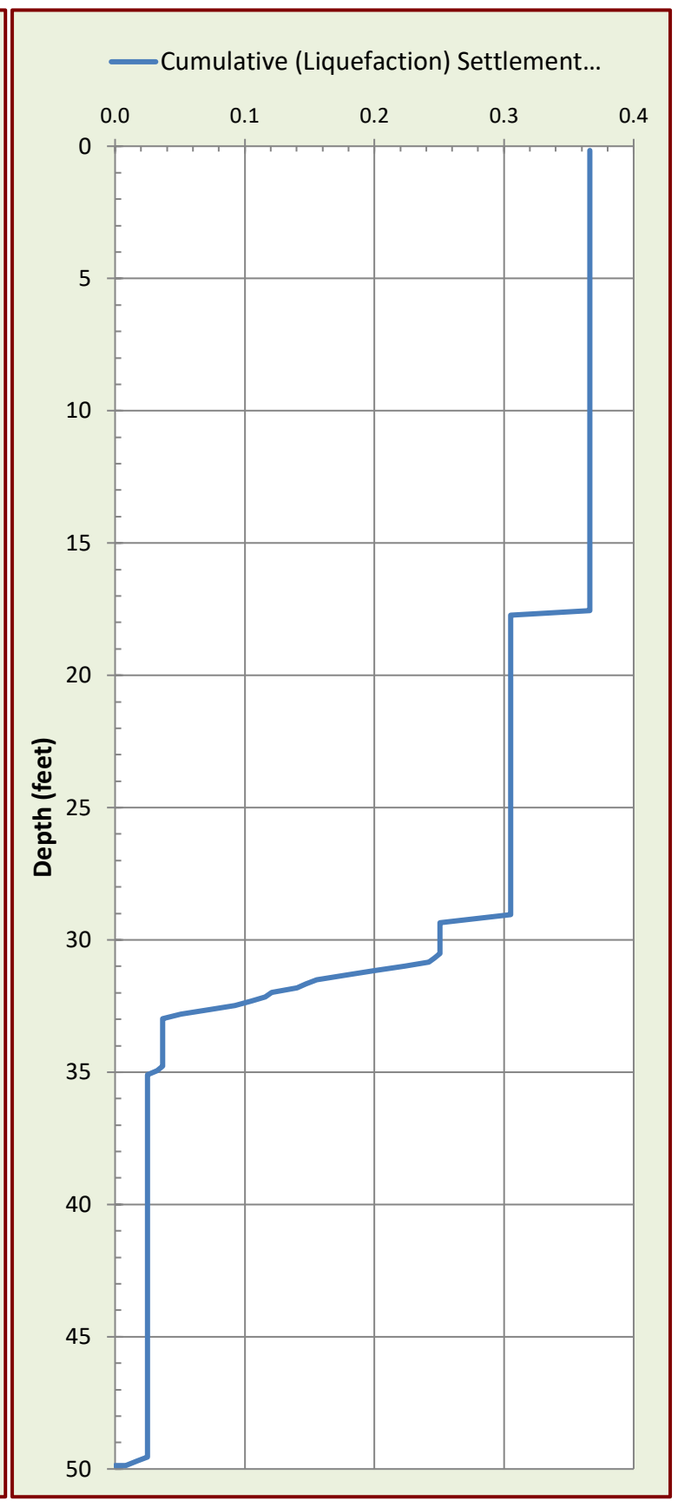
[1]Not Valid for L/H Values < 4 and > 40.

[2]LDI Values Only Summed to 2H Below Grade.









CORNERSTONE EARTH GROUP

FIGURE 4F

CPT NO. 6

© 2014 Cornerstone Earth Group, Inc.

PROJECT/CPT DATA

Project Title	**Montague Pointe Industrial**
Project No.	**1543-2-1**
Project Manager	**JDS**

SEISMIC PARAMETERS

Controlling Fault	**Hayward**	
Earthquake Magnitude (Mw)	**7.08**	
PGA (Amax)	**0.76**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**10**
Design Water Depth (feet)	**7**
Ave. Unit Weight Above GW (pcf)	**120**
Ave. Unit Weight Below GW (pcf)	**125**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 7 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.27 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.3 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.23 **L/H** 163.2

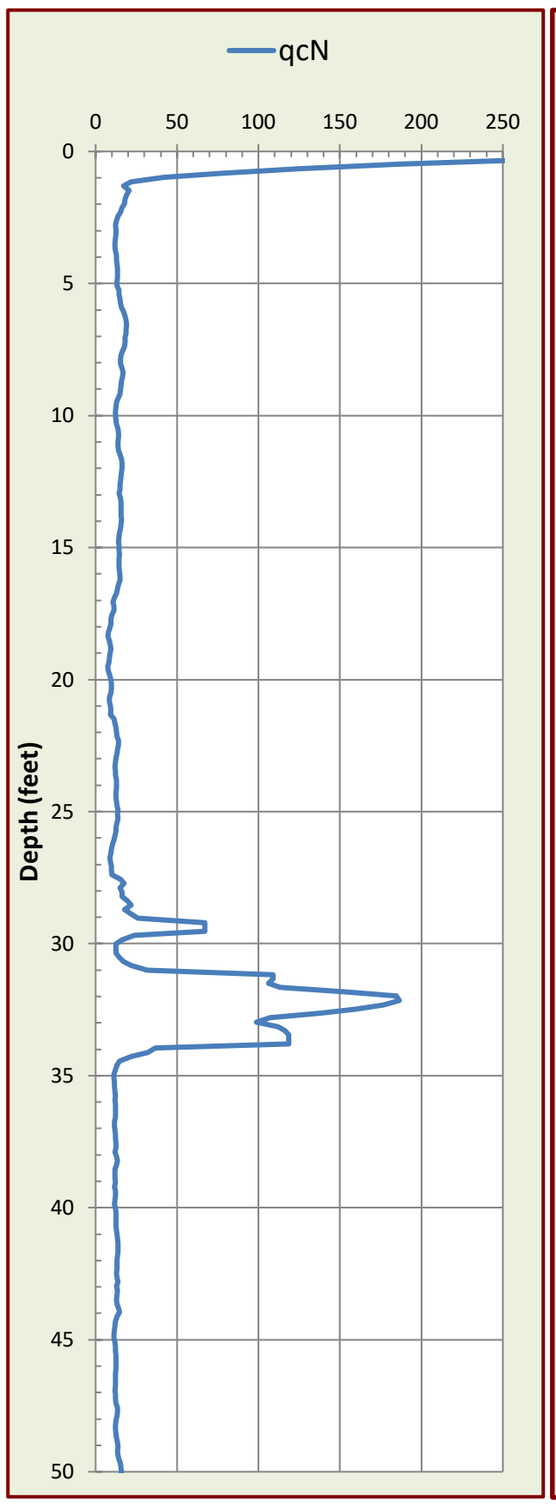
LDI[1] Corrected for Distance 0.02 (4 < L/H < 40)

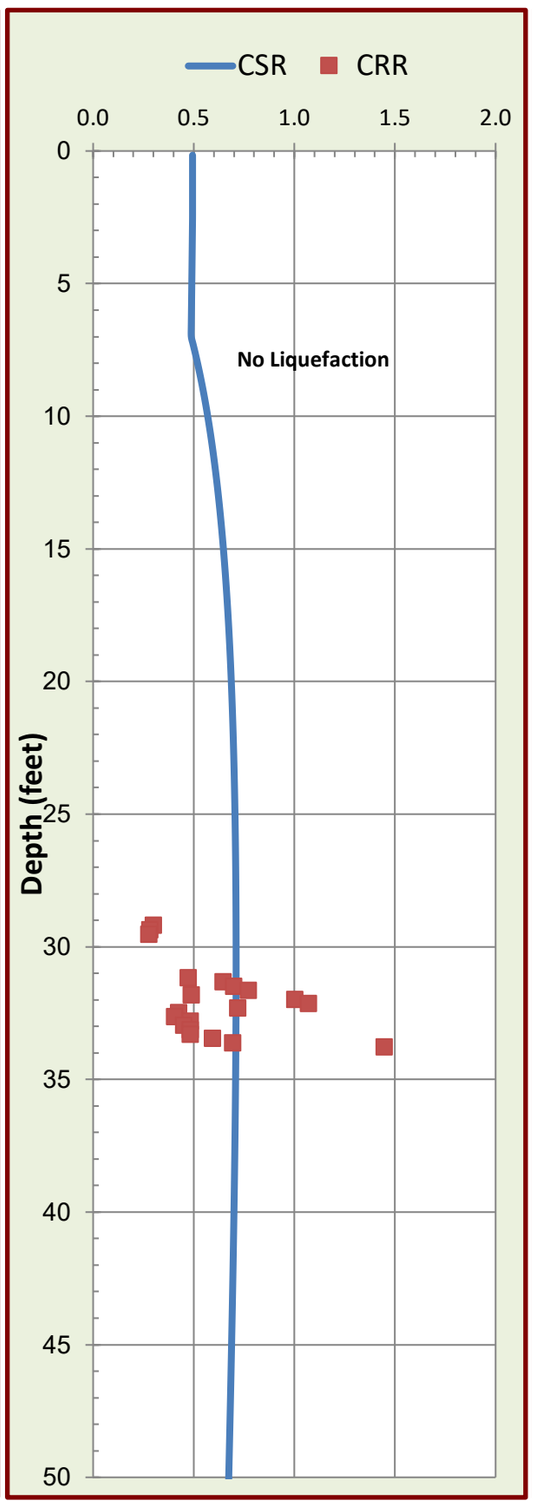
EXPECTED RANGE OF DISPLACEMENT

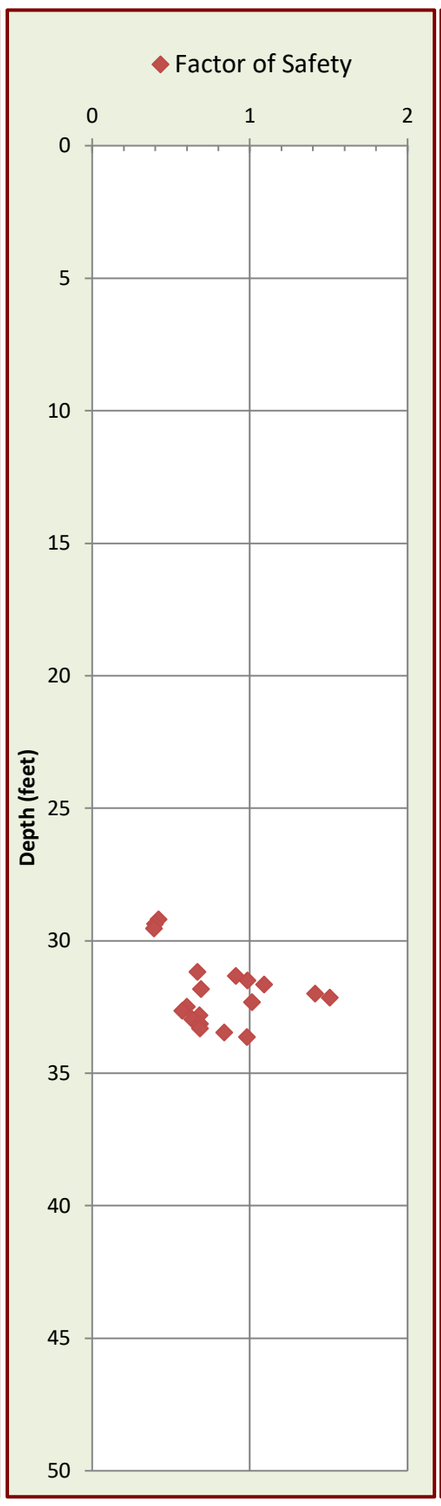
0.0 **to** **0.0** **feet**

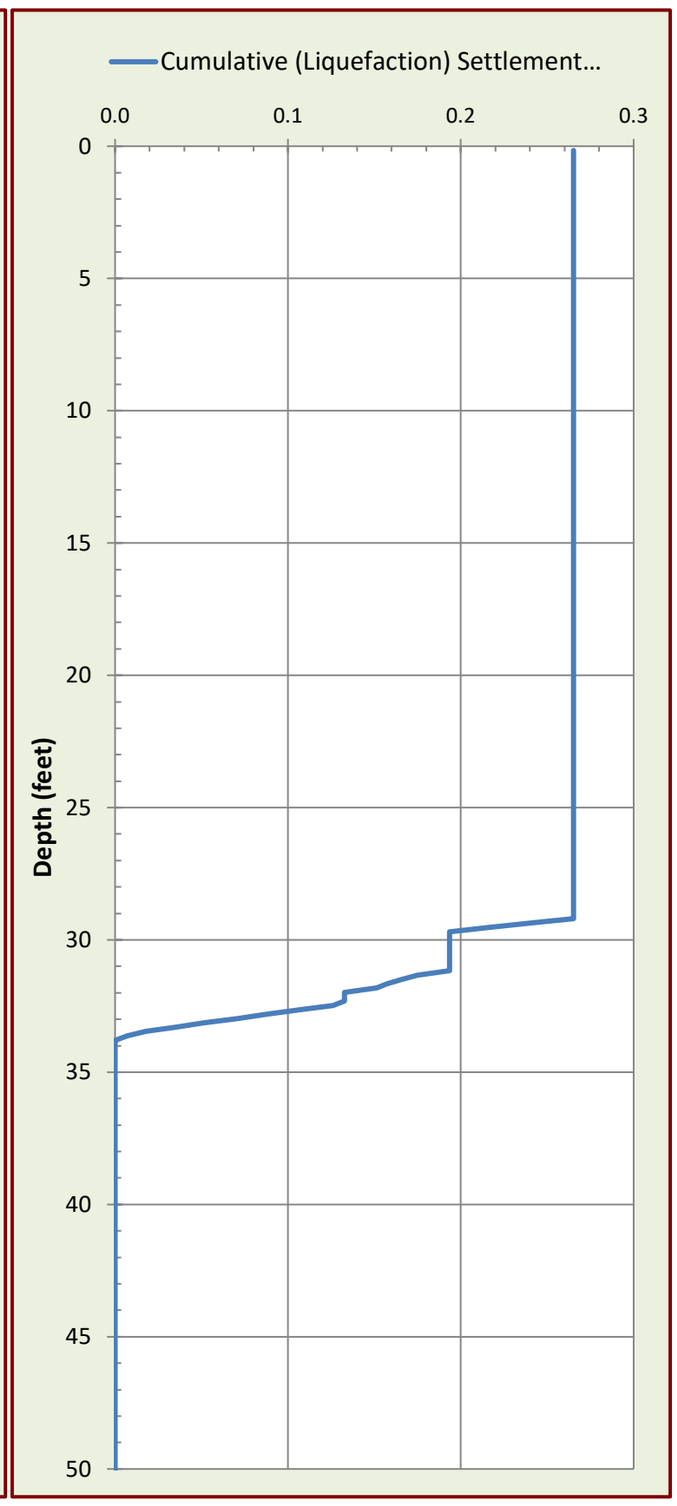
[1]Not Valid for L/H Values < 4 and > 40.

[2]LDI Values Only Summed to 2H Below Grade.









CORNERSTONE EARTH GROUP

FIGURE 4G

CPT NO. 7

© 2014 Cornerstone Earth Group, Inc.

PROJECT/CPT DATA

Project Title	**Montague Pointe Industrial**
Project No.	**1543-2-1**
Project Manager	**JDS**

SEISMIC PARAMETERS

Controlling Fault	**Hayward**	
Earthquake Magnitude (Mw)	**7.08**	
PGA (Amax)	**0.76**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**10**
Design Water Depth (feet)	**7**
Ave. Unit Weight Above GW (pcf)	**120**
Ave. Unit Weight Below GW (pcf)	**125**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 7 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.36 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.4 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.00 **L/H** 560.0

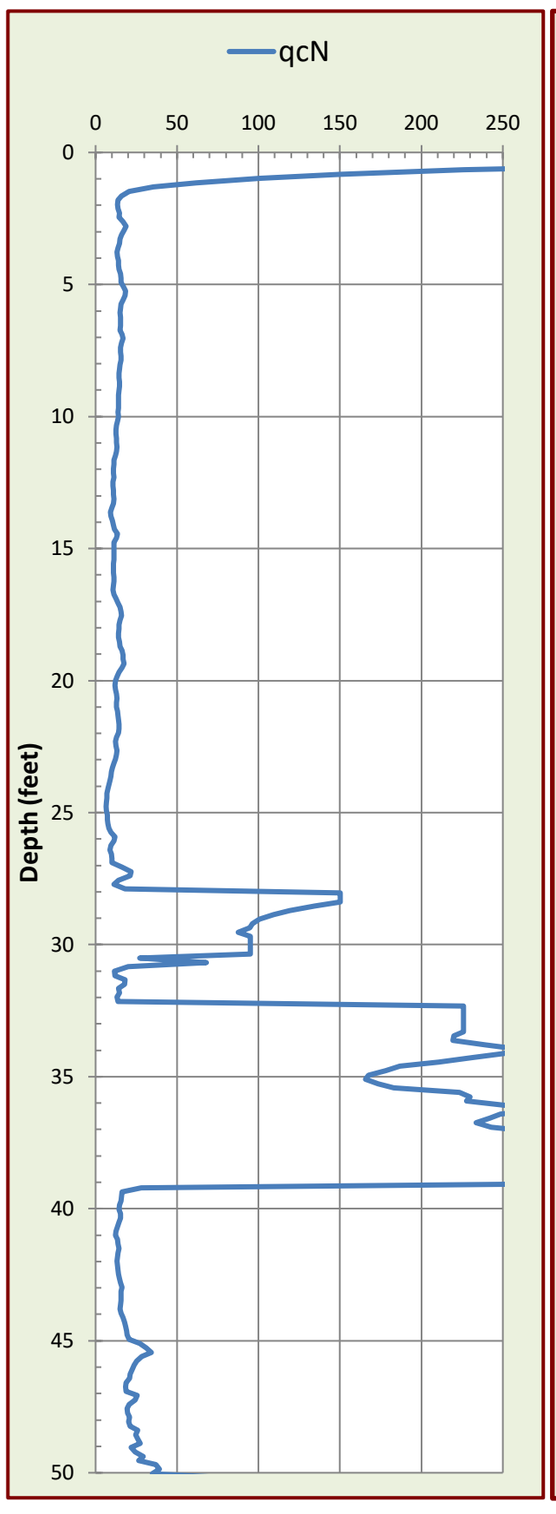
LDI[1] **Corrected for Distance** 0.00 (4 < L/H < 40)

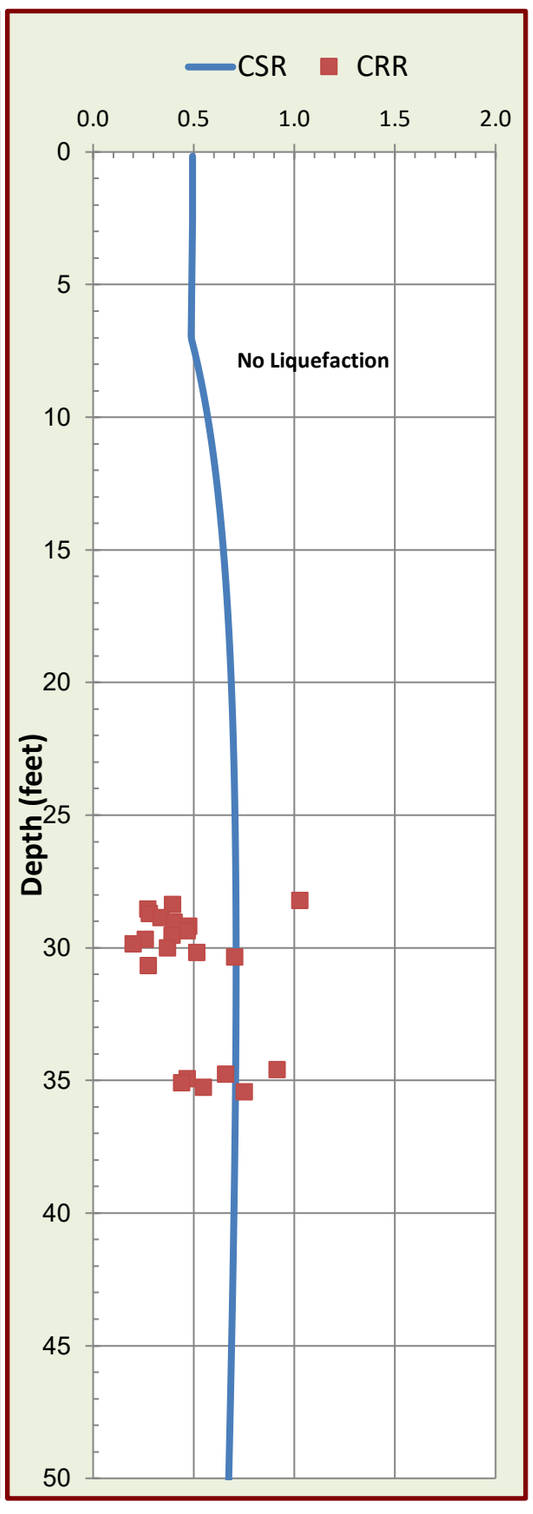
EXPECTED RANGE OF DISPLACEMENT

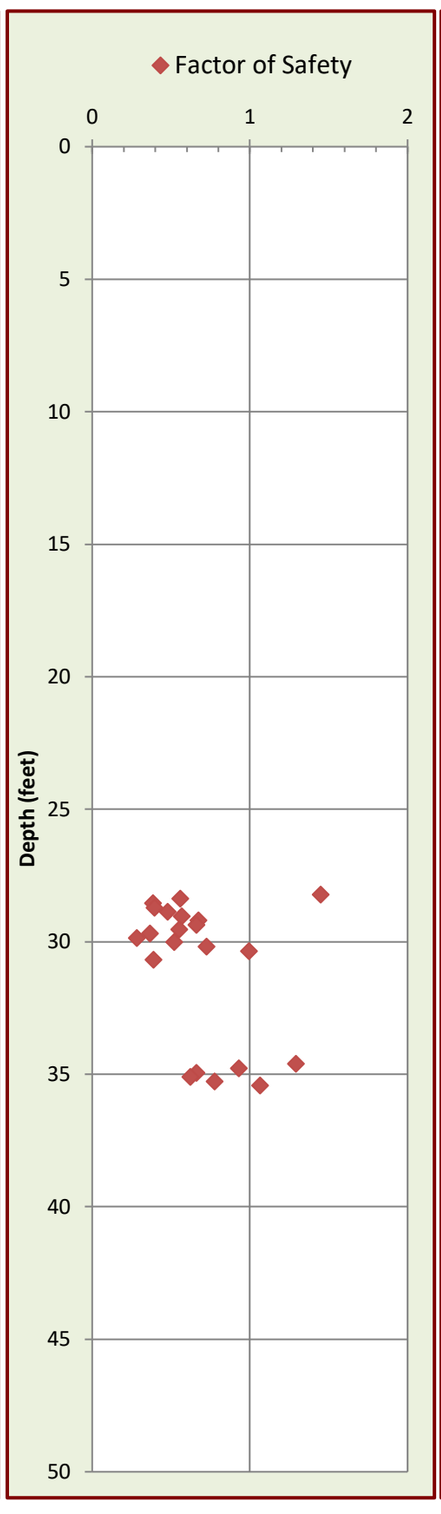
0.0 **to** **0.0** **feet**

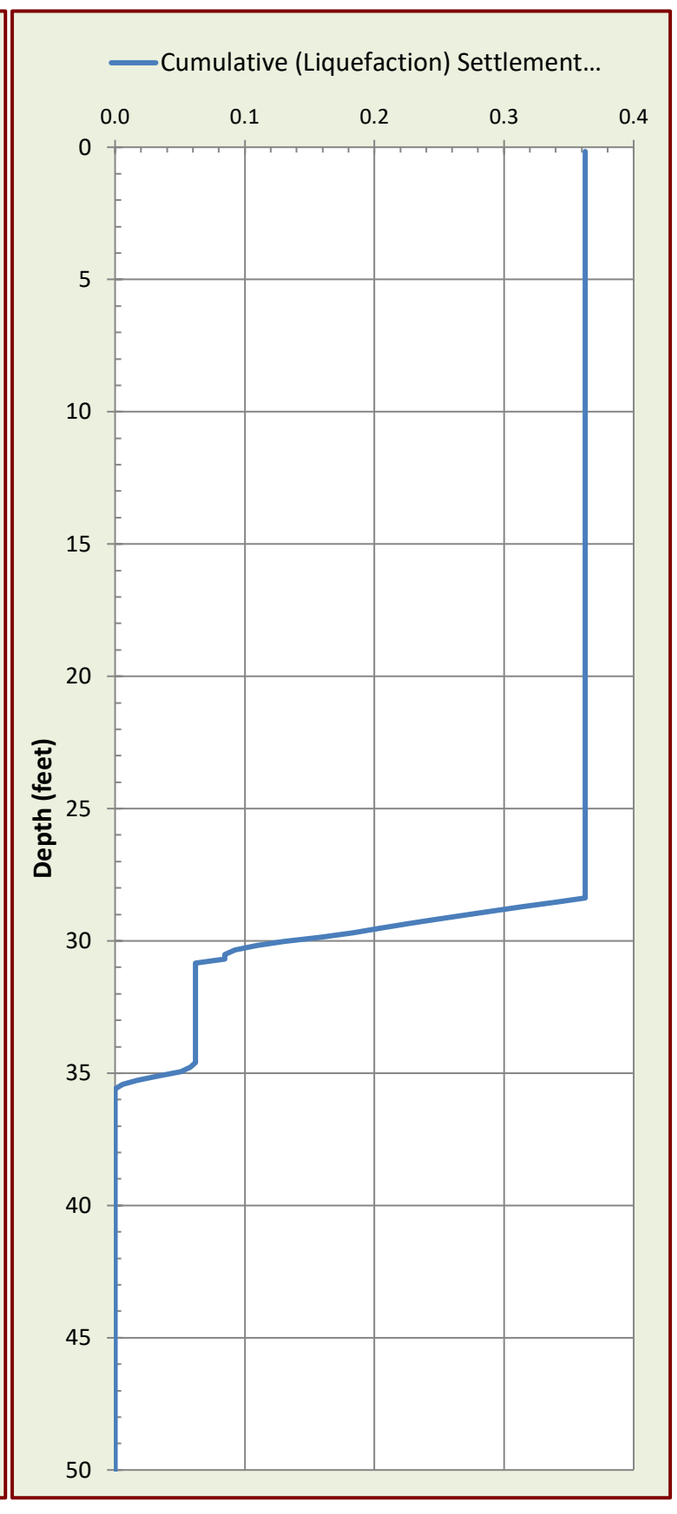
[1]Not Valid for L/H Values < 4 and > 40.

[2]LDI Values Only Summed to 2H Below Grade.









CORNERSTONE EARTH GROUP

FIGURE 4H

CPT NO. 8

© 2014 Cornerstone Earth Group, Inc.

PROJECT/CPT DATA

Project Title	**Montague Pointe Industrial**
Project No.	**1543-2-1**
Project Manager	**JDS**

SEISMIC PARAMETERS

Controlling Fault	**Hayward**	
Earthquake Magnitude (Mw)	**7.08**	
PGA (Amax)	**0.76**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**10**
Design Water Depth (feet)	**7**
Ave. Unit Weight Above GW (pcf)	**120**
Ave. Unit Weight Below GW (pcf)	**125**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 7 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.38 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.4 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.00 **L/H** 540.0

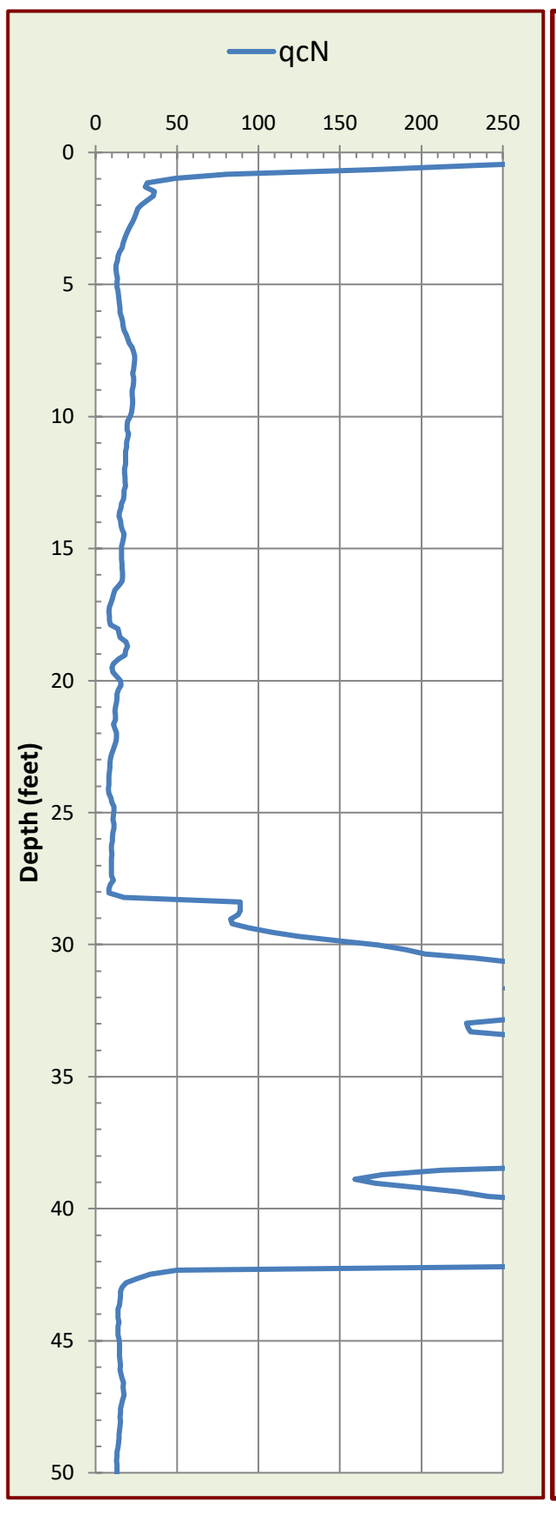
LDI[1] Corrected for Distance 0.00 (4 < L/H < 40)

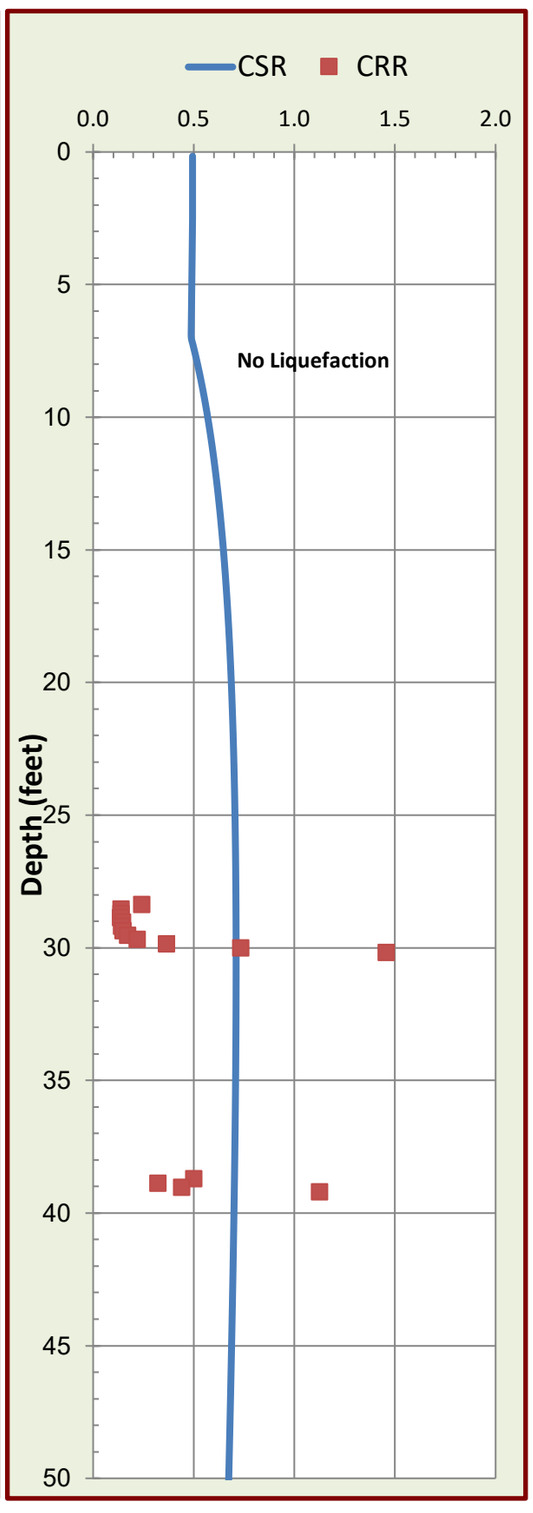
EXPECTED RANGE OF DISPLACEMENT

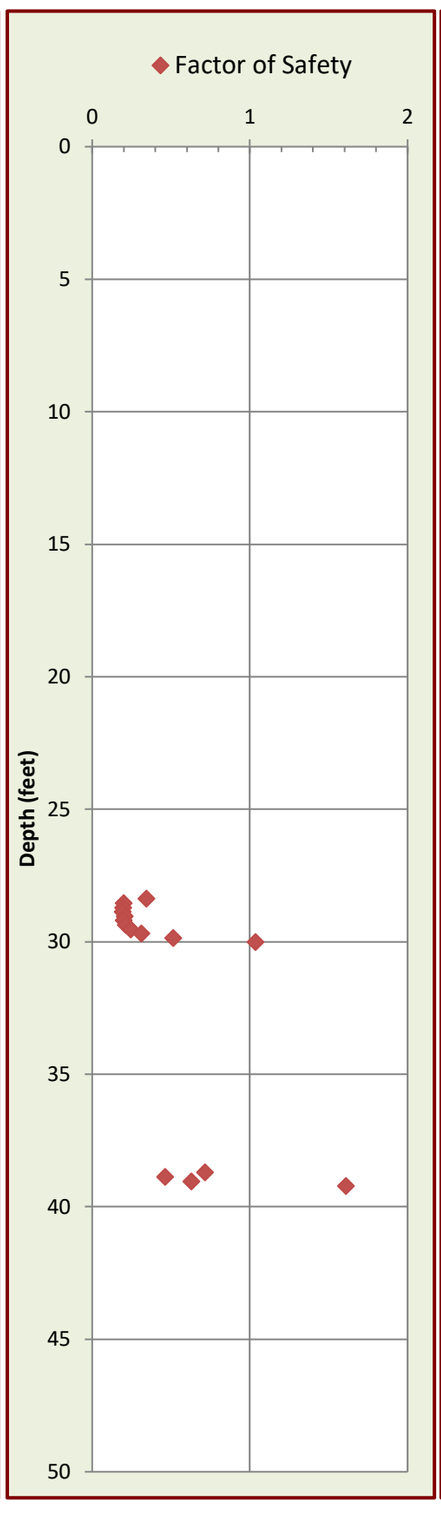
0.0 **to** **0.0** **feet**

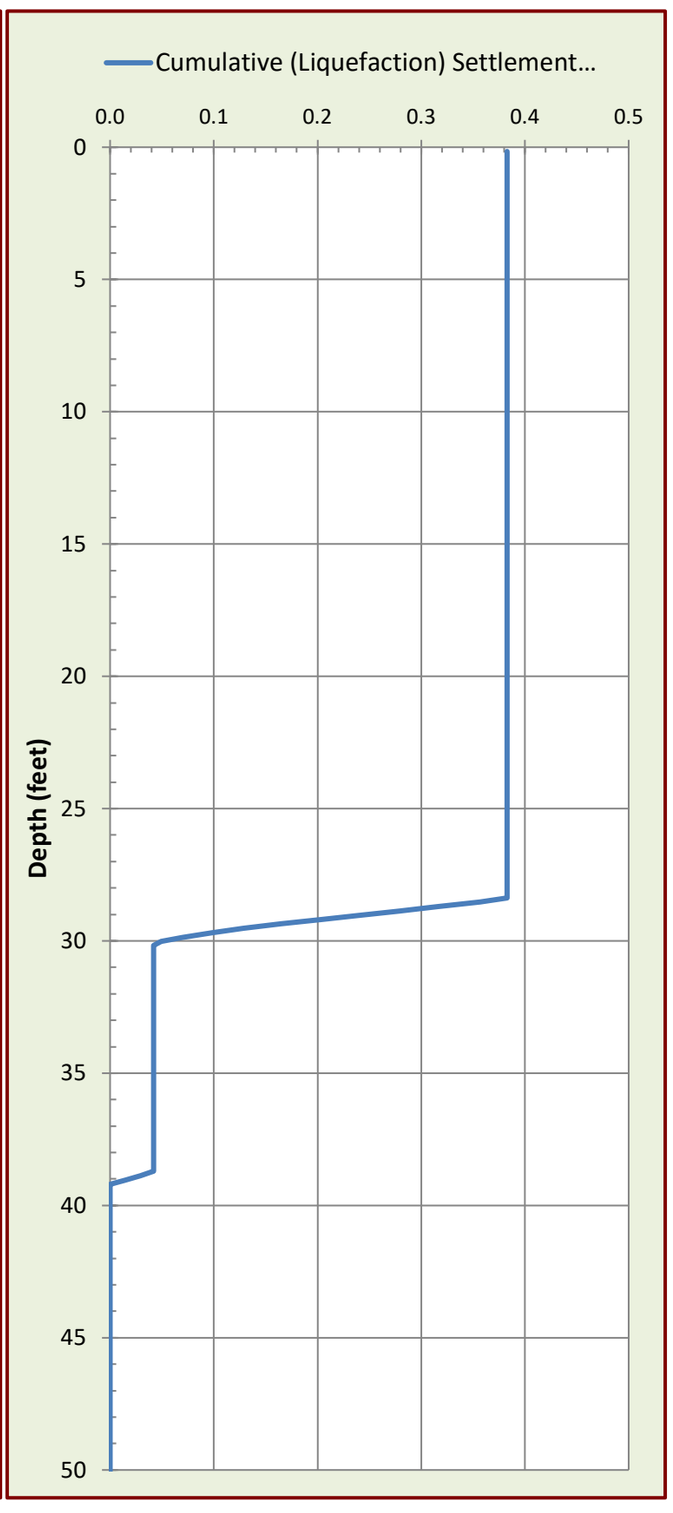
[1]Not Valid for L/H Values < 4 and > 40.

[2]LDI Values Only Summed to 2H Below Grade.









CORNERSTONE EARTH GROUP

FIGURE 4I

CPT NO. 9

© 2014 Cornerstone Earth Group, Inc.

PROJECT/CPT DATA

Project Title	**Montague Pointe Industrial**
Project No.	**1543-2-1**
Project Manager	**JDS**

SEISMIC PARAMETERS

Controlling Fault	**Hayward**	
Earthquake Magnitude (Mw)	**7.08**	
PGA (Amax)	**0.76**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**10**
Design Water Depth (feet)	**7**
Ave. Unit Weight Above GW (pcf)	**120**
Ave. Unit Weight Below GW (pcf)	**125**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 7 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.11 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.1 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.00 **L/H** 540.0

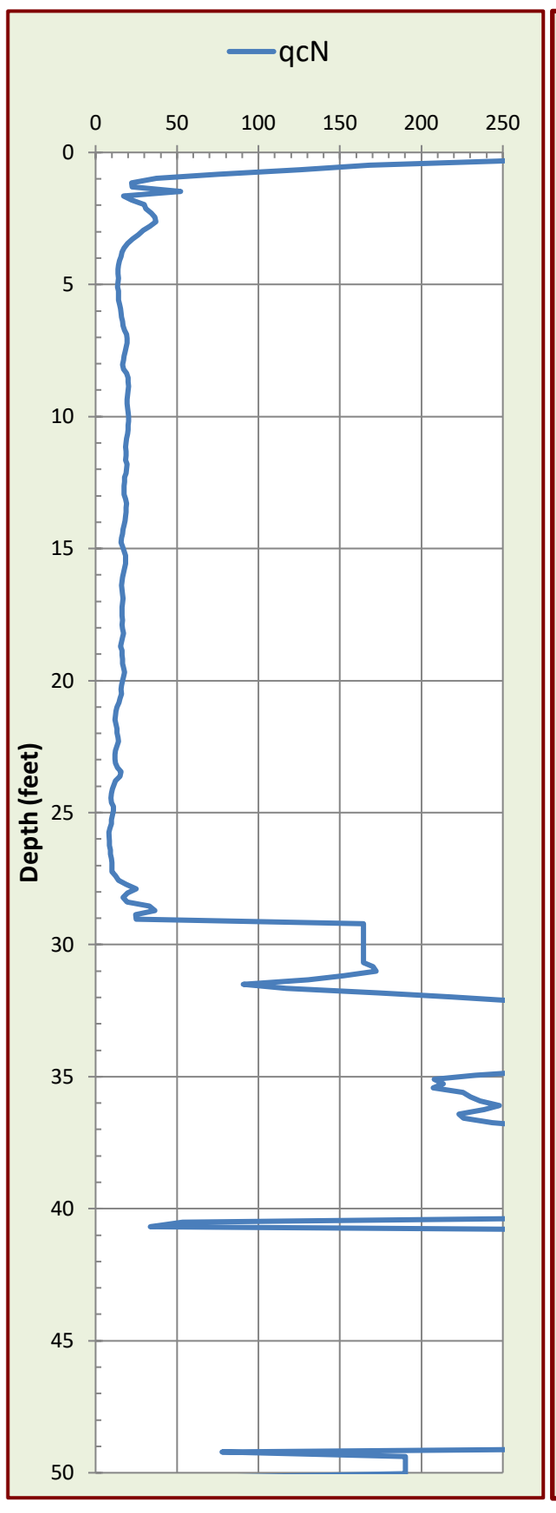
LDI[1] Corrected for Distance 0.00 (4 < L/H < 40)

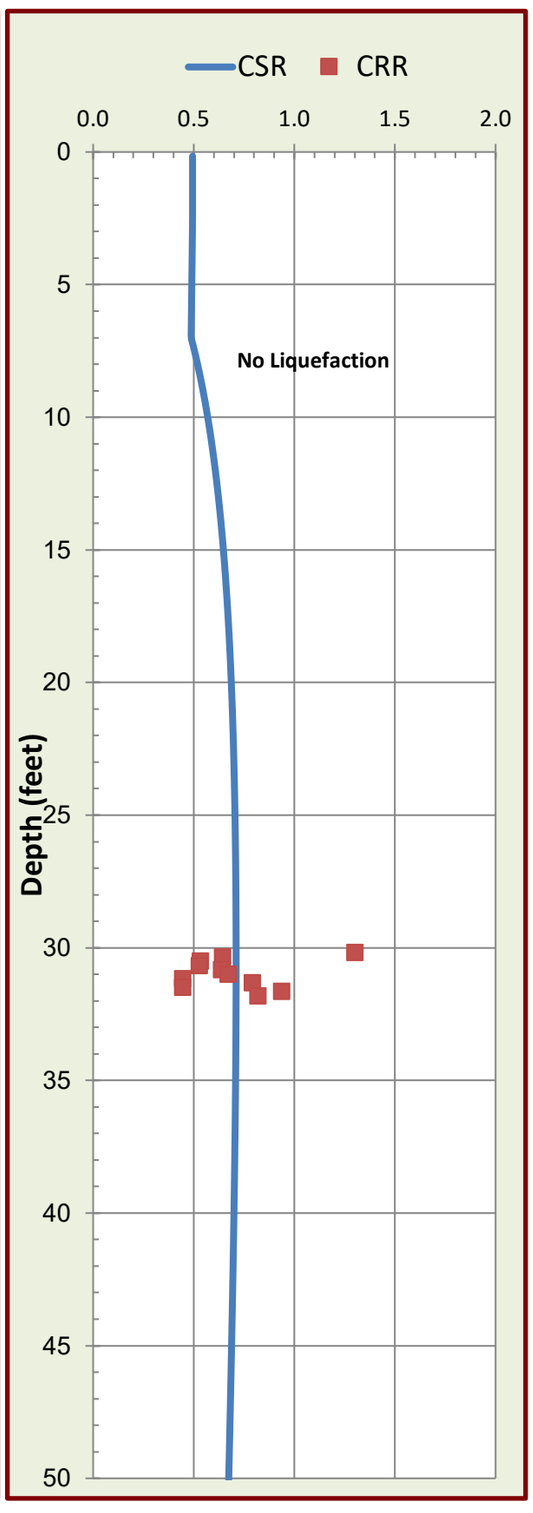
EXPECTED RANGE OF DISPLACEMENT

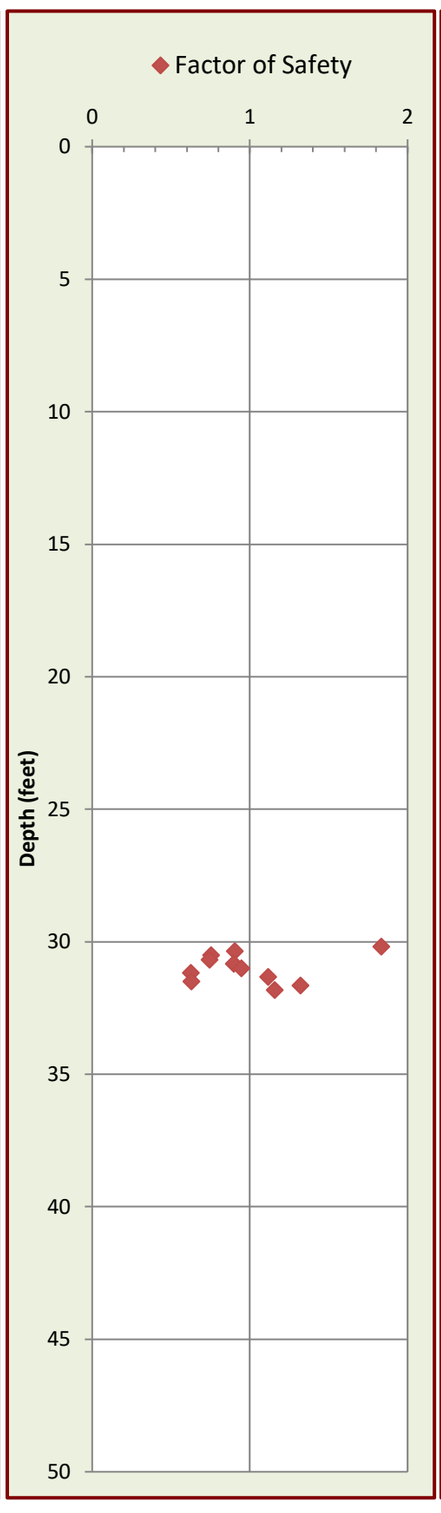
0.0 **to** **0.0** **feet**

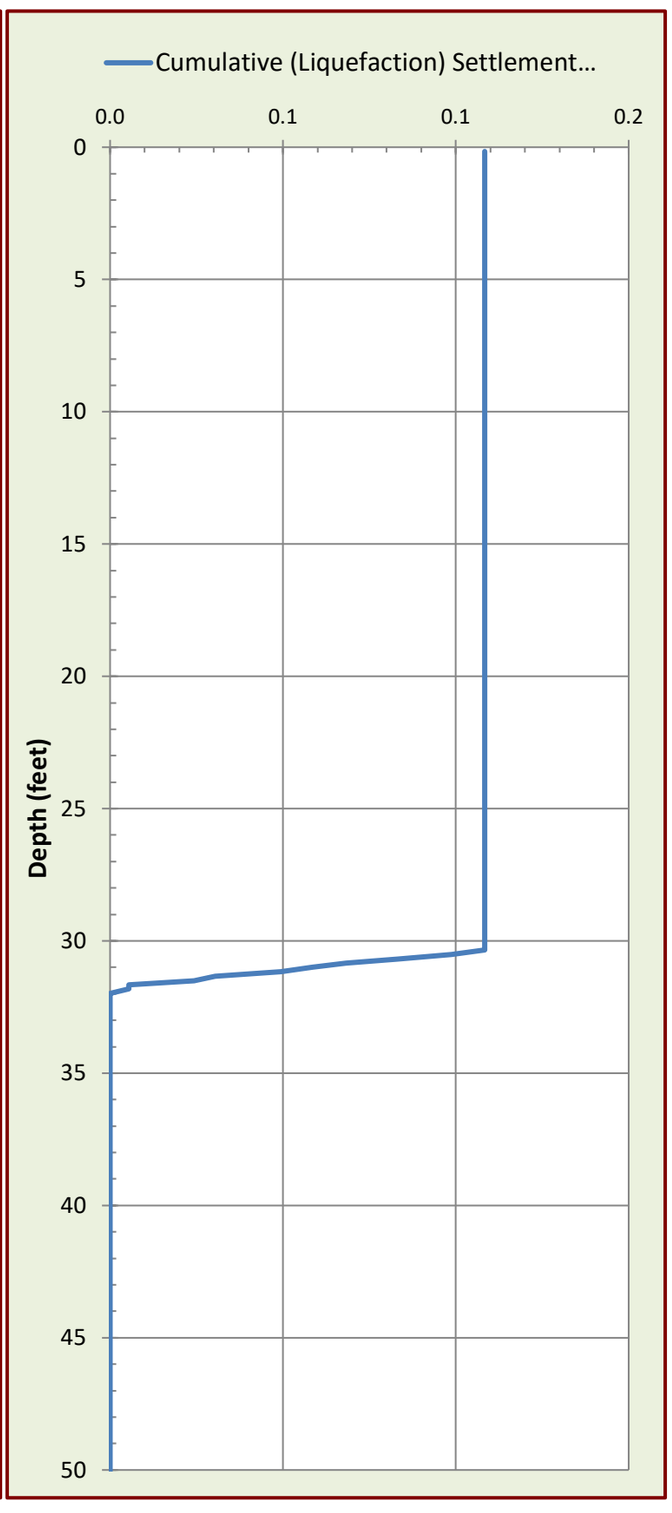
[1]Not Valid for L/H Values < 4 and > 40.

[2]LDI Values Only Summed to 2H Below Grade.









CORNERSTONE EARTH GROUP

FIGURE 4J

CPT NO. 10

© 2014 Cornerstone Earth Group, Inc.

PROJECT/CPT DATA

Project Title	**Montague Pointe Industrial**
Project No.	**1543-2-1**
Project Manager	**JDS**

SEISMIC PARAMETERS

Controlling Fault	**Hayward**	
Earthquake Magnitude (Mw)	**7.08**	
PGA (Amax)	**0.76**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**10**
Design Water Depth (feet)	**7**
Ave. Unit Weight Above GW (pcf)	**120**
Ave. Unit Weight Below GW (pcf)	**125**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 7 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.94 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.9 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.00 **L/H** 540.0

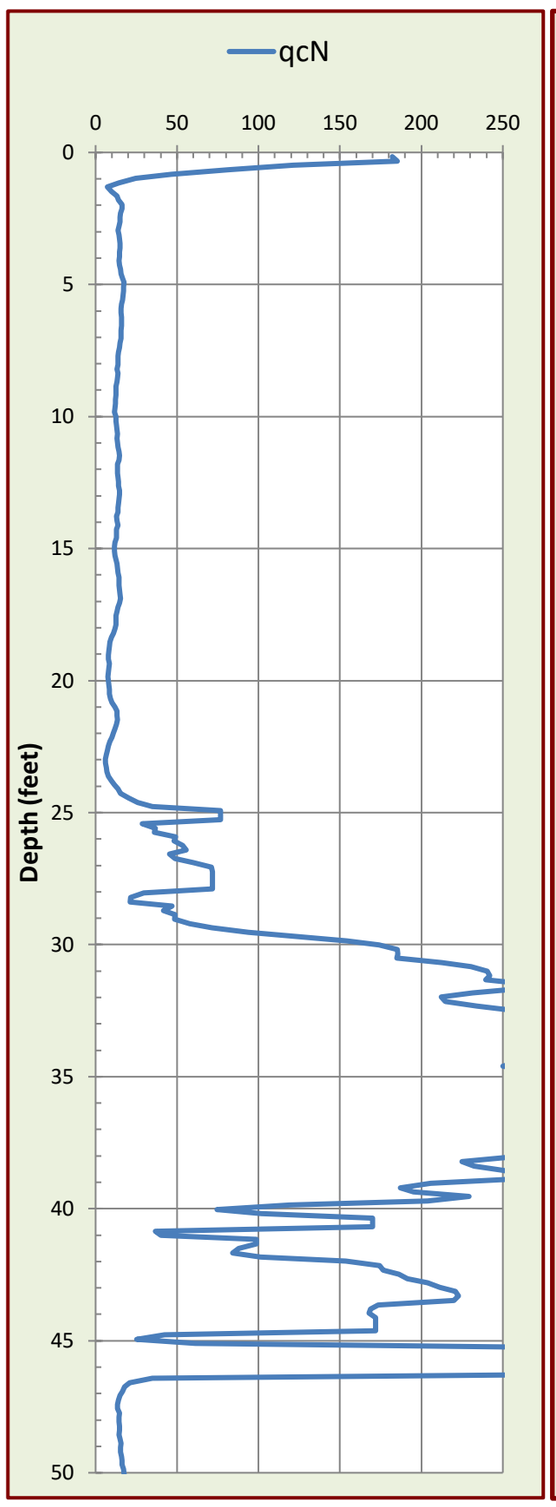
LDI[1] Corrected for Distance 0.00 (4 < L/H < 40)

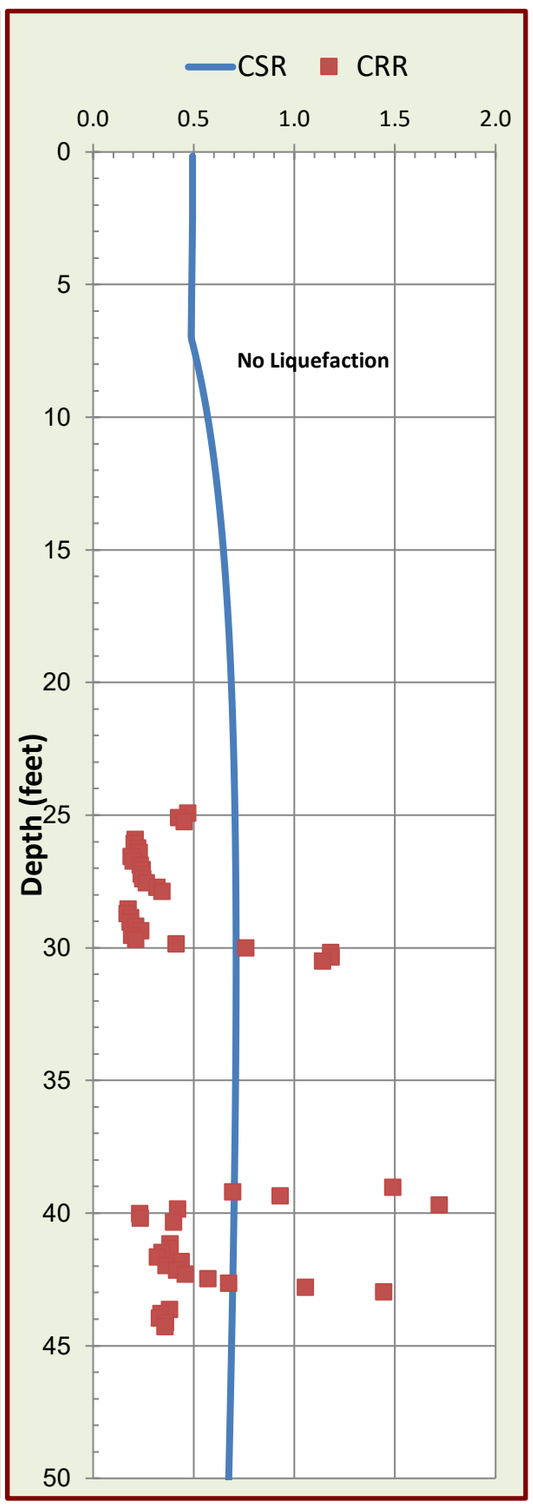
EXPECTED RANGE OF DISPLACEMENT

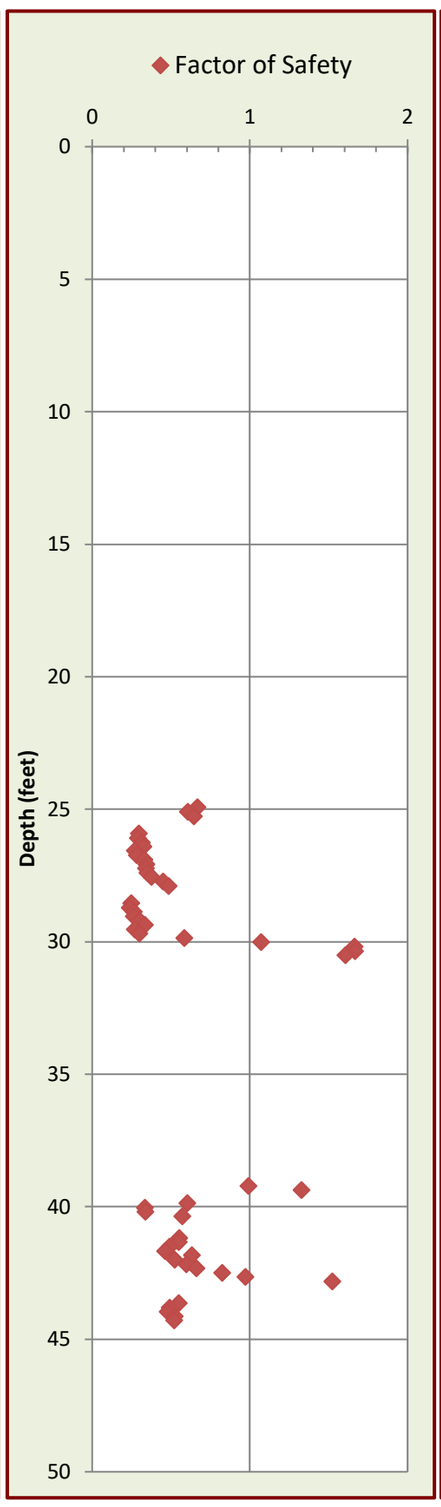
0.0 **to** **0.0** **feet**

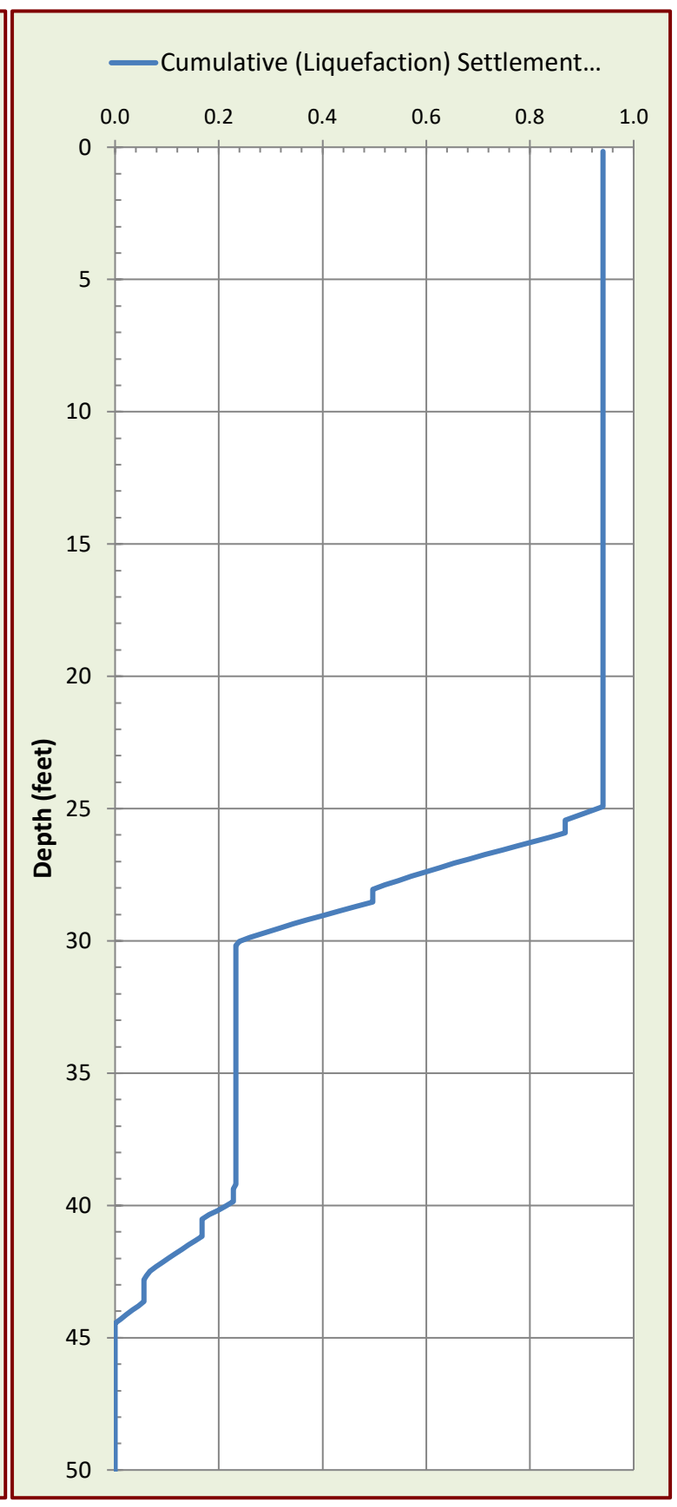
[1]Not Valid for L/H Values < 4 and > 40.

[2]LDI Values Only Summed to 2H Below Grade.









CORNERSTONE EARTH GROUP

FIGURE 4K

CPT NO. 11

© 2014 Cornerstone Earth Group, Inc.

PROJECT/CPT DATA

Project Title	**Montague Pointe Industrial**
Project No.	**1543-2-1**
Project Manager	**JDS**

SEISMIC PARAMETERS

Controlling Fault	**Hayward**	
Earthquake Magnitude (Mw)	**7.08**	
PGA (Amax)	**0.76**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**11.1**
Design Water Depth (feet)	**7**
Ave. Unit Weight Above GW (pcf)	**120**
Ave. Unit Weight Below GW (pcf)	**125**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 7 **FEET**

0.01 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

1.11 **(Inches)**

TOTAL SEISMIC SETTLEMENT 1.1 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 1.16 **L/H** 184.4

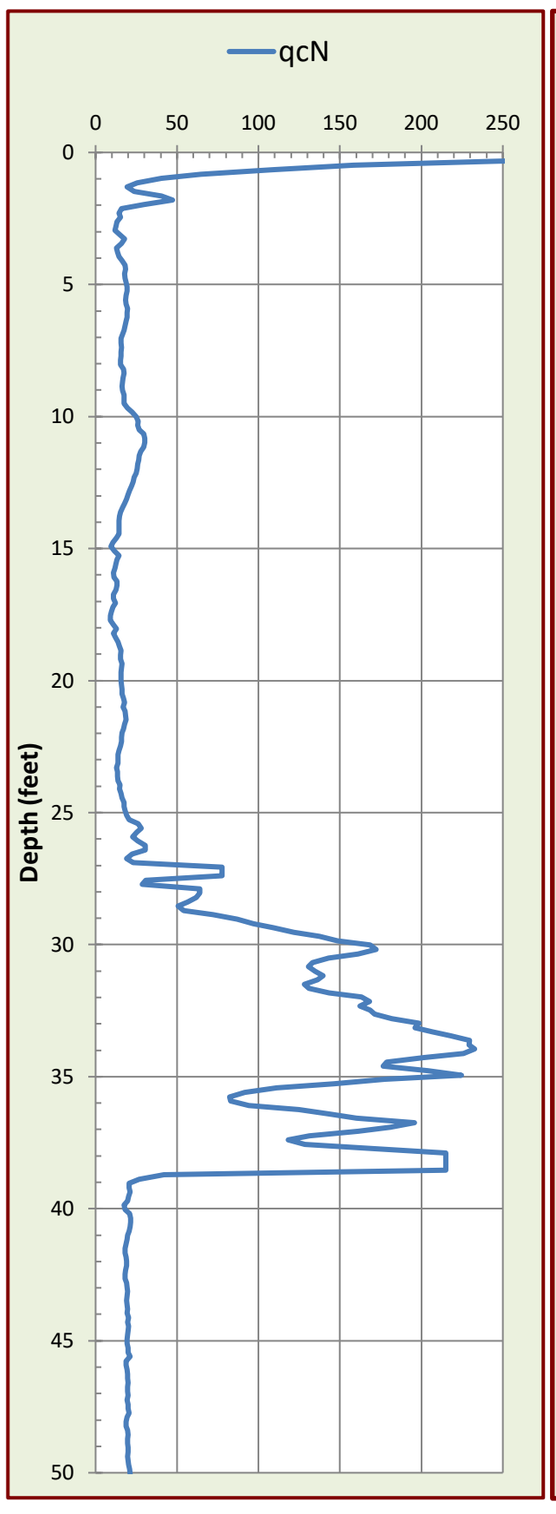
LDI[1] Corrected for Distance 0.11 (4 < L/H < 40)

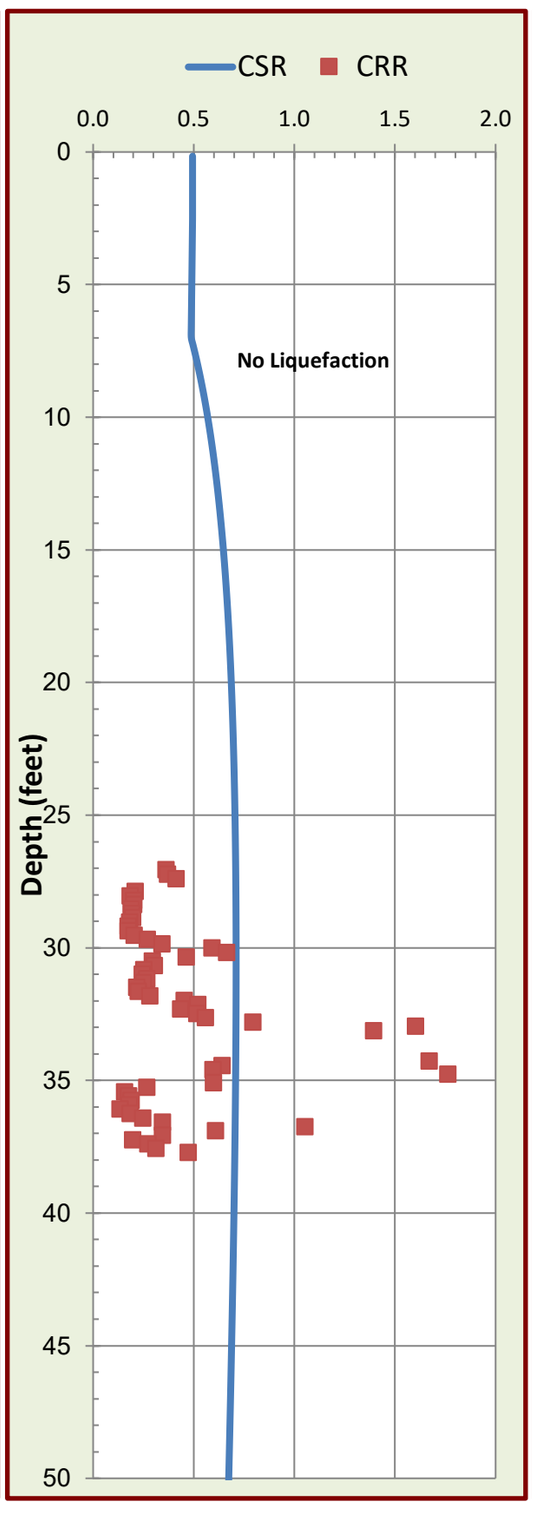
EXPECTED RANGE OF DISPLACEMENT

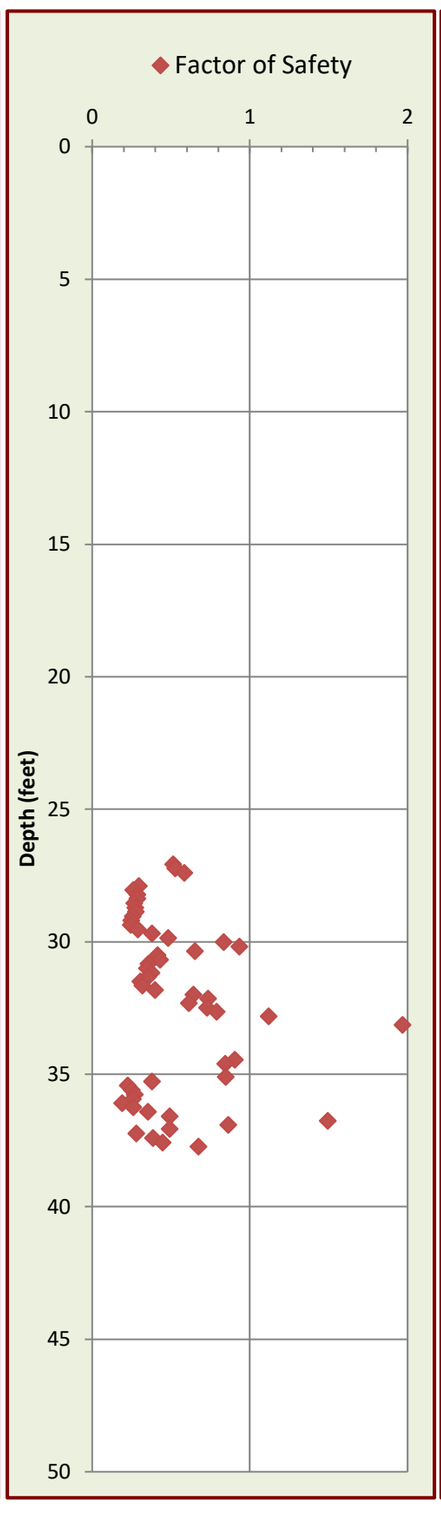
0.1 **to** **0.2** **feet**

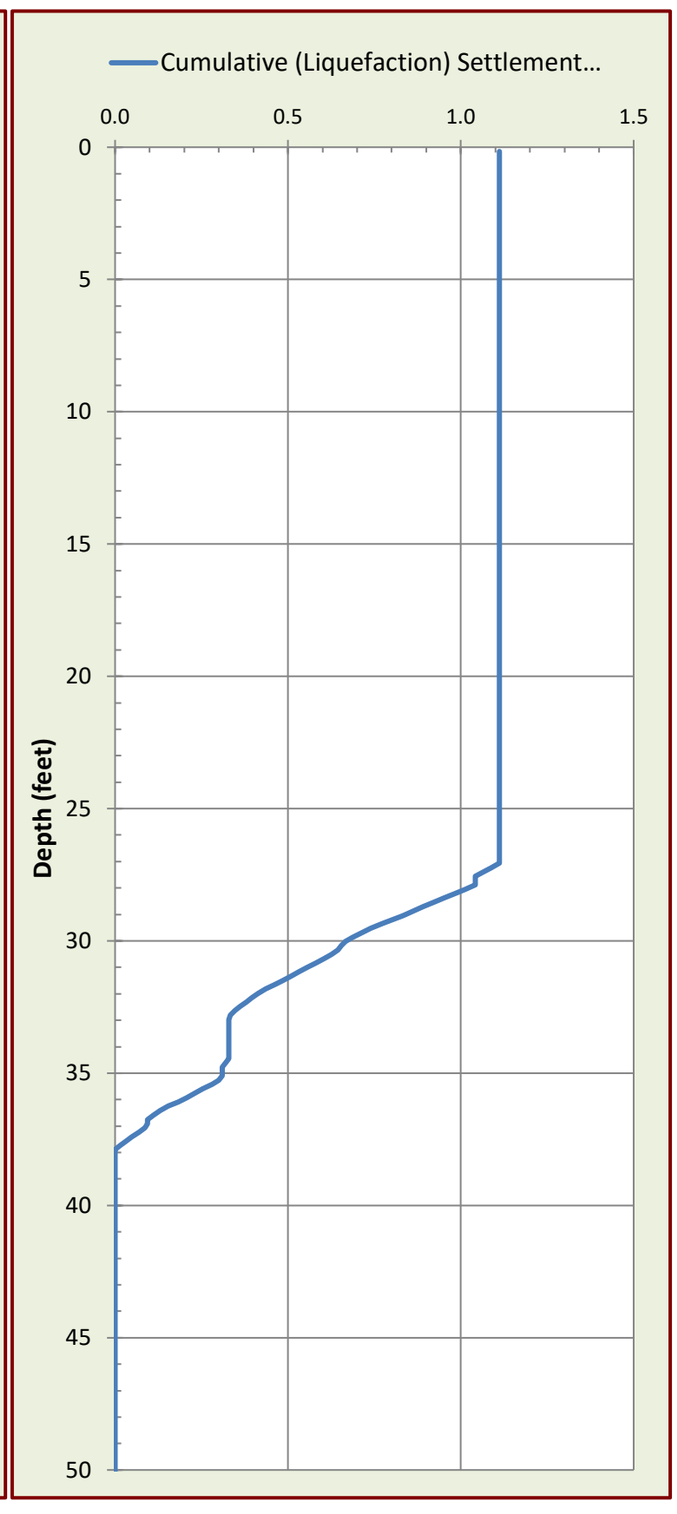
[1]Not Valid for L/H Values < 4 and > 40.

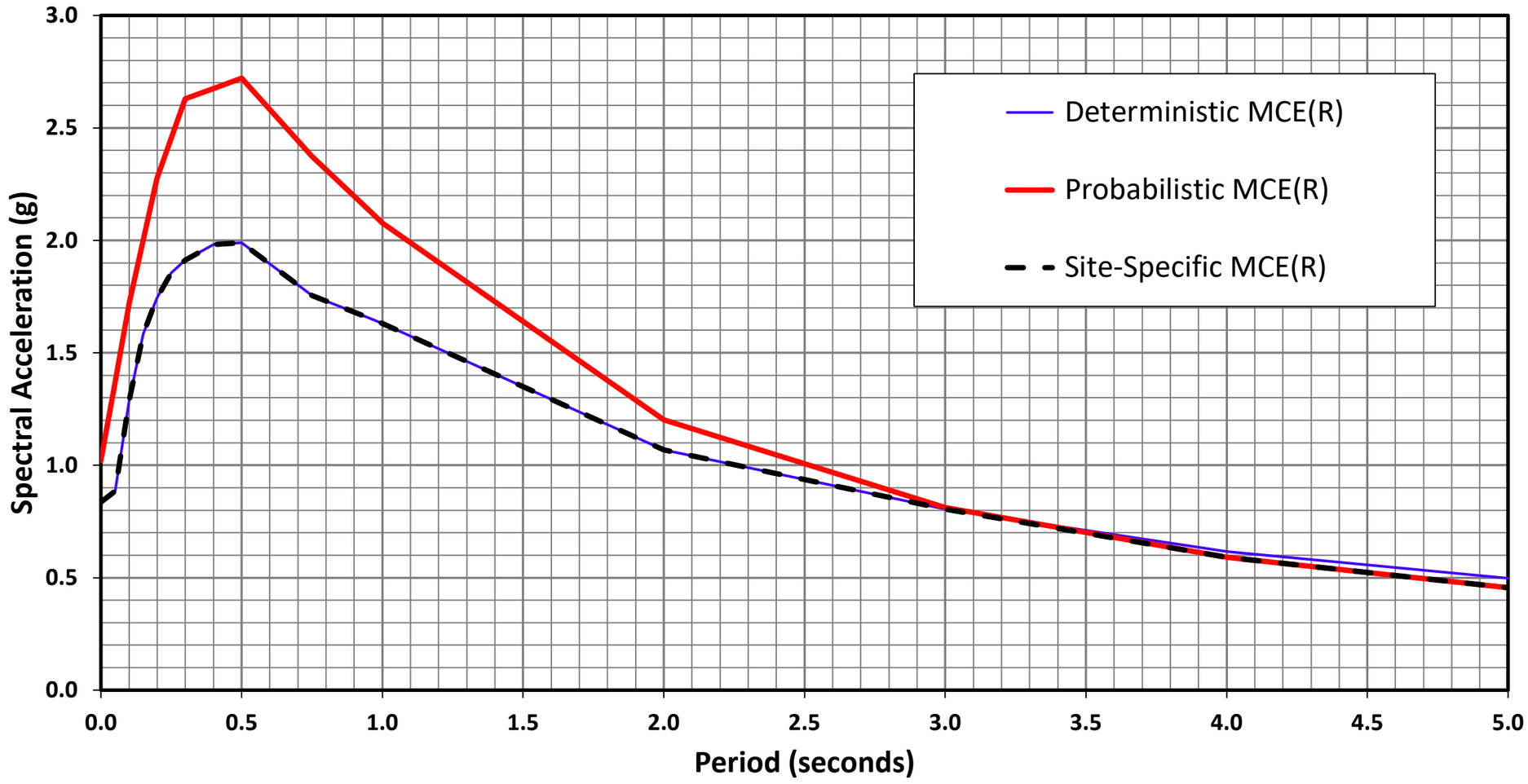
[2]LDI Values Only Summed to 2H Below Grade.











The Site-Specific Maximum Considered Earthquake (MCE_R) is defined as the lesser of the following at all periods:

- Deterministic MCE_R – maximum 84th percentile deterministic, or
- Probabilistic MCE_R – defined as the 2,475–year ground motion.

Site-Specific MCE_R	
Period (Seconds)	Spectral Acceleration (g)
0.00	0.836
0.05	0.885
0.10	1.288
0.15	1.582
0.19	1.723
0.20	1.746
0.25	1.856
0.30	1.912
0.40	1.982
0.50	1.989
0.75	1.754
0.97	1.648
1.00	1.630
2.00	1.068
3.00	0.804
4.00	0.590
5.00	0.456

References:

ASCE/SEI 7-16: Minimum Design Loads and Associated Criteria for Buildings and Other Strutures with Supplement No. 1.
2022 California Building Code, Title 24, Part 2, Volume 2

CORNERSTONE EARTH GROUP

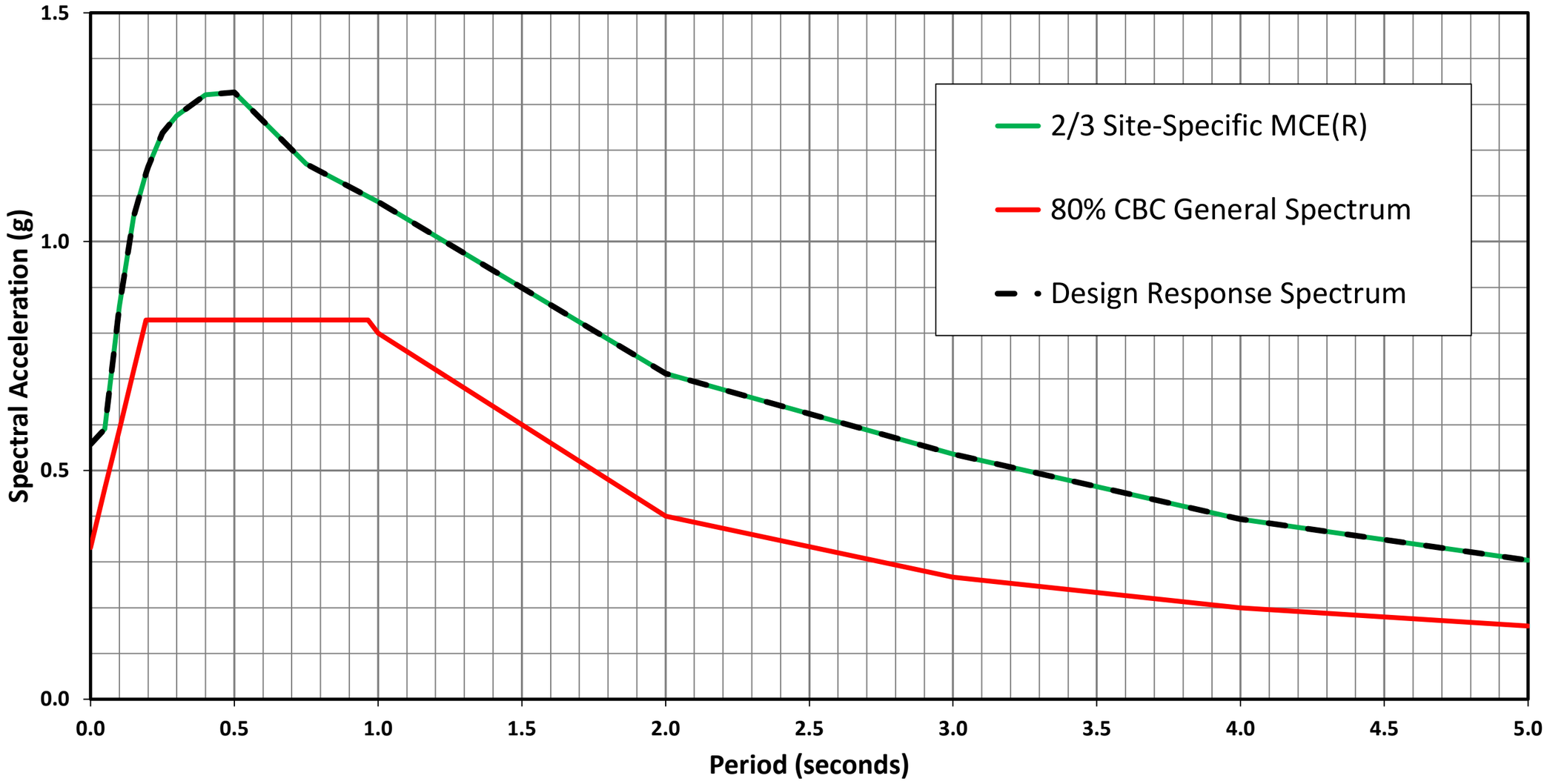
MCE_R RESPONSE SPECTRA

Montague Pointe Industrial
Zanker Rd and Montague Expwy
San Jose, CA

FIGURE 5

PROJECT NO. 1543-2-1

May 2025 | JDS



The Site-Specific Design Response Spectrum per Section 21.2, 21.3 and 21.4 of ASCE 7-16 is defined as the greater of the following at all periods:

- 2/3 of the Site-Specific MCE_R, or
- 80% of the CBC General Spectrum.

Design Response Spectra	
Period (Seconds)	Spectral Acceleration (g)
0.00	0.557
0.05	0.590
0.10	0.858
0.15	1.055
0.19	1.149
0.20	1.164
0.25	1.237
0.30	1.275
0.40	1.321
0.50	1.326
0.75	1.169
0.97	1.098
1.00	1.087
2.00	0.712
3.00	0.536
4.00	0.394
5.00	0.304

Site Design	Design Values
Site Class (Per Chapter 20 ASCE 7-16)	D
Shear Wave Velocity, V_{S30} (m/sec)	240
Site Latitude (degrees)	37.398655
Site Longitude (degrees)	-121.927685
Risk Category	II
Building Period (sec)	Unknown
Importance Factor, I_e	1
[1]Site Specific PGA_M (g)	0.76

Design Acceleration Parameters[1]	
S_{DS}	1.194
S_{D1}	1.608
S_{MS}	1.790
S_{M1}	2.412

[1] Lower of Deterministic and Probabilistic, but not less than 80% of mapped value of FM x PGA, determined in accordance with Section 21.5 of ASCE 7-16.

References:

ASCE/SEI 7-16: Minimum Design Loads and Associated Criteria for Buildings and Other Strutures with Supplement No. 1.
2022 California Building Code, Title 24, Part 2, Volume 2

CORNERSTONE EARTH GROUP

DESIGN RESPONSE SPECTRA

Montague Pointe Industrial
Zanker Rd and Montague Expwy
San Jose, CA

FIGURE 6

PROJECT NO. 1543-2-1

May 2025 | JDS

APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, hollow-stem auger drilling equipment and 20-ton truck-mounted Cone Penetration Test equipment. Eight 8-inch-diameter exploratory borings were drilled on March 21 and 25, 2025 to depths of 25 to 45 feet. Eleven CPT soundings were also performed in accordance with ASTM D 5778-95 (revised, 2002) on March 13 and 20, 2025, to depths ranging from approximately 50 to 110 feet. The approximate locations of exploratory borings and CPTs are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Boring logs, as well as a key to the classification of the soil and bedrock, are included as part of this appendix.

Boring locations were approximated using existing site boundaries, a hand-held GPS unit, and other site features as references. Boring elevations were based on elevations obtained from Google Earth. The locations and elevations of the borings should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. The standard penetration resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The 2-inch O.D. split-spoon sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration (ASTM D1586). 2.5-inch I.D. samples were obtained using a Modified California Sampler driven into the soil with the 140-pound hammer previously described. Relatively undisturbed samples were also obtained with 2.875-inch I.D. Shelby Tube sampler which were hydraulically pushed. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows required to drive the last 12 inches. The various samplers are denoted at the appropriate depth on the boring logs.

The CPT involved advancing an instrumented cone-tipped probe into the ground while simultaneously recording the resistance at the cone tip (q_c) and along the friction sleeve (f_s) at approximately 5-centimeter intervals. Based on the tip resistance and tip to sleeve ratio (R_f), the CPT classified the soil behavior type and estimated engineering properties of the soil, such as equivalent Standard Penetration Test (SPT) blow count, internal friction angle within sand layers, and undrained shear strength in silts and clays. A pressure transducer behind the tip of the CPT cone measured pore water pressure (u_2). Graphical logs of the CPT data is included as part of this appendix.

Field tests included an evaluation of the unconfined compressive strength of the soil samples using a pocket penetrometer device. The results of these tests are presented on the individual boring logs at the appropriate sample depths.

Attached boring and CPT logs and related information depict subsurface conditions at the locations indicated and on the date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these boring and CPT locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition,

any stratification lines on the logs represent the approximate boundary between soil types and the transition may be gradual.

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-10)

MATERIAL TYPES	CRITERIA FOR ASSIGNING SOIL GROUP NAMES			GROUP SYMBOL	SOIL GROUP NAMES & LEGEND
COARSE-GRAINED SOILS >50% RETAINED ON NO. 200 SIEVE	GRAVELS >50% OF COARSE FRACTION RETAINED ON NO 4. SIEVE	CLEAN GRAVELS <5% FINES	Cu>4 AND 1<Cc<3	GW	WELL-GRADED GRAVEL
			Cu>4 AND 1>Cc>3	GP	POORLY-GRADED GRAVEL
		GRAVELS WITH FINES >12% FINES	FINES CLASSIFY AS ML OR CL	GM	SILTY GRAVEL
			FINES CLASSIFY AS CL OR CH	GC	CLAYEY GRAVEL
	SANDS >50% OF COARSE FRACTION PASSES ON NO 4. SIEVE	CLEAN SANDS <5% FINES	Cu>6 AND 1<Cc<3	SW	WELL-GRADED SAND
			Cu>6 AND 1>Cc>3	SP	POORLY-GRADED SAND
		SANDS AND FINES >12% FINES	FINES CLASSIFY AS ML OR CL	SM	SILTY SAND
			FINES CLASSIFY AS CL OR CH	SC	CLAYEY SAND
FINE-GRAINED SOILS >50% PASSES NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT<50	INORGANIC	PI>7 AND PLOTS>"A" LINE	CL	LEAN CLAY
			PI>4 AND PLOTS<"A" LINE	ML	SILT
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OL	ORGANIC CLAY OR SILT
	SILTS AND CLAYS LIQUID LIMIT>50	INORGANIC	PI PLOTS >"A" LINE	CH	FAT CLAY
			PI PLOTS <"A" LINE	MH	ELASTIC SILT
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OH	ORGANIC CLAY OR SILT
HIGHLY ORGANIC SOILS		PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC ODOR		PT	PEAT

OTHER MATERIAL SYMBOLS

- Poorly-Graded Sand with Clay
- Clayey Sand
- Sandy Silt
- Artificial/Undocumented Fill
- Poorly-Graded Gravelly Sand
- Topsoil
- Well-Graded Gravel with Clay
- Well-Graded Gravel with Silt
- Sand
- Silt
- Well Graded Gravelly Sand
- Gravelly Silt
- Asphalt
- Boulders and Cobble

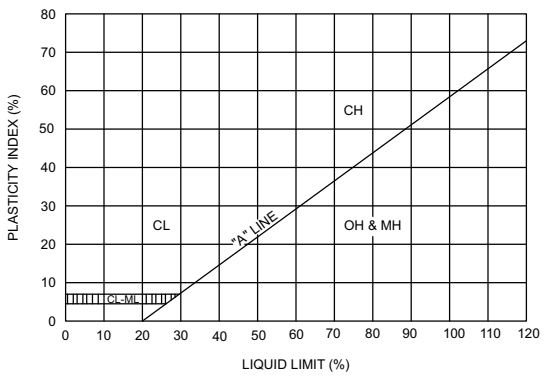
SAMPLER TYPES

- SPT
- Modified California (2.5" I.D.)
- Rock Core
- Shelby Tube
- No Recovery
- Grab Sample

ADDITIONAL TESTS

CA	-	CHEMICAL ANALYSIS (CORROSIVITY)	PI	-	PLASTICITY INDEX
CD	-	CONSOLIDATED DRAINED TRIAXIAL	SW	-	SWELL TEST
CN	-	CONSOLIDATION	TC	-	CYCLIC TRIAXIAL
CU	-	CONSOLIDATED UNDRAINED TRIAXIAL	TV	-	TORVANE SHEAR
DS	-	DIRECT SHEAR	UC	-	UNCONFINED COMPRESSION
PP	-	POCKET PENETROMETER (TSF)	(1.5)	-	(WITH SHEAR STRENGTH IN KSF)
(3.0)	-	(WITH SHEAR STRENGTH IN KSF)			
RV	-	R-VALUE	UU	-	UNCONSOLIDATED UNDRAINED TRIAXIAL
SA	-	SIEVE ANALYSIS: % PASSING #200 SIEVE			
▼	-	WATER LEVEL			

PLASTICITY CHART



PENETRATION RESISTANCE
(RECORDED AS BLOWS / FOOT)

SAND & GRAVEL		SILT & CLAY		
RELATIVE DENSITY	BLOWS/FOOT*	CONSISTENCY	BLOWS/FOOT*	STRENGTH** (KSF)
VERY LOOSE	0 - 4	VERY SOFT	0 - 2	0 - 0.25
LOOSE	4 - 10	SOFT	2 - 4	0.25 - 0.5
MEDIUM DENSE	10 - 30	MEDIUM STIFF	4 - 8	0.5-1.0
DENSE	30 - 50	STIFF	8 - 15	1.0 - 2.0
VERY DENSE	OVER 50	VERY STIFF	15 - 30	2.0 - 4.0
		HARD	OVER 30	OVER 4.0

* NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1-3/8 INCH I.D.) SPLIT-BARREL SAMPLER THE LAST 12 INCHES OF AN 18-INCH DRIVE (ASTM-1586 STANDARD PENETRATION TEST).

** UNDRAINED SHEAR STRENGTH IN KIPS/SQ. FT. AS DETERMINED BY LABORATORY TESTING OR APPROXIMATED BY THE STANDARD PENETRATION TEST, POCKET PENETROMETER, TORVANE, OR VISUAL OBSERVATION.

CORNERSTONE EARTH GROUP

LEGEND TO SOIL DESCRIPTIONS

Figure Number A-1

BORING NUMBER EB-1

CORNERSTONE EARTH GROUP

PROJECT NAME Montague Point Industrial
PROJECT NUMBER 1543-2-1
PROJECT LOCATION 3050 Zanker Road, San Jose, CA

DATE STARTED 3/21/25 **DATE COMPLETED** 3/21/25
GROUND ELEVATION 15 FT +/- **BORING DEPTH** 25 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc. **LATITUDE** 37.398578° **LONGITUDE** -121.931141°
DRILLING METHOD Mobile B-53B, 8 inch Hollow-Stem Auger **GROUND WATER LEVELS:**
LOGGED BY RA ▽ **AT TIME OF DRILLING** 19 ft.
NOTES ▼ **AT END OF DRILLING** 19 ft.

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
15.0	0	2 inches asphalt concrete over 5 inches aggregate base							
14.4		**Sandy Lean Clay with Gravel (CL) [Fill]** hard, moist, brown, fine to coarse sand, fine to coarse gravel, low plasticity	14	MC-1B	100	24			>4.5
12.5		**Lean Clay with Sand (CL)** very stiff, moist, dark gray to gray, fine sand, moderate plasticity	22	MC-2B	98	24			
	5		20	MC-3B	94	26			
8.0		**Lean Clay (CL)** very stiff, moist, brown with gray mottles, some fine sand, moderate plasticity	30	MC					
	10								
	15		18	MC-5B	102	25			
-2.0		**Lean Clay with Sand (CL)** very stiff, moist, gray with brown mottles, fine sand, moderate plasticity	18	MC					
	20								
-7.0		**Sandy Lean Clay (CL)** medium stiff, moist, brown, fine sand, low plasticity	13	MC-7B	105	20			
-10.0	25	Bottom of Boring at 25.0 feet.							

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 5/1/25 09:42 - P:\DRAFTING\GINT FILES\1543-2-1 MONTAGUE POINT IND.GPJ

BORING NUMBER EB-2

CORNERSTONE EARTH GROUP

PROJECT NAME Montague Point Industrial
PROJECT NUMBER 1543-2-1
PROJECT LOCATION 3050 Zanker Road, San Jose, CA

DATE STARTED 3/21/25 **DATE COMPLETED** 3/21/25
GROUND ELEVATION 15 FT +/- **BORING DEPTH** 31.5 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc. **LATITUDE** 37.398069° **LONGITUDE** -121.930258°
DRILLING METHOD Mobile B-53B, 8 inch Hollow-Stem Auger **GROUND WATER LEVELS:**
LOGGED BY RA ▽ **AT TIME OF DRILLING** 14 ft.
NOTES ▼ **AT END OF DRILLING** 14 ft.

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
15.0	0	2 inches asphalt concrete over 5 inches aggregate base						
14.4		**Lean Clay with Sand (CL)** very stiff, moist, dark gray to gray, fine sand, moderate plasticity Liquid Limit = 42, Plastic Limit = 20	22	MC-1B	101	23	22	
			24	MC-2B	101	23		
10.0	5	**Lean Clay (CL)** very stiff, moist, brown with gray mottles, some fine sand, moderate plasticity	26	MC-3B	103	23		
			42	MC-4B	103	23		
2.0	10	**Lean Clay with Sand (CL)** medium stiff, moist, brown with gray mottles, fine sand, moderate plasticity		ST				
	15		14	MC-6B	95	29		
	20	becomes stiff		ST-7	92	25		
-8.0		**Silty Sand (SM)** loose, wet, brown with gray mottles, fine sand						
	25		13	MC-8B	101	24		41
-11.0		*Continued Next Page*						

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 5/1/25 09:42 - P:\DRAFTING\GINT FILES\1543-2-1 MONTAGUE POINT IND.GPJ

BORING NUMBER EB-2

CORNERSTONE EARTH GROUP

PROJECT NAME Montague Point Industrial

PROJECT NUMBER 1543-2-1

PROJECT LOCATION 3050 Zanker Road, San Jose, CA

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
-11.0			**Silty Sand (SM)** medium dense, wet, brown with gray mottles, fine sand						
				12	SPT-9		26		36
	30		becomes dense	31	SPT-10		25		
-16.5			Bottom of Boring at 31.5 feet.						
	35								
	40								
	45								
	50								
	55								

UNDRAINED SHEAR STRENGTH, ksf

- ○ HAND PENETROMETER
- △ TORVANE
- ● UNCONFINED COMPRESSION
- ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL

1.0 2.0 3.0 4.0

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 5/1/25 09:42 - P:\DRAFTING\GINT FILES\1543-2-1 MONTAGUE POINT IND.GPJ

BORING NUMBER EB-3

CORNERSTONE EARTH GROUP

PROJECT NAME Montague Point Industrial
PROJECT NUMBER 1543-2-1
PROJECT LOCATION 3050 Zanker Road, San Jose, CA

DATE STARTED 3/21/25 **DATE COMPLETED** 3/21/25
GROUND ELEVATION 16 FT +/- **BORING DEPTH** 40 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.398936° **LONGITUDE** -121.929783°
DRILLING METHOD Mobile B-53B, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY RA
AT TIME OF DRILLING 13 ft.
NOTES
AT END OF DRILLING Not Encountered

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
16.0	0	3 inches asphalt concrete over 3 inches aggregate base						
15.7								
15.4		**Sandy Lean Clay (CL) [Fill]** very stiff, moist, brown, fine to coarse sand, some fine to coarse gravel, low plasticity	23	MC-1B	100	21		
13.5		**Lean Clay with Sand (CL)** very stiff, moist, dark gray to gray, fine sand, moderate plasticity	26	MC-2B	101	23		
11.0	5	**Lean Clay (CL)** very stiff, moist, brown with gray mottles, some fine sand, moderate plasticity	35	MC				
			23	MC-4B	103	23		
	10							
3.0		**Lean Clay with Sand (CL)** very stiff, moist, gray with brown mottles, fine sand, moderate plasticity	18	MC-5B	107	21		
	15							
		becomes stiff	10	MC-6B	93	31		
	20							
-7.0		**Sandy Lean Clay (CL)** stiff, moist, brown, fine sand, low plasticity	21	MC				
-9.0	25	**Clayey Sand (SC)** medium dense, moist, brown, fine sand	19	SPT-8		25		29
-10.0		*Continued Next Page*						

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 5/1/25 09:42 - P:\DRAFTING\GINT FILES\1543-2-1 MONTAGUE POINT IND.GPJ

BORING NUMBER EB-3

CORNERSTONE EARTH GROUP

PROJECT NAME Montague Point Industrial

PROJECT NUMBER 1543-2-1

PROJECT LOCATION 3050 Zanker Road, San Jose, CA

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
-10.0								
-11.0		**Silty Sand (SM)** dense, moist, brown, fine sand	32	SPT				
-13.0		**Poorly Graded Sand with Silt (SP-SM)** medium dense, wet, brown, fine sand	12	SPT-10		22		5
-15.5		**Poorly Graded Sand with Silt (SP-SM)** dense, wet, gray, fine to coarse sand, some fine subangular to subrounded gravel	40	SPT				
-19.0	35	**Poorly Graded Sand (SP)** medium dense, wet, gray, fine to coarse sand	21	SPT-12A		19		3
		becomes very dense, some fine subangular to subrounded gravel	54	SPT				
		becomes medium dense	45	MC-14B	123	8		4
-24.0	40	Bottom of Boring at 40.0 feet.						
	45							
	50							
	55							

UNDRAINED SHEAR STRENGTH, ksf
- ○ HAND PENETROMETER
- △ TORVANE
- ● UNCONFINED COMPRESSION
- ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL

1.0 2.0 3.0 4.0

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 5/1/25 09:42 - P:\DRAFTING\GINT FILES\1543-2-1 MONTAGUE POINT IND.GPJ

BORING NUMBER EB-4

CORNERSTONE EARTH GROUP

PROJECT NAME Montague Point Industrial
PROJECT NUMBER 1543-2-1
PROJECT LOCATION 3050 Zanker Road, San Jose, CA

DATE STARTED 3/21/25 **DATE COMPLETED** 3/21/25
GROUND ELEVATION 15 FT +/- **BORING DEPTH** 34.5 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.398663° **LONGITUDE** -121.928753°
DRILLING METHOD Mobile B-53B, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY RA
AT TIME OF DRILLING 10 ft.
NOTES
AT END OF DRILLING Not Encountered

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
15.0	0	2 inches asphalt concrete over 6 inches aggregate base						
14.8								
14.3		**Lean Clay with Gravel (CL) [Fill]** very stiff, moist, gray and dark gray mottled, fine to coarse subangular gravel, moderate plasticity	18	MC-1B	100	24		
12.8		**Lean Clay with Sand (CL)** very stiff, moist, dark gray to gray, fine sand, moderate plasticity	21	MC				
10.0	5	**Lean Clay (CL)** very stiff, moist, brown with gray mottles, some fine sand, moderate plasticity	23	MC-3B	100	24		
		becomes hard	38	MC-4B	106	21		
	10							
3.0		**Lean Clay with Sand (CL)** medium stiff, moist, gray with brown mottles, fine sand, moderate plasticity		ST-5	97	24		
0.0	15	**Sandy Lean Clay (CL)** medium stiff, moist, brown with gray mottles, fine sand, low to moderate plasticity	15	MC-6B	103	22		
		becomes stiff	20	MC				
	20							
-8.5		**Silty Sand (SM)** medium dense, wet, brown with gray mottles, fine sand	15	SPT-8B		26		66
	25		21	SPT-9		27		27
-11.0								

Continued Next Page

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 5/1/25 09:42 - P:\DRAFTING\GINT FILES\1543-2-1 MONTAGUE POINT IND.GPJ

BORING NUMBER EB-4

CORNERSTONE EARTH GROUP

PROJECT NAME Montague Point Industrial

PROJECT NUMBER 1543-2-1

PROJECT LOCATION 3050 Zanker Road, San Jose, CA

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
-11.0								
-11.5		**Lean Clay with Sand (CL)** stiff, moist, brown with gray mottles, fine sand, low plasticity	6	SPT-10B		25		
-13.5		**Clayey Sand (SC)** loose, wet, brown, fine sand	11	MC-11B	100	25		46
	30		9	SPT-12		29		
-17.0		**Lean Clay with Sand (CL)** very stiff, moist, brown with gray mottles, fine sand, low plasticity	24	MC-13B	98	26		
-19.5		Bottom of Boring at 34.5 feet.						
	35							
	40							
	45							
	50							
	55							

UNDRAINED SHEAR STRENGTH, ksf
- ○ HAND PENETROMETER
- △ TORVANE
- ● UNCONFINED COMPRESSION
- ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL

1.0 2.0 3.0 4.0

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 5/1/25 09:42 - P:\DRAFTING\GINT FILES\1543-2-1 MONTAGUE POINT IND.GPJ

BORING NUMBER EB-5

CORNERSTONE EARTH GROUP

PROJECT NAME Montague Point Industrial
PROJECT NUMBER 1543-2-1
PROJECT LOCATION 3050 Zanker Road, San Jose, CA

DATE STARTED 3/25/25 **DATE COMPLETED** 3/25/25
GROUND ELEVATION 19 FT +/- **BORING DEPTH** 31.5 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc. **LATITUDE** 37.398669° **LONGITUDE** -121.926157°
DRILLING METHOD Mobile B-53B, 8 inch Hollow-Stem Auger **GROUND WATER LEVELS:**
LOGGED BY RA **AT TIME OF DRILLING** Not Encountered
NOTES **AT END OF DRILLING** Not Encountered

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
19.0	0	2 inches asphalt concrete over 7 inches aggregate base							
18.8									
18.2		**Fat Clay (CH)** hard, moist, dark gray to gray, some fine sand, high plasticity	19	MC-1B	105	23			>4.5
		becomes very stiff	25	MC-2B	99	25			
14.0	5	**Lean Clay (CL)** very stiff, moist, brown with gray mottles, some fine sand, moderate plasticity	22	MC-3B	98	25			
			25	MC-4B	98	26			
	10								
			19	MC					
	15								
2.0		**Lean Clay with Sand (CL)** medium stiff, moist, gray with brown mottles, fine sand, low to moderate plasticity	13	MC-6B	100	24			
	20								
-2.5		**Lean Clay (CL)** medium stiff, moist, bluish gray with brown mottles, some fine sand, low plasticity	17	MC-7B	91	30			
	25								
-7.0									

Continued Next Page

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 5/1/25 09:42 - P:\DRAFTING\GINT FILES\1543-2-1 MONTAGUE POINT IND.GPJ

BORING NUMBER EB-5

CORNERSTONE EARTH GROUP

PROJECT NAME Montague Point Industrial

PROJECT NUMBER 1543-2-1

PROJECT LOCATION 3050 Zanker Road, San Jose, CA

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
-7.0	30	**Silty Sand (SM)** medium dense, wet, gray, fine sand	19	SPT-8		23		23
-11.0	30	**Poorly Graded Sand with Silt (SP-SM)** dense, wet, gray, fine to coarse sand	30	SPT-9		17		
-12.5		Bottom of Boring at 31.5 feet.						

UNDRAINED SHEAR STRENGTH, ksf
- ○ HAND PENETROMETER
- △ TORVANE
- ● UNCONFINED COMPRESSION
- ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL

1.0 2.0 3.0 4.0

35 40 45 50 55

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 5/1/25 09:42 - P:\DRAFTING\GINT FILES\1543-2-1 MONTAGUE POINT IND.GPJ

BORING NUMBER EB-6

CORNERSTONE EARTH GROUP

PROJECT NAME Montague Point Industrial
PROJECT NUMBER 1543-2-1
PROJECT LOCATION 3050 Zanker Road, San Jose, CA

DATE STARTED 3/25/25 **DATE COMPLETED** 3/25/25
GROUND ELEVATION 16 FT +/- **BORING DEPTH** 34 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.398331° **LONGITUDE** -121.927198°
DRILLING METHOD Mobile B-53B, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY RA
AT TIME OF DRILLING 14 ft.
NOTES
AT END OF DRILLING 14 ft.

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
16.0	0	2 inches asphalt concrete over 6 inches aggregate base						
15.3		**Fat Clay (CH)** very stiff, moist, dark gray to gray, some fine sand, high plasticity Liquid Limit = 54, Plastic Limit = 22	18	MC-1B	99	25	32	
			20	MC-2B	98	27		
11.5	5	**Lean Clay (CL)** very stiff, moist, brown with gray mottles, some fine sand, moderate plasticity	24	MC-3B	99	24		
			19	MC				
	10							
			20	MC-5B	96	27		
	15							
0.0		**Lean Clay with Sand (CL)** medium stiff, moist, gray with brown mottles, fine sand, moderate plasticity	11	MC-6B	99	25		
	20							
-6.0		**Lean Clay (CL)** very stiff, moist, bluish gray with brown mottles, some fine sand, low plasticity	22	MC				
	25							
-10.0								

Continued Next Page

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 5/1/25 09:42 - P:\DRAFTING\GINT FILES\1543-2-1 MONTAGUE POINT IND.GPJ

BORING NUMBER EB-6

CORNERSTONE EARTH GROUP

PROJECT NAME Montague Point Industrial

PROJECT NUMBER 1543-2-1

PROJECT LOCATION 3050 Zanker Road, San Jose, CA

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
-10.0		**Lean Clay (CL)** very stiff, moist, bluish gray with brown mottles, some fine sand, low plasticity						
-12.0		**Silt with Sand (ML)** stiff, moist, gray, fine sand, low plasticity	16	MC-8B	93	28		
	30							
-15.0		**Poorly Graded Sand with Silt (SP-SM)** very dense, moist, dark gray, fine to medium sand	50	SPT-9		15		11
-17.3			16	SPT-10B		27		92
-18.0		**Lean Clay (CL)** stiff, moist, gray brown mottled, some fine sand, moderate plasticity						
	35	Bottom of Boring at 34.0 feet.						
	40							
	45							
	50							
	55							

UNDRAINED SHEAR STRENGTH, ksf

○ HAND PENETROMETER

△ TORVANE

● UNCONFINED COMPRESSION

▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL

1.0 2.0 3.0 4.0

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 5/1/25 09:42 - P:\DRAFTING\GINT FILES\1543-2-1 MONTAGUE POINT IND.GPJ

BORING NUMBER EB-7

CORNERSTONE EARTH GROUP

PROJECT NAME Montague Point Industrial
PROJECT NUMBER 1543-2-1
PROJECT LOCATION 3050 Zanker Road, San Jose, CA

DATE STARTED 3/25/25 **DATE COMPLETED** 3/25/25
GROUND ELEVATION 18 FT +/- **BORING DEPTH** 31.5 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.396619° **LONGITUDE** -121.925708°
DRILLING METHOD Mobile B-53B, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY RA
AT TIME OF DRILLING 13 ft.
NOTES
AT END OF DRILLING Not Encountered

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
18.0	0	2 inches asphalt concrete over 6 inches aggregate base						
17.8								
17.3		**Fat Clay (CH)** very stiff, moist, dark gray to gray, some fine sand, high plasticity	19	MC-1B	98	25		
			23	MC-2B	100	25		
13.5	5	**Lean Clay (CL)** very stiff, moist, brown with gray mottles, some fine sand, moderate plasticity	23	MC				
		becomes stiff	16	MC-4B	91	30		
	10							
			17	MC-5B	88	34		
	15							
1.0		**Lean Clay with Sand (CL)** very stiff, moist, bluish gray with brown mottles, fine sand, moderate plasticity	25	MC-6B	98	27		
	20							
-5.0		**Sandy Lean Clay (CL)** medium stiff, moist, gray, fine sand, low plasticity	11	MC				
	25							
-8.0		*Continued Next Page*						

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 5/1/25 09:42 - P:\DRAFTING\GINT FILES\1543-2-1 MONTAGUE POINT IND.GPJ

BORING NUMBER EB-7

CORNERSTONE EARTH GROUP

PROJECT NAME Montague Point Industrial

PROJECT NUMBER 1543-2-1

PROJECT LOCATION 3050 Zanker Road, San Jose, CA

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
-8.0			**Sandy Lean Clay (CL)** medium stiff, moist, gray, fine sand, low plasticity						
-10.0			**Clayey Sand (SC)** medium dense, moist, gray, fine to coarse sand	21	SPT-8		17		21
	30			15	MC-9B	104	24		47
-13.5			Bottom of Boring at 31.5 feet.						
	35								
	40								
	45								
	50								
	55								

UNDRAINED SHEAR STRENGTH, ksf

○ HAND PENETROMETER

△ TORVANE

● UNCONFINED COMPRESSION

▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL

1.0 2.0 3.0 4.0

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 5/1/25 09:42 - P:\DRAFTING\GINT FILES\1543-2-1 MONTAGUE POINT IND.GPJ

BORING NUMBER EB-8

CORNERSTONE EARTH GROUP

PROJECT NAME Montague Point Industrial
PROJECT NUMBER 1543-2-1
PROJECT LOCATION 3050 Zanker Road, San Jose, CA

DATE STARTED 3/25/25 **DATE COMPLETED** 3/25/25
GROUND ELEVATION 18 FT +/- **BORING DEPTH** 45 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.397181° **LONGITUDE** -121.925130°
DRILLING METHOD Mobile B-53B, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY RA
▽ **AT TIME OF DRILLING** 13 ft.
NOTES
▼ **AT END OF DRILLING** 13 ft.

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
18.0	0	2 inches asphalt concrete over 6 inches aggregate base						
17.8								
17.3		**Fat Clay (CH)** very stiff, moist, dark gray to gray, some fine sand, high plasticity	27	MC-1B	102	24		
			26	MC-2B	99	25		
	5		27	MC-3B	101	26		
11.0		**Lean Clay with Sand (CL)** very stiff, moist, brown with gray mottles, fine sand, moderate plasticity	18	MC				
	10							
5.5		**Lean Clay (CL)** very stiff, moist, gray and brown with mottles, some fine sand, moderate plasticity	23	MC-5B	94	30		
	15		18	MC				
	20							
-4.0		**Lean Clay with Sand (CL)** very stiff, moist, brown with gray mottles, fine sand, moderate plasticity	16	MC-7B	100	24		
-7.0	25	**Sandy Lean Clay (CL)** medium stiff, moist, gray, fine sand, low	16	SPT-8		26		
-8.5								

Continued Next Page

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 5/1/25 09:42 - P:\DRAFTING\GINT FILES\1543-2-1 MONTAGUE POINT IND.GPJ

BORING NUMBER EB-8

CORNERSTONE EARTH GROUP

PROJECT NAME Montague Point Industrial

PROJECT NUMBER 1543-2-1

PROJECT LOCATION 3050 Zanker Road, San Jose, CA

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
-8.5		plasticity						
		Clayey Sand (SC)	19	SPT-9		26		39
-10.5		medium dense, moist, gray with brown mottles, fine sand						
		Poorly Graded Sand (SP)	30	SPT-10		16		4
	30	dense, wet, gray, fine to coarse sand, some fine subangular to subrounded gravel						
			41	SPT-11		13		
	35							
-19.0		**Poorly Graded Sand with Silt (SP-SM)**						
		very dense, wet, gray, fine to coarse sand	75	SPT-12		15		
	40		50/5"	SPT				
-24.5		**Silty Sand (SM)**						
		dense, wet, gray, fine to coarse sand	41	SPT-14		21		
-27.0	45	Bottom of Boring at 45.0 feet.						
	50							
	55							

UNDRAINED SHEAR STRENGTH, ksf

- ○ HAND PENETROMETER
- △ TORVANE
- ● UNCONFINED COMPRESSION
- ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL

1.0 2.0 3.0 4.0

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 5/1/25 09:42 - P:\DRAFTING\GINT FILES\1543-2-1 MONTAGUE POINT IND.GPJ

BORING NUMBER I-1

CORNERSTONE EARTH GROUP

PROJECT NAME Montague Point Industrial
PROJECT NUMBER 1543-2-1
PROJECT LOCATION 3050 Zanker Road, San Jose, CA

DATE STARTED 3/25/25 **DATE COMPLETED** 3/25/25
GROUND ELEVATION 16 FT +/- **BORING DEPTH** 5 ft.
DRILLING CONTRACTOR N/A **LATITUDE** 37.399137° **LONGITUDE** -121.928410°
DRILLING METHOD Coring Machine and Hand Auger **GROUND WATER LEVELS:**
LOGGED BY PT **AT TIME OF DRILLING** Not Encountered
NOTES **AT END OF DRILLING** Not Encountered

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
16.0	0								
15.8			1½ inches asphalt concrete over 6 inches aggregate base						
15.3			**Lean Clay with Gravel (CL) [Fill]** moist, gray and dark gray mottled, fine to coarse subangular gravel, moderate plasticity		GB-1		19		
13.5			**Lean Clay with Sand (CL)** moist, dark gray to gray, fine sand, moderate plasticity		GB-2		26		
					GB-3		25		
11.0	5		Bottom of Boring at 5.0 feet.						
	10								
	15								
	20								
	25								

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 5/1/25 09:43 - P:\DRAFTING\GINT FILES\1543-2-1 MONTAGUE POINT IND.GPJ

BORING NUMBER I-2

CORNERSTONE EARTH GROUP

PROJECT NAME Montague Point Industrial
PROJECT NUMBER 1543-2-1
PROJECT LOCATION 3050 Zanker Road, San Jose, CA

DATE STARTED 3/25/25 **DATE COMPLETED** 3/25/25
GROUND ELEVATION 18 FT +/- **BORING DEPTH** 3 ft.
DRILLING CONTRACTOR N/A **LATITUDE** 37.396476° **LONGITUDE** -121.925675°
DRILLING METHOD Coring Machine and Hand Auger **GROUND WATER LEVELS:**
LOGGED BY PT **AT TIME OF DRILLING** Not Encountered
NOTES **AT END OF DRILLING** Not Encountered

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
18.0	0								
17.8			2 inches asphalt concrete over 9 inches aggregate base						
17.1			**Fat Clay (CH)** moist, dark gray to gray, some fine sand, high plasticity		GB-1		16		
					GB-2		17		
15.0			Bottom of Boring at 3.0 feet.		GB-3		18		
	5								
	10								
	15								
	20								
	25								

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0

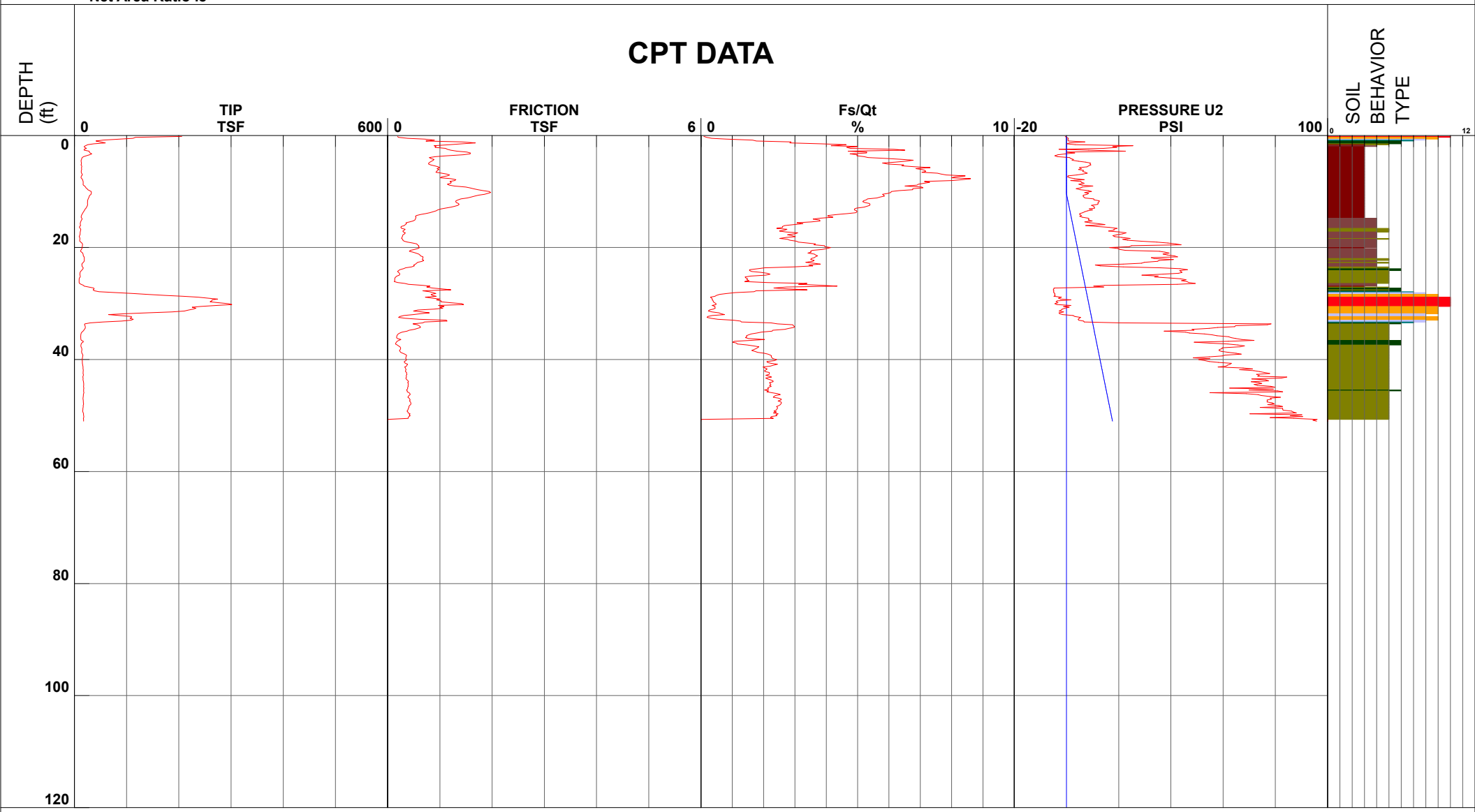
CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 5/1/25 09:43 - P:\DRAFTING\GINT FILES\1543-2-1 MONTAGUE POINT IND.GPJ

Cornerstone Earth Group

Project	Montague Pointe Industrial	Operator	AJ-SS	Filename	SDF(364).cpt
Job Number	1543-2-1	Cone Number	DDG1587	GPS	
Hole Number	CPT-01	Date and Time	3/13/2025 12:31:32 PM	Maximum Depth	51.02 ft
EST GW Depth During Test		10.40 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 15cm²

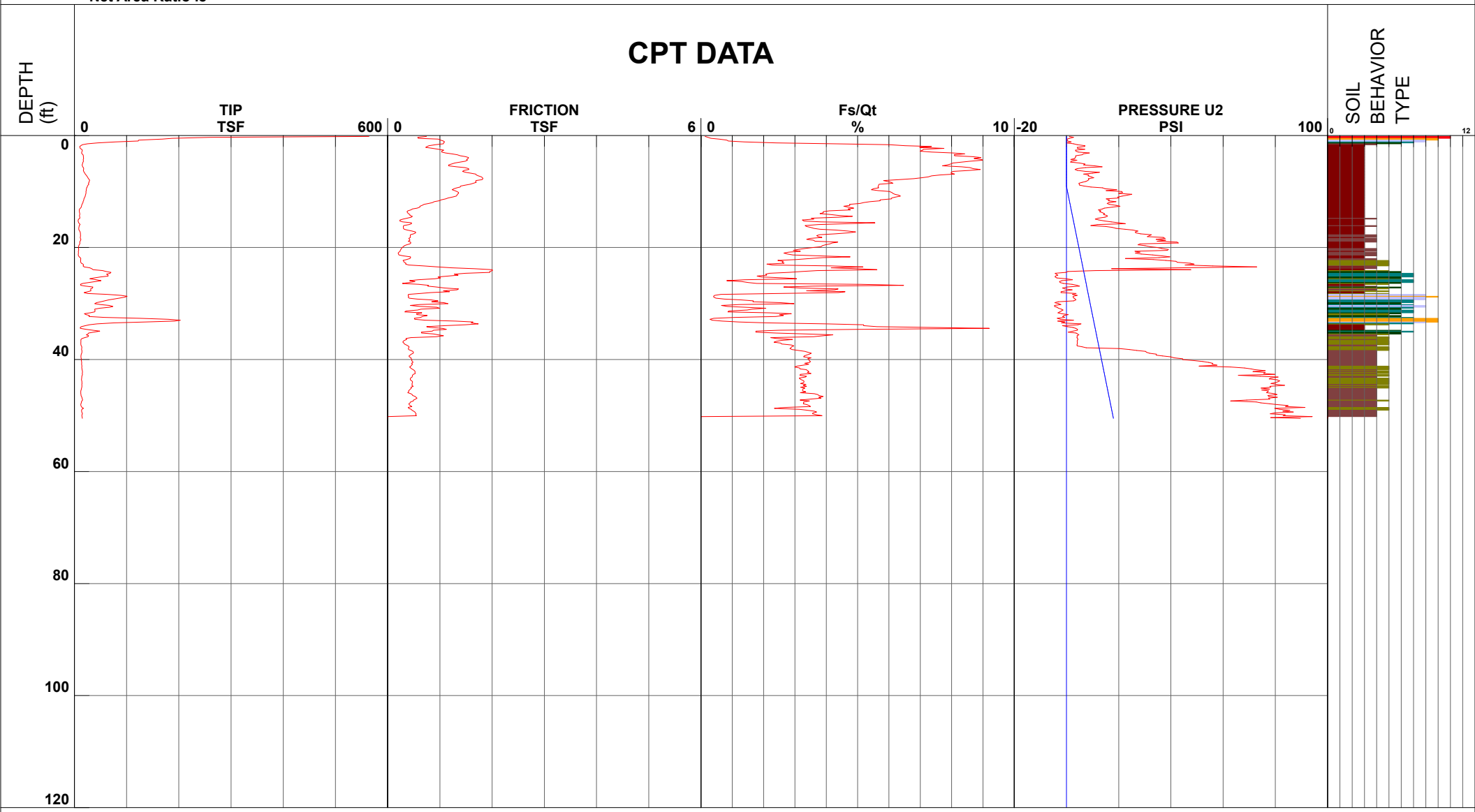
S*Soil behavior type and SPT based on data from UBC-1983

Cornerstone Earth Group

Project	Montague Pointe Industrial	Operator	AJ-SS	Filename	SDF(365).cpt
Job Number	1543-2-1	Cone Number	DDG1587	GPS	
Hole Number	CPT-02	Date and Time	3/13/2025 1:10:12 PM	Maximum Depth	50.52 ft
EST GW Depth During Test		9.10 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 15cm²

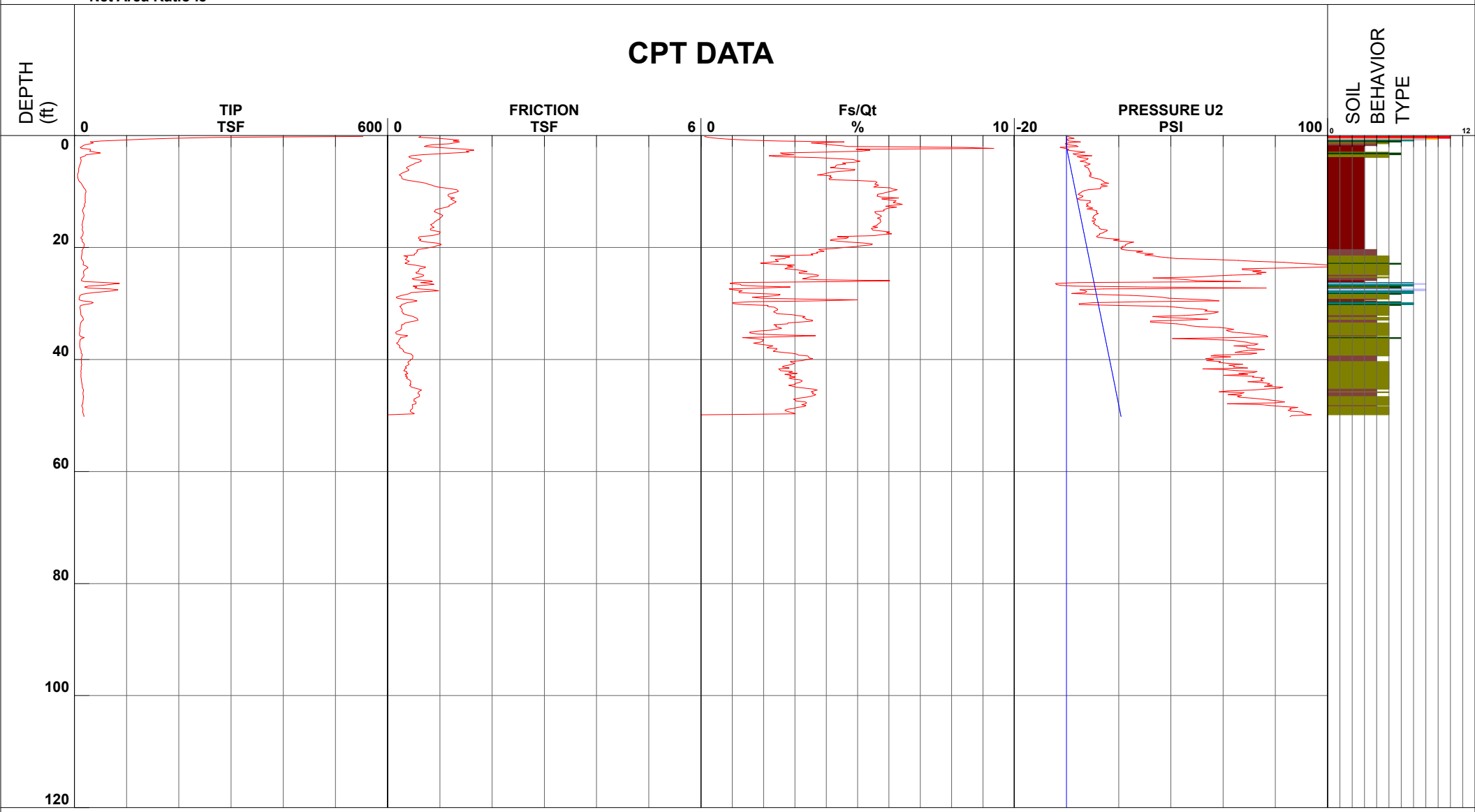
S*Soil behavior type and SPT based on data from UBC-1983

Cornerstone Earth Group

Project	Montague Pointe Industrial	Operator	AJ-SS	Filename	SDF(367).cpt
Job Number	1543-2-1	Cone Number	DDG1587	GPS	
Hole Number	CPT-03	Date and Time	3/13/2025 2:38:24 PM	Maximum Depth	50.20 ft
EST GW Depth During Test		2.00 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 15cm²

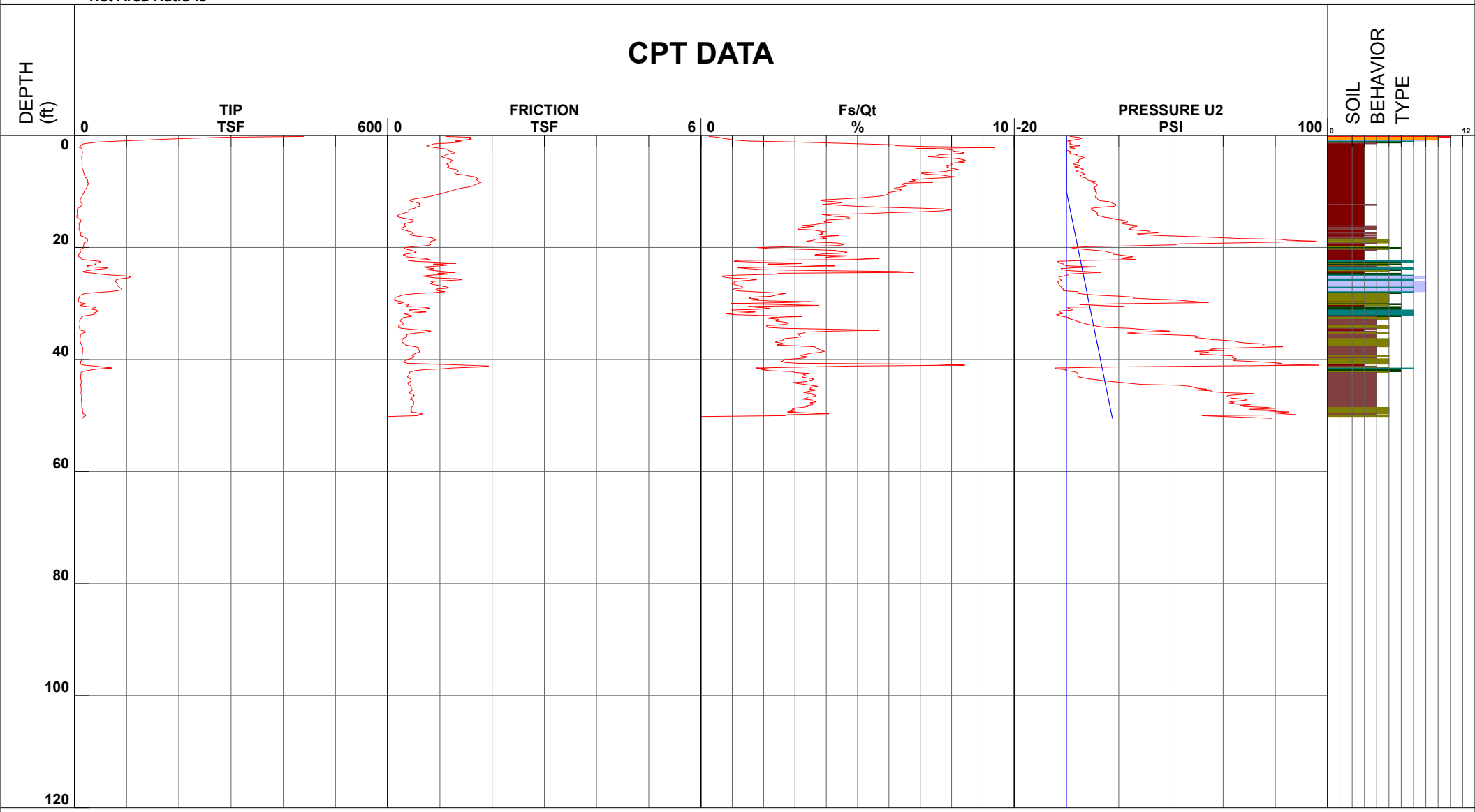
S*Soil behavior type and SPT based on data from UBC-1983

Cornerstone Earth Group

Project	Montague Pointe Industrial	Operator	AJ-SS	Filename	SDF(366).cpt
Job Number	1543-2-1	Cone Number	DDG1587	GPS	
Hole Number	CPT-04	Date and Time	3/13/2025 1:49:27 PM	Maximum Depth	50.52 ft
EST GW Depth During Test		10.00 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 15cm²

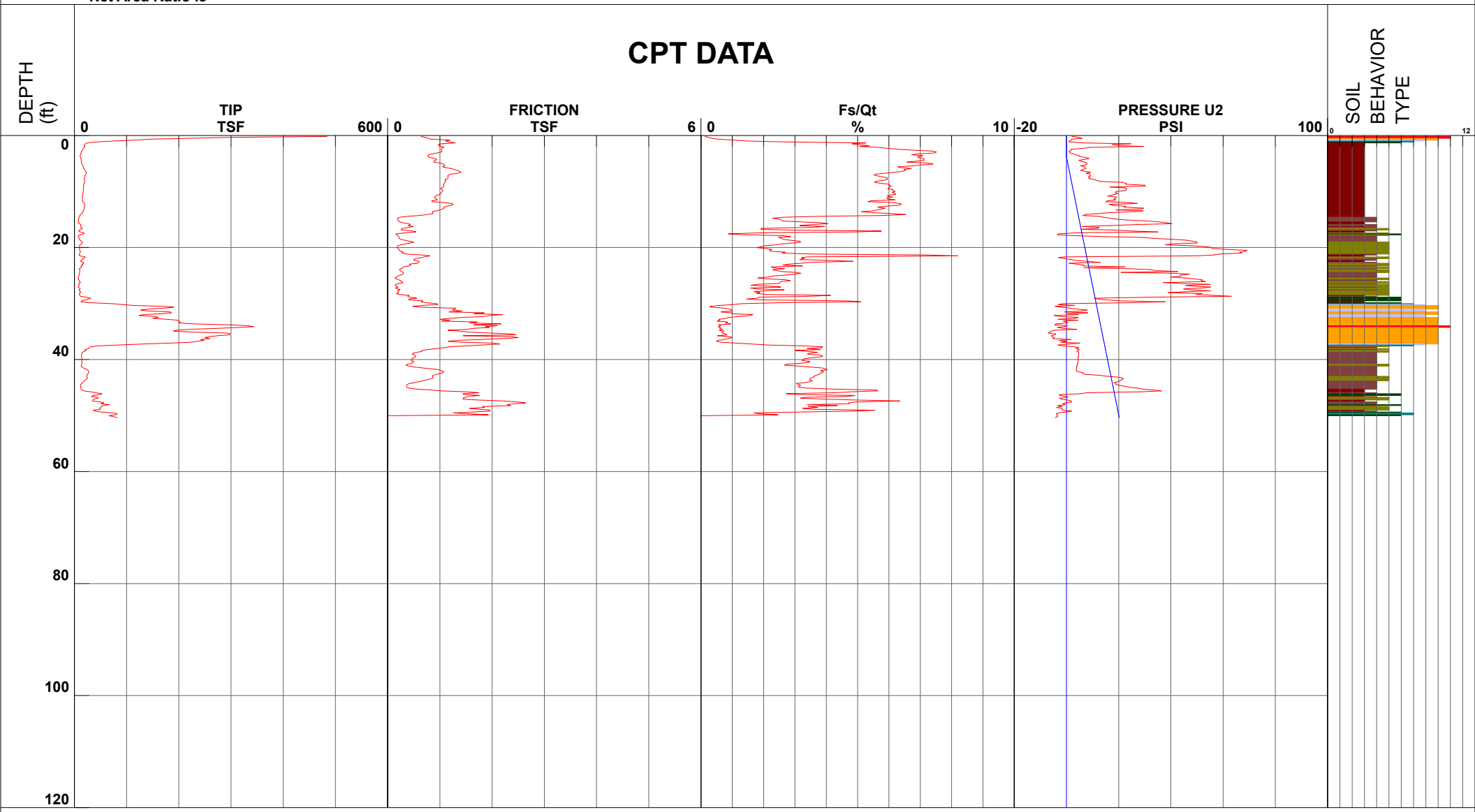
S*Soil behavior type and SPT based on data from UBC-1983

Cornerstone Earth Group

Project	Montague Pointe Industrial	Operator	AJ-SS	Filename	SDF(368).cpt
Job Number	1543-2-1	Cone Number	DDG1587	GPS	
Hole Number	CPT-05	Date and Time	3/13/2025 3:14:50 PM	Maximum Depth	50.36 ft
EST GW Depth During Test		3.80 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 15cm²

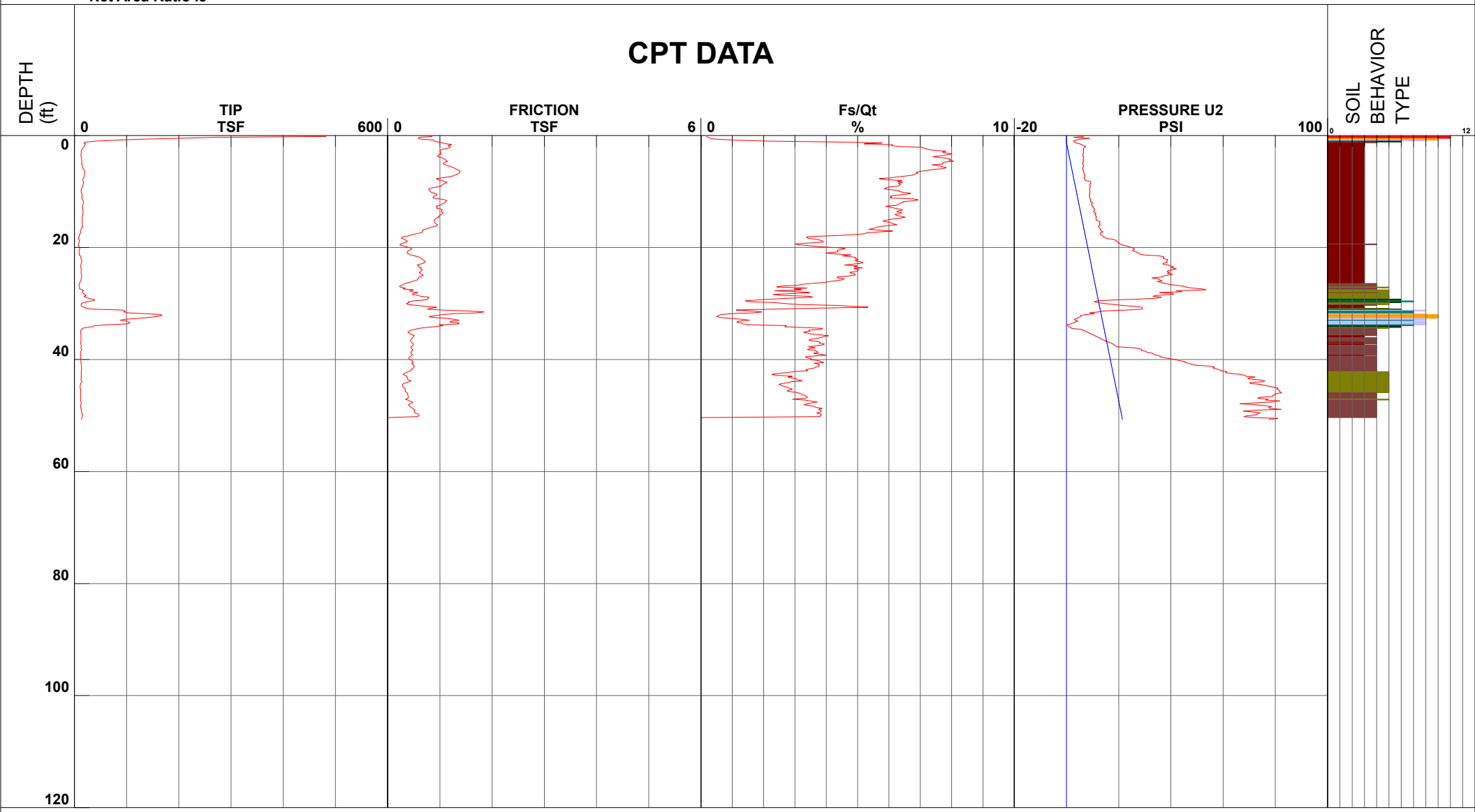
S*Soil behavior type and SPT based on data from UBC-1983

Cornerstone Earth Group

Project	Montague Pointe Industrial	Operator	JM-IY	Filename	SDF(147).cpt
Job Number	1543-2-1	Cone Number	DDG1596	GPS	
Hole Number	CPT-06	Date and Time	3/20/2025 10:38:51 AM	Maximum Depth	50.69 ft
EST GW Depth During Test		1.30 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 15cm²

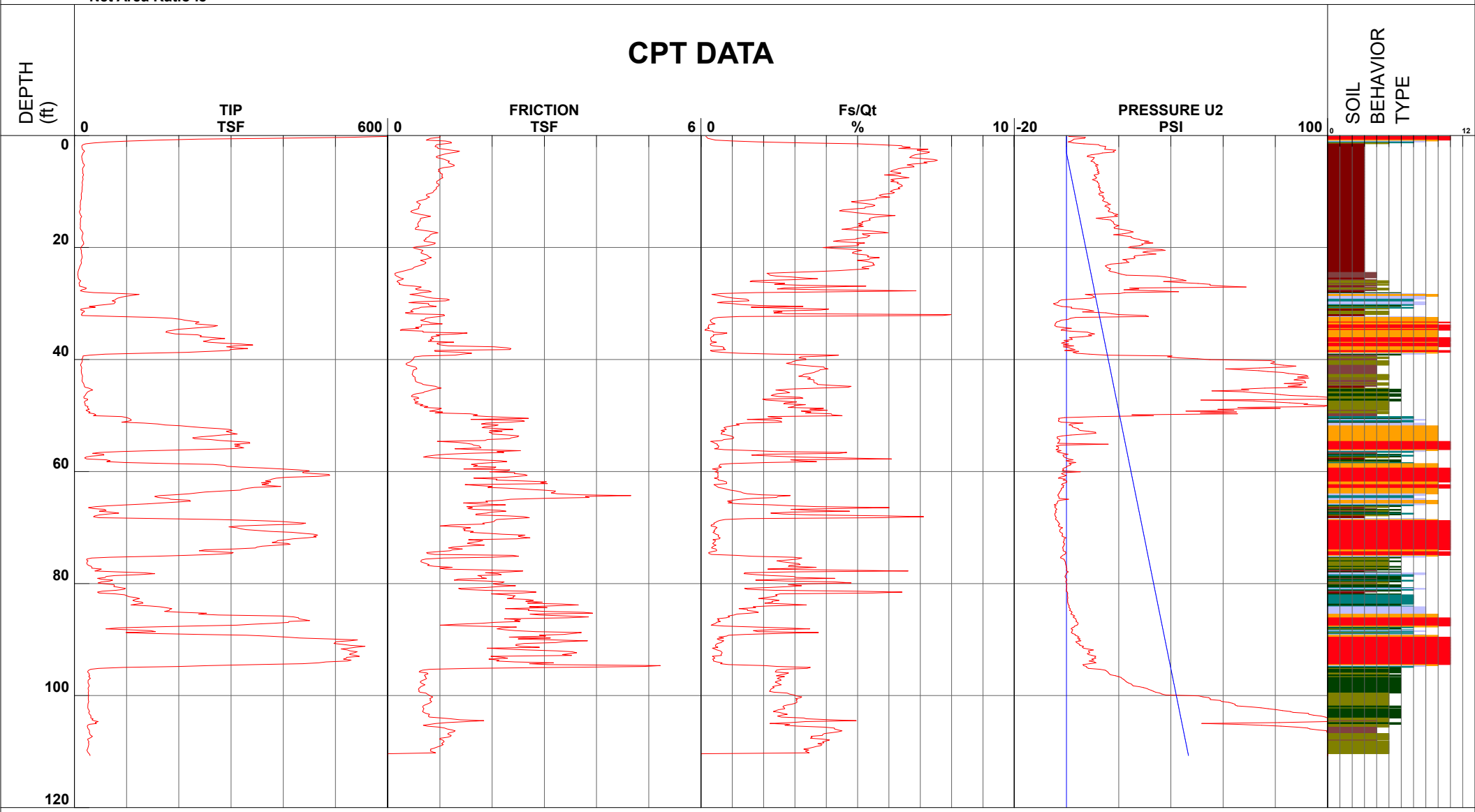
S*Soil behavior type and SPT based on data from UBC-1983

Cornerstone Earth Group

Project	Montague Pointe Industrial	Operator	JM-IY	Filename	SDF(146).cpt
Job Number	1543-2-1	Cone Number	DDG1596	GPS	
Hole Number	CPT-07	Date and Time	3/20/2025 7:50:02 AM	Maximum Depth	110.73 ft
EST GW Depth During Test		3.10 ft			

Net Area Ratio .8

CPT DATA



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

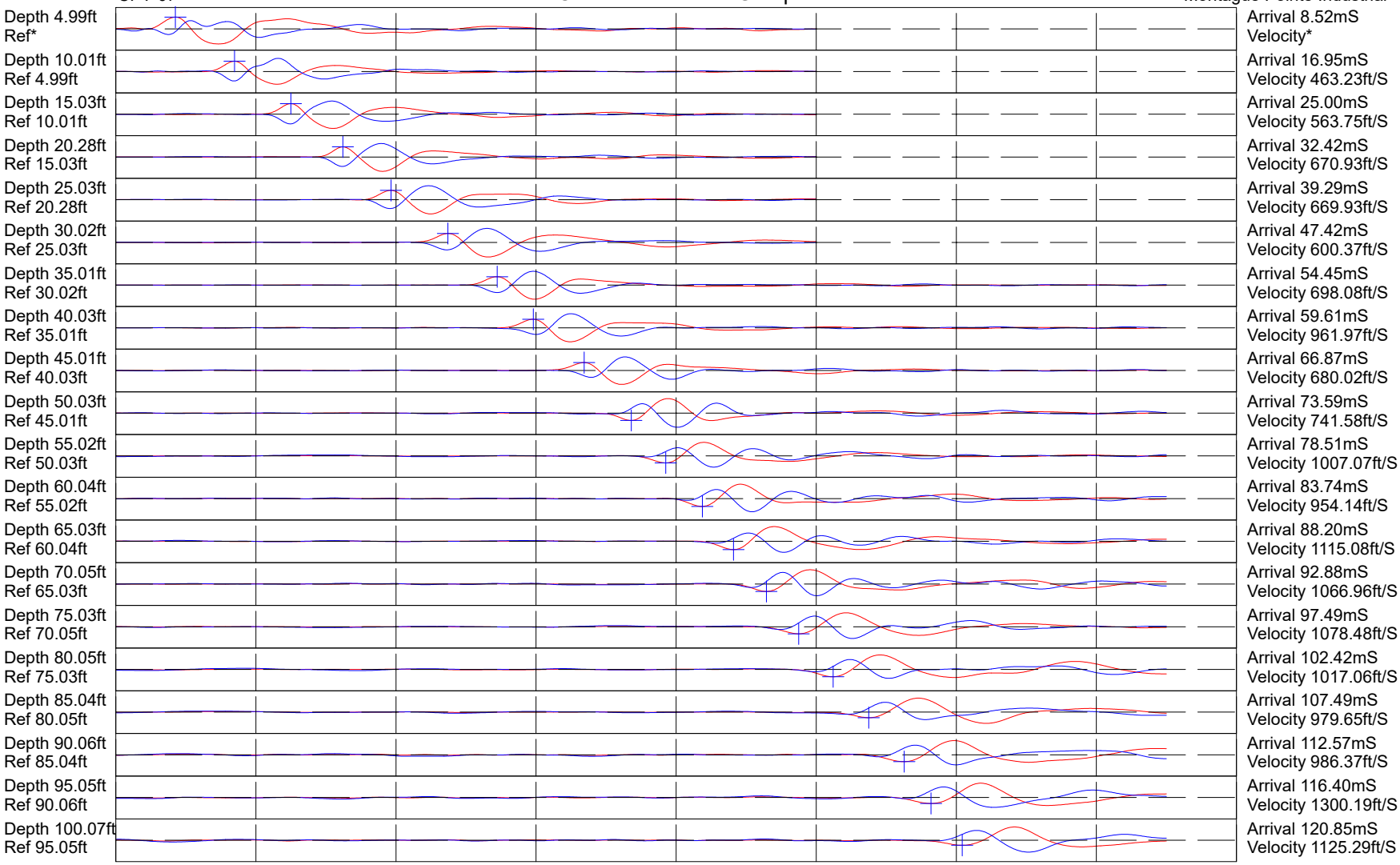
Cone Size 15cm²

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

CPT-07

Cornerstone Earth Group

Montague Pointe Industrial



Hammer to Rod String Distance (ft): 5.83

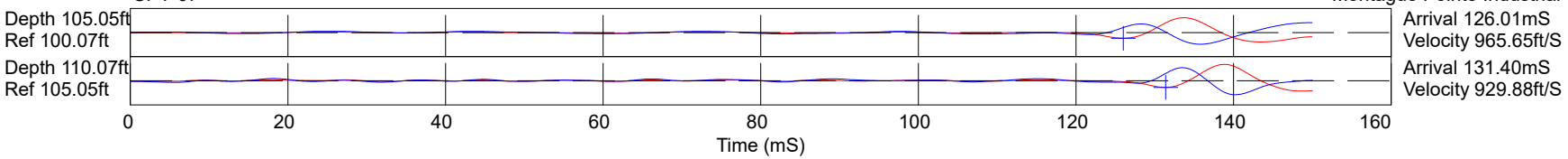
* = Not Determined

COMMENT:

CPT-07

SEISMIC TEST

Montague Pointe Industrial



Hammer to Rod String Distance (ft): 5.83
* = Not Determined

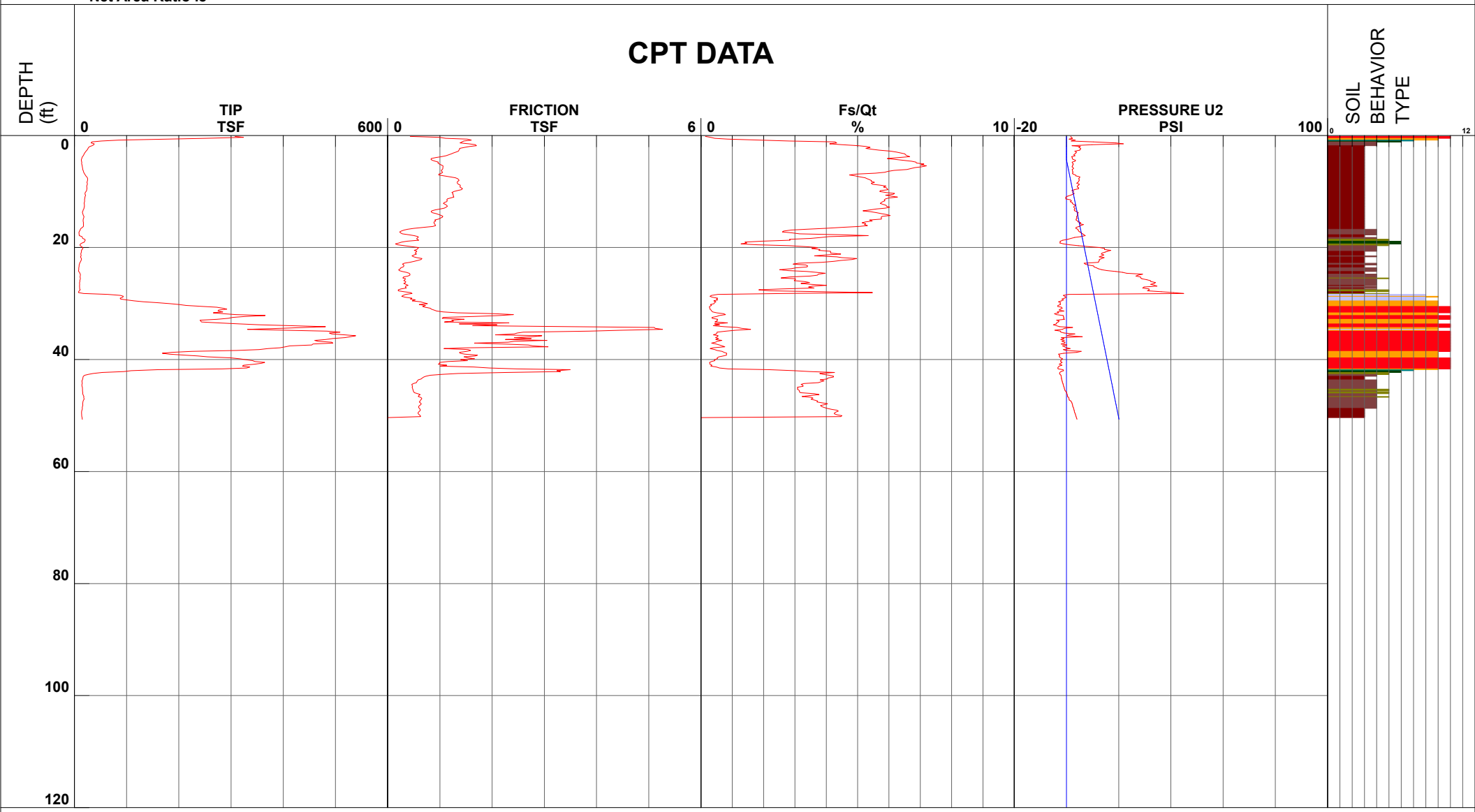
COMMENT:

Cornerstone Earth Group

Project	Montague Pointe Industrial	Operator	JM-IY	Filename	SDF(148).cpt
Job Number	1543-2-1	Cone Number	DDG1596	GPS	
Hole Number	CPT-08	Date and Time	3/20/2025 12:13:06 PM	Maximum Depth	50.69 ft
EST GW Depth During Test		4.30 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 15cm²

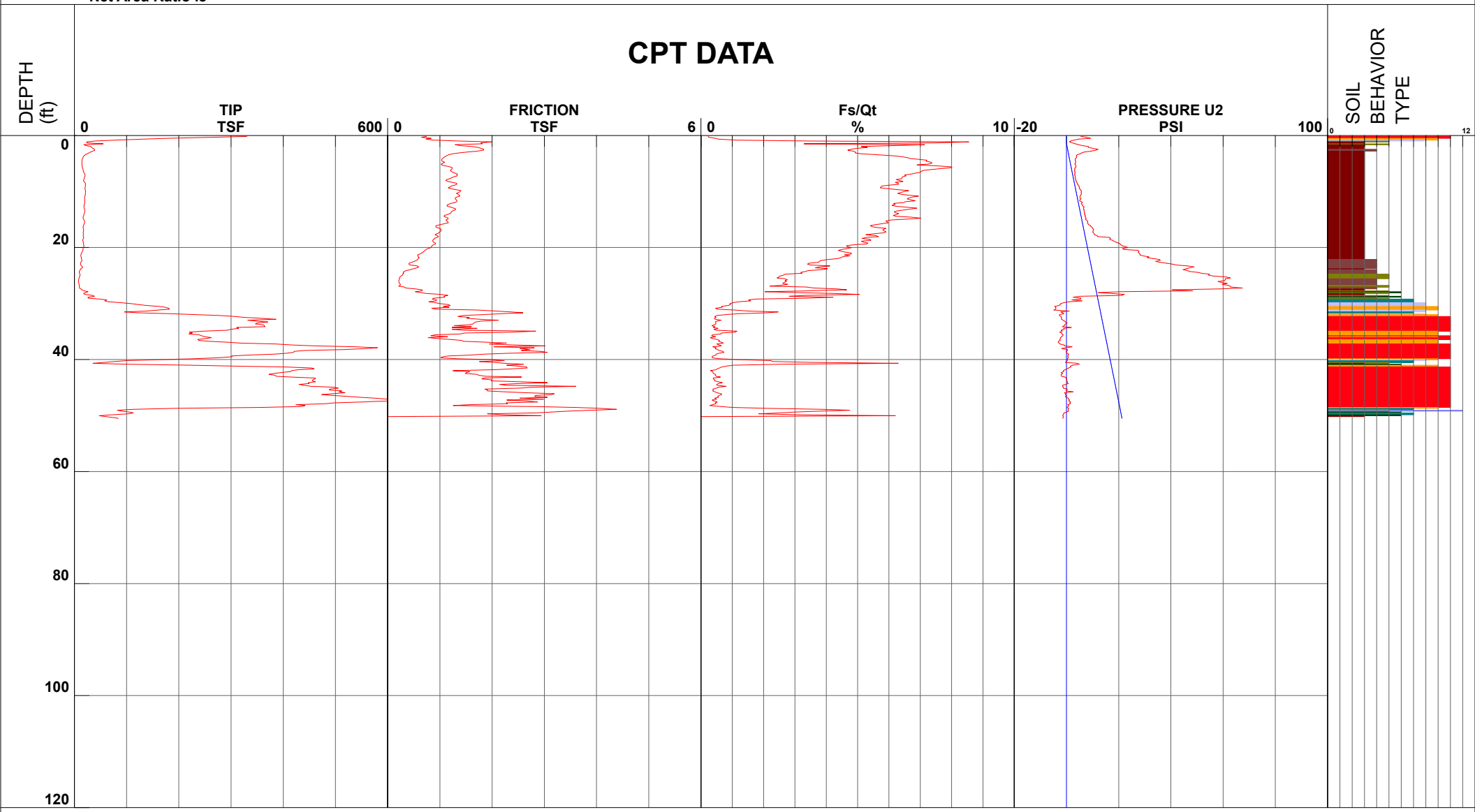
S*Soil behavior type and SPT based on data from UBC-1983

Cornerstone Earth Group

Project	Montague Pointe Industrial	Operator	JM-IY	Filename	SDF(149).cpt
Job Number	1543-2-1	Cone Number	DDG1596	GPS	
Hole Number	CPT-09	Date and Time	3/20/2025 12:52:47 PM	Maximum Depth	50.52 ft
EST GW Depth During Test		1.50 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 15cm²

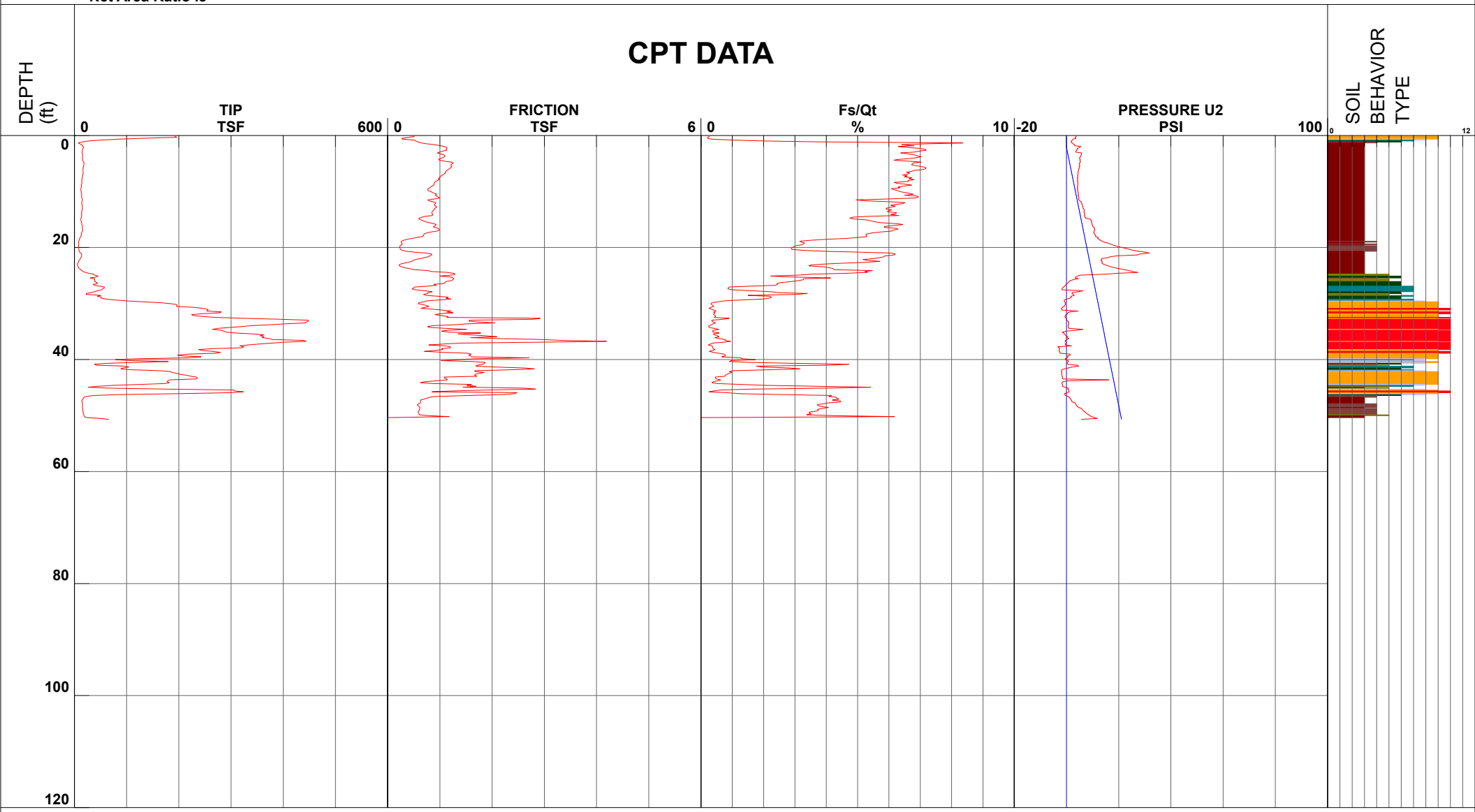
S*Soil behavior type and SPT based on data from UBC-1983

Cornerstone Earth Group

Project	Montague Pointe Industrial	Operator	JM-IY	Filename	SDF(150).cpt
Job Number	1543-2-1	Cone Number	DDG1596	GPS	
Hole Number	CPT-10	Date and Time	3/20/2025 1:34:40 PM	Maximum Depth	50.69 ft
EST GW Depth During Test		2.10 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 15cm²

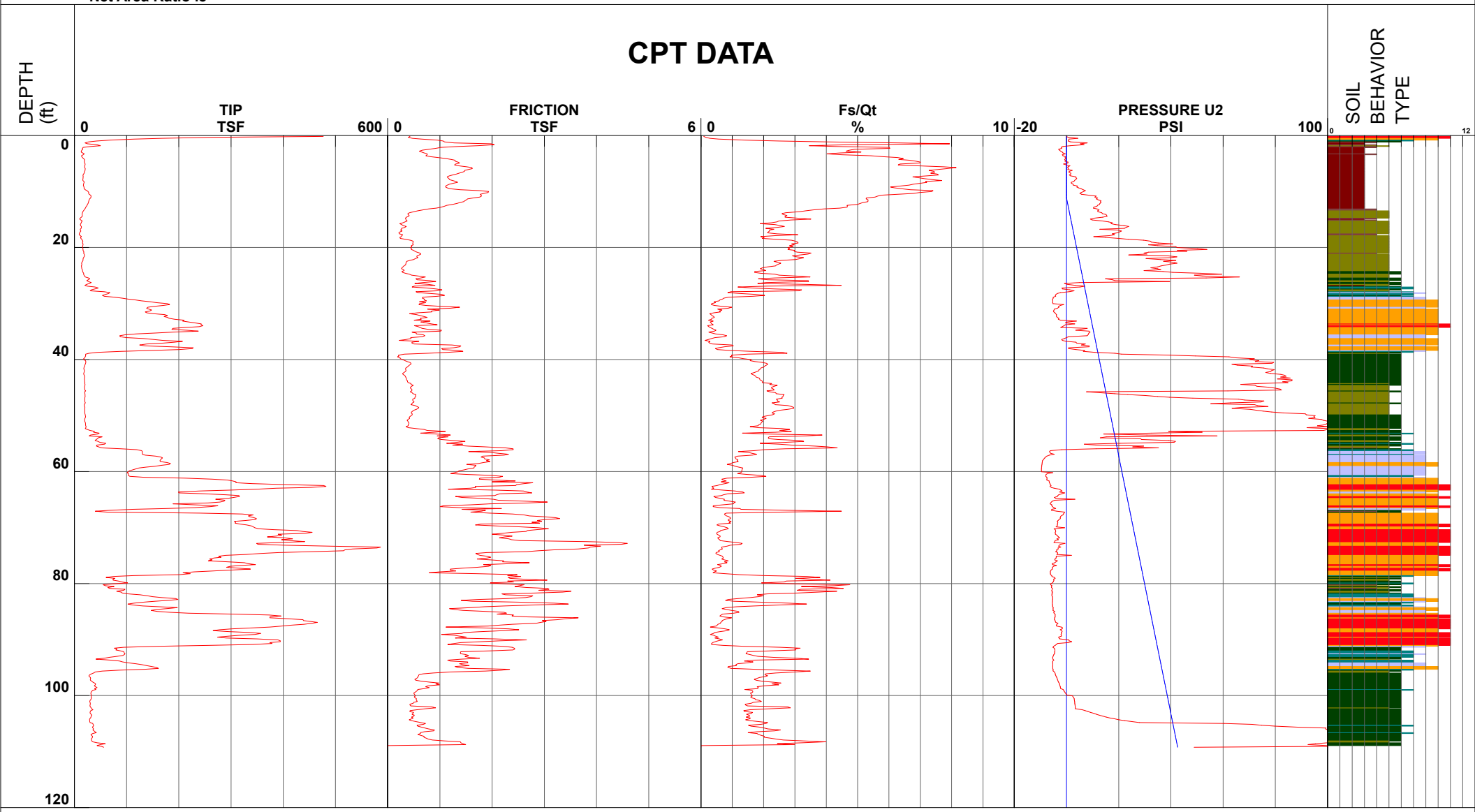
S*Soil behavior type and SPT based on data from UBC-1983

Cornerstone Earth Group

Project	Montague Pointe Industrial	Operator	AJ-SS	Filename	SDF(363).cpt
Job Number	1543-2-1	Cone Number	DDG1587	GPS	
Hole Number	CPT-11	Date and Time	3/13/2025 8:23:59 AM	Maximum Depth	109.25 ft
EST GW Depth During Test		11.10 ft			

Net Area Ratio .8

CPT DATA

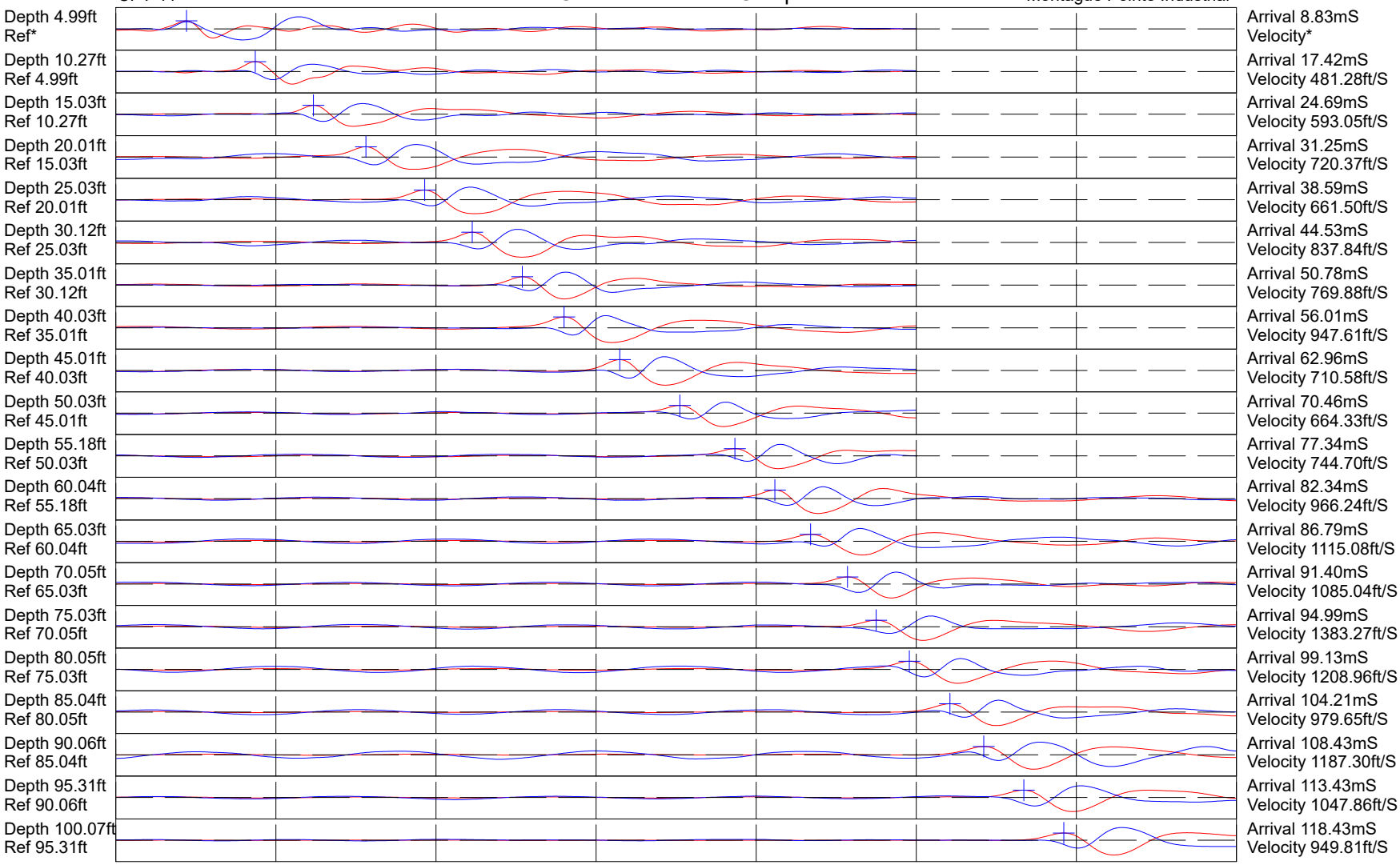


1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 15cm²

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

CPT-11 | Cornerstone Earth Group | Montague Pointe Industrial



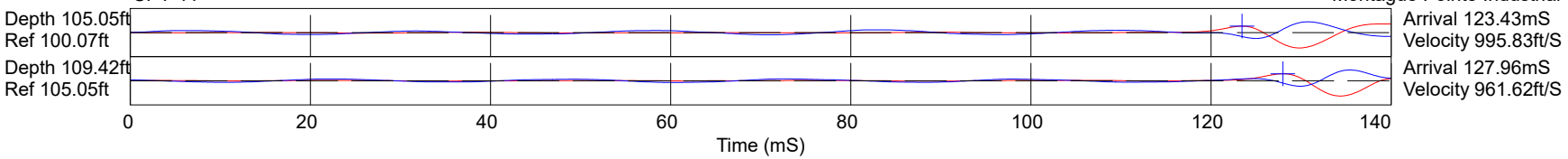
Hammer to Rod String Distance (ft): 5.83
* = Not Determined

COMMENT:

CPT-11

SEISMIC TEST

Montague Pointe Industrial



Hammer to Rod String Distance (ft): 5.83
* = Not Determined

COMMENT:

APPENDIX B: LABORATORY TEST PROGRAM

The laboratory testing program was performed to evaluate the physical and mechanical properties of the soils retrieved from the site to aid in verifying soil classification.

Moisture Content: The natural water content was determined (ASTM D2216) on 68 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

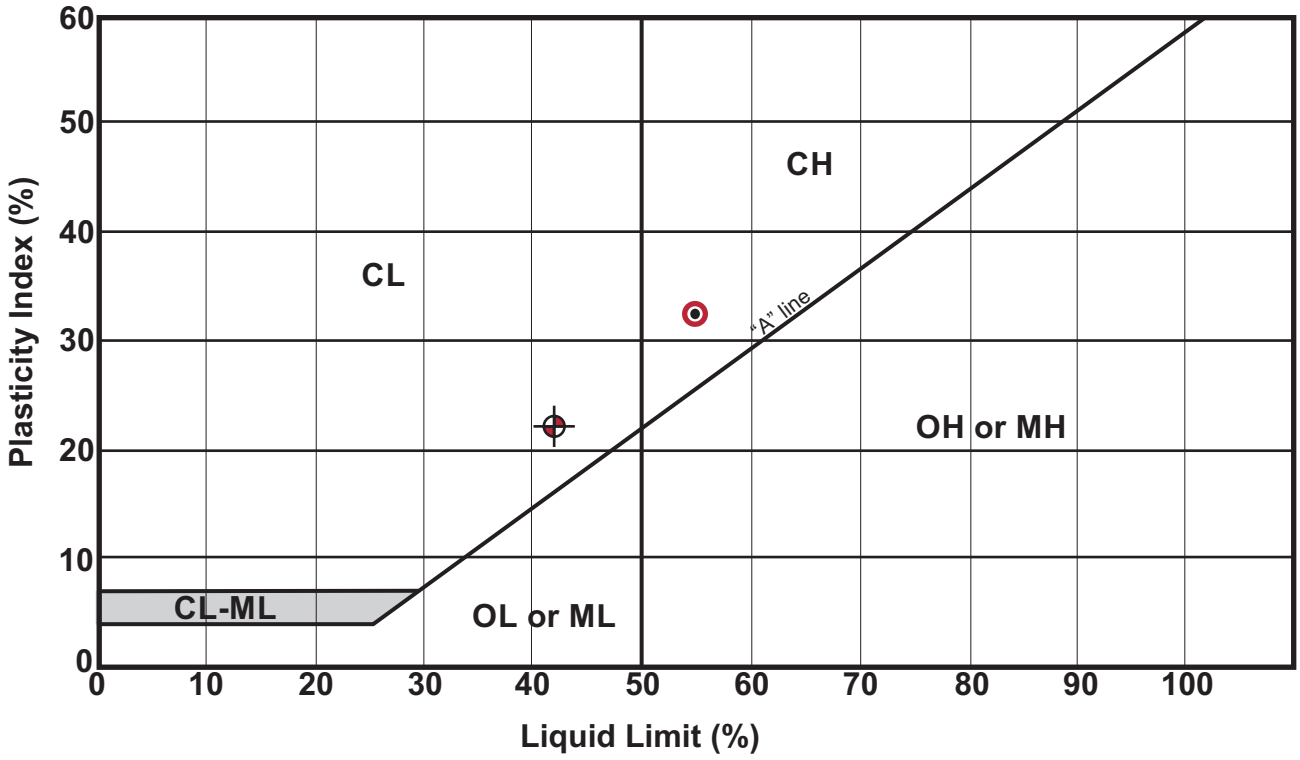
Dry Densities: In place dry density determinations (ASTM D2937) were performed on 47 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on 16 samples of the subsurface soils to aid in the classification of these soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Plasticity Index: Two Plasticity Index determinations (ASTM D4318) were performed on samples of the subsurface soils to measure the range of water contents over which this material exhibits plasticity. The Plasticity Index was used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil expansion potential. Results of these tests are shown on the boring logs at the appropriate sample depths.

Consolidation: One consolidation test (ASTM D2435) was performed on a relatively undisturbed sample of the subsurface clayey soils to assist in evaluating the compressibility property of this soil. Results of the consolidation test are presented graphically in this appendix.

Plasticity Index (ASTM D4318) Testing Summary



Symbol	Boring No.	Depth (ft)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index	Passing No. 200 (%)	Group Name (USCS - ASTM D2487)
⊕	EB-2	2.0	23	42	20	22	—	Lean Clay with Sand (CL)
◉	EB-6	2.0	25	54	22	32	—	Fat Clay (CH)

Samples prepared in accordance with ASTM D421

CORNERSTONE EARTH GROUP

Plasticity Index Testing Summary

Montague Pointe Industrial
San Jose, CA

Project Number: 1543-2-1

Figure Number: Figure B1

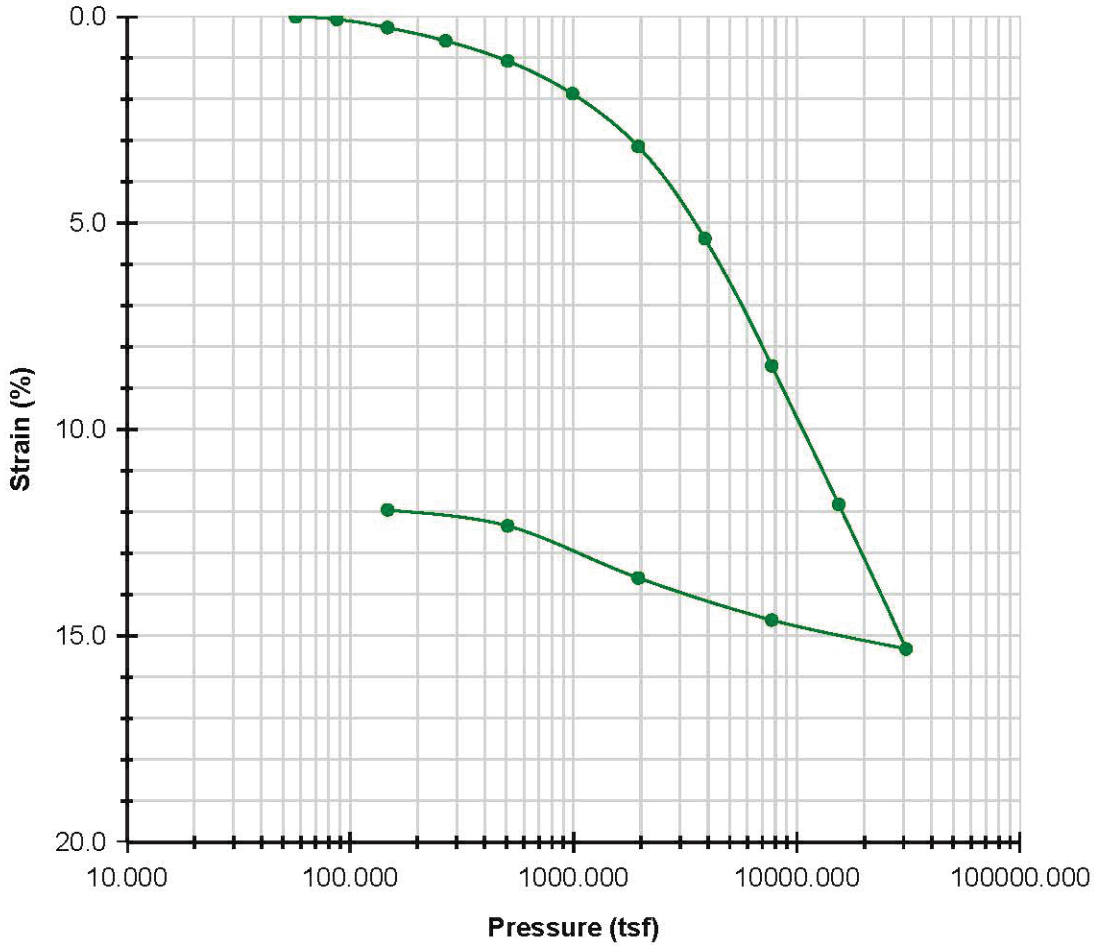
Date: May 2025

Drawn By: FLL

Consolidation Test ASTM D2435

Boring: EB-4 **Sample:** 5 **Depth:** 14.5'

Description: Lean Clay with Sand (CL)



	BEFORE	AFTER
Moisture (%)	23.8	18.3
Dry Density (pcf)	96.9	113.3
Saturation (%)	86.2	100.0
Void Ratio	0.75	0.50

(A) Stress Strain Curve

CORNERSTONE EARTH GROUP

Strain-Log Curve - EB-4 @ 14.5'

Montague Pointe Industrial
San Jose, CA

Project Number: 1543-2-1

Figure Number: Figure B2

Date: May 2025

Drawn By: FLL

APPENDIX C: SITE CORROSIVITY EVALUATION

JDH CORROSION CONSULTANTS REPORT DATED APRIL 21, 2025

April 21, 2025

Cornerstone Earth Group, Inc.
1259 Oakmead Parkway
Sunnyvale, CA 94085

Attention: **Jesse D Serpa, P.E.**
Project Engineer

Subject: **Site Corrosivity Evaluation**
Montague Pointe Industrial
San Jose, CA
Project: 1543-2-1

Dear Jesse,

In accordance with your request, we have reviewed the laboratory soils data for the above referenced project site. Our evaluation of these results and our corresponding recommendations for corrosion control for the above referenced project foundations and buried site utilities are presented herein for your consideration.

Soil Testing & Analysis

Soil Chemical Analysis

Six (6) soil samples from the project site were chemically analyzed for corrosivity by **Cornerstone Earth Group**. The samples were analyzed for chloride and sulfate concentration, pH and resistivity at 100% saturation. The test results are presented in Cornerstone Earth Group Corrosivity Test Summary dated 4/3,4/2025. The results of the chemical analysis were as follows:

Soil Laboratory Analysis

Chemical Analysis	Range of Results	Corrosion Classification*
Chlorides	5 - 43 mg/kg	Non-corrosive*
Sulfates	35 - 355 mg/kg	Non-corrosive to Mildly Corrosive**
pH	7.2 – 7.7	Non-corrosive*
Resistivity at 100% Saturation	683 – 1,893 ohm-cm	**Corrosive***

* With respect to bare steel or ductile iron.
** With respect to mortar coated steel

Discussion

Shallow Reinforced Concrete Foundations

Due to the low levels of water-soluble sulfates found in these soils, there is no special requirement for sulfate resistant concrete to be used at this site. The type of cement used should be in accordance with California Building Code (CBC) for soils which have less than 0.10 percent by weight of water soluble sulfate (SO_4) in soil and the minimum depth of cover for the reinforcing steel should be as specified in CBC as well.

Underground Metallic Pipelines

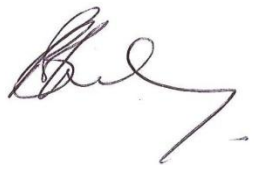
The soils at the project site are generally considered to be "corrosive" to ductile/cast iron, steel and dielectric coated steel based on the saturated resistivity measurements. Therefore, special requirements for corrosion control are required for buried metallic utilities at this site depending upon the critical nature of the piping. Pressure piping systems such as domestic and fire water should be provided with appropriate coating systems and cathodic protection, where warranted. In addition, all underground pipelines should be electrically isolated from above grade structures, reinforced concrete structures and copper lines in order to avoid potential galvanic corrosion problems.

LIMITATIONS

The conclusions and recommendations contained in this report are based on the information and assumptions referenced herein. All services provided herein were performed by persons who are experienced and skilled in providing these types of services and in accordance with the standards of workmanship in this profession. No other warrantees or guarantees, expressed or implied, is provided.

We thank you for the opportunity to be of service to **Cornerstone Earth Group** on this project and trust that you find the enclosed information satisfactory. If you have any questions, or if we can be of any additional assistance, please feel free to contact us at (925) 927-6630.

Respectfully submitted,



Brendon Hurley
JDH CORROSION CONSULTANTS, INC.
Field Technician



Mohammed Ali., P.E.
JDH Corrosion Consultants, Inc.
Senior Corrosion Engineer

CC: File 2025158

THIS PAGE INTENTIONALLY LEFT BLANK