APPENDIX 4

GEOTECHNICAL STUDY
PROPOSED RESIDENTIAL DEVELOPMENT
(UPDATED 9/8/2021)

38288-38594 CEDAR BOULEVARD
NEWARK, CALIFORNIA

SEPTEMBER 8, 2021
PROJECT PA20.1048.00

SUBMITTED TO:

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1 INTRODUCTION

This report presents the results of our geotechnical study for a proposed residential development on an approximately 7.75-acre property with addresses 38288 through 38594 Cedar Boulevard in Newark, California. The Assessor Parcel Number of the property is 92A-2375-2-6. The parcel is referenced as the "property," "site," or "project area" in this report. The approximate location of the project site is shown on the Vicinity Map included with Figures 1 and 2 of this report. Figure 1 shows a layout of the existing site conditions. Figure 2 shows a layout of the currently proposed development.

This report presents our findings, conclusions, and geotechnical recommendations for design and construction of the project. These findings, conclusions, and recommendations are based on information collected and reviewed during this investigation. The conclusions and recommendations in this report should not be extrapolated to other areas or used for other projects without our review.

1.1 Project Description

The property is occupied by miscellaneous businesses with buildings surrounded by at-grade asphalt concrete pavements. The proposed development will include demolition of the existing structures and improvements, followed by construction of residential units (single-family residences and duets). Associated improvements will include on-site roadways, underground utilities, and landscaping. The proposed residential buildings will be two- and three-story high, wood-framed structures. No basements are planned for the residential units.

Bioretention basins for on-site stormwater treatment and retention are proposed for this project. Each bioretention basin will consist of 18 inches of biotreatment soil mix underlain by 12 inches of Class 2 Permeable material.

Our review of the preliminary grading and drainage plans prepared by Ruggeri-Jensen-Azar (RJA), project civil engineer, dated July 29, 2021, indicates that site grading will involve cuts and fills of about 1 to 3 feet thick to construct the building pads and to achieve design grades.

The above project descriptions are based on information provided to us. If the actual project differs from those described above, Geo-Logic Associates (GLA) should be contacted to review our findings, conclusions, and recommendations and present any necessary modifications to address the different project development schemes.

1.2 Information Provided

For this investigation, Robson Homes provided us with the following.

- A drawing titled "Existing Conditions, Lands of Lebon and Freitas, City of Newark, Alameda County, California," prepared by RJA, dated September 12, 2018. This drawing shows a layout of the existing buildings and underground utilities.

- A set of drawings dated July 29, 2021, for submittal to City of Newark , including civil engineering, architectural, landscape, joint trench, and tentative tract maps.

1.3 Purpose and Scope of Services

The purpose of this geotechnical study was to explore subsurface conditions at the project site and to provide geotechnical recommendations for design and construction of the proposed improvements. The following work was performed.

1. Performed a site reconnaissance to observe site surface conditions and to mark locations of our exploration.

2. Reviewed available geologic and geotechnical information pertinent to the site.

3. Obtained two drilling permits from Alameda County Water District, one for exploratory borings and one for Cone Penetration Testing.

4. Notified Underground Service Alert (USA) for underground utility clearance and coordination of our drilling with Robson Homes.

5. Subcontracted with a private underground services locator to check the proposed exploration locations for presence of underground utilities.

6. Explored subsurface conditions by means of eight exploratory drill holes and three cone penetration test (CPT) probes to depths between approximately 20 and 50 feet below ground surface.

7. Collected two bulk samples of the near-surface soil.

8. Performed laboratory tests on selected soil samples from the drill holes and on the bulk samples to measure pertinent engineering properties of the samples.

9. Performed engineering analysis on the field and laboratory data.

10. Prepared a geotechnical study report.

11. Reviewed comments from the City of Newark and their geotechnical peer reviewer.

12. Prepared this updated geotechnical study report.

2 SITE INVESTIGATION

This investigation consists of a site reconnaissance and a subsurface exploration program. The site reconnaissance was to observe existing site surface conditions. The subsurface exploration program was to explore earth conditions at the project site. The observed surface and subsurface site conditions are discussed in Section 3 of this report.

2.1 Subsurface Exploration

Our geotechnical subsurface exploration program included eight exploratory drill holes (DH-1 through DH-8) and three CPT probes (CPT-1 through CPT-3). The exploratory drill holes and CPT probes were located in the field by referencing to existing site features and pacing; therefore, their locations are approximate. These approximate exploration locations are shown on Figures 1 and 2 of this report. The drill holes and CPT probes were backfilled in accordance with Alameda County Water District guidelines after completion of drilling and testing.

2.1.1 Exploratory Drill Holes

The eight exploratory drill holes were advanced on November 4 and 5, 2020, using a truck-mounted Mobile B-53 drilling rig equipped with 8-inch diameter hollow-stem augers. The depth of exploration ranged between approximately 20 and 50 feet below ground surface (bgs). In the field, our personnel visually classified the materials encountered and maintained a log of each drill hole.

Soil samples were obtained using a 2-inch outside diameter (O.D.; 1.4-inch inside diameter, I.D.) split-barrel sampler (also called a Standard Penetration Test sampler) and a 3-inch O.D. (2½-inch I.D.) split-barrel sampler. Soil samples were obtained by driving the sampler up to 18 inches into the earth material using a 140-pound hammer falling 30 inches. The number of blows required to drive the sampler was recorded for each 6-inch penetration interval. The number of blows required to drive the sampler the last 12 inches, or the penetration interval indicated on the log when harder material was encountered, is shown as blows per foot (blow count) on the drill hole logs. The samplers in DH-2 and DH-5, the two 50-foot deep holes, were driven with an automatic hammer and the samplers in the remaining drill holes were driven with a safety hammer on a wire winch.

In the field, our personnel visually classified the materials encountered and maintained a log of each drill hole. Visual classification of soils encountered in our drill holes was made in general accordance with the Unified Soil Classification System (ASTM D 2487 and D 2488). The results of our laboratory tests were used to refine our field classifications. Two Keys to Soil Classification, one for fine grained soils and one for coarse grained soils, are included in Appendix A, together with our drill hole logs.

2.1.2 Cone Penetration Test Probes

The three CPT probes were performed by Middle Earth Geo Testing on November 4, 2020, to a depth of approximately 50 feet bgs. CPT involves pushing a small diameter steel probe (15 cm^2 cross-sectional area cone was used for this project) into the ground using a hydraulic jack attached to a truck-mounted rig. The tip of the probe is instrumented and takes almost continuous measurements (roughly every 1 inch) of tip resistance, side friction, and pore water pressure. Graphic presentations of the CPT data are included in Appendix A of this report.

2.2 Laboratory Testing

Geotechnical laboratory testing was conducted on selected soil samples collected from our drill holes. These tests included moisture content, dry density, Atterberg limits, sieve analysis, hydrometer, and percentage passing a No. 200 sieve. An R-value test was performed on each of the two bulk samples collected from the site. The laboratory test results are presented on the drill hole logs at the corresponding sample depths. Graphic presentations of the results of the Atterberg limits, sieve analysis, hydrometer, and R-value tests are presented on separate sheets in Appendix B

In addition to geotechnical testing, three selected soil sample were sent to CERCO Analytical for corrosivity analysis. A report from CERCO Analytical with their corrosivity test results is included in Appendix B.

3 FINDINGS

3.1 Surface Conditions

The rectangular-shaped property is bordered by Highway 880 on the north and northeast, Cedar Boulevard on the south and southwest, commercial developments on the northwest, and an Alameda County Flood Control and Water Conservation District (ACFCWCD) channel on the southeast. The property is occupied by miscellaneous commercial and industrial business, with buildings mostly surrounded by asphalt concrete pavements. At 38288 Cedar Boulevard, much of the lot is unpaved.

We understand from the current property owner that the north/northeast portion of the property along Highway 880 was once occupied by a paved street. Old street pavements and underground utilities should be expected in the old street areas.

The ACFCWCD channel is about 10 feet in depth and both banks are lined with Gabion type retaining structures. The top of the western channel bank is about 20 to 25 feet from the eastern property line of the subject project.

3.2 Subsurface Conditions

Subsurface soils encountered in our drill holes and CPT probes can be generalized as alluvium. In drill holes DH-1 through DH-6, a pavement section consisting of roughly 2 to 3 inches of asphalt concrete over roughly 4 to 6 inches of base rock was encountered at ground surface.

In DH-1, subsurface soils below the pavement section consist of very stiff clay of intermediate plasticity to a depth of about 12 feet bgs and medium dense poorly graded sand to clayey sand to the maximum explored depth of about 20 feet bgs.

In DH-2, subsurface soils below the pavement section consist of stiff to very stiff clay of intermediate plasticity to a depth of about 17.5 feet bgs, medium dense poorly graded sand to a depth of about 19.5 feet bgs, medium dense to dense clayey sand to a depth of about 27 feet bgs, stiff sandy clay to a depth of about 37 feet bgs, firm clay with sand to a depth of about 42 feet bgs, medium dense to very dense clayey sand to the maximum explored depth of about 49 feet bgs.

In DH-3, subsurface soils below the pavement section consist of very stiff clay of intermediate plasticity to a depth of about 4.5 feet bgs, very stiff sandy clay to a depth of about 9.7 feet bgs, medium dense poorly graded sand to clayey sand to a depth of about 15.5 feet bgs, and very stiff clay to the maximum explored depth of about 21.5 feet bgs.

In DH-4, subsurface soils below the pavement section consist of very stiff to hard clay of intermediate plasticity to a depth of about 7 feet bgs, medium dense clayey sand to the depth of about 12 feet bgs, and very stiff clay to the maximum explored depth of about 20 feet bgs.

In DH-5, subsurface soils below the pavement section consist of hard clay of intermediate plasticity to a depth of about 7 feet bgs, medium dense poorly graded sand to clayey sand to a depth of about 12 feet bgs, firm to stiff clay to a depth of about 17 feet bgs, firm to stiff clay with sand to a depth of about 22 feet bgs, medium dense clayey sand to a depth of about 37 feet bgs, firm to stiff clay to the maximum explored depth of about 50 feet bgs.

In DH-6, subsurface soils below the pavement section consist of hard clay of intermediate plasticity to a depth of about 7 feet bgs, medium dense clayey sand to a depth of about 12 feet bgs, and very stiff clay to the maximum explored depth of about 20 feet bgs.

In DH-7, no pavement section was encountered at ground surface. Subsurface soils consist of hard clay of intermediate plasticity to a depth of about 7 feet bgs, and medium dense poorly graded sand with clay to the maximum explored depth of about 20 feet bgs.

In DH-8, no pavement section was encountered at ground surface. Subsurface soils consist of very stiff to hard clay of intermediate plasticity to a depth of about 3 feet bgs, hard sandy clay to a depth of about 7 feet bgs, medium dense clayey sand to a depth of about 12 feet bgs, and stiff clay with sand to the maximum explored depth of about 20 feet bgs.

In CPT-1, the interpreted soil behavior types consist of silty clay to clay to a depth of about 4 feet bgs, silty sand to sandy silt to a depth of about 12½ feet bgs, silty clay to clay to a depth of about 28½ feet bgs, silty sand to sandy silt to a depth of about 31 feet bgs, silty clay to clay to a depth of about 47 feet bgs, and clay sand to silty sand to the maximum explored depth of about 50 feet bgs.

In CPT-2, the interpreted soil behavior types consist of silty clay to clay to a depth of about 13 feet bgs, silty sand to sandy silt to a depth of about 15 feet bgs, silty clay to clay to a depth of about 27½ feet bgs, silty sand to sandy silt to a depth of about 31 feet bgs, and silty clay to clay to the maximum explored depth of about 50 feet bgs.

In CPT-3, the interpreted soil behavior types consist of silty clay to clay to a depth of about 12 feet bgs, silty sand to sandy silt to a depth of about 14 feet bgs, silty clay to clay to a depth of about 23 feet bgs, silty sand to sandy silt to a depth of about 32 feet bgs, and silty clay to clay to the maximum explored depth of about 50 feet bgs.

3.3 Groundwater

Groundwater was encountered in all eight drill holes and three CPTs, as shallow as 10 to 11 feet bgs, at the time of our investigation. Because the drill holes and CPT probes were backfilled soon after completion of drilling and testing, long-term stabilized groundwater level was not established.

Our review of Plate 1.2, “Historical liquefaction sites, depth to historically high ground water, and locations of boreholes used in this study, Newark 7.5-Minute quadrangle, California," Seismic Hazard Zone Report 090, California Geologic Survey, 2003, indicates that historically high groundwater level at the site was about 10 feet.

It should be noted that fluctuations in the groundwater level may occur due to seasonal variations in rainfall and temperature, water level in the adjacent creek, pumping from wells, regional groundwater recharge program, irrigation, or other factors that were not evident at the time of our investigation.

3.4 Variations in Subsurface Conditions

Our interpretations of soil and groundwater conditions, as described in this report, are based on information obtained during this study. Our conclusions and recommendations are based on these interpretations. Please realize the site has undergone different phases of development and grading. Therefore, it is likely that undisclosed variations in subsurface conditions exist at the site, particularly old foundations, abandoned utilities and localized areas of deep and loose fill.

Careful observations should be made during construction to verify our interpretations. Should variations from our interpretations be found, we should be notified to evaluate whether any revisions should be made to our recommendations.

4 SEISMIC CONSIDERATIONS

4.1 Earthquake Faulting

The Greater San Francisco Bay Area is seismically dominated by the active San Andreas Fault system, the tectonic boundary between the northward moving Pacific Plate (west of the fault) and the North American Plate (east of the fault). This movement is distributed across a complex system of generally strike-slip, right-lateral, and subparallel faults.

Potential sources of significant earthquake ground shaking at the site include several active and potentially active faults in the Greater San Francisco Bay area, as well as faults farther afield. The faults were first compiled on the State's Fault Activity Map (Jennings, 1974; Jennings and Bryant, 2010). This map has now been integrated into the US Geological Survey's Quaternary Fault and Fold Database and made available as a .kmz "drape" over Google Earth terrain files.

The distance to a seismic source (fault) is defined by the NGA relationships as the closest distance to the seismogenic zone, be it in the subsurface or at the surface; distances may therefore differ from distances measured on the ground surface. The distances shown on the table below are for reference only, as they are horizontal distances from the site to the surface trace of the seismic source, and not necessarily the closest distance to a (dipping) seismogenic zone. These distances were measured using the US Geological Survey's Quaternary Fault and Fold Database, with major faults listed in approximate order of distance from the site; not all sources are listed in the summary table below.

Table 4.1-1: Distance and Orientation to Nearby Faults

Fault Name	Approximate Distance	Orientation from Site
Hayward (southern section)	4 km	Northeast
Calaveras (northern section)	11 km	Northeast
San Andreas (Peninsula section)	25.5 km	Southwest
San Gregorio	41.4 km	Southwest

4.2 Ground Accelerations

According to the 2019 California Building Code (CBC) and American Society of Civil Engineers (ASCE) Standard 7-16, the spectral response acceleration at any period can be taken as the lesser of the spectral response accelerations from the probabilistic and deterministic ground motion approaches. The U.S. Seismic Design Maps tool available at the Structural Engineers Association of California (SEAOC) website was used for this purpose to retrieve seismic design parameter values for design of buildings at the subject site. Two levels of ground motions are considered in the Application: Risk-targeted Maximum Considered Earthquake (MCE_R) and Design Earthquake (DE), with both probabilistic and deterministic values defined in terms of maximum-direction rather than geometric-mean, horizontal spectral acceleration (S_a). The probabilistic MCE_R spectral response accelerations are represented by a 5 percent damped acceleration response spectrum having a 1 percent probability of collapse within a 50-year period and in the direction of the maximum horizontal response. The probabilistic Design Earthquake (DE) S_a value at any period can be taken as two-thirds of the MCE_R S_a value at the same period.

Using the Seismic Design Maps application at the Structural Engineers Association of California (SEAOC) website, a site Class D, and the latitude and longitude of the site (latitude 37.5330426º N, longitude -122.0098198º W), the calculated geometric mean peak ground acceleration adjusted for site class effects (PGA_M) for the MCE_G (Geometric Mean Maximum Considered Earthquake) is 0.84g.

4.3 Seismicity

The Working Group on California Earthquake Probabilities' (WGCEP) estimates of the probabilities of major earthquakes are now in their sixth iteration, with the greatest changes in approach being the inclusion of multifold rupture scenarios, in the progressive consideration of more potential seismic sources, the possibility of earthquakes on unrecognized faults, and the inclusion of the notion of fault "readiness". Current estimates (WGCEP, 2014) for the San Francisco region indicate a 72% probability of a large (magnitude 6.7 or greater) earthquake in the San Francisco Bay area as a whole over the 30-year period beginning in 2014; this overall probability is greater than the previous (WGCEP, 2007) probability of 63%, due mainly to the inclusion of multi-fault rupture scenarios. The estimate for the Calaveras fault alone is 14.4% (revised up from the 7% presented by WGCEP, 2007); for the (northern) San Andreas fault alone, 27.4% (revised upward from the WGCEP (2007) value of 21%); and for the Hayward fault, 45.3% (revised upward from the WGCEP (2007) value of 31%).

4.4 Liquefaction

Soil liquefaction is a phenomenon in which saturated granular soils, and certain fine-grained soils, lose their strength due to the build-up of excess pore water pressure during cyclic loading, such as that induced by earthquakes. Soils most susceptible to liquefaction are saturated, clean, loose, fine-grained sands and non-plastic silts. Certain gravels, plastic silts, and clays are also susceptible to liquefaction. The primary factors affecting soil liquefaction include: 1) intensity and duration of seismic shaking; 2) soil type; 3) relative density of granular soils; 4) moisture content and plasticity of fine-grained soils; 5) overburden pressure; and 6) depth to ground water.

Our review of the California Geological Survey (CGS) Earthquake Zones of Required Investigation indicates the project area is located in a liquefaction hazard zone.

CPT-1, CPT-2, and CPT-3 were used in a site-specific liquefaction assessment, with computer program CLiq V.2.1.6.11 from GeoLogismiki Geotechnical Software. Our liquefaction assessment was based on a PGA value of 0.84g, earthquake moment magnitude of 7.5, and a groundwater level of 10 feet bgs. The results of our analysis suggest that some of the underlying soils may liquefy when subject to shaking by the design earthquake, as shown in the table below.

Table 4.4-1: Potential Liquefaction Zones and Estimated Total Settlements		
CPT	**Potential Liquefaction Zones (feet)***	**Estimated Settlements (inches)**
CPT-1	10-12.3; 29-30.7	1.6
CPT-2	13.5-14.4; 28.0-30.5; 41.5-42.2	1.5
CPT-3	10.8-11; 12.3-12.6; 20-20.3; 24.1-25.4; 26.9-31.7	2.1
* very thin sand lenses not included		

The estimated liquefaction-induced ground settlement is on the order of 1½ to 2¼ inches. The estimated Liquefaction Potential Index (LPI) ranges between 6.8 and 10.8. For LPI between 5 and 15, the risk of liquefaction is high. The results of our liquefaction analysis are presented in Appendix C.

CGS Special Publication 117A "Guidelines for Evaluating and Mitigating Seismic Hazards in California" mentions that structural mitigation may be acceptable where liquefaction-related displacement of less than 1 foot horizontal and less than 4 inches vertical are predicted. Table 6 of SP 117A "Recommended Liquefaction Mitigation Techniques (Modified from Mitchell, 1995 and Hayward Baker, 1997)" lists post-tensioned slab as one of the options under the "Structural" category. We have discussed the use of post-tensioned slab foundations for the proposed structures with the project structural engineer and we understand that the post-tensioned slabs can be designed to mitigate the potential liquefaction-induced settlements. Refer to the "Recommendations" section of this report for recommended design parameters.

Project design should consider the potential liquefaction-induced settlements in surface drainage, gravity flow utilities, pipe connections, etc.

4.5 Lateral Spreading

Lateral spreading is horizontal movement of soil toward a free face, such as a creek bank, typically associated with liquefaction. Liquefaction-induced lateral spreading can also occur on mild slopes (flatter than 5%) underlain by loose sands and a shallow groundwater table. If liquefaction occurs, the unsaturated overburden soil can slide as intact blocks over the lower, liquefied deposit, creating fissures and scarps. The potential for lateral spreading in general mirrors the potential for liquefaction, and the depth of the liquefiable soil layers with respect to the creek banks.

The property is bordered by an Alameda County Flood Control and Water Conservation District (ACFCWCD) channel on the southeast. This channel is about 10 feet in depth and both banks are lined with Gabion type retaining structures. The top of the western channel bank is about 20 to 25 feet from the eastern property line of the subject project, and about 38 to 42 feet horizontally from the proposed houses along the eastern property boundary.

To evaluate the stability of the channel bank, we performed a static and pseudo-static slope stability analysis using the computer program SLOPE/W. The soil properties used in our analysis were developed using our drill hole and CPT data and are shown in Table 4.5-1 below. A seismic coefficient of 0.25 was used in our analysis.

Table 4.5-1: Soil Properties for Slope Stability Analysis

Depth Below Ground Surface (feet)	Soil	Static Analysis		Pseudo-static Analysis	
		Cohesion (psf)	Friction Angle (degrees)	Cohesion (psf)	Friction Angle (degrees)
0 – 4.5	Clay	1700	0	1700	0
4.5 – 10 (1)	Silty Sand to Sandy Silt	0	34	0	34
10 – 12.3	Silty Sand to Sandy Silt	0	34	800(2)	0
12.3 - 23	Clay	1700	0	1700	0
23 - 23.8	Sand to Silty Sand	0	34	800(2)	0
23.8 - 28.5	Clay	1700	0	1700	0
28.5 - 31	Silty Sand to Sandy Silt	0	34	800(2)	0
31- 47	Clay	1700	0	1700	0
47 - 50	Sand to Silty Sand	0	34	800(2)	0

Notes:
1. Groundwater at 10 feet below ground surface.
2. Estimated residual shear strength for liquefiable soils based on CPT-1.

The results of our analysis indicates a static factor of safety of 5.6 and pseudo-static factors of safety of 2.8 under a global search and 2.4 when the failure plane was forced to go through the potentially liquefiable sand layer between 10 and 12.3 feet bgs. Based on these results, it appears the potential for lateral spreading to affect the project site is low. The results of our slope stability analysis are presented in Appendix D.

4.6 Seismic Design Parameters

Design of the proposed structures should comply with design for structures located in seismically active areas. Structures should be designed in accordance with the requirements of governing jurisdictions and applicable building codes. GLA evaluated ASCE 7-16 seismic design parameters for the site using the SEAOC U.S. Design Maps application. The table below lists the seismic design parameters for the site. Note that, in accordance with Section 11.4.8 of ASCE 7-16, a ground motion hazard analysis is required because the Mapped Spectral Acceleration at 1.0-second Period (S_1) value for the site is greater than or equal to 0.2 g, unless the exceptions in Section 11.4.8 are met. This should be verified by the project structural engineer.

Table 4.5-1: Seismic Design Parameters for Buildings Based on 2019 CBC & ASCE 7-16	
Seismic Design Parameter	**Value**
Site Class	D
Site Coefficient, F_a	1.0
Site Coefficient, F_v	1.7
Mapped Spectral Acceleration at 0.2-second Period, S_s	1.825 g
Mapped Spectral Acceleration at 1.0-second Period, S_1	0.695 g
Spectral Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	1.825 g
Spectral Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	1.18 g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	1.217 g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.787 g
Long-period Transition Period, T_L	12 sec.

Note: The site would be Site Class F because it is underlain by potentially liquefiable soils. But if the fundamental period of vibration of the proposed structures is equal to or less than 0.5 second, the Site Class can be determined by assuming there is no liquefaction (ASCE 7-16 Section 20.3.1). Therefore, Site Class D was selected for this project. If the fundamental period of vibration of the proposed structures is larger than 0.5 second as determined by the project structural engineer, GLA should be contacted for a site-specific seismic response analysis.

5 CONCLUSIONS AND DISCUSSION

Based on our geotechnical evaluation, it is our opinion the project site may be developed as discussed in this report, provided our geotechnical recommendations are incorporated in the design and construction of the project. Our opinions, conclusions, and recommendations are based on our understanding of the proposed development, data review, properties of soils encountered in subsurface exploration, laboratory test results, and engineering analyses. Geotechnical considerations for this project are discussed below.

5.1 Ground Rupture

The project site is not located in an Alquist-Priolo Earthquake Fault Zone. Because no active or potentially active faults are known to cross the site, it is reasonable to conclude that the risk of fault rupture through the project site is low.

5.2 Seismic Shaking

The project site is located in an area of high seismicity. Based on general knowledge of the site seismicity, it should be anticipated that, during their useful life, the proposed structures will be subject to at least one severe earthquake (magnitude 7 to 8+) that could cause considerable ground shaking at the site. It is also anticipated that the site will periodically experience small to moderate magnitude earthquakes.

5.3 Liquefaction

The results of our liquefaction analysis indicate that some of the subsurface soils are prone to liquefaction when subject to seismic shaking. The estimated liquefaction-induced total ground settlements are on the order of 1½ to 2¼ inches. The potential differential settlements would be on the order of 1 inch. Based on our discussion with the project structural engineer, the post-tensioned slab foundations can be designed to accommodate the potential settlements from building loads and liquefaction.

5.4 Lateral Spreading

As discussed in Section 4.5 above, the results of our stability analysis indicates a static factor of safety of 5.6 and pseudo-static factors of safety of 2.8 under a global search and 2.4 when the failure plane was forced to go through the potentially liquefiable sand layer between 10 and 12.3 feet bgs. Based on these results, it appears the potential for lateral spreading to effect the project is low.

5.5 Expansion Potential of Surficial Soils

The results of two Atterberg limits tests performed on near-surface soil samples collected in our drill holes indicate the soil has an intermediate plasticity which generally corresponds to a moderate to high expansion potential.

Expansive soils are characterized by their ability to undergo significant volume change (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from rainfall, landscape irrigation, perched groundwater, drought or other factors. Changes in soil moisture may result in unacceptable settlement or heave of structures, concrete slabs and pavements supported on these materials. Depending on the extent and location below finished subgrade, these soils could have a detrimental effect on the proposed construction.

To reduce its potential impact on the proposed structures, the upper 30 inches of soil below design grade in the proposed building and concrete slab-on-grade areas should be moisture conditioned with controlled compaction per the “Geotechnical Recommendations” section of this report. The post-tensioned slab foundations for the proposed structures should be designed using the recommended parameters in this report to accommodate the potential effect of soil expansion.

5.6 Shallow Groundwater

Groundwater was encountered at 10 to 11 feet bgs at the site (see Section 3.3 of this report). Typically, groundwater levels fluctuate between the dry summer and wet winter months. Design and construction of underground improvements should consider the relatively shallow groundwater level at the site. Groundwater can reduce stability of excavation side-walls, impede construction, and induce buoyancy force on buried pipes. Soils below groundwater table will be wet and require drying before the material can be used as fill.

Excavations extending below groundwater will require dewatering and special considerations so construction can proceed in a "dry" condition. Refer to the “Recommendations” section of this report.

5.7 Existing Improvements

Existing improvements at the site include miscellaneous structures, underground utilities, chain link fences, and isolated trees. Prior to construction, the existing structures and improvements should be removed and the resulting excavations should be properly backfilled with engineered fill under the observation and testing of the project Geotechnical Engineer.

6 GEOTECHNICAL RECOMMENDATIONS

6.1 Earthwork

6.1.1 Site Preparation, Clearing and Stripping

Prior to grading, construction areas should be cleared of all structures and foundations, obstructions, deleterious materials, abandoned or designated utility lines, designated trees, and other below grade obstacles encountered during the clearing operation. The northern portion of the site was once occupied by a street and utilities are expected to remain under the street. Tree stumps should be grubbed. Roots with diameter of about 1 inch or larger or length of about 3 feet or longer should be removed. Depressions, excavations, and holes that extend below the planned finish grades should be cleaned and backfilled with engineered fill compacted to the requirements given under the section of "Engineered Fill Placement and Compaction."

After clearing, vegetated areas should be stripped to sufficient depth to remove vegetation and organic-laden topsoil. Stripped material may be stockpiled for use in landscape areas if approved by the project landscape architect; otherwise, it should be removed from the site. For planning purposes, an estimated stripping depth of 1 to 3 inches may be assumed in unpaved areas. The actual stripping depth should be determined in the field by the Geotechnical Engineer at the time of construction.

6.1.2 Excavation, Temporary Construction Slopes, Shoring and Dewatering

Excavations for this project are expected to include demolition excavations, cuts to achieve design grades, over-excavations to remove loose and/or disturbed soils, trenching to construct new underground utilities, and foundation excavations. Excavation walls in clayey soil and less than 5 feet in height should be able to stand near vertical with minimal bracing, provided proper moisture content in the soil is maintained. Granular (sand and gravel) soils, typically have little or no cohesion, will require more extensive bracing or laying back because they are prone to sudden collapse. Excavations and temporary construction slopes should be constructed in accordance with the current CAL-OSHA safety standards and local jurisdiction. The stability and safety of excavations, braced or unbraced, is the responsibility of the contractor. Care should be exercised when excavating in the proximity of existing structures and improvements.

Contractors are responsible for the design, installation, maintenance, and removal of temporary shoring and bracing systems. The presence of existing structures, pavements, and underground utilities must be incorporated in the design of the shoring and bracing systems.

The relatively shallow groundwater level should be considered in the design and construction of excavations. Excavations extending below groundwater will require dewatering. Dewatering should lower the groundwater level to at least 2 feet below the bottom of the excavations. The design, installation, permitting, maintenance and removal of the dewatering system are the responsibility of the contractor. Wet and soft soils, if encountered in the bottom of the excavations, should be over-excavated and replaced with ¾-inch by No. 4, clean, crushed rock to create a stable working surface. The depth of over-excavations should be a function of the depth of wet and soft soils. A geotextile fabric may be necessary to help stabilize the wet and soft soil subgrade.

Trench excavations adjacent to existing or proposed foundations should be above an imaginary plane having an inclination of 1½:1 (horizontal to vertical) extending down from the bottom edge of the foundations.

6.1.3 Subgrade Preparation

After site clearing and stripping, the soil subgrades should be prepared as recommended below.

Building and concrete slab-on-grade areas: Soils in building and concrete slab-on-grade areas should be over-excavated to at least 18 inches below design pad grade, but not less than 12 inches below existing grade. The soil surfaces exposed by over-excavation should be scarified to a depth of 12 inches, moisture-conditioned, and compacted in accordance with the recommendations given in the "Engineered Fill Placement and Compaction" section below. In structure areas to receive concrete slabs-on-grade or foundations, subgrade preparation should extend a minimum of 5 feet horizontally beyond the limits of the proposed structures and any adjoining flatwork, unless it is restricted by existing improvements.

Pavement areas: Soils in pavement areas should be over-excavated to at least 12 inches below existing ground surface. The soil surfaces exposed by over-excavation should be scarified to a depth of 8 inches, moisture-conditioned, and compacted in accordance with the recommendations given in the "Engineered Fill Placement and Compaction" section below. Subgrade preparation should extend a minimum of 3 feet beyond the back of the curbs or pavements.

Prepared soil subgrades should be non-yielding when proof-rolled by a fully loaded water truck or similar weight equipment. Moisture conditioning of subgrade soils should consist of adding water if the soils are too dry and allowing the soils to dry if the soils are too wet. After the subgrades are properly prepared, the areas may be raised to design grades by placement of engineered fill.

Wet soils should be anticipated during and after rainy months. Where encountered, unstable, wet or soft soil will require processing before compaction can be achieved. If construction schedule does not allow for air-drying, other means such as lime or cement treatment of the soil or excavation and replacement with suitable material may be considered. Geotextile fabrics may also be used to help stabilize the subgrade. The method to be used should be determined at the time of construction based on the actual site conditions. We recommend obtaining unit prices for subgrade stabilization during the construction bid process.

6.1.4 Materials for Fill

In general, on-site soils with an organic content of less than 3 percent by weight, free of deleterious materials or hazardous substances, and meeting the gradation requirements below may be used as engineered fill except where special material is required. The existing asphalt concrete, if properly pulverized to meet the gradation requirements below, and the existing aggregate base may also be used as general engineered fill.

Engineered fill material should not contain rocks or lumps larger than 3 inches in greatest dimension, should not contain more than 15 percent of the material larger than 1½ inches, and should contain at least 20 percent passing the No. 200 sieve. In addition to these requirements, import fill, including "non-expansive" fill, should have a low expansion potential as indicated by Plasticity Index of 15 or less (per ASTM D4318), or Expansion Index of less than 20 (per ASTM D4829).

All fills should be approved by the project Geotechnical Engineer prior to delivery to the site. At least 5 working days prior to importing to the site, a representative sample of the proposed import fill should be delivered to our laboratory for evaluation. Import fills should be tested and approved for residential use per the California Department of Toxic Substances Control (DTSC) guidelines.

6.1.5 Engineered Fill Placement and Compaction

Engineered fill should be placed in horizontal lifts each not exceeding 8 inches in thickness, moisture conditioned to the required moisture content, and mechanically compacted to the recommendations below. Relative compaction or compaction is defined as the in-place dry density of the compacted soil divided by the laboratory maximum dry density as determined by ASTM Test Method D1557, latest edition, expressed as a percentage. Moisture conditioning of soils should consist of adding water to the soils if they are too dry and allowing the soils to dry if they are too wet.

Engineered fills consisting of expansive soils should be compacted to between 87 and 92 percent relative compaction at moisture content between 3 and 6 percent above the laboratory optimum value. Engineered fills consisting of soils of low expansion potential, including "non-expansive" fill, should be compacted to a minimum of 90 percent relative compaction with moisture content between about 1 and 3 percent above the laboratory optimum value. In pavement areas, the upper 8 inches of subgrade soil should be compacted to a minimum of 95 percent relative compaction. Aggregate base in vehicle pavement areas should be compacted at slightly above the optimum moisture content to a minimum of 95 percent relative compaction.

6.1.6 Utility Trench Backfill

Backfilling of utility trenches in public right-of-way and street areas should comply with requirements of City of Newark and relevant utility agencies.

Backfilling of utility trenches in private areas may consist of pipe bedding extending from the bottom of the trench to about 1 foot above the top of pipe and backfill material above. Pipe bedding may consist of free-draining sand (less than 5% passing a No. 200 sieve), lean concrete or sand cement slurry. Sand, if used as bedding, should be compacted to a minimum of 90 percent relative compaction.

Above the pipe bedding, utility trenches should be backfilled per requirements of City of Newark or relevant utility agencies. Trench backfill above the pipe bedding should be compacted to the requirements given in the section of "Engineered Fill Placement and Compaction." The backfill material should be placed in lifts not exceeding about 6 inches in uncompacted thickness. Thinner lifts may be necessary to achieve the recommended level of compaction of the backfill due to equipment limitations. Compaction should be performed by mechanical means only. Water jetting or flooding to attain compaction of backfill should not be permitted.

6.1.7 Considerations for Soil Moisture and Seepage Control

Subgrade soil and engineered fill should be compacted at moisture content meeting our recommendations. Consideration should be given to reducing the potential for water infiltration from the exterior to under the buildings through utility lines crossing the building perimeter. In utility lines crossing beneath perimeter foundations, permeable backfill should be terminated at least 1 foot outside of the perimeter foundation. Impermeable material, such as concrete or clay soil, should be used for the entire trench depth to act as a seepage cutoff.

Where concrete slabs or pavements abut against landscaped areas, the base rock layer and subgrade soil should be protected against saturation. Water if allowed to seep into the subgrade soil or pavement section could reduce the service life of the improvements. Methods that may be considered to reduce infiltration of water include: 1) subdrains installed behind curbs and slabs in landscape areas; 2) vertical cut-offs, such as a deepened curb section, or equivalent, extending at least 2 inches into the subgrade soil; and 3) use of a drip or controlled irrigation system for landscape watering.

6.1.8 Wet Weather Construction

If site grading and construction is to be performed during the winter rainy months, the owner and contractors should be fully aware of the potential impact of wet weather. Rainstorms can cause delay to construction and damage to previously completed work by saturating compacted pads or subgrades, or flooding excavations.

Earthwork during rainy months will require extra effort and caution by the contractors. The contractors are responsible for protecting their work to avoid damage by rainwater. Standing pools of water should be pumped out immediately. Construction during wet weather conditions should be addressed in the project construction bid documents and/or specifications. We recommend the contractors submit a wet weather construction plan outlining procedures they will employ to protect their work and to minimize damage to their work by rainstorms.

6.2 Foundations

6.2.1 General

The Geotechnical Engineer should review the foundation plans and details before construction and observe the foundation excavations during construction to determine if the foundation excavations extend into suitable bearing material. Prior to placement of concrete, foundation excavations should be cleaned of loose soils. If unsuitable soils are encountered in the foundation excavations, the soils should be removed as recommended by our Geotechnical Engineer and replaced with approved material such as compacted engineered fill or lean concrete.

Foundation excavations should not be allowed to dry before placement of concrete. If visible cracks appear in the foundation excavations, the excavations should be thoroughly moisture conditioned beginning at least 2 days prior to placement of concrete to close all cracks. It is also important that the base of the foundation excavations not be allowed to become excessively wet, resulting in soft soils. Water should not be allowed to pond in the bottom of the excavations. Areas that become water damaged should be over-excavated to a firm base. The foundation excavations should be monitored by our representative for compliance with appropriate moisture control and to confirm the adequacy of the bearing materials.

To maintain the desired support, the bottom of foundations and other structural improvements (e.g. curbs, sidewalks, etc.) adjacent to below-ground improvements, including utility trenches and bio-retention facilities, should be below an imaginary plane having an inclination of 1.5 horizontal to 1 vertical, extending upward from the bottom edge of the adjacent buried improvements.

6.2.2 Post-tensioned Slabs (Buildings)

The proposed residential structures may be supported on post-tensioned (PT) slab foundations bearing on properly moisture-conditioned and compacted on-site soil. Preparation of soil subgrade, moisture conditioning, and compaction of soil and engineered fill should be as recommended in the “Earthwork” section of this report. At least one week prior to slab construction, the moisture content of the subgrade soil should be evaluated. If the soil’s moisture content is lower than the recommended value of at least 3 percent above the laboratory optimum moisture content, water should be added to bring the soil’s moisture content to above the recommended value.

The following parameters may be used with the 2004 PTI "Design of Post-Tensioned Slabs-on-Ground, Third Edition" manual for design of the PT slabs. Per request of the structural engineer, two sets of parameters are provided – scenario 1 based on potential effects from on-site expansive soils and scenario 2 based on potential effects of liquefaction-induced settlements. These parameters are based on ASCE 7-16 Section 12.13.9.2.1.1 which indicates that "for sites with expansive soils, movements from both expansive soils and liquefied soils need not be considered concurrently."

Table 6.2-1: Parameters for Design of Post-tensioned Slabs Constructed on Native Expansive Soil

Parameters	Scenario 1 – effects from expansive soil	Scenario 2 – effects from liquefaction-induced settlements
e_m (center lift)	8.5 feet	8.5 feet
e_m (edge lift)	5 feet	8.5 feet
y_m (center lift)	0.85 inch	1.1 inch
y_m (edge lift)	2.3 inch	1.1 inch

Allowable soil bearing pressure = 1,500 psf for dead plus live loads, with a one-third increase when including transient loads, such as wind or seismic

A deepened edge, minimum 6 inches wide, should be constructed along the perimeter of the PT slabs. The deepened edge should extend to at least 18 inches below the bottom of the PT slabs (see Figure 3). The deepened edge can help reduce moisture infiltration to under the PT slabs.

Where interior building grades are higher than the exterior grades, the perimeter foundation elements should be designed to resist the lateral soil pressure and surcharge loads acting on the foundations. The bottom of the perimeter foundations should extend at least 18 inches below the lowest finish grades, excluding landscaping soils which are typically not compacted and should not be considered for structural support.

The PT slabs may be constructed on 1 to 2 inches of sand over a 15-mil visqueen vapor barrier over compacted subgrade soil provided a lower water-cement ratio (0.45 to 0.50) is used to help reduce the permeability of the concrete and, hence, vapor transmission through the PT slabs. Sand has been used for protection of the vapor barrier during construction and to allow dissipation of concrete mix water during curing. The use of sand, or equivalent material, should be determined by the project structural engineer or architect.

Settlements under building loads are expected to be primarily elastic. Post construction total and differential settlements of the PT slabs under non-seismic conditions are anticipated to be less than 1 and ½ inch, respectively. Refer to the "Liquefaction" section of this report for estimated liquefaction-induced ground settlement.

6.2.3 Conventional Footings (Retaining Walls, etc.)

Footings, continuous and isolated, may be used to support site landscaping retaining walls, anticipated to be less than about 3 feet in exposed height. The perimeter sound walls which may retain up to about 2 feet of soil may be supported on conventional footings or driller piers (see below). Footings should bear on undisturbed native soil and/or properly compacted engineered fill. Preparation of soil subgrade, moisture conditioning, and compaction of soil and engineered fill should be as recommended in the "Earthwork" section of this report.

Footings may be designed for a net allowable bearing pressure of 3,000 pounds per square foot due to dead plus live loads, with a one-third increase when including transient loads such as wind or seismic. The footing bottom should extend at least 18 inches below pad grade or lowest adjacent finish grade, whichever provides a deeper embedment. Footings should be at least 12 inches wide. Footings should be reinforced as determined by the project Structural Engineer.

Resistance to lateral loads may be developed from a combination of friction between the bottom of foundations and the supporting subgrade, and by passive resistance acting against the vertical sides of the foundations. Footings bearing on native soil or engineered fill may be designed using an ultimate friction coefficient of 0.3 between the foundations and supporting subgrade, and an ultimate passive resistance of 300 pounds per cubic foot (pcf, equivalent fluid weight) acting against the embedded sides of the foundations. The passive pressure can be assumed to act starting at the top of the lowest adjacent grade in paved areas. In unpaved areas, the passive pressure can be assumed to act starting at a depth of 1 foot below grade. It should be noted that the passive resistance value discussed above is only applicable where the concrete is placed directly against undisturbed soil or engineered fills. Voids created by the use of forms should be backfilled with property compacted engineered fill or with concrete.

Total post-construction settlement of the foundations under non-seismic conditions is anticipated to be up to about 1 inch, with up to about ½ inch of differential settlement over a distance of about 30 feet.

6.2.4 Drilled Pier Foundations

Drilled, cast-in-place, reinforced concrete piers may be considered for support of the proposed perimeter sound walls which may retain up to 2 feet of soil. Piers should be designed to derive their vertical supporting capacity from "skin friction" between the pier shafts and the surrounding earth materials. Piers should have a diameter of 12 inches or greater. Center to center spacing of the piers should be a minimum of 3 pier diameters. Reinforcement in the piers should be determined by the structural engineer.

Recommended allowable skin friction values for design of drilled piers are shown in Table 6.2-2 below. These values are for dead plus live vertical loads, and may be increased by one-third when including transient loads, such as wind or seismic. End bearing capacity of the piers should be ignored.

Resistance to lateral loads may be calculated based on passive soil pressure acting against the piers. The values in Table 6.2-2 may be assumed to act on 2 times the pier diameter, for level ground surface in front of the piers in the direction of load application. The upper 1 foot of soil should be ignored in the calculation of passive pressure. It should be noted that passive resistance is only applicable where the concrete is placed directly against undisturbed soil or engineered fill.

Table 6.2-2: Recommended Adhesion Values for Drilled Piers

Depth (feet)	Allowable Skin Friction (psf)	Ultimate Passive Value (psf/ft)
0 - 7	500	300
7 – 12	300	175
10 – 20	400	175

The presence of groundwater should be considered in the design and construction of the foundation piers. If piers extend below groundwater level, concrete should be placed by the “tremie” method to replace the water in the pier holes. The presence of granular soils should also be considered in the design and construction of drilled piers.

6.3 Concrete Slabs-on-Grade

The interior building slabs will be the PT slabs.

Exterior concrete slabs-on-grade for this project will be limited to driveways and exterior flatwork. Concrete for driveways should be at least 6 inches thick and concrete for exterior walks and patios should be at least 4 inches thick. The concrete slabs should be constructed on a 4-inch minimum thick section of Class 2 Aggregate Base over properly prepared subgrade soil as recommended in the “Earthwork” section of this report. At least one week prior to slab construction, the moisture content of the subgrade soil should be evaluated. If the soil’s moisture content is lower than the recommended value of at least 3 percent above the laboratory optimum moisture content, water should be added to bring the soil’s moisture content to above the recommended value. Design of reinforcement, joint spacing, etc. is the responsibility of the design engineer.

Exterior concrete slabs-on-grade should be cast free from adjacent foundations or other non-heaving edge restraints. This may be accomplished by using a strip of 1/2-inch asphalt-impregnated felt divider material between the slab edges and the adjacent structure. Frequent construction or control joints should be provided in all concrete slabs where cracking is objectionable. Continuous reinforcing or dowels at the construction and control joints will also aid in reducing uneven slab movements.

6.4 Retaining Walls

Retaining walls for this project are anticipated to be landscaping walls with exposed height up to about 3 feet. The proposed perimeter sound walls may also retain soil about 2 feet thick.

Retaining walls should be designed to resist lateral earth pressure and surcharge forces acting on the walls. Lateral pressures will depend on the degree of movement the walls are allowed (or desired), the type of backfill, the magnitude of external loads, and subsurface drainage provisions.

For static loading conditions, the walls may be designed using at-rest or active soil pressure. At-rest soil pressure should be used for walls where movement at the top of walls is restrained or undesirable. Wall movements could cause settlement of backfill and structures supported on the backfill. Active soil pressure may be used for retaining walls where the top of walls is free to deflect and resulting movement of the backfill is acceptable. The at-rest and active soil pressures given below are for level backfill surface and do not include hydrostatic pressure caused by water behind the walls.

Table 6.4-1: Recommended Lateral Soil Pressures for Retaining Walls

Condition	Lateral Soil Pressure (Equivalent Fluid Weight) for Level Backfill
Active	45 pcf
At-rest	55 pcf

Note: To develop active soil pressures, wall movements of about 0.005H to 0.01H may be necessary for cohesive soils, with up to 0.005H for cohesionless soils.

Pressures due to static external loads should be added to the soil pressures recommended above in the wall design. For uniform vertical load at the ground surface, the additional lateral pressure on the walls should be calculated as a uniform pressure equal to the magnitude of the vertical load multiplied by a factor. For level backfill slope, the factor is 0.38 for active soil condition and 0.5 for at-rest soil condition. For other slope inclinations and other types of surcharge loads, such as vehicle loads, point loads, strip loads, consult our office for specific recommendations.

Foundations for retaining walls may consist of footings or drilled piers designed using the recommendations in the "Foundations" Section of this report.

To achieve a drained backfill condition, a subsurface drain should be installed behind each wall extending from the wall bottom to about 1 foot below finished grade. The drain should consist of a 12-inch minimum wide blanket of drainage material consisting of either Class 2 Permeable material (Caltrans Standard Specifications, Section 68) or clean, 1/2 to 3/4-inch maximum size crushed rock or gravel. If crushed rock or gravel is used, it should be encapsulated in a geotextile filter fabric, such as Mirafi 140N or equivalent. Filter fabric is optional if Class 2 Permeable material is used. The top 1 foot below finish grade should be backfilled with compacted clayey soil to reduce infiltration of surface water.

A 4-inch minimum diameter, perforated, schedule 40 PVC (or equivalent) pipe should be installed (with perforations facing down) along the base of each wall on a 2-inch thick bed of drain rock, regardless whether drain rock or pre-fabricated drainage panel is used. The pipes should be sloped to drain by gravity to a proper collection system and be discharged at a proper outlet as designed by the project Civil Engineer.

Backfill against retaining walls should be compacted as discussed in the "Earthwork" Section of this report. Over-compaction should be avoided because increased compaction effort can result in lateral pressures significantly higher than those recommended above. Backfill placed within 3 feet of the walls should be compacted with hand-operated equipment.

6.5 Vehicle Pavements

Vehicle pavements for this project will include interior streets, primarily serving automobiles and light pickup trucks, with occasional heavy vehicles, such as delivery and garbage trucks. If the pavements are constructed prior to completion of construction, the pavements will be subject to construction traffic including heavy delivery and concrete trucks.

R-values of 6 and 8 were measured on the two bulk soil samples collected at the site. For design purposes, an R-value of 5 was used to calculate the pavement sections tabulated below using the Caltrans pavement section design procedures.

We understand from the City of Newark that residential interior streets should be designed for a traffic index of at least 5.0, with minimum asphalt concrete thickness of 4 inches. The table below presents our recommended minimum flexible pavement sections for traffic indices between 5.0 and 6.5.

Table 6.5-1: Recommended Minimum Flexible Pavement Section

DESIGN TRAFFIC INDEX	HOT MIX ASPHALT (inches)	CLASS 2 AGGREGATE BASE (inches)	TOTAL (inches)
5.0	4.0	8.0	12.0
5.5	4.0	9.5	13.5
6.0	4.0	11.5	15.5
6.5	4.0	13.5	17.5
6.5	4.5	12.5	17.0

Pavement sections should be constructed on soil subgrades that have been prepared as outlined in the "Earthwork" section of this report. The upper 8 inches of soil subgrade in pavement areas should be compacted to a minimum of 95 percent relative compaction. The full section of aggregate base and aggregate subbase should be compacted to a minimum of 95 percent relative compaction. Evaluation of relative compaction should be based on ASTM D1557, latest edition. The Class 2 Aggregate Base material should conform to Section 26 of the

Caltrans Standard Specifications and the Class 2 Aggregate Subbase material should conform to Section 25 of the Caltrans Standard Specifications.

6.6 Surface and Subsurface Drainage

Engineering design of grading and drainage at the site is the responsibility of the project Civil Engineer. We suggest the following for consideration by the project Civil Engineer, as appropriate.

Sufficient surface drainage should be provided to direct water away from buildings, foundations, concrete slabs-on-grade and pavements, and towards suitable collection and discharge facilities. Ponding of surface water should be avoided by establishing positive drainage away from all improvements.

6.7 Stormwater Treatment System

Bioretention basins for on-site stormwater treatment and retention are proposed for this project. The bioretention basins will consist of an 18-inch thick layer of bio-treatment soil mix (BSM) underlain by a 12-inch thick layer of Caltrans Class 2 Permeable material. We recommend the following guidelines be incorporated in the planning and design of the bioretention system.

- Underground vaults, bioretention basins, pipes, etc. should be constructed above an imaginary plane extending down at an inclination of 1.5:1 (horizontal:vertical) from the bottom edge or corner of nearby foundations. This may require deepening of the nearby foundations.

- Bioretention basins should be constructed above an imaginary plane extending down at an inclination of 1.5:1 (horizontal:vertical) from the bottom edge of nearby exterior flatwork or pavements. If this minimum set back is not met, the following should be considered.

    - Line the sides of the bioretention basins with an impermeable barrier to reduce lateral migration of water.

    - Install one or more layers of geogrids in the soil adjacent to the bioretention basins for added lateral support. If the vertical distance between the bottom of the bioretention basins and the adjacent finish grade (H) is 5 feet or less, one layer of geogrid at least 6 feet wide should be installed at mid-height (H/2). If H is greater than 5 feet, additional layers of geogrids should be installed at not more than 2 feet vertical spacing. The length and elevation of multi geogrid layers should be determined by the Geotechnical Engineer after review of the basin design.

- Construct concrete curbs for pavements. The concrete curbs should extend below the bottom of the bioretention basins and should be designed to resist the lateral soil pressure recommended in this report.

7 PLAN REVIEW, EARTHWORK AND FOUNDATION OBSERVATION

Post-report geotechnical services by Geo-Logic Associates (GLA), typically consisting of pre-construction design consultations and reviews and construction observation and testing services, are necessary for GLA to confirm the recommendations contained in this report. This report is based on limited sampling and investigation, and by those constraints may not have discovered local anomalies or other varying conditions that may exist on the project site. Therefore, this report is only preliminary until GLA can confirm that actual conditions in the ground conform to those anticipated in the report. Accordingly, as an integral part of this report, GLA recommends post-report, construction related geotechnical services to assist the project team during design and construction of the project. GLA requires that it perform these services if it is to remain as the project Geotechnical Engineer-of-record.

During design, GLA can provide consultation and supplemental recommendations to assist the project team in design and value engineering, especially if the project design has been modified after completion of our report. It is impossible for us to anticipate every design scenario and use of construction materials during preparation of our report. Therefore, retaining GLA to provide post-report consultation will help address design changes, answer questions and evaluate alternatives proposed by the project designers and contractors.

Prior to issuing project plans and specifications for construction bidding purposes, GLA should review the grading, drainage and foundation plans and the project specifications to determine if the intent of our recommendations has been incorporated in these documents. We have found that such a review process will help reduce the likelihood of misinterpretation of our recommendations which may cause construction delay and additional cost.

Construction phase services can include, among other things, the observation and testing during site clearing, stripping, excavation, mass grading, subgrade preparation, fill placement and compaction, backfill compaction, foundation construction and pavement construction activities.

Geo-Logic Associates would be pleased to provide cost proposals for follow-up geotechnical services. Post-report geotechnical services may include additional field and laboratory services.

8 LIMITATIONS

In preparing the findings and professional opinions presented in this report, Geo-Logic Associates (GLA) has endeavored to follow generally accepted principles and practices of the engineering geologic and geotechnical engineering professions in the area and at the time our services were performed. No warranty, either express or implied, is provided.

The conclusions and recommendations contained in this report are based, in part, on information that has been provided to us. In the event that the general development concept or general location and type of structures are modified, our conclusions and recommendations shall not be considered valid unless we are retained to review such changes and to make any necessary additions or changes to our recommendations. To remain as the project Geotechnical Engineer-of-record, GLA must be retained to provide geotechnical services as discussed under the Post-report Geotechnical Services section of this report.

Subsurface exploration is necessarily confined to selected locations and conditions may, and often do, vary between these locations. Should conditions different from those described in this report be encountered during project development, GLA should be consulted to review the conditions and determine whether our recommendations are still valid. Additional exploration, testing, and analysis may be required for such evaluation.

Should persons concerned with this project observe geotechnical features or conditions at the site or surrounding areas which are different from those described in this report, those observations should be reported immediately to GLA for evaluation.

It is important that the information in this report be made known to the design professionals involved with the project, that our recommendations be incorporated into project drawings and documents, and that the recommendations be carried out during construction by the contractor and subcontractors. It is not the responsibility of GLA to notify the design professionals and the project contractors and subcontractors.

The findings, conclusions, and recommendations in this report are applicable only to the specific project development on this specific site. These data should not be used for other projects, sites, or purposes unless they are reviewed by GLA or a qualified geotechnical professional.

Report prepared by,
Geo-Logic Associates

DRAFT FOR CLIENT REVIEW ONLY

Chalerm (Beeson) Liang
GE 2031

Fs/csl

Copies: Robson Homes, Dominic Boitano

APPROXIMATE SCALE (feet)

0 80

N

Legend

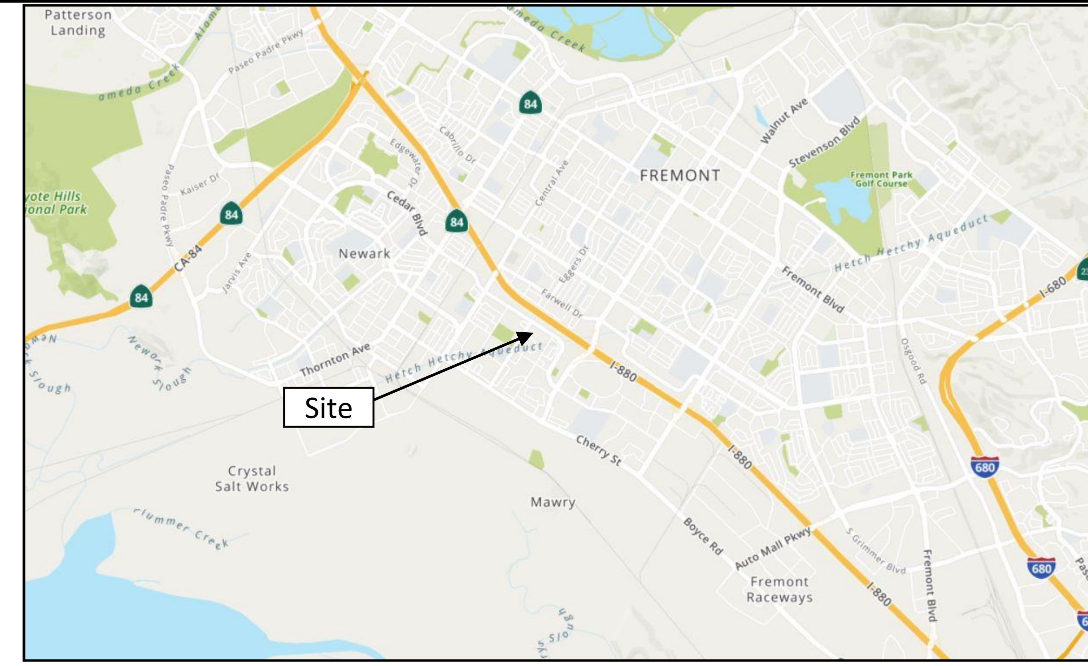
DH-8 Number & approximate location of exploratory drill hole

CPT-3 Number & approximate location of cone penetration test probe

B-2 Number & approximate location of bulk sample

Base

GoogleEarth



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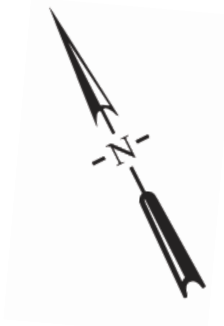
Geo-Logic ASSOCIATES	16055 Caputo Drive, Suite D Morgan Hill, California 95037 Phone (408) 778-2818 Fax (408) 779-6879	Drafted By: Date: Checked By: Revision:	**SITE PLAN (EXISTING CONDITIONS)** **38288-38594 CEDAR BOULEVARD** **NEWARK, CALIFORNIA**	**FIGURE** 1 **PROJECT** PA20.1048

APPROXIMATE SCALE (feet)

0 80



Legend

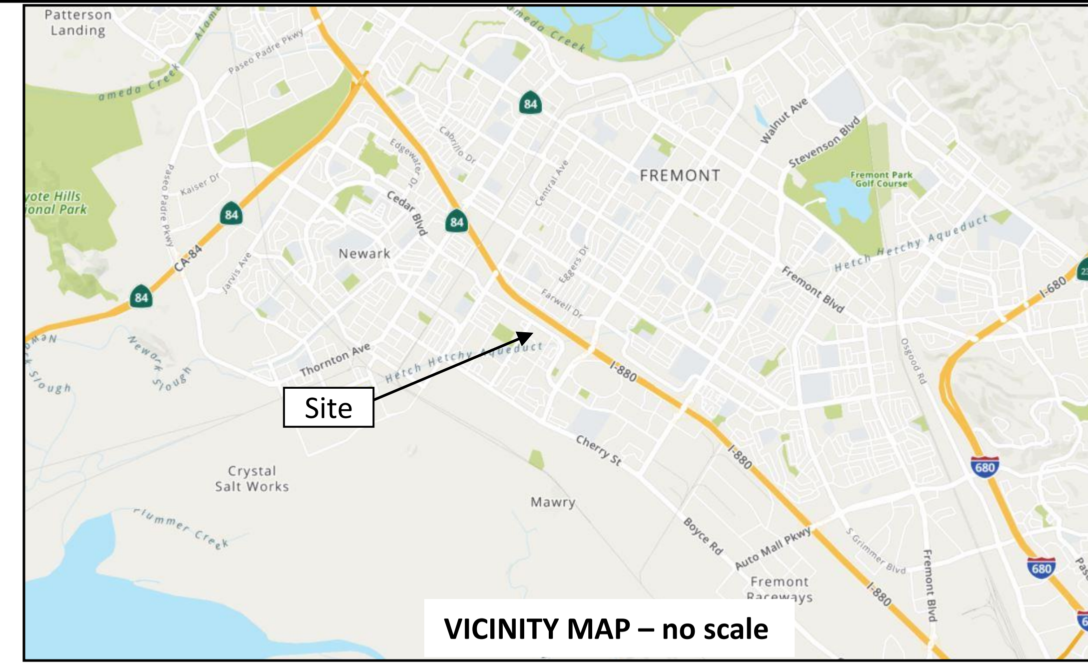
DH-8 Number & approximate location of exploratory drill hole

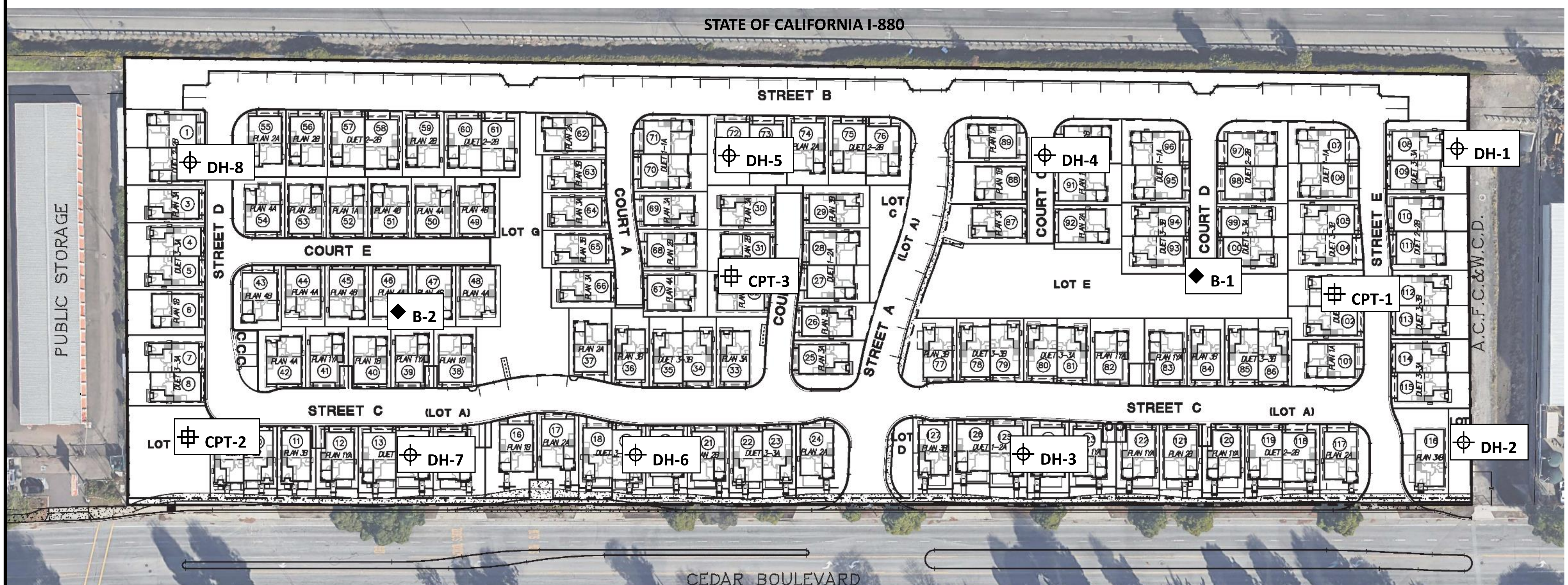
CPT-3 Number & approximate location of cone penetration test probe

B-2 Number & approximate location of bulk sample

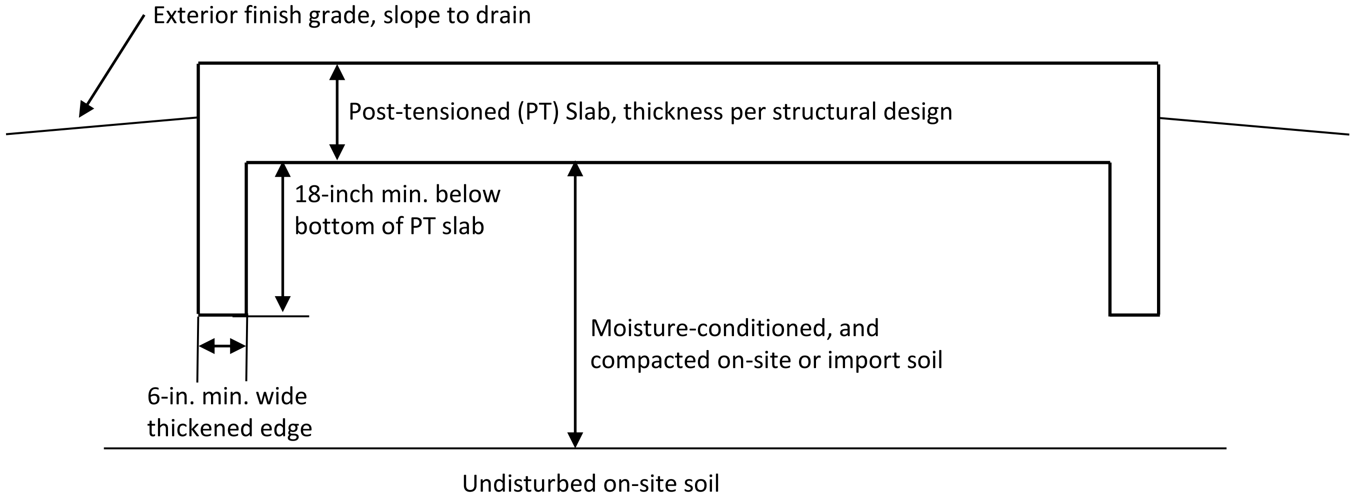
Base

Planned Development Permit, 38478 Cedar Boulevard, City of Newark, Alameda County, California, prepared by Ruggeri-Jensen-Azar, dated July 29, 2021

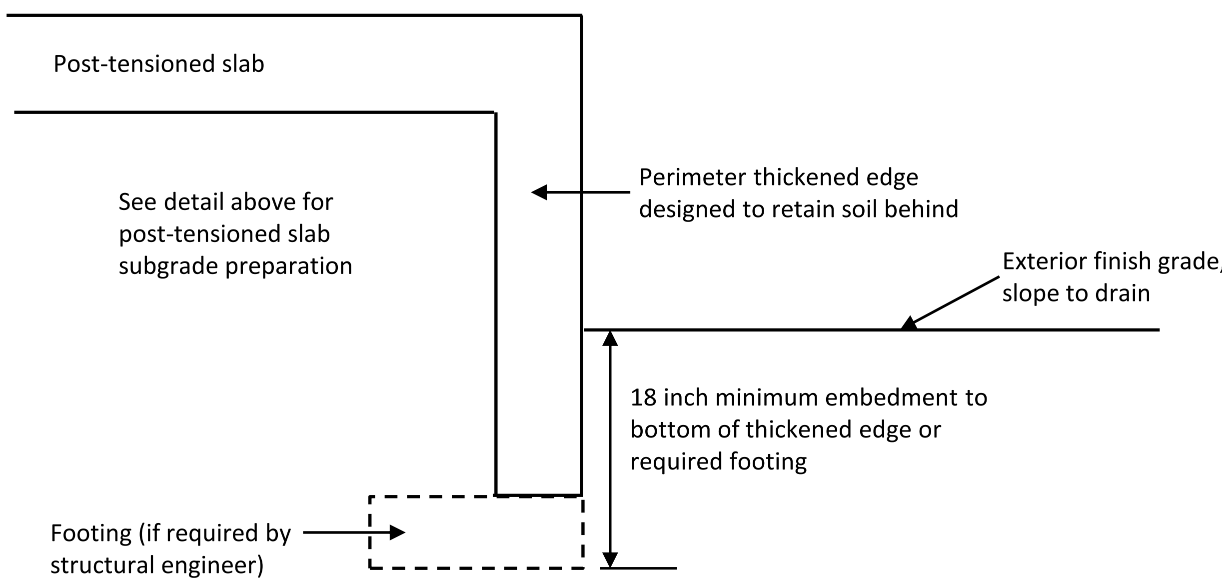




Geo-Logic ASSOCIATES	16055 Caputo Drive, Suite D Morgan Hill, California 95037 Phone (408) 778-2818 Fax (408) 779-6879	Drafted By: Date: Checked By: Revision:	**SITE PLAN (PROPOSED DEVELOPMENT) 38288-38594 CEDAR BOULEVARD NEWARK, CALIFORNIA**	**FIGURE** 2 **PROJECT** PA20.1048



Subgrade Preparation and Thickened Edge for Post-tensioned Slab Foundations



Foundation Embedment for Post-tensioned Slabs with Differential Grades

Note:

1. Refer to geotechnical report for detailed recommendations.

Schematic Only – Not to Scale

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Compiled by:	Date:
Reviewed by:	Revision:

**POST-TENSIONED SLAB
TYPICAL SECTION
38288-38594 CEDAR BOULEVARD
NEWARK, CALIFORNIA**

FIGURE
3

PROJECT
PA19.1048

APPENDIX A

KEYS TO SOIL CLASSIFICATION

DRILL HOLE LOGS,

AND

CONE PENETRATION TEST PLOTS

KEY TO SOIL CLASSIFICATION - FINE GRAINED SOILS

(50% OR MORE IS SMALLER THAN NO. 200 SIEVE SIZE)

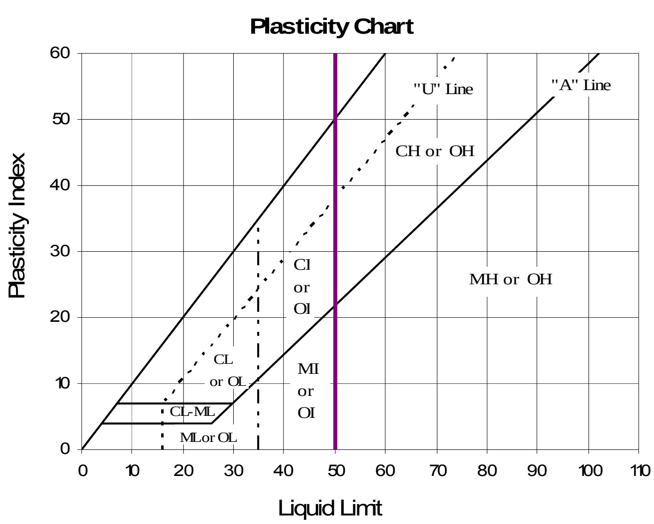
(modified from ASTM D2487 to include fine grained soils with intermediate plasticity)

MAJOR DIVISIONS			GROUP SYMBOLS	GROUP NAMES
SILTS AND CLAYS (Liquid Limit less than 35) Low Plasticity	Inorganic	PI < 4 or plots below “A” line	ML	Silt, Silt with Sand or Gravel, Sandy or Gravelly Silt, Sandy or Gravelly Silt with Sand or Gravel
	Inorganic	PI > 7 or plots on or above “A” line	CL	Lean Clay, Lean Clay with Sand or Gravel, Sandy or Gravelly Lean Clay, Sandy or Gravelly Lean Clay with Sand or Gravel
	Inorganic	PI between 4 and 7	CL-ML	Silty Clay, Silty Clay with Sand or Gravel, Sandy or Gravelly Silty Clay, Sandy or Gravelly Silty Clay with Sand or Gravel
	Organic	See footnote 3	OL	Organic Silt (below “A” Line) or Organic Clay (on or above “A” Line) (1,2)
SILTS AND CLAYS (35 ≤ Liquid Limit < 50) Intermediate Plasticity	Inorganic	PI < 4 or plots below “A” line	MI	Silt, Silt with Sand or Gravel, Sandy or Gravelly Silt, Sandy or Gravelly Silt with Sand or Gravel
	Inorganic	PI > 7 or plots on or above “A” line	CI	Clay, Clay with Sand or Gravel, Sandy or Gravelly Clay, Sandy or Gravelly Clay with Sand or Gravel
	Organic	See footnote 3	OI	Organic Silt (below “A” Line) or Organic Clay (on or above “A” Line) (1,2)
SILTS AND CLAYS (Liquid Limit 50 or greater) High Plasticity	Inorganic	PI plots below “A” line	MH	Elastic Silt, Elastic Silt with Sand or Gravel, Sandy or Gravelly Elastic Silt, Sandy or Gravelly Elastic Silt with Sand or Gravel
	Inorganic	PI plots on or above “A” line	CH	Fat Clay, Fat Clay with Sand or Gravel, Sandy or Gravelly Fat Clay, Sandy or Gravelly Fat Clay with Sand or Gravel
	Organic	See note 3 below	OH	Organic Silt (below “A” Line) or Organic Clay (on or above “A” Line) (1,2)

1. If soil contains 15% to 29% plus No. 200 material, include “with sand” or “with gravel” to group name, whichever is predominant.
2. If soil contains ≥30% plus No. 200 material, include “sandy” or “gravelly” to group name, whichever is predominant. If soil contains ≥15% of sand or gravel sized material, add “with sand” or “with gravel” to group name.
3. Ratio of liquid limit of oven dried sample to liquid limit of not dried sample is less than 0.75.

CONSISTENCY	UNCONFINED SHEAR STRENGTH (KSF)	STANDARD PENETRATION (BLOWS/FOOT)
VERY SOFT	< 0.25	< 2
SOFT	0.25 – 0.5	2 – 4
FIRM	0.5 – 1.0	5 – 8
STIFF	1.0 – 2.0	9 – 15
VERY STIFF	2.0 – 4.0	16 – 30
HARD	> 4.0	> 30

MOISTURE	CRITERIA
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp, but no visible water
Wet	Visible free water, usually soil is below the water table



GEO-LOGIC ASSOCIATES

KEY TO SOIL CLASSIFICATION – COARSE GRAINED SOILS

(MORE THAN 50% IS LARGER THAN NO. 200 SIEVE SIZE)

(modified from ASTM D2487 to include fines with intermediate plasticity)

MAJOR DIVISIONS			GROUP SYMBOLS	GROUP NAMES[1]
GRAVELS (more than 50% of coarse fraction is larger than No. 4 sieve size)	Gravels with less than 5% fines	Cu ≥ 4 and 1 ≤ Cc ≤ 3	GW	Well Graded Gravel, Well Graded Gravel with Sand
		Cu < 4 and/or 1 > Cc > 3	GP	Poorly Graded Gravel, Poorly Graded Gravel with Sand
	Gravels with 5% to 12% fines	ML, MI or MH fines	GW-GM	Well Graded Gravel with Silt, Well Graded Gravel with Silt and Sand
			GP-GM	Poorly Graded Gravel with Silt, Poorly Graded Gravel with Silt and Sand
		CL, CI or CH fines	GW-GC	Well Graded Gravel with Clay, Well Graded Gravel with Clay and Sand
			GP-GC	Poorly Graded Gravel with Clay, Poorly Graded Gravel with Clay and Sand
	Gravels with more than 12% fines	ML, MI or MH fines	GM	Silty Gravel, Silty Gravel with Sand
		CL, CI or CH fines	GC	Clayey Gravel, Clayey Gravel with Sand
		CL-ML fines	GC-GM	Silty Clayey Gravel; Silty, Clayey Gravel with Sand
SANDS (50% or more of coarse fraction is smaller than No. 4 sieve size)	Sands with less than 5% fines	Cu ≥ 6 and 1 ≤ Cc ≤ 3	SW	Well Graded Sand, Well Graded Sand with Gravel
		Cu < 6 and/or 1 > Cc > 3	SP	Poorly Graded Sand, Poorly Graded Sand with Gravel
	Sands with 5% to 12% fines	ML, MI or MH fines	SW-SM	Well Graded Sand with Silt, Well Graded Sand with Silt and Gravel
			SP-SM	Poorly Graded Sand with Silt, Poorly Graded Sand with Silt and Gravel
		CL, CI or CH fines	SW-SC	Well Graded Sand with Clay, Well Graded Sand with Clay and Gravel
			SP-SC	Poorly Graded Sand with Clay, Poorly Graded Sand with Clay and Gravel
	Sands with more than 12% fines	ML, MI or MH fines	SM	Silty Sand, Silty Sand with Gravel
		CL, CI or CH fines	SC	Clayey Sand, Clayey Sand with Gravel
		CL-ML fines	SC-SM	Silty, Clayey Sand; Silty, Clayey Sand with Gravel

US STANDARD SIEVES 3 Inch ¾ Inch No. 4 No. 10 No. 40 No. 200

	COARSE	FINE	COARSE	MEDIUM	FINE	
COBBLES & BOULDERS	GRAVELS		SANDS			SILTS AND CLAYS

RELATIVE DENSITY (SANDS AND GRAVELS)	STANDARD PENETRATION (BLOWS/FOOT)
Very Loose	0 - 4
Loose	5 – 10
Medium Dense	11 – 30
Dense	31 - 50
Very Dense	50+

1. Add “with sand” to group name if material contains 15% or greater of sand-sized particle. Add “with gravel” to group name if material contains 15% or greater of gravel-sized particle.

MOISTURE	CRITERIA
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp, but no visible water
Wet	Visible free water, usually soi is below the water table

GEO-LOGIC ASSOCIATES

DATE: 11/4/2020	LOG OF EXPLORATORY DRILL HOLE	DH- 1

PROJECT NAME: 38288-38594 Cedar Boulevard, Newark, CA **PROJECT NUMBER:** PA20.1048

DRILL RIG: Mobile B-53R **LOGGED BY:** FS

HOLE DIAMETER: 8-inch hollow stem auger **HOLE ELEVATION:** ---

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** 11 ft **Final:** 11 ft

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
Pavement Section (±3" AC over ±4" AB)												
CLAY: Brown, moist, very stiff	Cl	1										
		2	S D D	28								
		3										
		4										
		5	S D D	29	3.25			19		109		
		6										
		7										
		8										
light brown		9										
		10	S D D	22				18		111		
		11										
POORY GRADED SAND with CLAY to CLAYEY SAND: Brown, moist, medium dense; mostly fine to medium sand	SP-SC/ SC	12										
		13										
		14										
		15	S D D	27				17		111		
		16										
		17										
		18										
medium dense to dense		19										
		20	S D D	32								
BOTTOM OF HOLE = 20 Feet												

DATE: 11/4/2020	**LOG OF EXPLORATORY DRILL HOLE**	**DH- 2**

PROJECT NAME: 38288-38594 Cedar Boulevard, Newark, CA **PROJECT NUMBER:** PA20.1048

DRILL RIG: Mobile B-53R, automatic hammer **LOGGED BY:** FS

HOLE DIAMETER: 8-inch hollow stem auger **HOLE ELEVATION:** ---

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** 18 ft **Final:** 39.5 ft

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
Pavement Section (±2.5" AC over ±4" AB)												
CLAY: Dark brown, moist, stiff to very stiff	CI	1										
		2	SDD	37	1.25			18		111		
		3										
dark gray brown		4	SDD	41	1.5			17		115		
		5										
		6										
		7										
		8										
gray, wet, stiff		9	SDD	16	1.0			25		100		
		10										
		11										
		12										
		13										
medium gray brown, moist, stiff to very stiff		14	SDD	43	1.75			19		113		
		15										
		16										
		17										
POORLY GRADED SAND: Gray, wet, medium dense to dense; mostly fine sand	SP	18										
		19	SDD	32				43		106		
CLAYEY SAND: see next page	SC	20										

DATE: 11/4/2020	LOG OF EXPLORATORY DRILL HOLE	DH- 2

PROJECT NAME:	38288-38594 Cedar Boulevard, Newark, CA	**PROJECT NUMBER:**	PA20.1048
DRILL RIG:	Mobile B-53R, automatic hammer	**LOGGED BY:**	FS
HOLE DIAMETER:	8-inch hollow stem auger	**HOLE ELEVATION:**	---
SAMPLER:	D = 3" OD, 2½" ID Split-spoon X = 2½" OD, 2" ID Split-spoon I = Standard Penetrometer (2" OD SPT) S = Slough in sample	**GROUND WATER DEPTH:**	**Initial:** 18 ft **Final:** 39.5 ft

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
CLAYEY SAND: Brown, wet, medium dense to dense; mostly fine sand	SC											
		21										
		22										
		23										
dense		24	S I I	37		18		25				
		25										
		26										
		27										
SANDY CLAY: Gray, wet, stiff; with mostly fine sand	CI											
		28										
		29	S I I	10		67		31				
		30										
		31										
		32										
		33										
		34	S I I	18			42	34	20			
		35										
		36										
		37										
CLAY with SAND: Gray, moist to wet, firm; with mostly fine sand	CI											
		38										
		39	S I I	4				24				
		40										

DATE: 11/4/2020	LOG OF EXPLORATORY DRILL HOLE	DH- 2

PROJECT NAME: 38288-38594 Cedar Boulevard, Newark, CA **PROJECT NUMBER:** PA20.1048

DRILL RIG: Mobile B-53R, automatic hammer **LOGGED BY:** FS

HOLE DIAMETER: 8-inch hollow stem auger **HOLE ELEVATION:** ---

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** 18 ft **Final:** 39.5 ft

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
CLAY with SAND (continued)	CI	41										
CLAYEY SAND: Gray, wet, medium dense; mostly	SC	42										
		43										
		44	S									
		45	I I	17		44		26				
		46										
CLAYEY SAND: Gray, moist to wet, very dense; fine to coarse sand; with fine gravel	SC	47										
		48										
		49	I	50/6"		19		14				
BOTTOM OF HOLE = 49 Feet		50										
		51										
		52										
		53										
		54										
		55										
		56										
		57										
		58										
		59										
		60										

DATE: 11/4/2020	LOG OF EXPLORATORY DRILL HOLE	DH- 3

PROJECT NAME: 38288-38594 Cedar Boulevard, Newark, CA **PROJECT NUMBER:** PA20.1048

DRILL RIG: Mobile B-53R **LOGGED BY:** FS

HOLE DIAMETER: 8-inch hollow stem auger **HOLE ELEVATION:** ---

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** 11.5 ft

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
Pavement Section (±2" AC over ±6" AB)												
CLAY: Dark brown, moist, very stiff	CI	1										
		2	SDD	33	3.5			15		117		
		3										
light brown		4	SDD	28	3.0							
SANDY CLAY: Light brown, moist, very stiff; with	CI	5						16		115		
		6										
		7										
		8										
		9	SDD	21	2.5							
POORLY GRADED SAND with CLAY to CLAYEY SAND: Gray, moist, medium dense; mostly fine sand	SP-SC/SC	10						12		101		
		11										
		12										
		13										
		14	SDD	16	(no sample recovery)							
		15	SDD									
CLAY: Brown, moist, very stiff	CI	16		25								
		17										
		18										
		19	SDD	22	(no sample recovery)							
		20										

DATE: 11/4/2020	LOG OF EXPLORATORY DRILL HOLE	DH- 3

PROJECT NAME: 38288-38594 Cedar Boulevard, Newark, CA **PROJECT NUMBER:** PA20.1048

DRILL RIG: Mobile B-53R **LOGGED BY:** FS

HOLE DIAMETER: 8-inch hollow stem auger **HOLE ELEVATION:** ---

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** 11.5 ft

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
CLAY (continued): wet	CI		S									
		21	I I	16								
BOTTOM OF HOLE = 21.5 Feet		22										
		23										
		24										
		25										
		26										
		27										
		28										
		29										
		30										
		31										
		32										
		33										
		34										
		35										
		36										
		37										
		38										
		39										
		40										

DATE: 11/4/2020	LOG OF EXPLORATORY DRILL HOLE	DH- 4

PROJECT NAME: 38288-38594 Cedar Boulevard, Newark, CA **PROJECT NUMBER:** PA20.1048

DRILL RIG: Mobile B-53R **LOGGED BY:** FS

HOLE DIAMETER: 8-inch hollow stem auger **HOLE ELEVATION:** ---

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** 11 ft

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
Pavement Section (±2" AC over ±4" AB)												
LEAN CLAY to CLAY: Dark brown, moist, very stiff to hard	CL/ CI	1										
		2	SDD	37		85	35	19	20	111		
		3										
		4	SDD	43								
hard		5			4.5			15		116		
		6										
		7										
CLAYEY SAND: Brown, moist, medium dense; mostly fine sand	SC	8										
		9	SDD	21								
		10				28		9		92		
		11										
		12										
CLAY: Brown, moist, stiff to very stiff	CI	13										
		14	SDD	25								
		15										
		16										
		17										
		18										
with some fine sand		19	SDD	19								
		20										
BOTTOM OF HOLE = 20 Feet												

GEO-LOGIC ASSOCIATES	PAGE: 1 of 1

DATE: 11/5/2020	LOG OF EXPLORATORY DRILL HOLE	DH- 5

PROJECT NAME: 38288-38594 Cedar Boulevard, Newark, CA **PROJECT NUMBER:** PA20.1048

DRILL RIG: Mobile B-53R, automatic hammer **LOGGED BY:** FS

HOLE DIAMETER: 8-inch hollow stem auger **HOLE ELEVATION:** ---

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** 32 ft

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
Pavement Section (±2.5" AC over ±5" AB)												
CLAY: Gray brown, moist, hard	CI	1										
			S									
		2	D	20	4.5							
			D		4.5+			14		111		
		3										
			S									
brown		4	D	22								
		5	D		4.5+			7		126		
		6										
		7										
POORLY GRADED SAND with CLAY to CLAYEY SAND: Brown, moist, medium dense; fine to medium sand	SP-SC/ SC	8										
		9	S									
			D	12								
		10	D					10		110		
		11										
		12										
CLAY: Brown, moist, firm to stiff	CI	13										
		14	S									
			D	6								
		15	D		1.25			15		119		
		16										
		17										
CLAY with SAND: Brown, moist, firm to stiff; with mostly fine sand	CI	18										
		19	S									
			D	9								
		20	D		1.25			24		102		

DATE: 11/5/2020	LOG OF EXPLORATORY DRILL HOLE	DH- 5

PROJECT NAME: 38288-38594 Cedar Boulevard, Newark, CA **PROJECT NUMBER:** PA20.1048

DRILL RIG: Mobile B-53R, automatic hammer **LOGGED BY:** FS

HOLE DIAMETER: 8-inch hollow stem auger **HOLE ELEVATION:** ---

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** 32 ft

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
CLAY with SAND (continued)	CI											
		21										
CLAYEY SAND: Brown, wet, medium dense; mostly fine sand	SC	22										
		23										
		24	S D D	12		32		24		95		
		25										
		26										
		27										
		28										
gray, mostly fine to medium sand		29	S I I	8		26		23				
		30										
		31										
		32										
		33										
		34	S I I	7		34		24				
		35										
		36										
CLAY: Gray, moist to wet, firm to stiff	CI	37										
		38										
		39	S I I	11			41	32	19			
		40										

DATE: 11/5/2020	LOG OF EXPLORATORY DRILL HOLE	DH- 5

PROJECT NAME:	38288-38594 Cedar Boulevard, Newark, CA	PROJECT NUMBER:	PA20.1048
DRILL RIG:	Mobile B-53R, automatic hammer	LOGGED BY:	FS
HOLE DIAMETER:	8-inch hollow stem auger	HOLE ELEVATION:	---
SAMPLER:	D = 3" OD, 2½" ID Split-spoon X = 2½" OD, 2" ID Split-spoon I = Standard Penetrometer (2" OD SPT) S = Slough in sample	GROUND WATER DEPTH:	Initial: --- Final: 32 ft

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
CLAY (continued)	CI											
		41										
		42										
		43										
		44	S									
stiff			I	17				20				
		45	I									
		46										
		47										
		48										
		49	S									
			I	19				23				
		50	I									
BOTTOM OF HOLE = 50 Feet		51										
		52										
		53										
		54										
		55										
		56										
		57										
		58										
		59										
		60										

DATE: 11/5/2020 | **LOG OF EXPLORATORY DRILL HOLE** | **DH- 6**

PROJECT NAME: 38288-38594 Cedar Boulevard, Newark, CA **PROJECT NUMBER:** PA20.1048

DRILL RIG: Mobile B-53R **LOGGED BY:** FS

HOLE DIAMETER: 8-inch hollow stem auger **HOLE ELEVATION:** ---

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** 12 ft

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
Pavement Section (±2" AC over ±5" AB)												
CLAY: Dark brown, moist, hard	CI	1										
		2	SDD	34	4.5+			16		113		
		3										
		4										
brown		5	SDD	35	4.5+			15		116		
		6										
		7										
CLAYEY SAND: Gray, moist, medium dense; mostly fine sand	SC	8										
		9										
		10	SDD	17		24		11		94		
		11										
		12										
CLAY: Brown, wet, very stiff	CI	13										
		14										
		15	SDD	17								
		16										
		17										
		18										
		19										
		20	SDD	25								
BOTTOM OF HOLE = 20 Feet												

DATE: 11/5/2020	LOG OF EXPLORATORY DRILL HOLE	DH- 7

PROJECT NAME: 38288-38594 Cedar Boulevard, Newark, CA **PROJECT NUMBER:** PA20.1048

DRILL RIG: Mobile B-53R **LOGGED BY:** FS

HOLE DIAMETER: 8-inch hollow stem auger **HOLE ELEVATION:** ---

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** 11 ft **Final:** 12 ft

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
CLAY: Dark brown, moist, hard	CI											
		1										
		2	SDD	35		90	39	14	24	112		
		3										
light brown		4	SDD	50	4.5+			12		117		
		5										
		6										
		7										
POORLY GRADED SAND with CLAY: Brown, moist, medium dense; mostly fine to medium sand	SP-SC	8										
		9	SDD	18				15		106		
		10										
		11										
		12										
		13										
gray		14	SDD	16		11		20		99		
		15										
		16										
		17										
		18										
		19	SDD	23								
BOTTOM OF HOLE = 20 Feet		20										

DATE: 11/5/2020	LOG OF EXPLORATORY DRILL HOLE	DH- 8

PROJECT NAME: 38288-38594 Cedar Boulevard, Newark, CA **PROJECT NUMBER:** PA20.1048

DRILL RIG: Mobile B-53R **LOGGED BY:** FS

HOLE DIAMETER: 8-inch hollow stem auger **HOLE ELEVATION:** ---

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

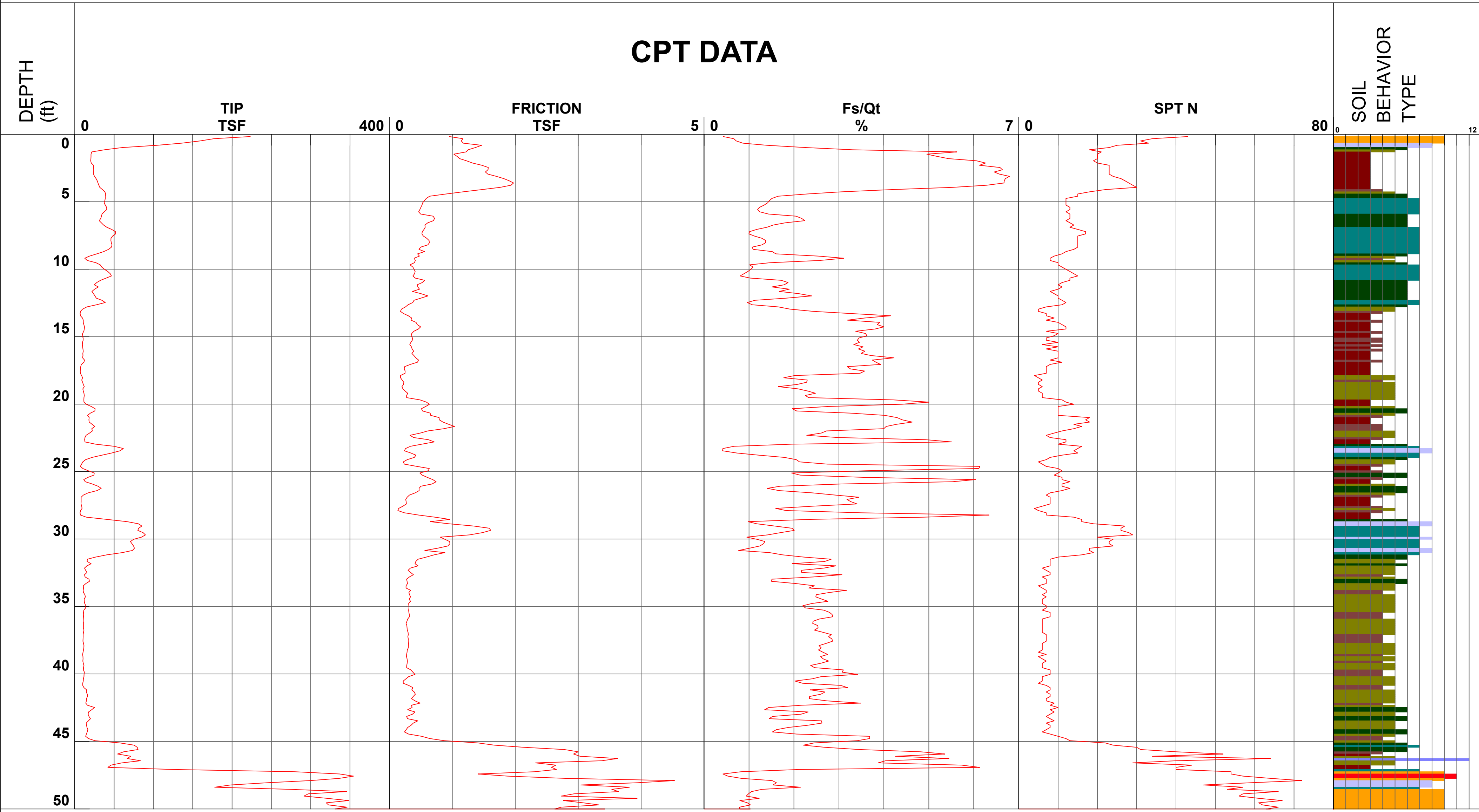
GROUND WATER DEPTH: **Initial:** 12 ft **Final:** 11 ft 7 in

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
CLAY: Dark brown, moist, very stiff to hard	CI											
		1										
		2	SDD	30	3.3			19		110		
SANDY CLAY: Medium to light brown, moist, hard	CI	3										
		4	SDD	36	4.5+			15		114		
		5										
		6										
CLAYEY SAND: Brown, moist, medium dense; mostly fine sand	SC	7										
		8										
		9	SDD	16				21		104		
		10										
		11										
CLAY with SAND: Medium brown, moist, stiff	CI	12										
		13										
		14	SDD	9	1.1			25		99		
		15										
		16										
		17										
		18										
		19	SDD	24	1.1							
BOTTOM OF HOLE = 20 Feet		20										

Geo-Logic Associates

Project	38478 Cedar Boulevard	Operator	JM-ZG	Filename	SDF(237).cpt
Job Number	PA20.1048.00	Cone Number	DDG1530	GPS	
Hole Number	CPT-01	Date and Time	11/4/2020 1:33:07 PM	Maximum Depth	50.69 ft
EST GW Depth During Test		11.00 ft			

Net Area Ratio .8



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

Cone Size 15cm squared

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

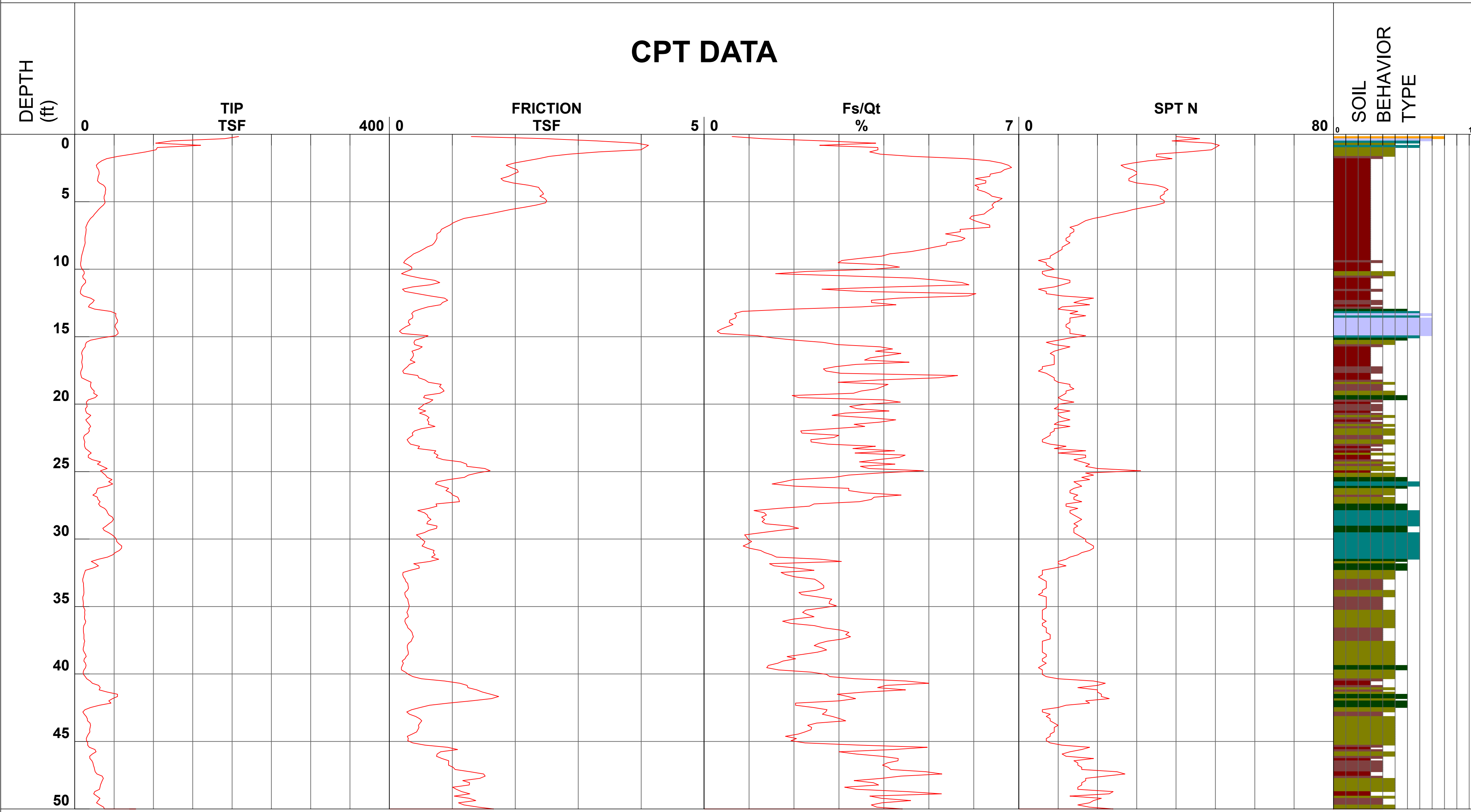
Geo-Logic Associates

Middle Earth Geo Testing Inc.

Project	38478 Cedar Boulevard	Operator	JM-ZG	Filename	SDF(238).cpt
Job Number	PA20.1048.00	Cone Number	DDG1530	GPS	
Hole Number	CPT-02	Date and Time	11/4/2020 3:06:10 PM	Maximum Depth	50.52 ft
EST GW Depth During Test		9.70 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 15cm squared

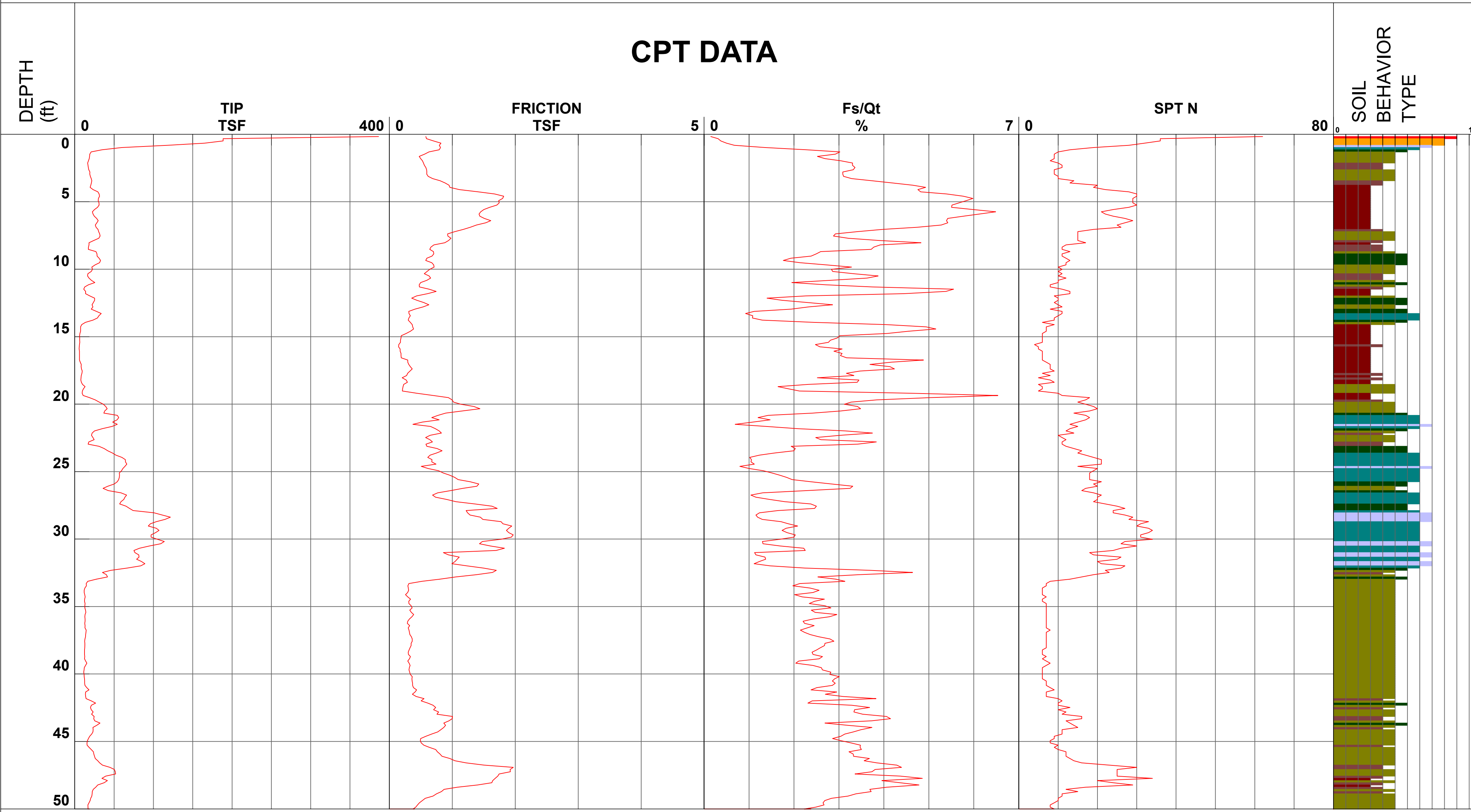
S*Soil behavior type and SPT based on data from UBC-1983

Geo-Logic Associates

Middle Earth Geo Testing Inc.

Project	38478 Cedar Boulevard	Operator	JM-ZG	Filename	SDF(236).cpt
Job Number	PA20.1048.00	Cone Number	DDG1530	GPS	
Hole Number	CPT-03	Date and Time	11/4/2020 11:58:55 AM	Maximum Depth	50.69 ft
EST GW Depth During Test		11.80 ft			

Net Area Ratio .8



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

Cone Size 15cm squared

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

APPENDIX B

LABORATORY TEST RESULTS

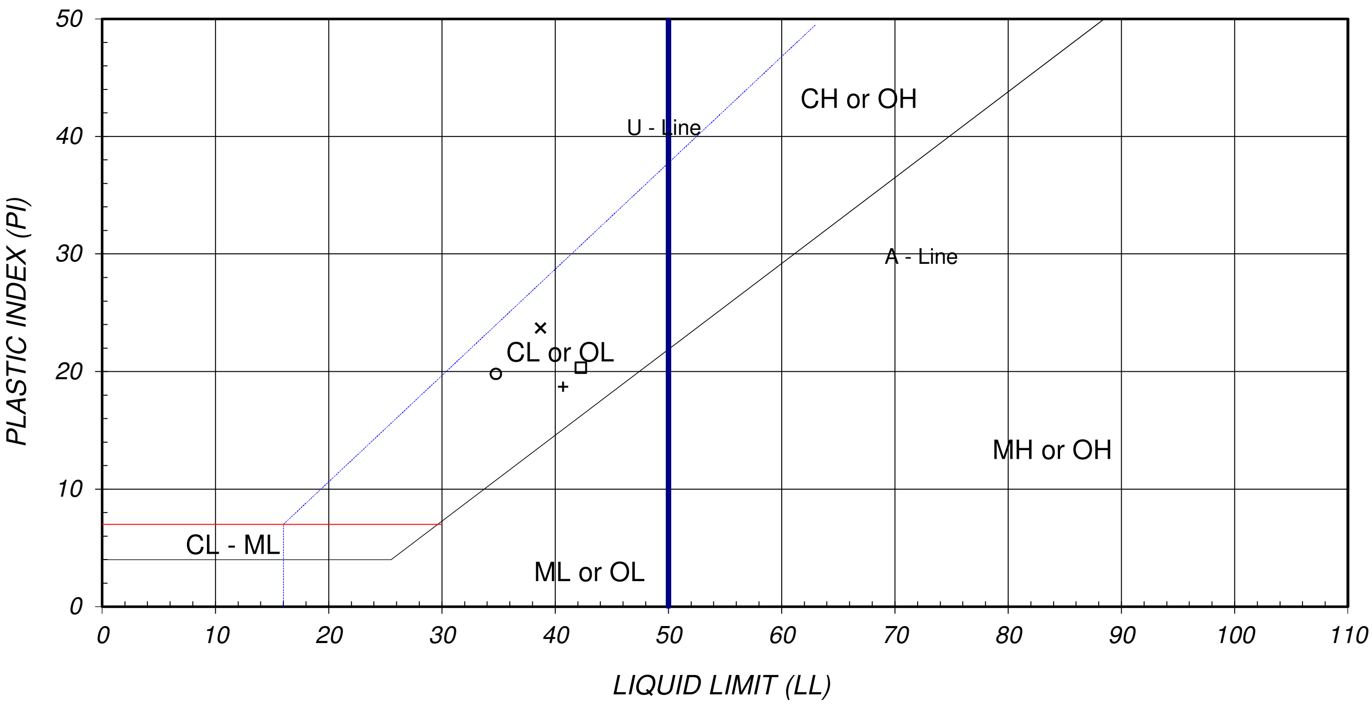
ATTERBERG LIMITS
Summary Report
ASTM D-4318

Client : Robson Homes LLC
Project No: PA20.1048.00
Lab Log No.: **4701**
Project Name: 38478 Cedar Blvd.
Report Date: December 10, 2020

LSN	SYMBOL	SAMPLE IDENTIFICATION	SAMPLE DESCRIPTION	LIQUID LIMIT	PLASTIC LIMIT	PLASTIC INDEX
4701A	□	DH-2 @ 34-35'	* gray lean clay	42	22	20
4701B	○	DH-4 @ 2-2.5'	dark brown lean clay (CL)	35	15	20
4701C	+	DH-5 @ 39-40'	* gray lean clay	41	22	19
4701D	x	DH-7 @ 2-2.5'	dark brown lean clay (CL)	39	15	24

* = based on visual classification. No sieve performed.

PLASTICITY CHART



This testing is based upon accepted industry practice as well as the test method listed. These results apply only to the samples supplied and tested for the above referenced job.

L : Labexcel \ Projects \ Client \ Robson Homes LL \ PA20.1048.00 | Print Date: 12/10/20 | Entered By: PP | Reviewed By: RZS | LLN: **4701**

DCN: PI-rp (rev. 9/18/12)

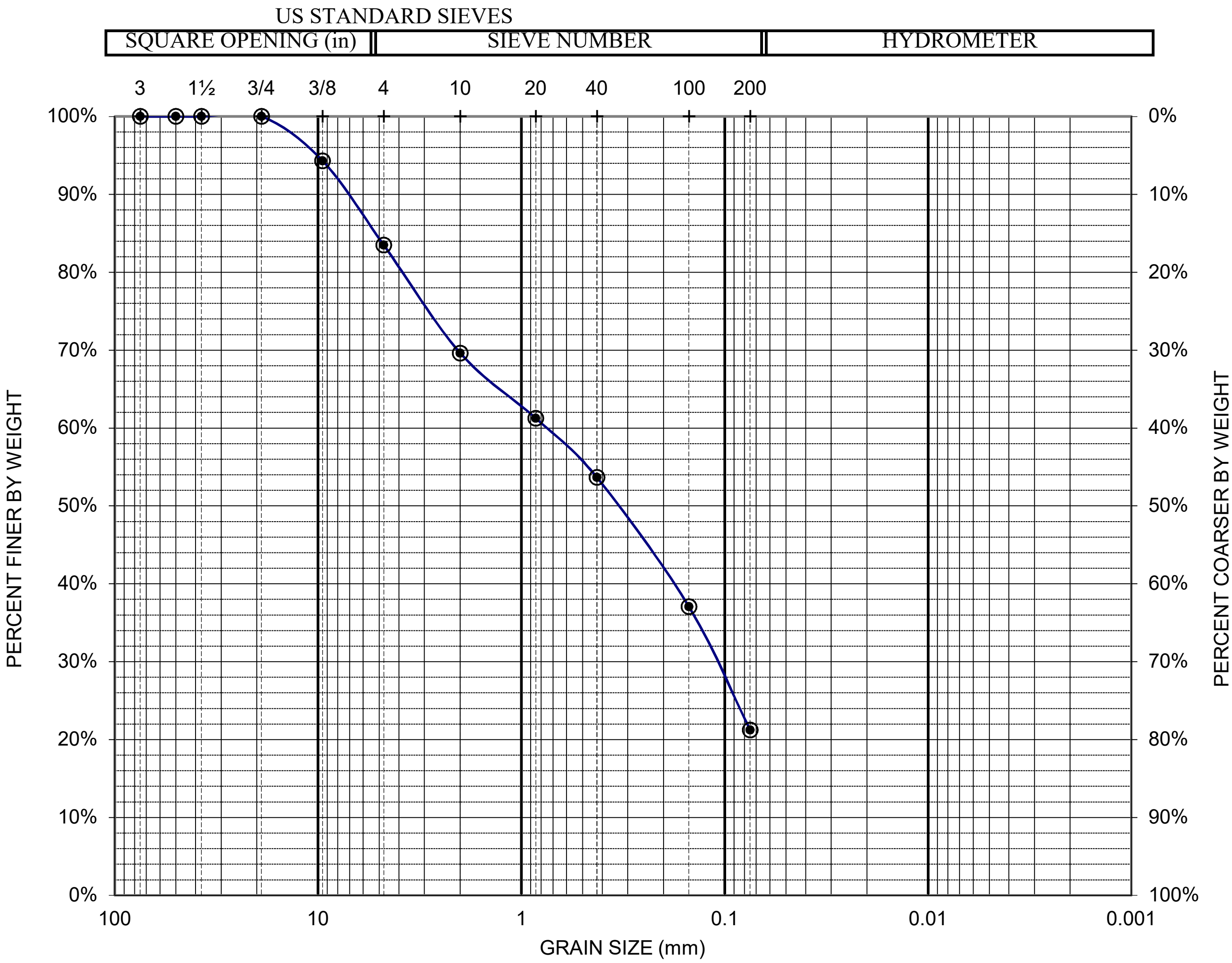
Figure B-1

GRAIN SIZE TEST RESULTS

PROJECT NAME 38288-38594 Cedar Boulevard		**PROJECT No.**	PA20.1048
DRILL HOLE No. 2	**DEPTH (ft)** 48.5	**SAMPLE** 0	**DATE OF TEST** 11/25/2020

SOURCE/QUARRY: ---

DESCRIPTION OF SOIL: Clayey Sand with Gravel



	COARSE	FINE	COARSE	MEDIUM	FINE	
COBBLES	GRAVEL		SAND			SILT & CLAY
	16.5%		62.3%			21.2%

REMARKS:

RevOct20070

GEO-LOGIC ASSOCIATES

Figure B-2

Geo-Logic ASSOCIATES

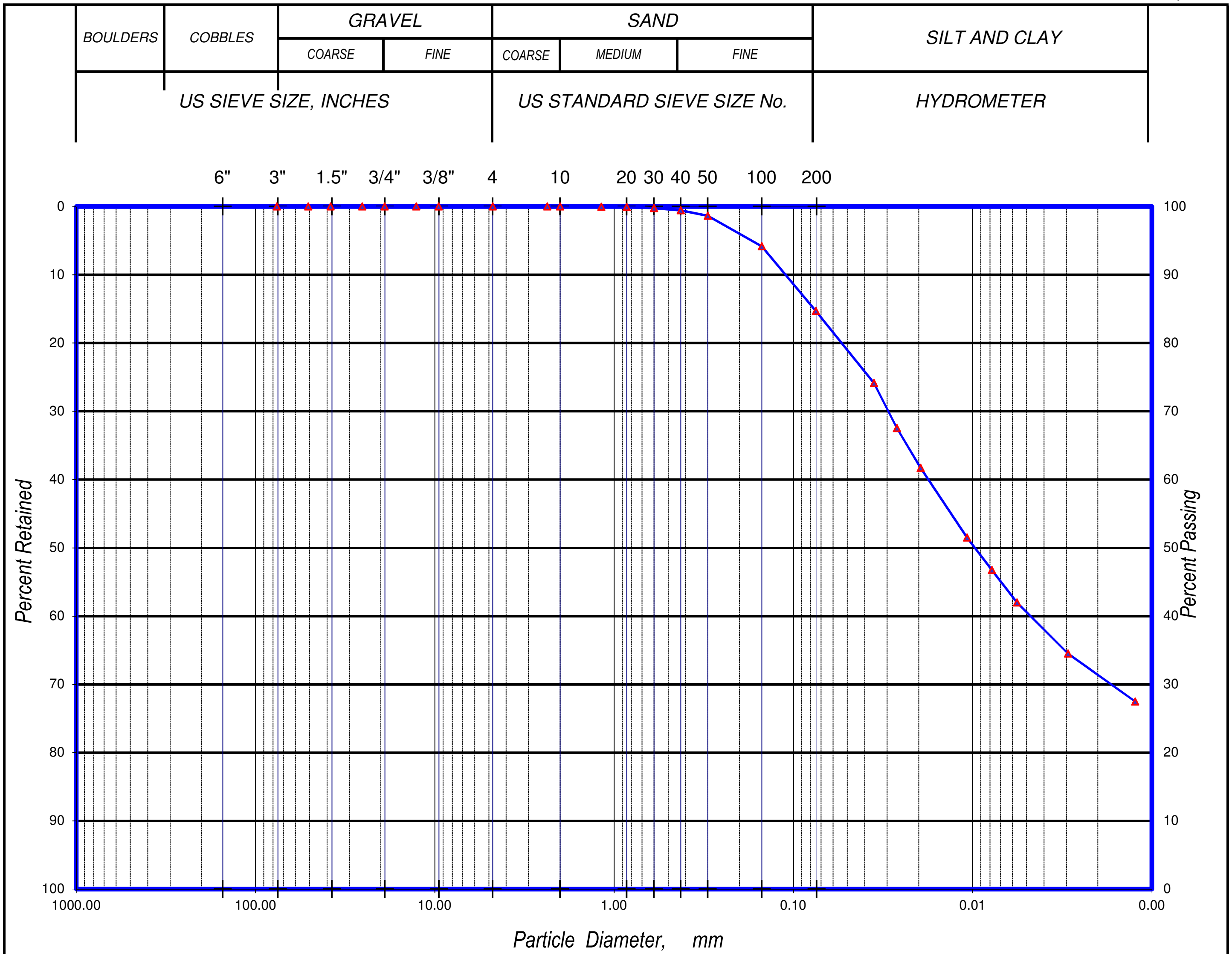
PARTICLE SIZE ANALYSIS

Test Report

ASTM D-6913 / D-7928, (replacing D-422)
Method A: (+/-1%)

Client :	Project No:	Lab Sample No:
ROBSON HOMES LLC	PA20.1048.00	4701B

Project Name:	Report Date:
38478 CEDAR BLVD.	December 10, 2020



Symbol	Sample ID	Description	% Gravel	% Sand	% Silt - Clay
▲	DH-4 @ 2-2.5'	dark brown lean clay (CL)	0.0	15.3	84.7

Size Passing, mm D_{60} = 0.02 D_{30} = 0.00 D_{10} = N/A 5 micron (%) = 43

Coefficient of Curvature, C_c: N/A Coefficient of Uniformity, C_u: N/A Fineness Modulus = 0.07

Note: * Percentages are +/- 0.1% based on computer rounding as allowed by ASTM D-6026-01 Section 5.2.3.

This testing is based upon accepted industry practice as well as the test method listed. These results apply only to the samples supplied and tested for the above referenced job.

L : Labexcel \ Projects \ Client \ Client Name \ 4701 \ 4701B-ma	Print Date:	Entered By:	Reviewed By:	LSN:
DCN: MA-rp (rev. 6/27/12)	12/10/20	PP	RZS	4701B

Figure B-3

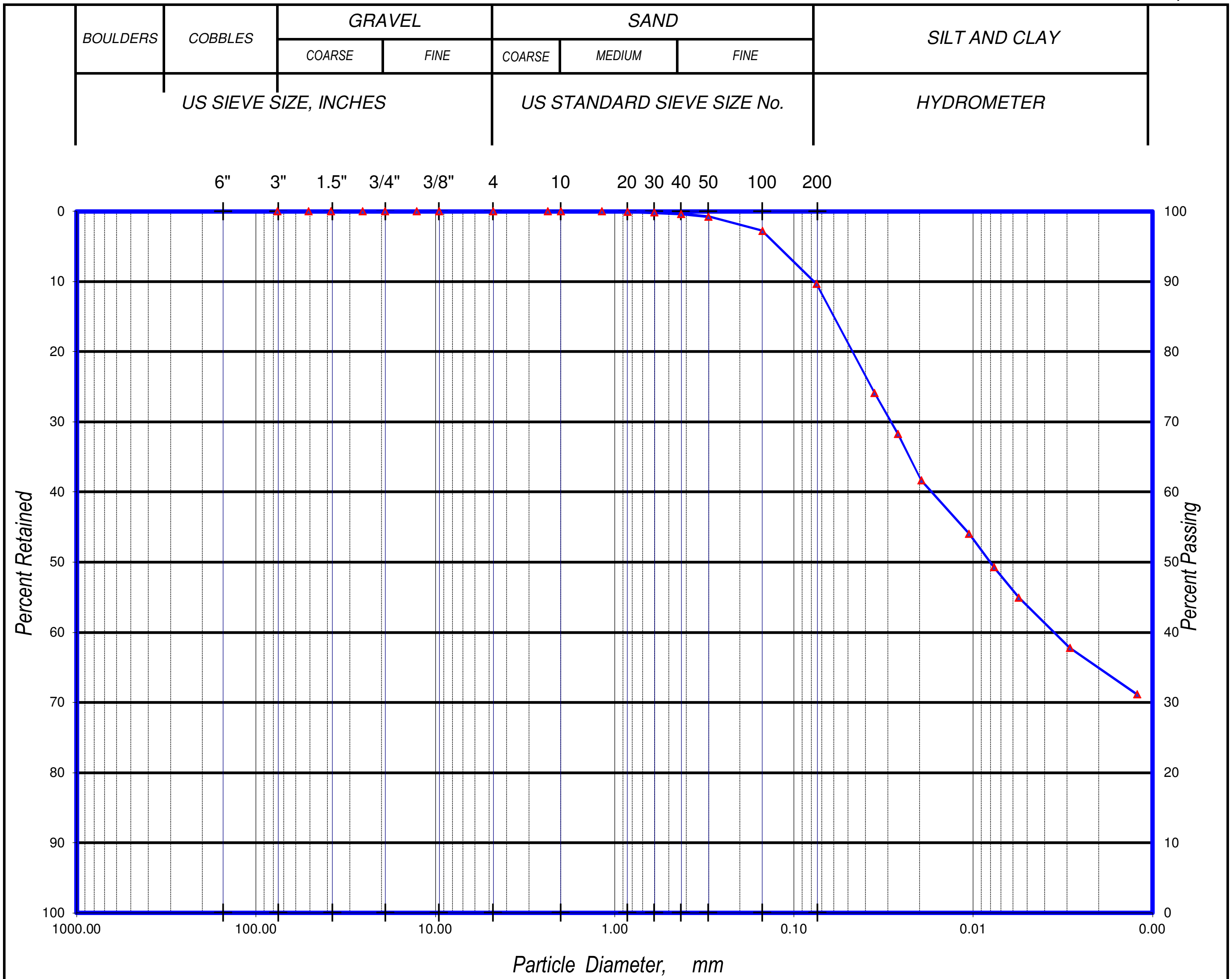
PARTICLE SIZE ANALYSIS

Test Report

ASTM D-6913 / D-7928, (replacing D-422)
Method B:(+/-0.1%)

Client:	Project No:	Lab Sample No:
ROBSON HOMES LLC	PA20.1048.00	**4701D**

Project Name:	Report Date:
38478 CEDAR BLVD.	December 10, 2020



Symbol	Sample ID	Description	% Gravel	% Sand	% Silt - Clay
▲	DH-7 @ 2-2.5'	dark brown lean clay (CL)	0.0	10.4	89.6

Size Passing, mm D_{60} = 0.02 D_{30} = 0.00 D_{10} = N/A 5 micron (%) = 46

Coefficient of Curvature, C_c: N/A Coefficient of Uniformity, C_u: N/A Fineness Modulus = 0.04

Note: * Percentages are +/- 0.1% based on computer rounding as allowed by ASTM D-6026-01 Section 5.2.3.

This testing is based upon accepted industry practice as well as the test method listed. These results apply only to the samples supplied and tested for the above referenced job.

L : Labexcel \ Projects \ Client \ Client Name \ 4701 \ 4701D-ma	Print Date:	Entered By:	Reviewed By:	LSN:
DCN: MA-rp (rev. 6/27/12)	12/10/20	PP	RZS	**4701D**

Figure B-4

'R' VALUE CA 301

Project # PA20.1048 Date: 11/12/20 By: LD

Sample : Bulk #2 Soil Type: Brown, Silty Clay

TEST SPECIMEN		A	B	C	D
Compactor Air Pressure	psi	**100**	**60**	**75**	
Initial Moisture Content	%	**7.6**	**7.6**	**7.6**	
Water Added	ml	**80**	**120**	**100**	
Moisture at Compaction	%	14.8	18.4	16.6	
Sample & Mold Weight	gms	**3210**	**3162**	**3189**	
Mold Weight	gms	**2103**	**2092**	**2100**	
Net Sample Weight	gms	1107	1070	1089	
Sample Height	in.	**2.445**	**2.48**	**2.48**	
Dry Density	pcf	119.5	110.5	114.1	
Pressure	lbs	**9360**	**3430**	**6050**	
Exudation Pressure	psi	745	273	482	
Expansion Dial	x 0.0001	**100**	**1**	**48**	
Expansion Pressure	psf	433	4	208	
Ph at 1000lbs	psi	**35**	**65**	**50**	
Ph at 2000lbs	psi	**96**	**143**	**119**	
Displacement	turns	**3.18**	**4.5**	**3.92**	
R' Value		34	6	18	
Corrected 'R' Value		**34**	**6**	**18**	

FINAL 'R' VALUE	
By Exudation Pressure (@ 300 psi):	**8**
By Epansion Pressure :	**13**
TI = 5	

Figure B-5

Geo-Logic ASSOCIATES

'R' VALUE CA 301

Project # PA20.1048 Date: 11/12/20 By: LD

Sample : Bulk #1 Soil Type: Brown, Silty Clay

TEST SPECIMEN		A	B	C	D
Compactor Air Pressure	psi	**200**	**70**	**50**	
Initial Moisture Content	%	**6.7**	**6.7**	**6.7**	
Water Added	ml	**80**	**120**	**150**	
Moisture at Compaction	%	13.8	17.4	20.0	
Sample & Mold Weight	gms	**3248**	**3168**	**3152**	
Mold Weight	gms	**2114**	**2075**	**2098**	
Net Sample Weight	gms	1134	1093	1054	
Sample Height	in.	**2.49**	**2.51**	**2.53**	
Dry Density	pcf	121.2	112.4	105.2	
Pressure	lbs	**9475**	**5260**	**3085**	
Exudation Pressure	psi	754	419	246	
Expansion Dial	x 0.0001	**118**	**20**	**0**	
Expansion Pressure	psf	511	87	0	
Ph at 1000lbs	psi	**28**	**60**	**68**	
Ph at 2000lbs	psi	**87**	**130**	**145**	
Displacement	turns	**3.05**	**4.72**	**5.06**	
R' Value		41	11	5	
Corrected 'R' Value		**41**	**11**	**5**	

FINAL 'R' VALUE	
By Exudation Pressure (@ 300 psi):	**6**
By Epansion Pressure :	**14**
TI = 5	

Figure B-6

Geo-Logic ASSOCIATES

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 **462 2771** Fax. 925 **462 2775**
www.cercoanalytical.com

11 December, 2020

Job No. 2011220
Cust. No. 10854

Ms. Francesca Senes
Geo-Logic Associates
16055-D Caputo Drive
Morgan Hill, CA 95037

Subject: Project No.: PA 20.1048.00
Project Name: 38478 Cedar Blvd., Newark
Corrosivity Analysis – ASTM Test Methods

Dear Ms. Senes:

Pursuant to your request, CERCO Analytical has analyzed the soil sample submitted on November 30, 2020. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, samples -001 & -002 are classified as "corrosive" and sample -003 is classified as "moderately corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentrations ranged from none detected to 46 mg/kg. Because the chloride ion concentrations are less than 300 mg/kg, they are determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentration ranges from 22 to 32 mg/kg and are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The pH of the soils ranged from 7.75 to 9.61, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials ranged from 320 to 380-mV which is indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client: Geo-Logic Associates
Client's Project No.: PA 20.1048.00
Client's Project Name: 38478 Cedar Blvd., Newark
Date Sampled: 3-Nov-20
Date Received: 30-Nov-20
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 11-Dec-2020

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
2011220-001	Composite 1	380	9.61	980	-	46	32
2011220-002	Composite 2	320	8.64	1,100	-	29	22
2011220-003	Composite 3	360	7.75	2,900	-	N.D.	28
Method:		ASTM D1498	ASTM D4972	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:		-	-	-	50	15	15
Date Analyzed:		9-Dec-2020	9-Dec-2020	10-Dec-2020	-	9-Dec-2020	9-Dec-2020

* Results Reported on "As Received" Basis
N.D. - None Detected

Cheryl McMillen
Laboratory Director

Quality Control Summary - **All laboratory quality control parameters were found to be within established limits**

APPENDIX C

RESULTS OF LIQUEFACTION ANALYSES

GeoLogismiki
Geotechnical Engineers
Merarhias 56
http://www.geologismiki.gr

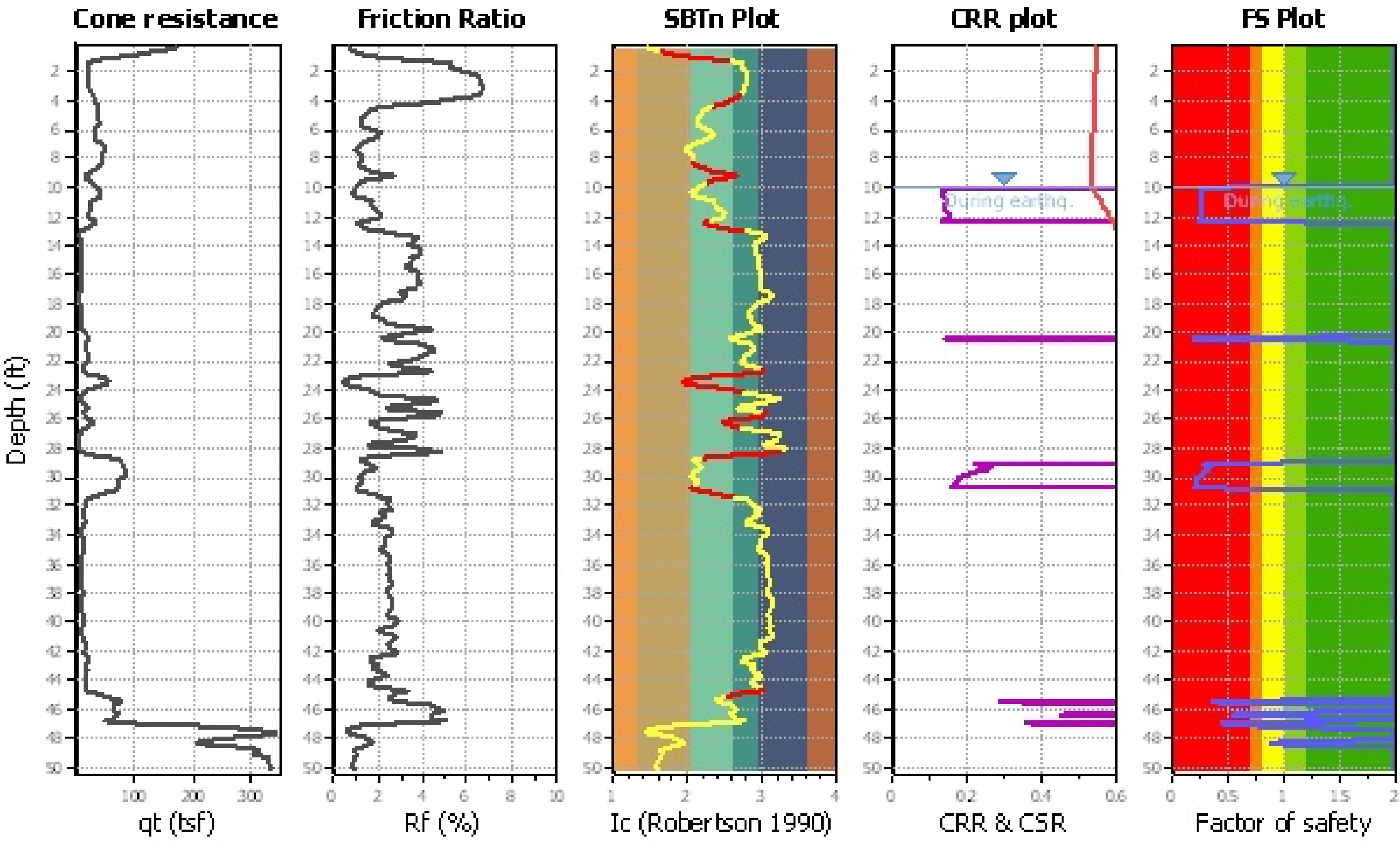
LIQUEFACTION ANALYSIS REPORT

Project title : 38288-38594 Cedar Blvd. **Location : Newark, CA**

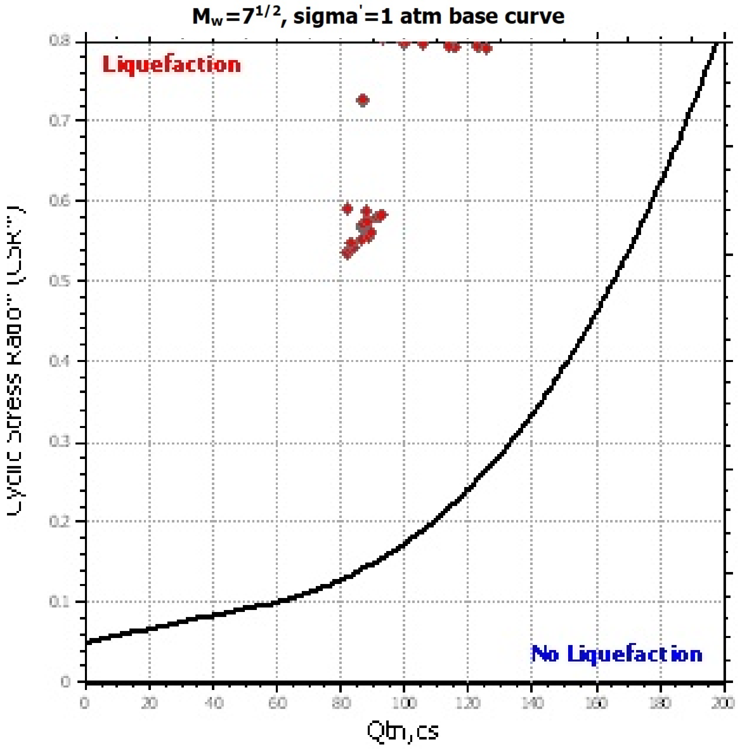
CPT file : CPT-01

Input parameters and analysis data

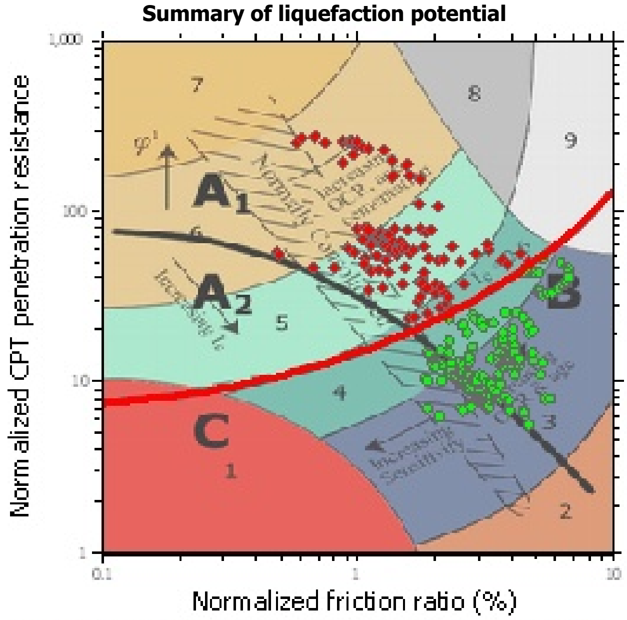
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.50	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.84	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



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

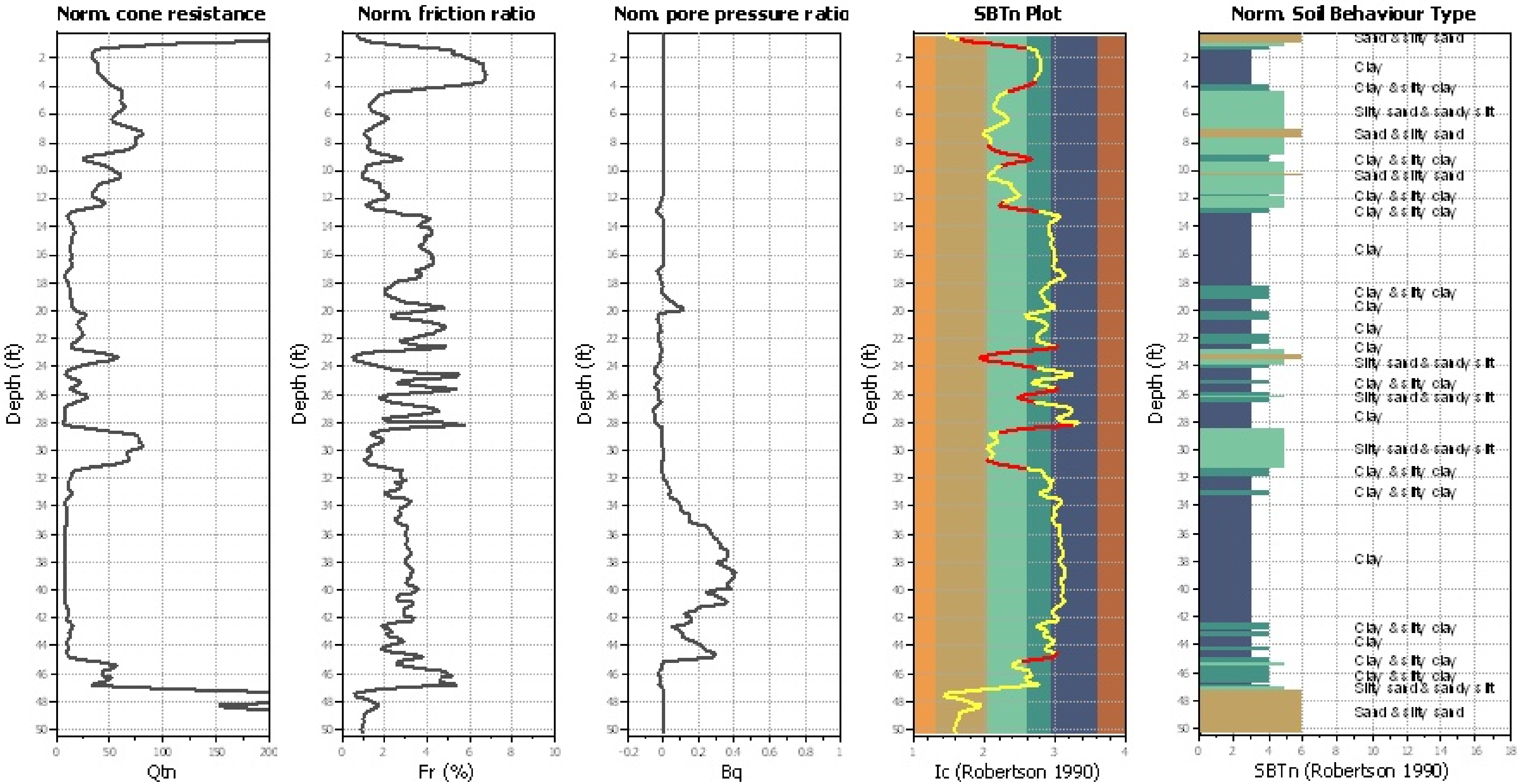


Summary of liquefaction potential



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

CPT basic interpretation plots (normaliz



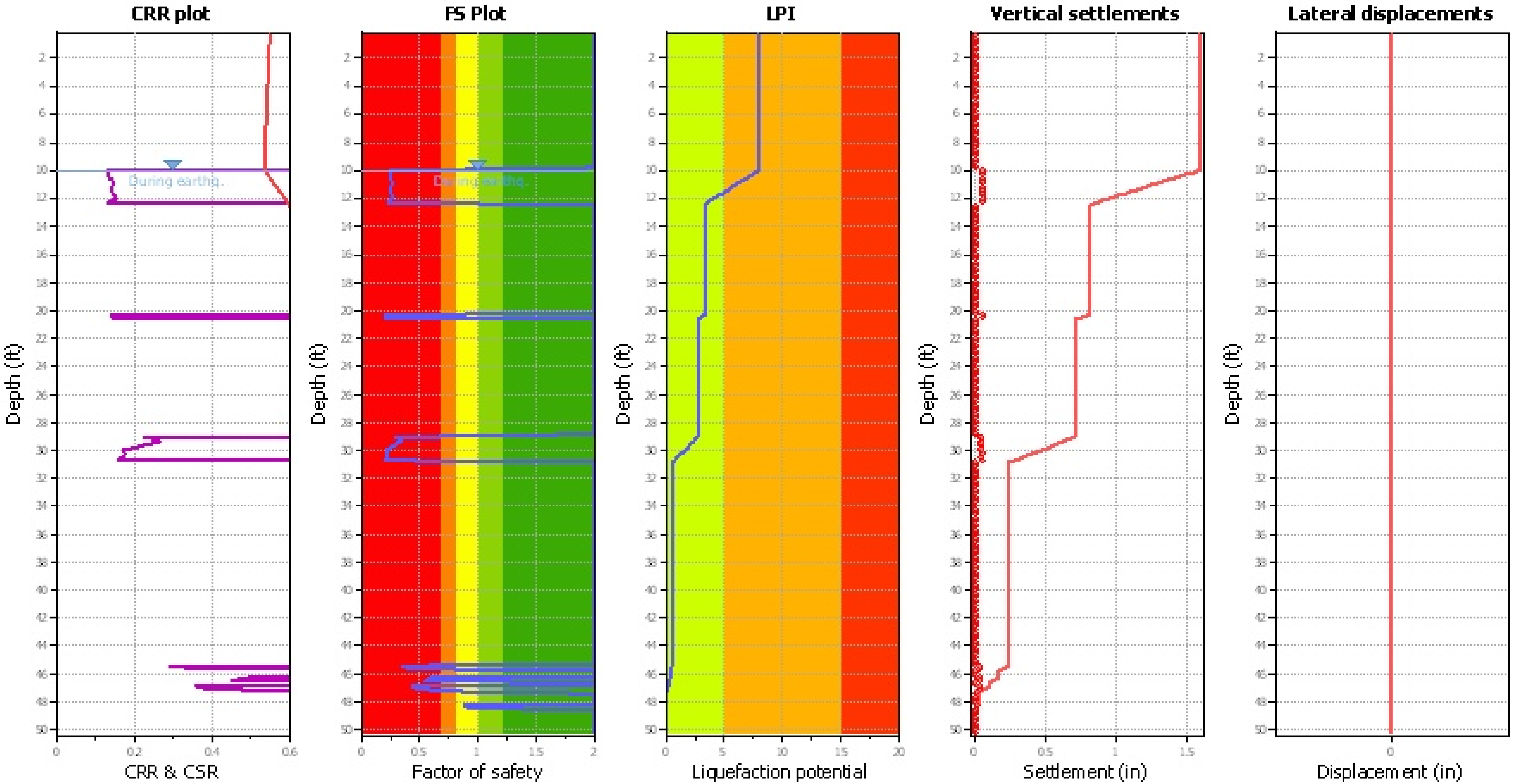
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.84	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.84	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

GeoLogismiki
Geotechnical Engineers
Merarhias 56
http://www.geologismiki.gr

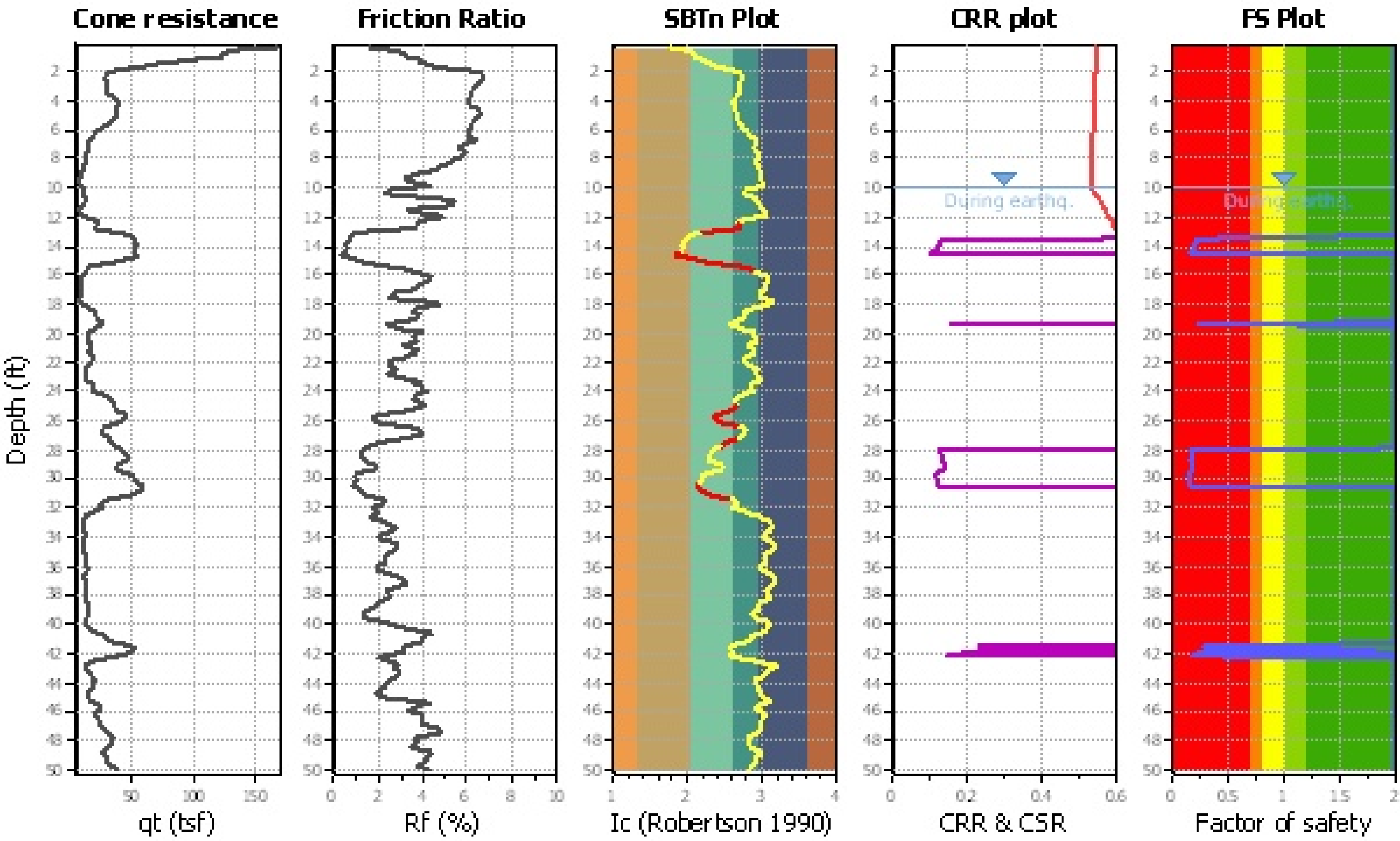
LIQUEFACTION ANALYSIS REPORT

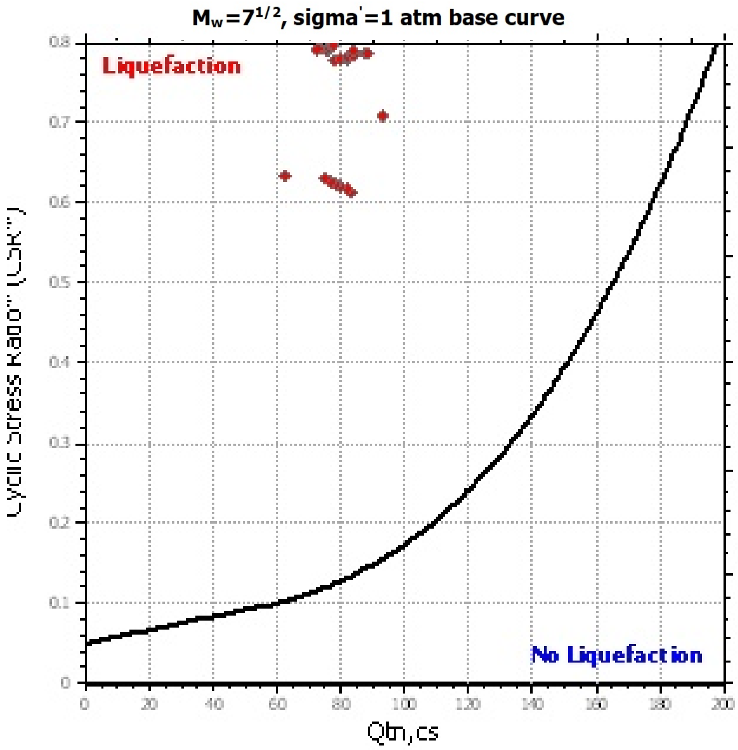
Project title : 38288-38594 Cedar Blvd. **Location : Newark, CA**

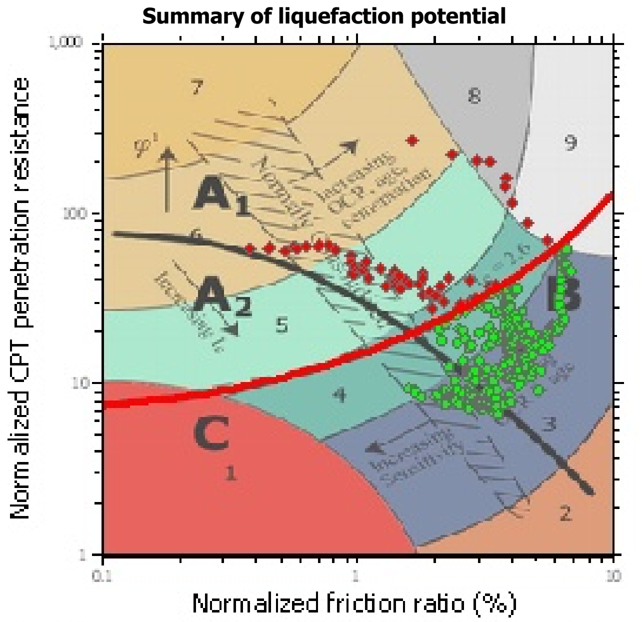
CPT file : CPT-02

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.50	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.84	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

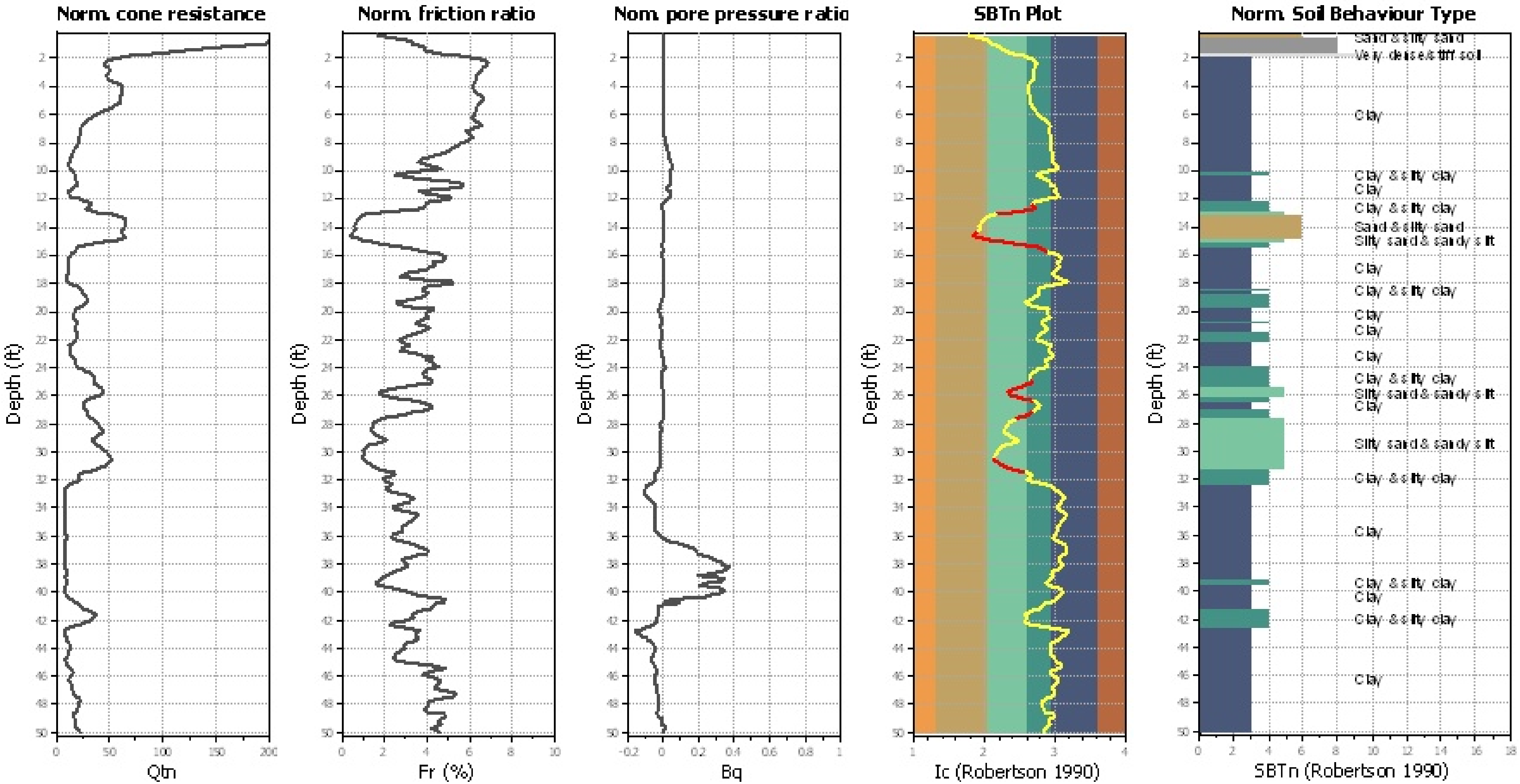






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normaliz



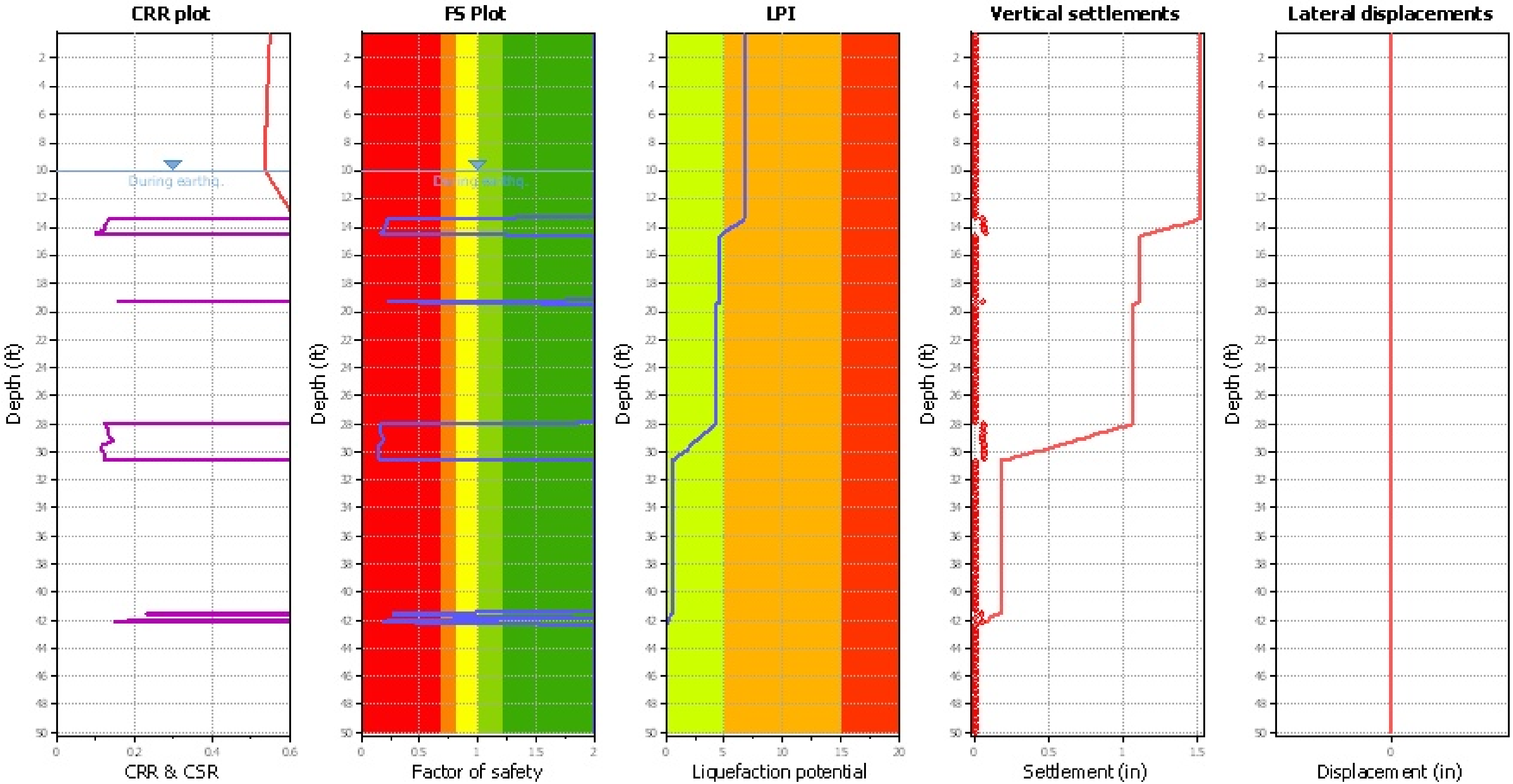
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.84	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.84	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

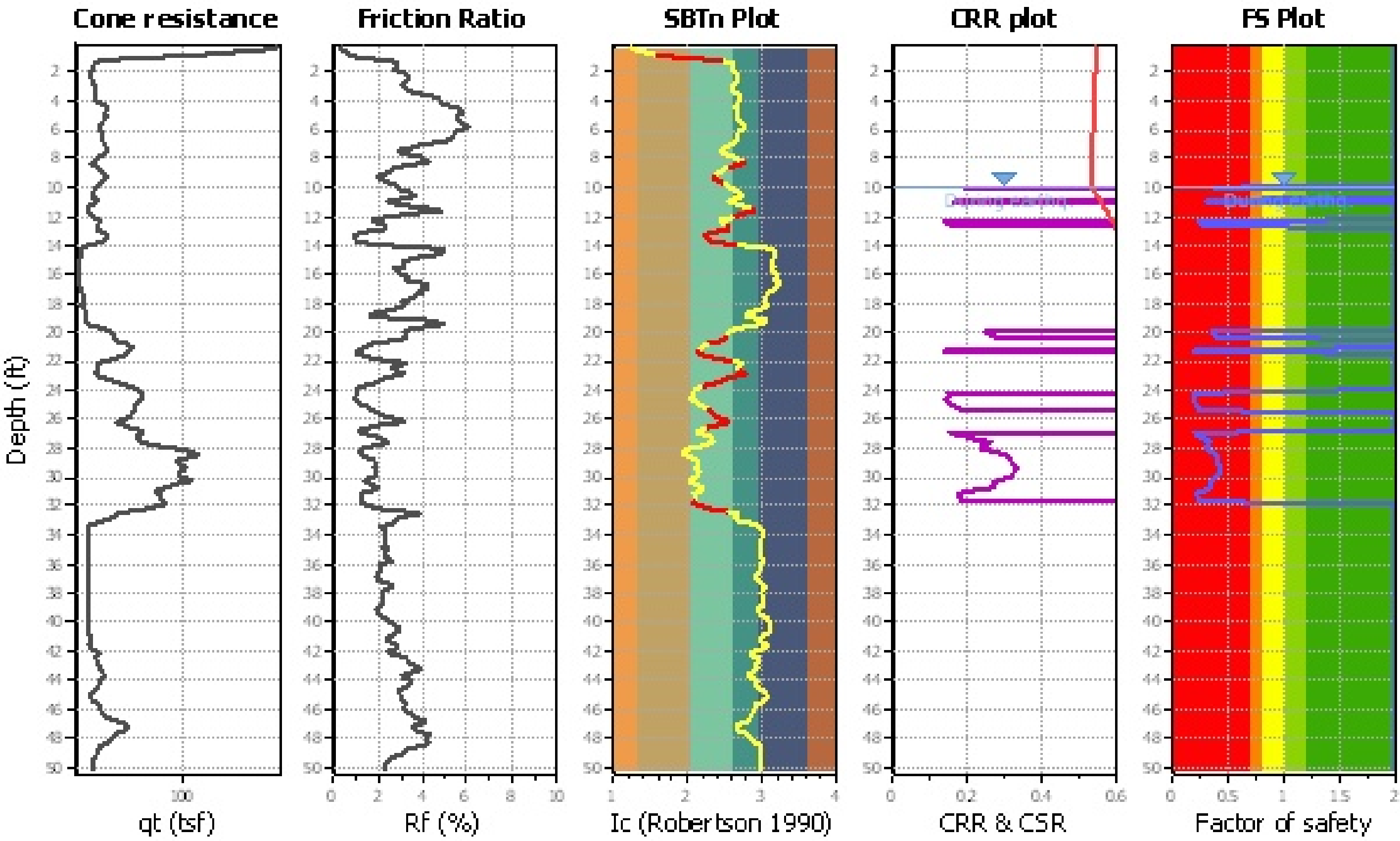
LIQUEFACTION ANALYSIS REPORT

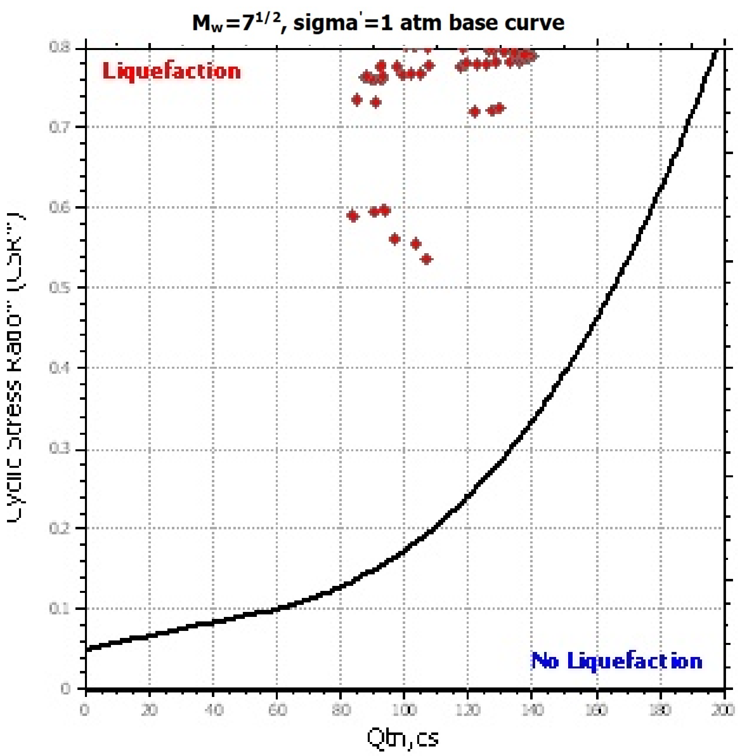
Project title : 38288-38594 Cedar Blvd. **Location : Newark, CA**

CPT file : CPT-03

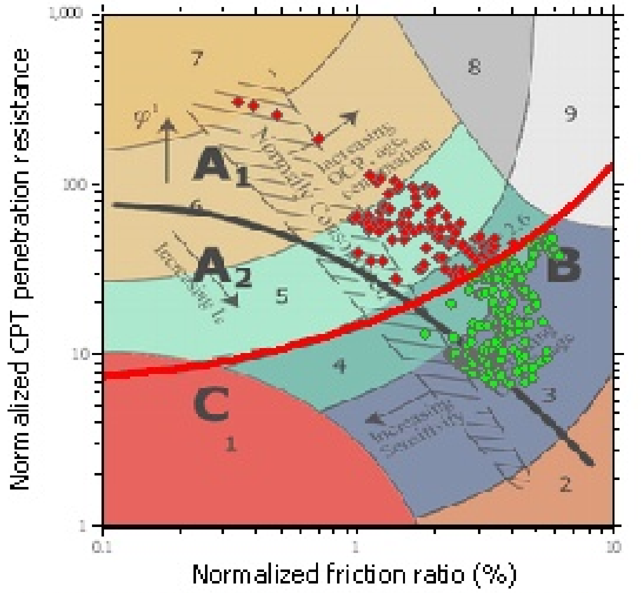
Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.50	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.84	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		





Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
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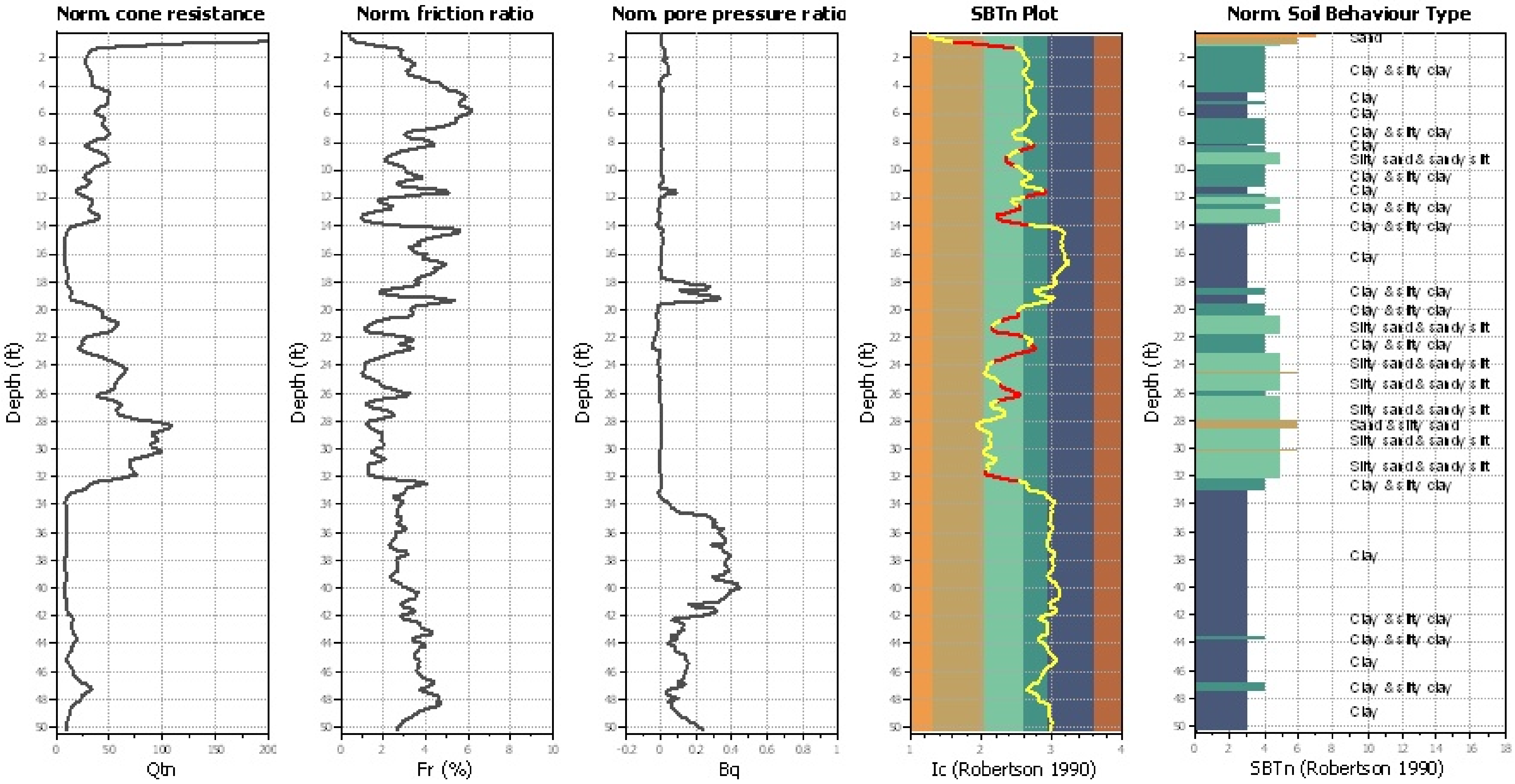
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normaliz



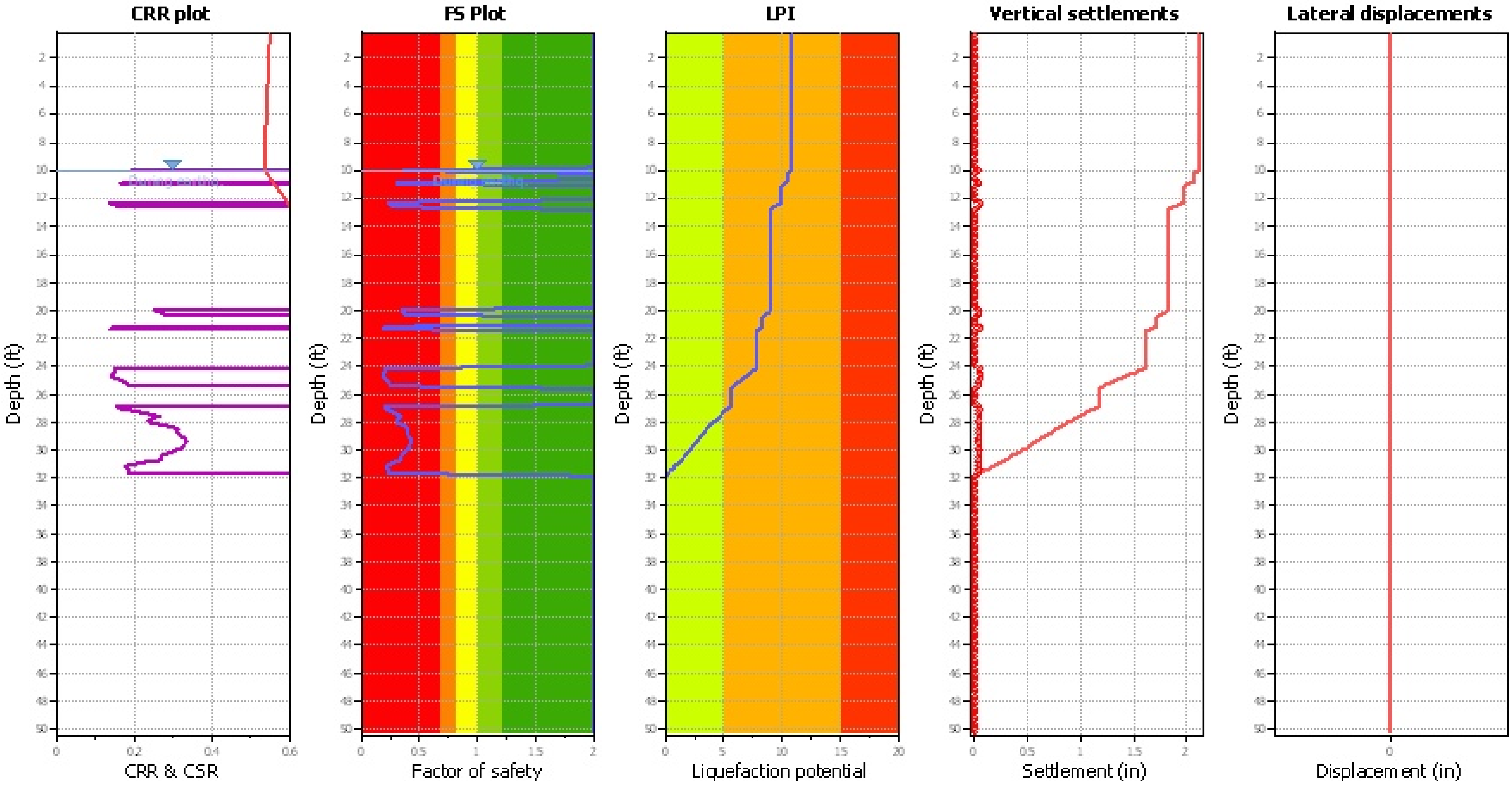
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.84	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to
- 9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.84	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

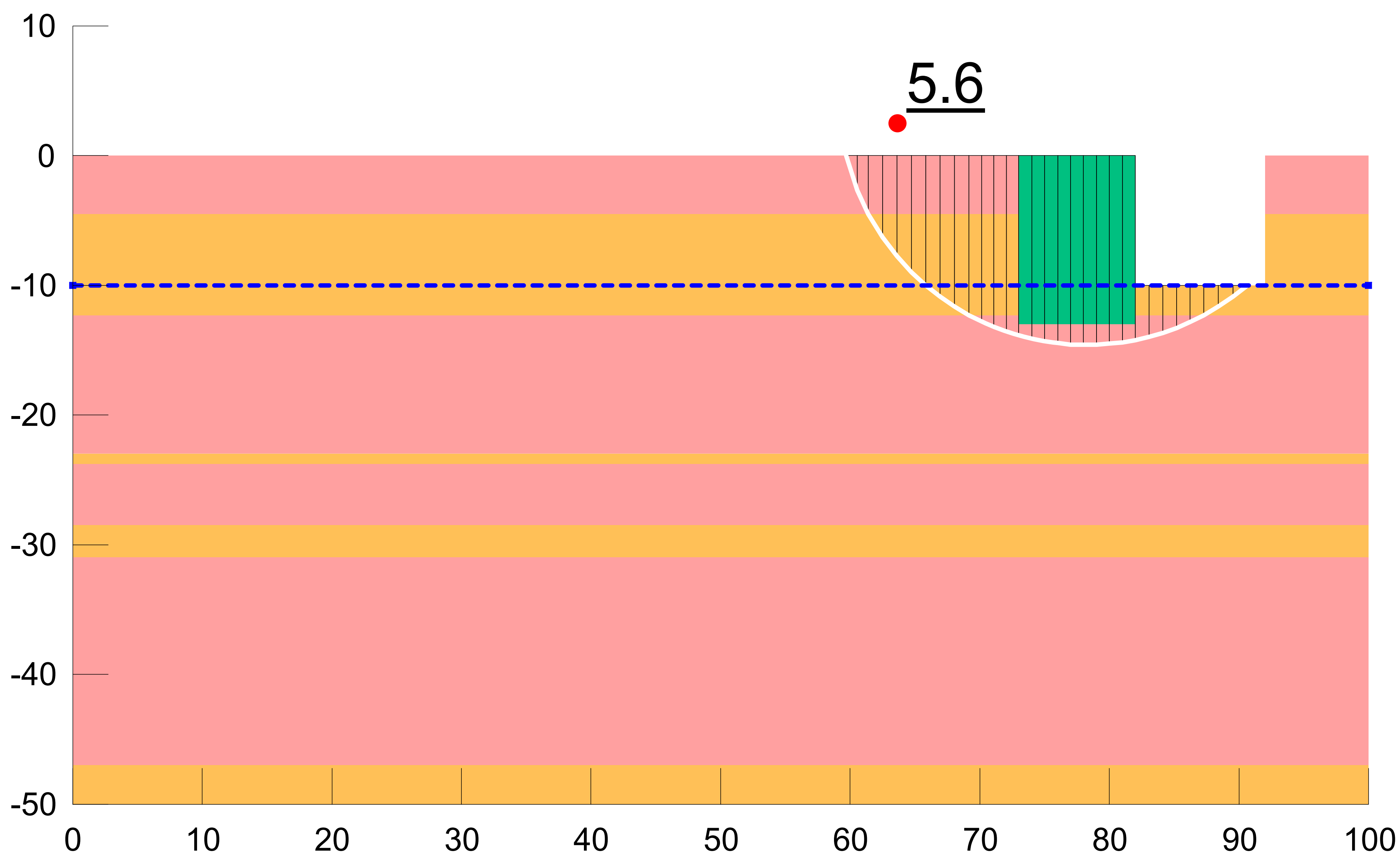
- Very high risk
- High risk
- Low risk

APPENDIX D

RESULTS OF SLOPE STABILITY ANALYSIS

Color	Name	Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Cohesion (psf)
	Gabion Wall	High Strength	140			
	Silty Clay to Clay	Undrained (Phi=0)	128			1,700
	Silty Sand to Sandy Silt	Mohr-Coulomb	131	0	34	

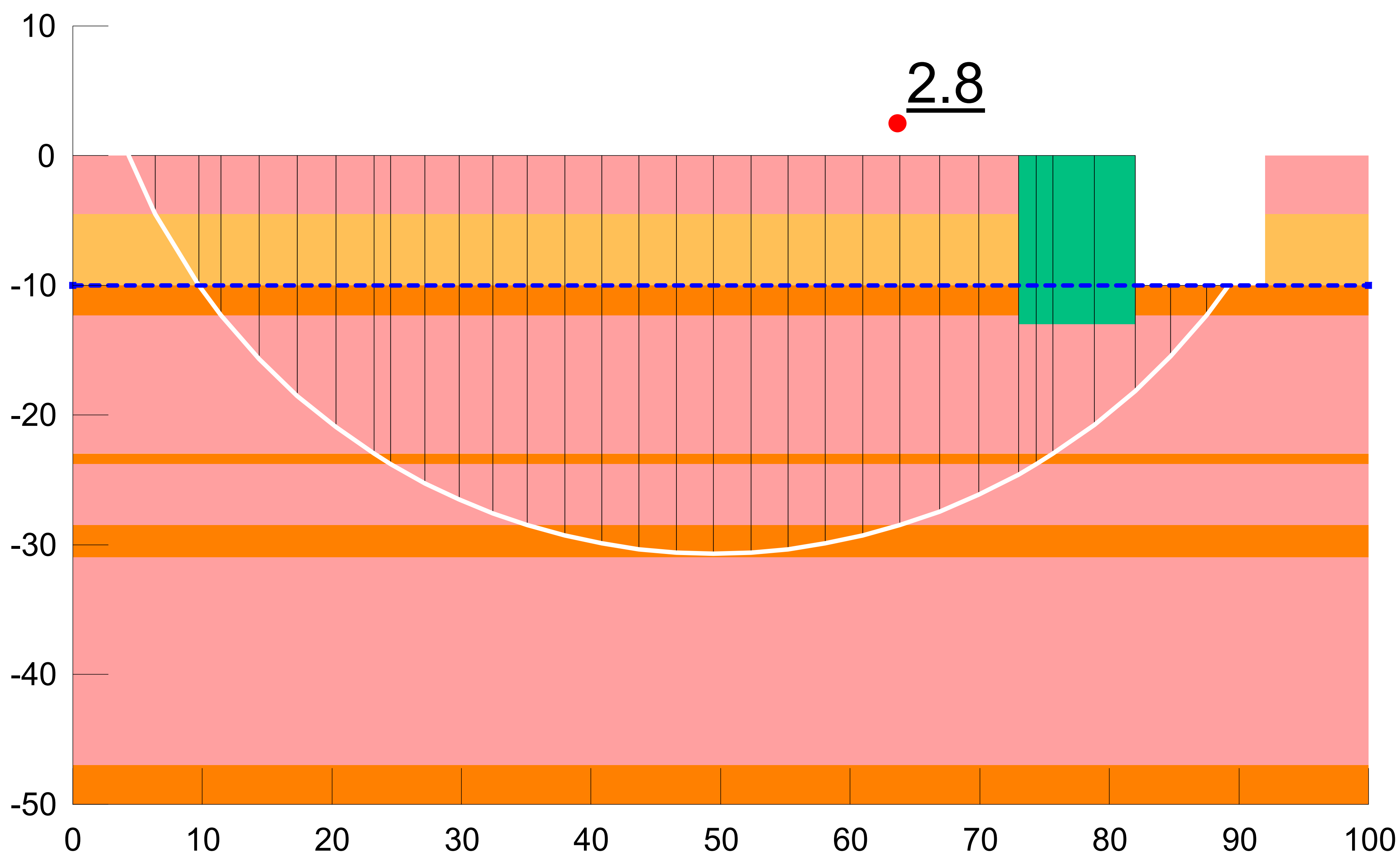
Horz Seismic Coef.: 0



Cedar Blvd Slope Stability Description: Static
Cedar Blvd Stability_01.gsz
09/07/2021 1:130

Color	Name	Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Cohesion (psf)
	Gabion Wall	High Strength	140			
	Silty Clay to Clay	Undrained (Phi=0)	128			1,700
	Silty Sand to Sandy Silt	Mohr-Coulomb	131	0	34	
	Silty Sand to Sandy Silt (Below GW)	Undrained (Phi=0)	131			850

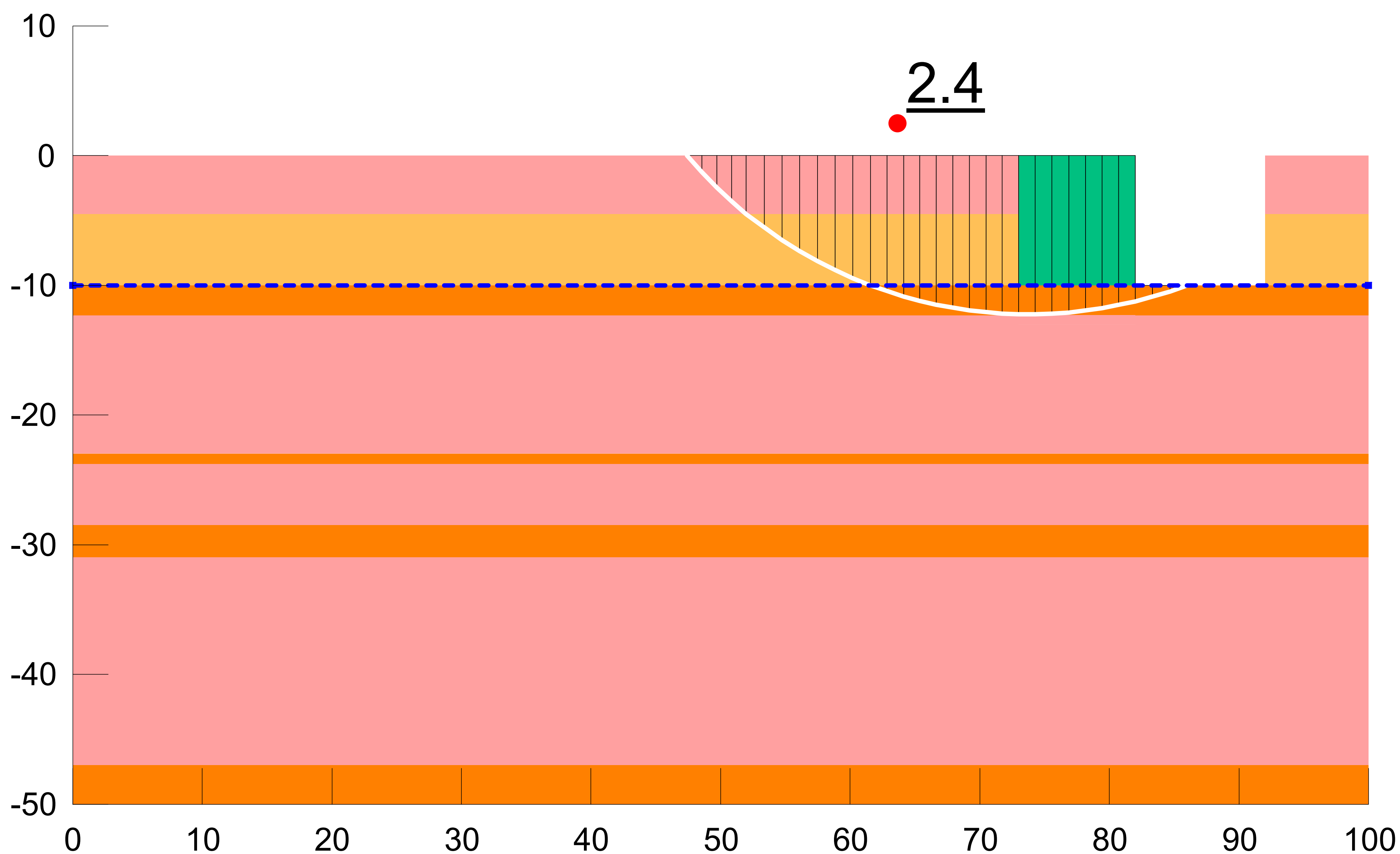
Horz Seismic Coef.: 0.25



Cedar Blvd Slope Stability Description: Seismic	
Cedar Blvd Stability_01_seismic.gsz	
09/07/2021	1:130

Color	Name	Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Cohesion (psf)
	Gabion Wall	High Strength	140			
	Silty Clay to Clay	Undrained (Phi=0)	128			1,700
	Silty Sand to Sandy Silt	Mohr-Coulomb	131	0	34	
	Silty Sand to Sandy Silt (Below GW)	Undrained (Phi=0)	131			850

Horz Seismic Coef.: 0.25



Cedar Blvd Slope Stability Description: Seismic	
Cedar Blvd Stability_02_seismic.gsz	
09/07/2021	1:130