APPENDIX D

PRELIMINARY GEOTECHNICAL INVESTIGATION

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CORNERSTONE
EARTH GROUP

Type of Services	Preliminary Geotechnical Investigation
Project Name	IPT Winton Avenue
Location	2663, 2695, 2707, 2711, 2725, and 2893 West Winton Avenue Hayward, California
Client	Industrial Property Trust
Client Address	4675 MacArthur Court, Suite 625 Newport Beach, California
Project Number	855-7-1
Date	March 10, 2017

DRAFT

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SECTION 1: INTRODUCTION

This preliminary geotechnical investigation was prepared for the sole use of Industrial Property Trust (IPT) for the warehouse project located at 2663, 2695, 2707, 2711, 2725, and 2893 West Winton Avenue in Hayward, California. The purpose of this study was to evaluate the existing subsurface conditions and develop an opinion regarding potential geotechnical concerns that could impact the proposed development. The preliminary geotechnical recommendations contained in this report are for your forward planning, cost estimating, and preliminary project design. For our use, we were provided with the following document:

- A topographic survey titled, “Topographic Survey of Winton Avenue for I.P.T., Hayward, California”, prepared by Kier & Wright Civil Engineers and Surveyors, Inc., dated February 2017.
- A site plan titled, “Schematic Design of Winton Avenue for Industrial Property Trust”, prepared by Kier & Wright Civil Engineers and Surveyors, dated February 14, 2017.
- A site plan titled, “Conceptual Site Plan, Winton Avenue, Hayward, CA,” prepared by HPA Architecture, dated December 16, 2016.

1.1 PROJECT DESCRIPTION

Based on our review of the above referenced conceptual plan prepared by HPA Architecture, the project will consist of a one-story, high bay, at-grade, warehouse of concrete tilt-up construction. The new warehouse will total about 502,500 square feet with a footprint of approximately 540 feet by 930 feet. Approximately 15,000 square feet of the building will be office space. At-grade parking for autos, trucks, and trailers will be located along the east, west, and south sides of the building. Loading docks will be located along the east and west sides of the building. Appurtenant utilities, landscaping and other improvements necessary for site development are also planned.

Structural loads were not available at the time of this proposal but are anticipated to be typical of this type of structure. Column spacing for the proposed warehouse will be from 50 to 60 feet. Based on the referenced plans, grading will include cuts of 1 to 5 feet in truck dock, storm water retention, and parking areas and fill of 1 to 6 feet for construction of the building pad.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated January 24, 2017, and consisted of field and laboratory programs to evaluate physical and engineering properties of the subsurface soils, engineering analysis to prepare preliminary recommendations for site work and grading, building foundations, flatwork, and pavements, and preparation of this report. Brief descriptions of our exploration and laboratory programs are presented below.

1.3 EXPLORATION PROGRAM

Field exploration consisted of two borings drilled on February 8, 2017, with truck-mounted hollow-stem auger drilling equipment and five Cone Penetration Tests (CPTs) advanced on February 3, 2017. The borings were drilled to depths of 30 and 50 feet; the CPTs were advanced to depths of 50½ to 100 feet. Seismic shear wave velocity measurements were collected from CPT-3. Boring EB-1 was advanced adjacent to CPT-1, and EB-2 was advanced adjacent to CPT-4 for direct evaluation of physical samples to correlated soil behavior.

The borings and CPTs were backfilled with cement grout in accordance with local requirements; exploration permits were obtained as required by local jurisdictions. The approximate locations of our exploratory borings and CPTs are shown on the Site Plan, Figure 2. Details regarding our field program are included in Appendix A.

1.4 LABORATORY TESTING PROGRAM

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

1.5 ENVIRONMENTAL SERVICES

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

SECTION 2: REGIONAL SETTING

2.1 GEOLOGICAL SETTING

San Francisco Bay is a northwesterly trending structural depression that lies along the boundary of the Pacific and North America tectonic plates. The Bay is within the Coast Ranges geomorphic province of California, which is characterized by a series of nearly parallel mountain ranges. Active faults, including the San Andreas, Hayward, and Calaveras Faults, roughly parallel the western and eastern limits of the Bay. The Bay began forming during the Pleistocene Epoch, approximately 2 million years ago, the San Francisco-Marin block began to tilt eastward along the Hayward Fault. The eastern side of the block became a depression and filled with sediment and water.

Bedrock units exposed in the eastern portions of the Bay range from Jurassic-Cretaceous to Quaternary age (approximately 135 million years old to recent). The oldest bedrock units (Jurassic-Cretaceous age) include the Franciscan Formation, which consists of interbedded sandstone and shale, limestone, radiolarian chert, and metavolcanic rocks (Graymer et al. 1994). The Franciscan Formation is west of the Hayward Fault, and is exposed in the hills along the Peninsula. East of the Hayward Fault, a thick sequence of Tertiary age sandstones and shales of the Great Valley Sequence overlies the Franciscan Formation. Along the eastern shoreline of the Bay, layers of Quaternary-age alluvial sediments mantle the Franciscan Formation. Since Cretaceous time, the Bay Area has undergone numerous episodes of faulting and folding. As such, rock units exposed along fault zones are typically sheared and highly weathered.

The site is mapped as being underlain by Holocene and Pleistocene deposits described as undivided surficial deposits (Qu) including stream, estuary, and levee deposits (Graymer, Jones, and Brabb, 1997). The site is also mapped as being underlain by Holocene alluvial fine-grained deposits (Qhaf) that interfinger with and grade into Bay Mud and medium grained alluvium (Helley and LaJoie, 1979). Although, Bay Mud was not encountered within our exploratory borings or indicated by our CPT data, maps of the area indicate the western portion of the site may be underlain by Bay Mud. The presence of Bay Mud should be further evaluated for the design-level geotechnical investigation.

2.2 REGIONAL SEISMICITY

The San Francisco Bay area region is one of the most seismically active areas in the Country. While seismologists cannot predict earthquake events, the U.S. Geological Survey's Working Group on California Earthquake Probabilities 2015 revises earlier estimates from their 2008 (2008, UCERF2) publication. Compared to the previous assessment issued in 2008, the estimated rate of earthquakes around magnitude 6.7 (the size of the destructive 1994 Northridge earthquake) has gone down by about 30 percent. The expected frequency of such events statewide has dropped from an average of one per 4.8 years to about one per 6.3 years. However, in the new study, the estimate for the likelihood that California will experience a magnitude 8 or larger earthquake in the next 30 years has increased from about 4.7 percent for UCERF2 to about 7.0 percent for UCERF3.

UCERF3 estimates that each region of California will experience a magnitude 6.7 or larger earthquake in the next 30 years. Additionally, there is a 63 percent chance of at least one magnitude 6.7 or greater earthquake occurring in the Bay Area region between 2007 and 2036.

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

Table 1: Approximate Fault Distances

Fault Name	Distance	
	(miles)	(kilometers)
Hayward (Total Length)	3.1	5.0
Calaveras	11.5	18.5
San Andreas (1906)	15.2	24.5

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

SECTION 3: SITE CONDITIONS

3.1 SITE BACKGROUND

Based on aerial images of the site provided by the websites HistoricAerials.com (2017) and Google Earth (2013), the site was occupied by two ranch-style homes and West Winton Avenue is visible in an image dated 1946. Subdivision of the site and additional residences and a commercial development in the southwestern portion of the site is visible in an image dated 1958. The southern portion of the site is occupied by commercial development consisting of fenced yards (auto wrecking), and the northern portion of the site is relatively undeveloped with several objects scattered throughout in an image dated 1966. The site was occupied by an automobile wrecking facility including shop buildings in the southern and northern portions of the site in an image dated 1968. Additional buildings are visible in an image dated 1980. The northeast portion of the site is occupied by paved parking areas and driveways in an image dated 1987. The site is occupied by numerous wrecked vehicles in images dated 1993 to 2005. The western portion of the site was cleared of vehicles and items in an image dated 2009. The north, east, and west portions of the site were cleared of vehicles and buildings in images dated 2010 and 2012. The central portion of the site was cleared after 2012 and prior to February 2014.

3.2 SURFACE DESCRIPTION

The site is currently occupied by a shop/commercial building located in the southern portion of the site. The site is relatively level and at or near the elevations of the adjacent roadways and properties. Based on the preliminary grading plan prepared by Kier & Wright (2017), the elevation of the site ranges from approximately Elevation 6¾ to 13 feet above mean sea level (MSL) in the southwestern and northeastern portions of the site, respectively. The site is trapezoidal in shape and bounded by West Winton Avenue to the south, San Francisco Bay wetlands to the west, and commercial developments to the north and east.

The ground surface observed during our field explorations predominantly consisted of crushed aggregate; however, concrete pavement and slabs were observed in the southern portion of the site near the existing shop/commercial building. Approximately 6 inches of granular base was encountered within EB-1. Short retaining walls were also observed within the central portion of the site. Areas of grass, weeds, and bushes were also observed in the central portion of the site.

3.3 SUBSURFACE CONDITIONS

Below the ground surface, our explorations generally encountered undocumented fill underlain by native alluvial soil. Undocumented fill was encountered within our exploratory borings to depths of approximately 2¾ to 4 feet below the existing grades and consisted of medium dense clayey sand with gravel and very stiff sandy lean clay. The underlying alluvium consisted of medium stiff to stiff high plasticity clay, medium stiff to very stiff sandy lean clay, medium stiff to very stiff lean clay and lean clay with sand, loose to medium dense silty sand, and medium dense poorly graded sand with silt. Loose silty sand was encountered at depths of 12 to 16½ feet and medium dense silty sand was encountered at depths of 21½ to 23½ feet and 22 to 25½ feet. Medium dense poorly graded sand with silt was encountered at depths of 16½ to 22 feet below the existing grades.

Our CPTs generally indicated soil behavior types including clay, silty clay to clay, clayey silt to silty clay, sandy silt to clayey silt, silty sand to sandy silt, sand to silty sand, sand, medium stiff to very stiff fine grained soil, and sand to clayey sand to a depth of 100 feet below the existing grades, the maximum depth explored.

3.3.1 Plasticity/Expansion Potential

We performed one Plasticity Index (PI) test on a representative soil sample. Test results were used to evaluate expansion potential of surficial soil. The surficial PI test resulted in a PI of 25, indicating moderate expansion potential to wetting and drying cycles.

3.3.2 In-Situ Moisture Contents

Laboratory testing indicated the in-situ moisture contents within the upper 10 feet range from about 0 to 15 percent above the estimated laboratory optimum moisture.

3.4 GROUND WATER

Ground water was encountered in EB-1 and EB-2 at depths of 5¼ to 9 feet below the existing grades (corresponding to Elevations 4¼ and -1 feet MSL). Additionally, three pore pressure dissipation tests were performed at CPT-1 through CPT-3 and indicated ground water depths of 5 to 6 feet (corresponding to Elevations 4½ and 1 feet MSL). Ground water maps for the site indicate a historic high ground water level of 5 feet (CGS, 2003). Additionally, ground water level data provided on the Geotracker website (http://www.envirostor.dtsc.ca.gov/public/) for monitoring wells located approximately 2,300 feet east of the site indicate high ground water levels of 4½ to 7½ feet below the surface. Based on the proximity of the site to the San

Francisco Bay, fluctuations in the elevation of the ground water level at the site is likely to occur and should be planned for during design and construction. All measurements were taken at the time of drilling and may not represent the stabilized levels that can be higher than the initial levels encountered. For our preliminary analysis, we assumed a design ground water depth of 5 feet below existing site grades.

Fluctuations in ground water levels occur due to many factors including seasonal fluctuation, underground drainage patterns, regional fluctuations, and other factors.

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT RUPTURE

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

4.2 ESTIMATED GROUND SHAKING

Moderate to severe (design-level) earthquakes can cause strong ground shaking, which is the case for most sites within the Bay Area. A peak ground acceleration (PGA) of 0.65g was estimated for analysis using a value equal to $PGA_M = F_{PGA} \times PGA_G$ (Equation 11.8-1) as allowed in the 2016 California Building Code (CBC).

4.3 LIQUEFACTION POTENTIAL

The site is within a State-designated Liquefaction Hazard Zone (CGS, San Leandro Quadrangle, 2003). Our field and laboratory programs addressed this issue by testing and sampling potentially liquefiable layers to a depth of 50 feet, performing visual classification on sampled materials, evaluating CPT data, and performing various tests to further classify soil properties.

4.3.1 Background

During strong seismic shaking, cyclically induced stresses can cause increased pore pressures within the soil matrix that can result in liquefaction triggering, soil softening due to shear stress loss, potentially significant ground deformation due to settlement within sandy liquefiable layers as pore pressures dissipate, and/or flow failures in sloping ground or where open faces are present (lateral spreading) (NCEER 1998). Limited field and laboratory data is available regarding ground deformation due to settlement; however, in clean sand layers settlement on the order of 2 to 3 percent of the liquefied layer thickness can occur. Soils most susceptible to liquefaction are loose, non-cohesive soils that are saturated and are bedded with poor drainage, such as sand and silt layers bedded with a cohesive cap.

4.3.2 Analysis

As discussed in the "Subsurface" section above, several sand layers were encountered below the design ground water depth of 5 feet. Following the procedures in the 2008 monograph, *Soil Liquefaction During Earthquakes* (Idriss and Boulanger, 2008) and in accordance with CDMG Special Publication 117A guidelines (CDMG, 2008) for quantitative analysis, these layers were analyzed for liquefaction triggering and potential post-liquefaction settlement. These methods compare the ratio of the estimated cyclic shaking (Cyclic Stress Ratio - CSR) to the soil's estimated resistance to cyclic shaking (Cyclic Resistance Ratio - CRR), providing a factor of safety against liquefaction triggering. Factors of safety less than or equal to 1.3 are considered to be potentially liquefiable and capable of post-liquefaction re-consolidation.

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

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

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

The results of our CPT analyses (CPT-1 through CPT-5) are presented on Figures 4A through 4E of this report.

4.3.3 Summary

Our analyses indicate that several layers could potentially experience liquefaction triggering that could result in soil softening and post-liquefaction total settlement at the ground surface of less than ¼ inch to 1 inch based on the Yoshimine et al. (2006) method. As discussed in SP 117A, differential movement for level ground sites over deep soil sites will be up to about two-thirds of the total settlement between independent foundation elements. In our opinion, differential settlement is anticipated to be less than ¼- to ⅔-inch between independent foundation elements, or over a horizontal distance of 50 feet. As the soil conditions at CPT-1 may be localized, we recommend performing additional subsurface explorations (i.e. CPTs and borings) in the vicinity of CPT-1 for the design-level geotechnical investigation in order to further evaluate the lateral extent of liquefaction settlement.

4.3.4 Ground Rupture Potential

The methods used to estimate liquefaction settlements assume that there is a sufficient cap of non-liquefiable material to prevent ground rupture or sand boils. For ground rupture to occur, the pore water pressure within the liquefiable soil layer will need to be great enough to break through the overlying non-liquefiable layer, which could cause significant ground deformation and settlement. The work of Youd and Garris (1995) indicates that the 14-foot thick layer of non-liquefiable cap is sufficient to prevent ground rupture; therefore the above total settlement estimates are reasonable. In addition, we understand that 1 to 6 feet of fill will be placed during development of the site, thus increasing the thickness of non-liquefiable soil.

4.4 LATERAL SPREADING

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

Wetlands of the San Francisco Bay are located adjacent to the site and Sulfur Creek is located approximately 900 feet north of the site; however, there are no significant open faces within a distance considered susceptible to lateral spreading; therefore, in our opinion, the potential for lateral spreading to affect the site is low.

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose unsaturated sandy soils can settle during strong seismic shaking. As the soil encountered at the site was predominantly medium stiff to very stiff clay above the preliminary design ground water depth of 5 feet, in our opinion, the potential for significant differential seismic settlement affecting the proposed improvements is low.

4.6 TSUNAMI/SEICHE

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

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

A tsunami or seiche originating in the Pacific Ocean would lose much of its energy passing through San Francisco Bay. Based on the study of tsunami inundation potential for the San Francisco Bay Area (Ritter and Dupre, 1972), areas most likely to be inundated are marshlands, tidal flats, and former bay margin lands that are now artificially filled, but are still at or below sea level, and are generally within 1½ miles of the shoreline. Based on the tsunami inundation map by CGS (2009), the majority of the site is within an inundation zone. The potential for tsunami inundation to impact the site should be considered in design.

4.7 FLOODING

Based on our internet search of the Federal Emergency Management Agency (FEMA) flood map public database, the site is predominately located within Zone X described as “0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile”; however, the southwestern portion of the site is located within Zone AE, described as “Base Flood Elevations Determined” with a base flood elevation of 10 feet. We recommend the project civil engineer be retained to confirm this information and verify the base flood elevation, if appropriate.

4.8 COMPRESSIBLE SOIL

As discussed, the site lies at the margin of young, moderately compressible alluvial deposits. Based on our recent exploration and review of information in our files, the western portion of the site and the western portion of the proposed building may be underlain by soft to medium stiff, moderately compressible silty clay and/or Bay Mud. Preliminary settlement analyses were performed to estimate future long-term settlement due to areal fill placement. Preliminary settlement estimates are based on the site history previously described and our limited field explorations and are intended to provide rough settlement estimates for a period of 50 years after construction. Additional settlement analysis will be required during the design-level geotechnical investigation.

SECTION 5: CONCLUSIONS

5.1 SUMMARY

From a geotechnical viewpoint, the project is feasible provided the concerns listed below are addressed in the project design. The preliminary recommendations that follow are intended for conceptual planning and preliminary design. A design-level geotechnical investigation should be performed once site development plans are final. The design-level investigation findings will be used to confirm the preliminary recommendations and develop detailed recommendations for design and construction. Descriptions of each geotechnical concern with brief outlines of our preliminary recommendations follow the listed concerns.

- Potential for static settlement due to the placement of fill
- Potential for liquefaction-induced settlement
- Shallow ground water
- Presence of undocumented fill
- Presence of expansive soil

5.1.1 Potential for Significant Static Settlement with the Addition of Fill

As discussed, the site is predominately underlain by clay that will consolidate under the weight of new fill and foundation loads resulting in settlement. The preliminary static settlement estimates presented in Table 2 are based on the addition of up to 7 feet of fill (including slab-on-grade section). Additionally, portions of the site may be underlain by highly compressible Bay Mud. The presence of Bay Mud at the site will result in increased settlement as a result of foundation loads and placement of fill. Therefore, the static settlement estimates present below are preliminary and should be further evaluated during the design-level geotechnical investigation. The thickness of new fill assumed in our analysis includes all materials (e.g. aggregate base, crushed rock, and concrete slabs-on-grade) from existing grade up to finish floor elevation. Our preliminary analysis assumed an average soil profile across the site; however, post-construction settlement is anticipated to be less near the northwest corner of the building (EB-1/CPT-1) and potentially higher near the southwest corner of the building (EB-2/CPT-4).

Table 2: Preliminary Average Static Settlement Estimates

Thickness of New Fill (feet)[1]	Total Static Settlement (inch)	Differential Static Settlement (inch)
1	½	¼
2	⅝	<⅜
3	¾	⅜
4	1	½
5	1¼	⅝
6	1½	¾
7	1¾	⅞

[1] Depth of fill includes materials from existing grade up to finish floor.

As an option to reduce the impact of settlement on the proposed structure, a surcharge program consisting of placing fill in the proposed building location prior to the start of construction may be selected. On a preliminary basis, the period of the surcharge program would be approximately 6 to 9 months prior to the start of construction. The installation of wick drains may be selected to reduce the estimated time period of the surcharge program. If a surcharge program is selected, recommendations will be provided in the design-level geotechnical report.

In addition to the preliminary static settlement estimates presented in Table 2 above, foundations should also be designed to tolerate the total and differential seismic settlement discussed in Section 4.3 and in the following section. Preliminary foundation recommendations are presented in the “Foundations” section of this report.

5.1.2 Potential for Liquefaction-Induced Settlement

As discussed, our preliminary liquefaction analysis indicates that there is a potential for liquefaction of localized sand layers during a significant seismic event. Although the potential for liquefied sand to vent to the ground surface through cracks in the surficial soil is low, our preliminary analysis indicates that liquefaction-induced settlement of less than ¼ to 1 inch could occur, resulting in differential settlement of less than ¼ to ⅔ inch between adjacent foundation elements. As discussed, total liquefaction settlement of about 1 inch was calculated at CPT-1 whereas the total liquefaction settlement calculated at CPT-2 through CPT-5 was less than ¼ inch. Based on our analysis, significant liquefaction settlement is potentially localized in the northwestern portion of the site; however, other areas of significant liquefaction potential may be present and should be further evaluated during the design-level geotechnical investigation. Additionally, foundations should be designed to tolerate the estimated total and differential settlement (static and seismic). Preliminary foundation recommendations are presented in the “Foundations” section of this report.

5.1.3 Shallow Ground Water

Shallow ground water was measured at depths of 5¼ to 13 feet below the existing ground surface (corresponding to Elevations 4¼ and -1 feet MSL). Due to the proximity to the Bay, the ground water level at the site is anticipated to fluctuate. For this project, we recommend that a preliminary design ground water depth of 5 feet be used. Our experience with similar sites indicates that shallow ground water could significantly impact grading and underground construction. These impacts typically consist of potentially wet and unstable pavement subgrade, difficulty achieving compaction, and difficult underground utility installation. Dewatering and shoring of utility trenches may be required in some isolated areas of the site.

5.1.4 Presence of Undocumented Fill

As discussed in Section 3.3, up to 4 feet of undocumented fill was encountered in our borings, deeper fill is possible and should be anticipated and planned for by the contractor. Since the proposed structure can possibly be supported on shallow foundations, we recommend that remedial grading include over-excavation and re-compaction of undocumented fill within the building footprint. Additionally, the presence of shallow ground water will likely impact the stability of the bottom of the over-excavation. Therefore, stabilization fabric or chemical treatment may be needed to bridge wet and unstable soil.

5.1.5 Presence of Expansive Soil

Moderately expansive fill soils generally blanket the site. Expansive soils can undergo significant volume change with changes in moisture content. They shrink and harden when dried and expand and soften when wetted. To reduce the potential for damage to the planned structures, slabs-on-grade should have sufficient reinforcement and be supported on a layer of non-expansive fill; footings should extend below the zone of seasonal moisture fluctuation. In addition, it is important to limit moisture changes in the surficial soils by using positive drainage away from buildings as well as limiting landscaping watering. Preliminary grading and foundation recommendations addressing this concern are presented in the following sections.

5.2 DESIGN-LEVEL GEOTECHNICAL INVESTIGATION

The preliminary recommendations contained in this preliminary study were based on limited site development information and limited exploration, review of available subsurface information and our experience in the area. As site conditions may vary significantly between the small-diameter borings performed during this investigation, we also recommend that we be retained to 1) perform a design-level geotechnical investigation, once detailed site development plans are available; 2) to review the geotechnical aspects of the project structural, civil, and landscape plans and specifications, allowing sufficient time to provide the design team with any comments prior to issuing the plans for construction; and 3) be present to provide geotechnical observation and testing during earthwork and foundation construction.

SECTION 6: EARTHWORK

6.1 ANTICIPATED EARTHWORK MEASURES

On a preliminary basis, we recommend that any remnant foundations, slabs, and/or abandoned underground utilities be removed entirely and the resulting excavations backfilled with engineered fill. Additionally, any native soils that are disturbed during demolition of the existing improvements should also be removed and replaced as engineered fill. Existing undocumented fills should be over-excavated and re-compacted prior to placing new fill for the building pad. For preliminary planning, we suggest an average over-excavation depth of 3 feet be assumed for initial cost estimating.

All on-site soils below the surficial organic soil layer may be suitable for use as fill at the site, unless determined to be unsuitable due to previous environmental impacts. On a preliminary basis, imported fill material for use as general fill should have a Plasticity Index of 15 or less. All fill as well as scarified surface soils in those areas to receive fill or slabs-on-grade should be compacted to at least 90 percent relative compaction as determined by ASTM Test Designation D-1557, latest edition; and be at least 3 percent above optimum. The upper 6 inches of subgrade in pavement areas and all aggregate base materials should be compacted to at least 95 percent relative compaction (ASTM D-1557, latest edition). Utility trench backfill should be compacted to at least 90 percent relative compaction (ASTM D-1557, latest edition) by mechanical means only.

Surface water runoff should not be allowed to pond adjacent to building foundations, slabs-on-grade, or pavements. Hardscape surfaces should slope at least 1 to 2 percent towards suitable discharge facilities; landscape areas should slope at least 2 to 3 percent away from buildings.

SECTION 7: FOUNDATIONS

7.1 SUMMARY OF RECOMMENDATIONS

On a preliminary basis, the proposed structure can likely be supported on shallow foundations provided the building can be designed to tolerate higher total and differential settlement due to seismic and static loading, and provided the recommendations in the “Earthwork” section and the sections below are considered for preliminary planning.

7.2 SEISMIC DESIGN CRITERIA

The project structural design should be based on the 2016 California Building Code (CBC), which provides criteria for the seismic design of buildings in Chapter 16. The “Seismic Coefficients” used to design buildings are established based on a series of tables and figures addressing different site factors, including the soil profile in the upper 100 feet below grade and mapped spectral acceleration parameters based on distance to the controlling seismic source/fault system. Shear wave velocity measurements performed for our investigation at CPT-3 to a depth of approximately 100 feet below the existing grade resulted in average shear wave velocities of 681 feet per second (or 208 meters per second). Based on the shear wave velocity measurements, our borings, and review of the local geology, on a preliminary basis, we have classified the site as Soil Classification D. The mapped spectral acceleration parameters S_S and S_1 were calculated using the USGS computer program *Design Maps*, located at http://earthquake.usgs.gov/hazards/designmaps/usdesign.php, based on the site coordinates presented below and the site classification. The table below lists the various factors used to determine the seismic coefficients and other parameters.

Table 3: CBC Site Categorization and Site Coefficients

Classification/Coefficient	**Design Value**
Site Class	D
Site Latitude	37.65285°
Site Longitude	-122.14244°
0.2-second Period Mapped Spectral Acceleration[1], S_S	1.680g
1-second Period Mapped Spectral Acceleration[1], S_1	0.663g
Short-Period Site Coefficient – Fa	1.0
Long-Period Site Coefficient – Fv	1.5
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - S_{MS}	1.680g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	0.995g
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.120g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	0.663g

[1] For Site Class B, 5 percent damped.

7.3 SHALLOW FOUNDATIONS

7.3.1 Spread Footings

On a preliminary basis, the proposed structure may be supported on conventional shallow footings that should bear on natural, undisturbed soil or engineered fill, be at least 18 inches wide, and extend at least 18 inches below the lowest adjacent grade. Lowest adjacent grade is defined as the deeper of the following: 1) bottom of the adjacent interior slab-on-grade, or 2) finished exterior grade, excluding landscaping topsoil. The deeper footing embedment is recommended due to the presence of moderately to highly expansive soils, and is intended to embed the footing below the zone of significant seasonal moisture fluctuation, reducing the potential for differential movement.

On a preliminary basis, footings should be designed for allowable bearing pressures of 2,000 pounds per square foot (psf) for dead loads, 3,000 psf for combined dead plus live loads, and 4,000 psf for all loads including wind and seismic.

7.3.2 Footing Settlement

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

Table 4: Assumed Structural Loading

Foundation Area	Range of Assumed Loads
Interior Isolated Column Footing	100 to 120 kips
Perimeter Strip Footing	4 to 6 kips per lineal foot

Based on the above loading and the allowable bearing pressures presented above, we estimate the total static footing settlement will be on the order of ¼ inch, with less than ¼ inch of post-construction differential settlement between adjacent foundation elements. In addition we estimate that differential seismic movement will be on the order of ⅔ inch between independent foundation elements or over a horizontal distance of 50 feet, resulting in a total estimated differential footing movement on the order of 1 inch between foundation elements, assumed to be on the order of 50 feet. As discussed, up to about 7 feet of fill (including slab-on-grade section) will be placed for construction of the building pad. For preliminary estimates of total and differential static settlement due to the assumed foundation loads and the addition of new fill, refer to Table 2. For estimating total (static and seismic) settlement for different fill depths, the total and differential seismic settlement discussed above should be added to the preliminary static settlement estimates presented in Table 2. Additionally, the presence of highly compressible Bay Mud will significantly impact our preliminary static settlement estimates. Our preliminary static settlement estimates are based on the fill depths indicated on the preliminary grading plan and our assumed footing loads. Therefore we recommend our preliminary settlement estimates be reevaluated during the design-level geotechnical investigation after the anticipated structural loading is known.

As previously discussed, the preliminary settlement estimates may not be tolerable for shallow spread footing foundations. Therefore, a surcharge program may be selected to reduce the impact of static settlement on the proposed structure. As an alternative, ground improvement may be selected to reduce the impacts of static and seismic settlement on the proposed building. Preliminary recommendations for ground improvement are discussed below.

7.3.3 Ground Improvement

If foundations designed in accordance with the above recommendations are not capable of resisting such differential movement, ground improvement may be considered to reduce the estimated static and seismic settlement. On a preliminary basis, ground improvement may be used to reduce the estimated differential settlement due to static and seismic loading to less than 1 inch.

The design allowable bearing pressures will be dependent on the final ground improvement details including the stiffness and spacing of ground improvement elements; however, substantial improvement in bearing capacity would be expected. For your preliminary design, we expect allowable bearing pressures on the order of 4,000 psf for combined dead plus live loads would be feasible following ground improvement (this assumes that ground improvement is performed after fill placement). The above estimates should be evaluated further during the design-level geotechnical investigation and after a designed-build ground improvement contractor is chosen.

Ground improvement and the replacement of disturbed near-surface soil as engineered fill, if necessary, would be designed to reduce total settlement due to static and seismic conditions to a tolerable level.

SECTION 8: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

8.1 WAREHOUSE SLABS-ON-GRADE

Warehouse slabs-on-grade should be at least 6 inches thick should have a minimum compressive strength of 3,500 psi. The warehouse slab should also be supported on at least 6 inches of non-expansive, crushed granular base having an R-value of at least 50 and no more than 10 percent passing the No. 200 sieve, such as Class 2 aggregate base. Due to the moderate plasticity of the surficial soils, an additional 6 inches of non-expansive fill (NEF) should underlie the upper granular base. All base and sub-base materials should be placed and compacted in accordance with the “Compaction” section of this report. If there will be areas within the warehouse that are moisture sensitive, such as equipment and elevator rooms, a vapor barrier may be placed over the upper granular base prior to slab construction. Please refer to the recommendations in the “Interior Slabs Moisture Protection Considerations” section for vapor barrier construction. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness.

8.2 EXTERIOR SLABS-ON-GRADE

Exterior concrete flatwork subject to pedestrian and/or occasional light pick up loading should be at least 4 inches thick and supported on at least 6 inches of Class 2 aggregate base overlying subgrade prepared in accordance with the “Earthwork” recommendations of this report. Flatwork that will be subject to heavier or frequent vehicular loading should be designed in accordance with the recommendations in the “Vehicular Pavements” section below.

SECTION 9: VEHICULAR PAVEMENTS

9.1 ASPHALT CONCRETE

The following asphalt concrete pavement recommendations tabulated below are based on the Procedure 608 of the Caltrans Highway Design Manual, estimated traffic indices for various pavement-loading conditions, and on a preliminary design R-value of 5. The preliminary design R-value was chosen based on experience with similar clay soil conditions and engineering judgment considering the variable surface conditions.

Table 5: Preliminary Asphalt Concrete Pavement Recommendations

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base* (inches)	Total Pavement Section Thickness (inches)
4.0	2.5	7.5	10.0
4.5	2.5	9.5	12.0
5.0	3.0	10.0	13.0
5.5	3.0	12.0	15.0
6.0	3.5	12.5	16.0
6.5	4.0	14.0	18.0
7.0	4.0	15.5	19.5
7.5	4.5	17.0	21.5
8.0	5.0	17.5	22.5

*Caltrans Class 2 aggregate base; minimum R-value of 78

As discussed, pavement subgrade support could potentially be impacted by the presence of shallow, wet subgrade conditions. Wet subgrade soils will likely require chemical stabilization using lime or cement to create a stiff bridging layer and to dry out exposed soils. Otherwise, subgrade soil may need to be over-excavated to depths on the order of 12 to 24 inches and replaced with imported crushed rock, aggregate base or subbase underlain by geotextile stabilization fabric or geogrid.

9.2 PORTLAND CEMENT CONCRETE

The exterior Portland Cement Concrete (PCC) pavement recommendations tabulated below are preliminary and based on methods presented in the Portland Cement Association (PCA) design manual (PCA, 1984). We have provided a few pavement alternatives as an anticipated Average Daily Truck Traffic (ADTT) was not provided. An allowable ADTT should be chosen that is greater than what is expected for the development.

Table 6: Preliminary PCC Pavement Recommendations

Allowable ADTT	Minimum PCC Thickness (inches)
13	5.5
130	6

The PCC thicknesses above are based on a concrete compressive strength of at least 3,500 psi, supporting the PCC on at least 6 inches of Class 2 aggregate base, and laterally restraining the PCC with curbs or concrete shoulders. Adequate expansion and control joints should be included. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness.

SECTION 10: LIMITATIONS

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

Preliminary recommendations in this report are based upon the soil and ground water conditions encountered during our limited subsurface exploration. Preparation of a design-level investigation is anticipated to provide additional information and refine the preliminary recommendations presented herein. If variations or unsuitable conditions are encountered during the construction phase, Cornerstone must be contacted to provide supplemental recommendations, as needed.

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

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

Conclusions and recommendations presented in this report are valid as of the present time for the development as currently planned. Changes in the condition of the property or adjacent properties may occur with the passage of time, whether by natural processes or the acts of other persons. In addition, changes in applicable or appropriate standards may occur through legislation or the broadening of knowledge. Therefore, the conclusions and recommendations presented in this report may be invalidated, wholly or in part, by changes beyond Cornerstone's control. This report should be reviewed by Cornerstone after a period of three (3) years has elapsed from the date of this report. In addition, if the current project design is changed, then Cornerstone must review the proposed changes and provide supplemental recommendations, as needed.

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

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

SECTION 11: REFERENCES

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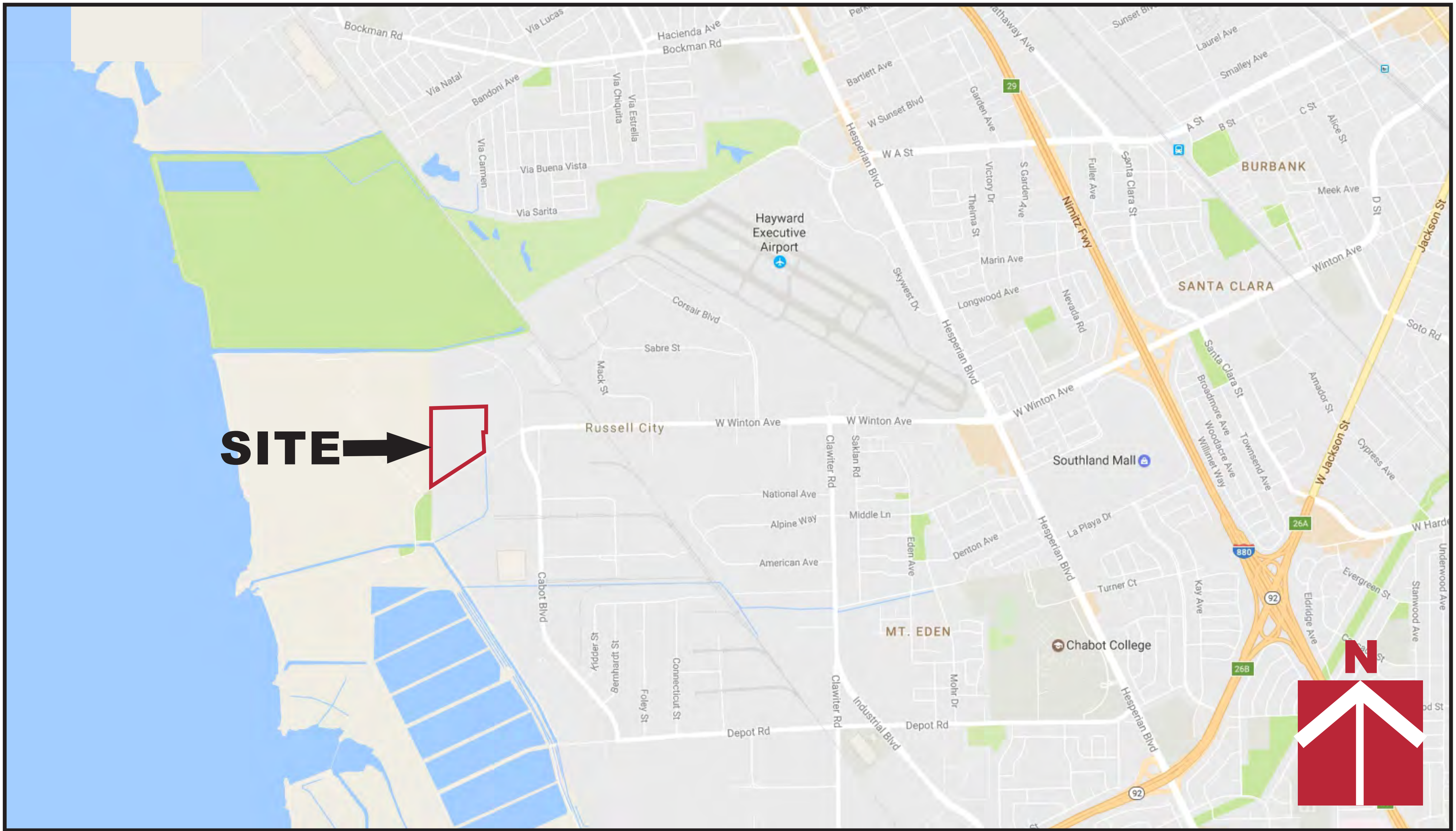

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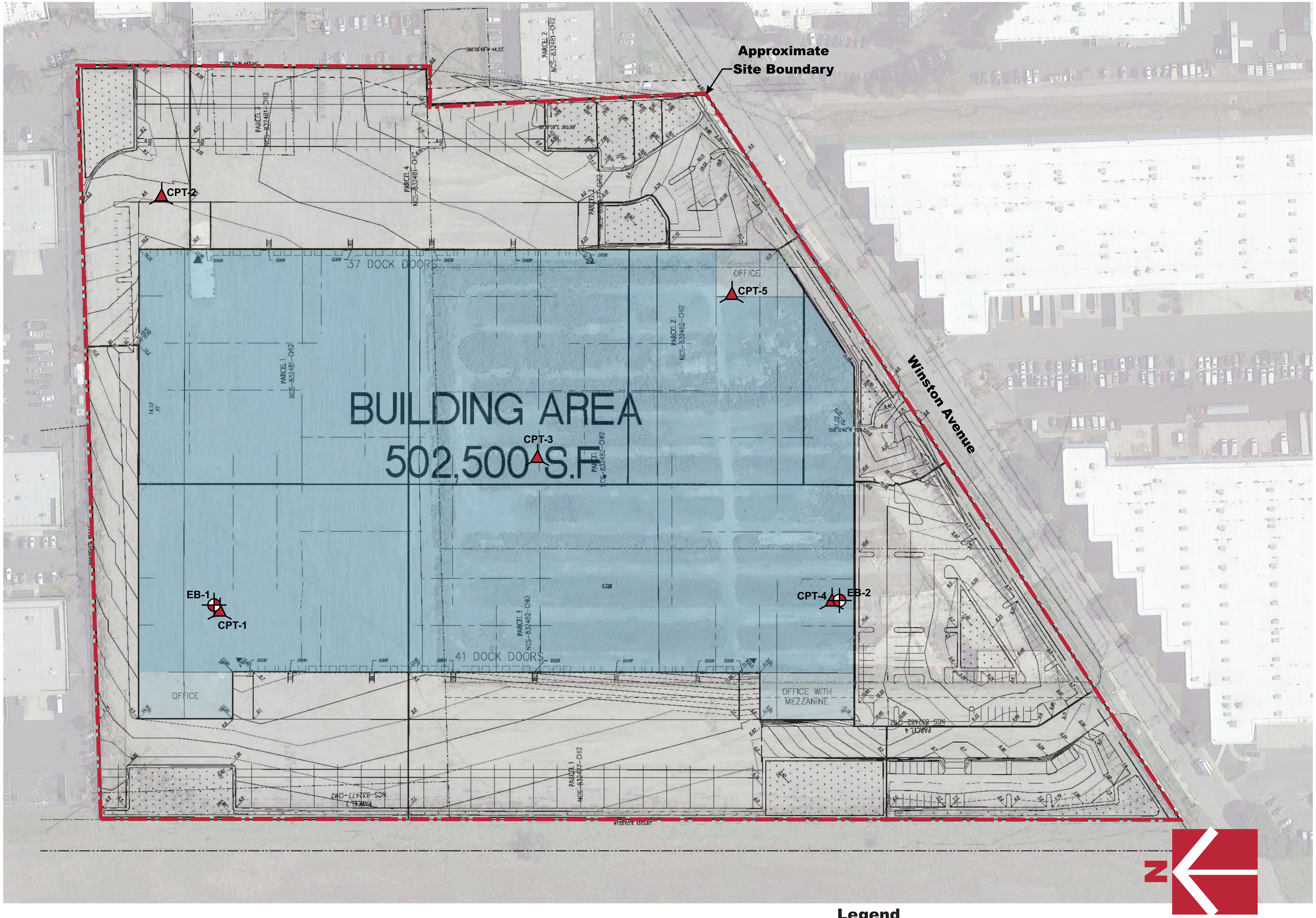
Vicinity Map

IPT Winton Avenue
Hayward, CA

Project Number	855-7-1
Figure Number	Figure 1
Date	February 2017
Drawn By	RRN



Legend

Approximate location of exploratory boring (EB)

Approximate location of cone penetration test (CPT)

0	120	240

APPROXIMATE SCALE (FEET)

Base by Google Earth, dated 11/2/2016
Overlay by Kier & Wright, Schematic Design - C2, dated 2/14/2017

Project Number 855-7-1

Figure Number Figure 2

Date February 2017

Drawn By RRN

Site Plan

IPT Winton Avenue
Hayward, CA

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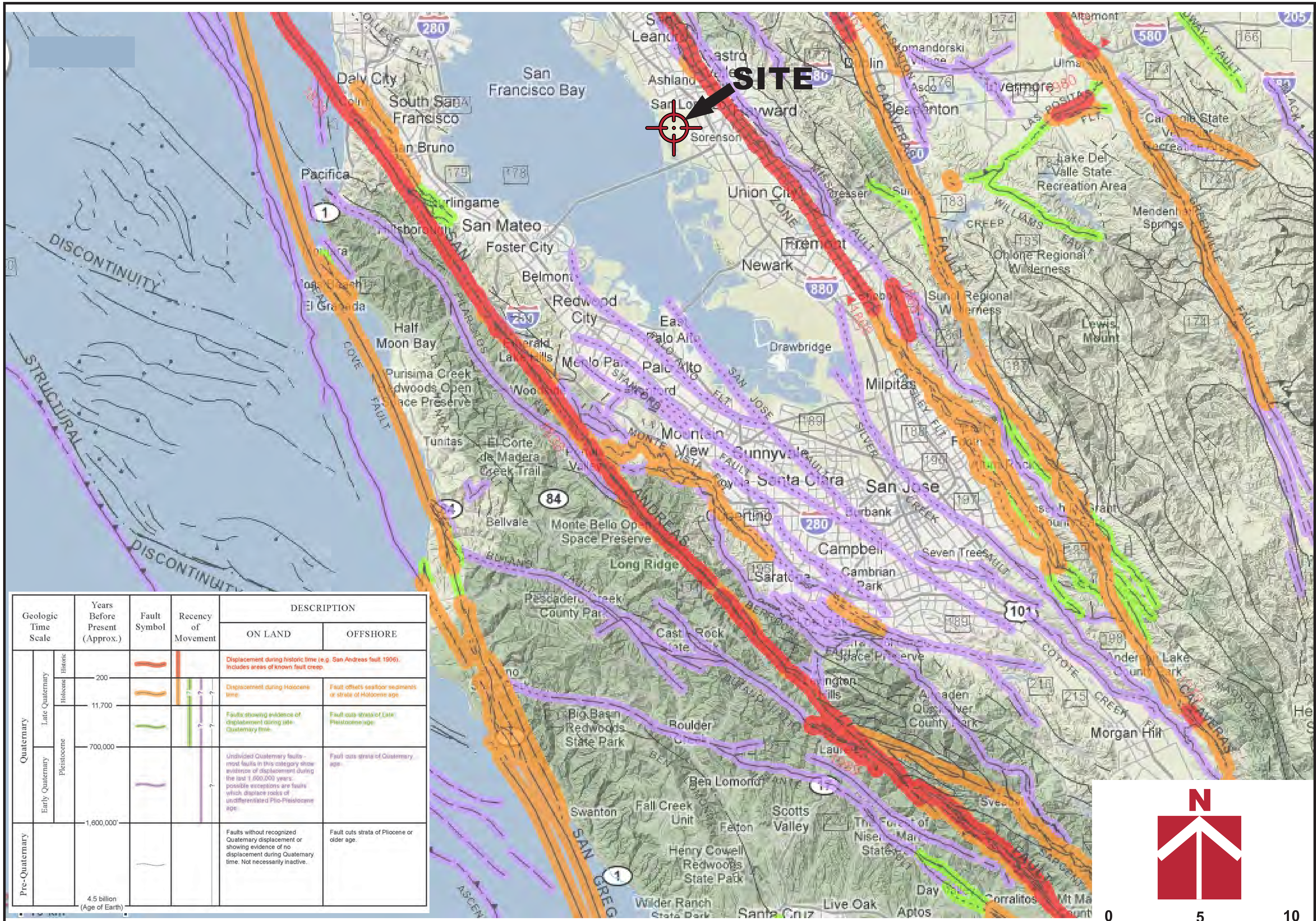

Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION ON LAND	DESCRIPTION OFFSHORE
Quaternary	Late Quaternary	Historic	200	🟥		Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	11,700	🟧	? ? ?	Displacement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

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

Regional Fault Map

IPT Winton Avenue
Hayward, CA

Project Number 855-7-1

Figure Number Figure 3

Date February 2017

Drawn By RRN

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FIGURE 4A

CPT NO. 1

PROJECT/CPT DATA

Project Title: **IPT - Winton Avenue**

Project No.: **855-7-1**

Project Manager: **NSD**

SEISMIC PARAMETERS

Controlling Fault: **Haward**

Earthquake Magnitude (Mw): **7.9**

PGA (Amax): **0.649** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet): **5**

Design Water Depth (feet): **5**

Ave. Unit Weight Above GW (pcf): **125**

Ave. Unit Weight Below GW (pcf): **120**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 5 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

1.15 **(Inches)**

TOTAL SEISMIC SETTLEMENT 1.1 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.76 **L/H** 160.0

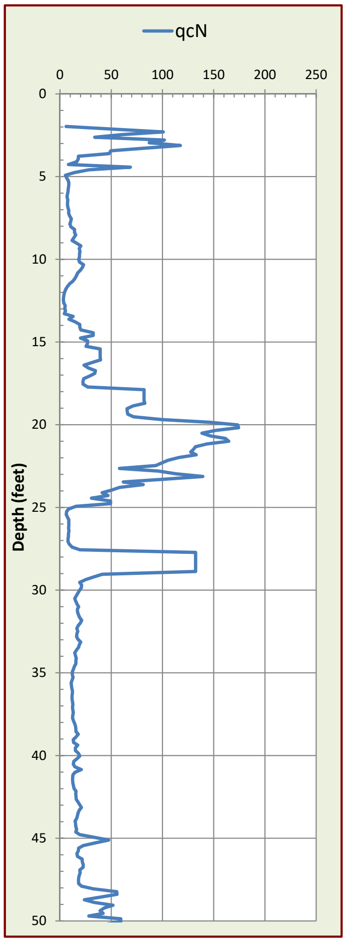
LDI[1] **Corrected for Distance** 0.08 (4 < L/H < 40)

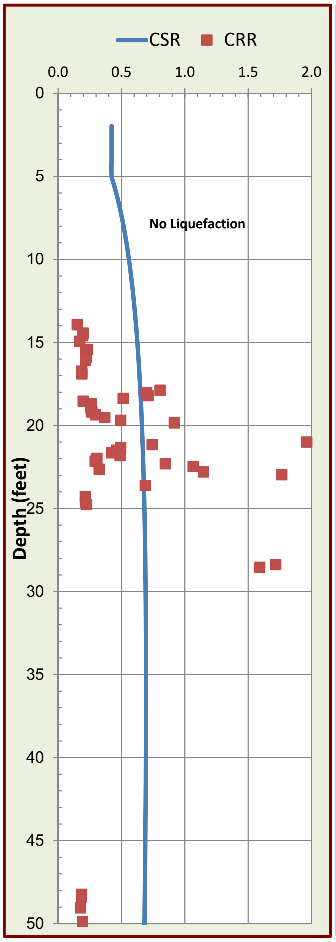
EXPECTED RANGE OF DISPLACEMENT

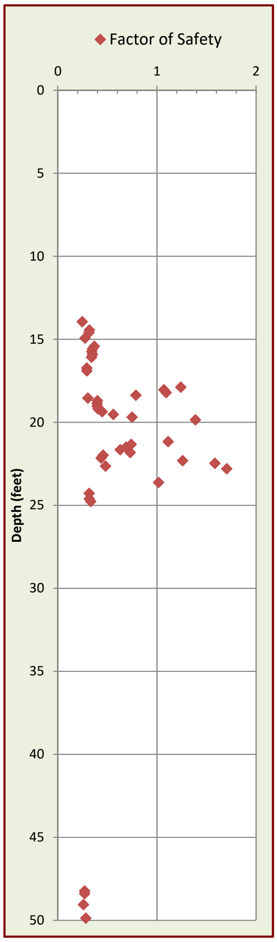
0.0 **to** **0.2** **feet**

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

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







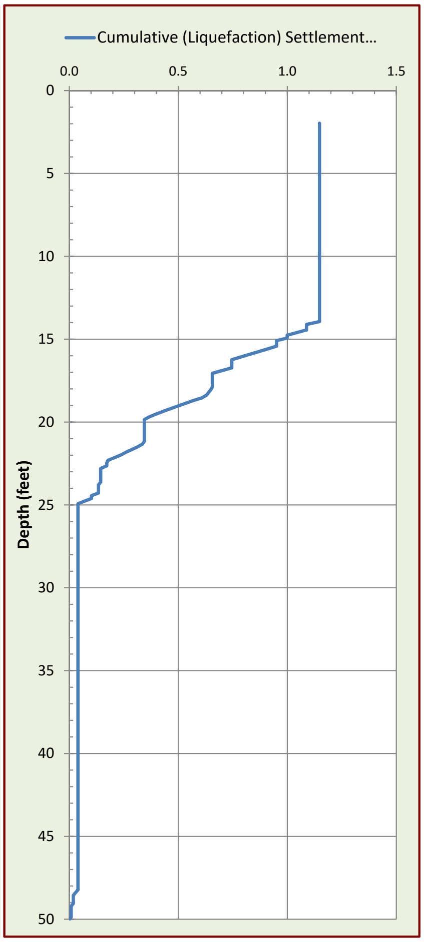


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FIGURE 4B

CPT NO. 2

PROJECT/CPT DATA

Project Title **IPT - Winton Avenue**

Project No. **855-7-1**

Project Manager **NSD**

SEISMIC PARAMETERS

Controlling Fault **Haward**

Earthquake Magnitude (Mw) **7.9**

PGA (Amax) **0.649** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet) **9.7**

Design Water Depth (feet) **5**

Ave. Unit Weight Above GW (pcf) **125**

Ave. Unit Weight Below GW (pcf) **120**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 5 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.14 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.1 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.06 **L/H** 160.0

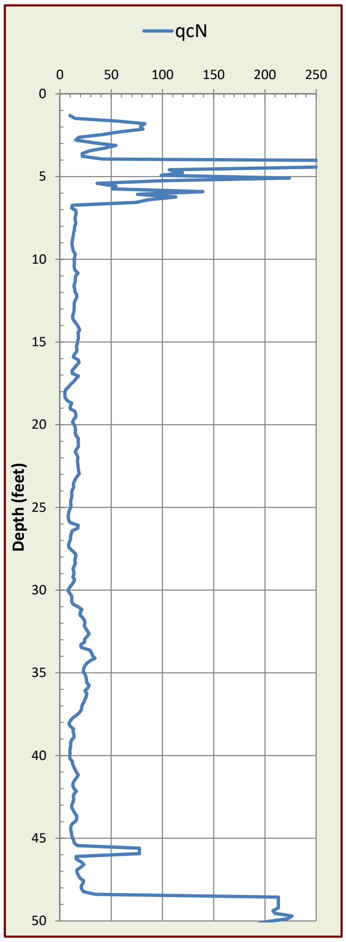
LDI[1]**Corrected for Distance** 0.01 (4 < L/H < 40)

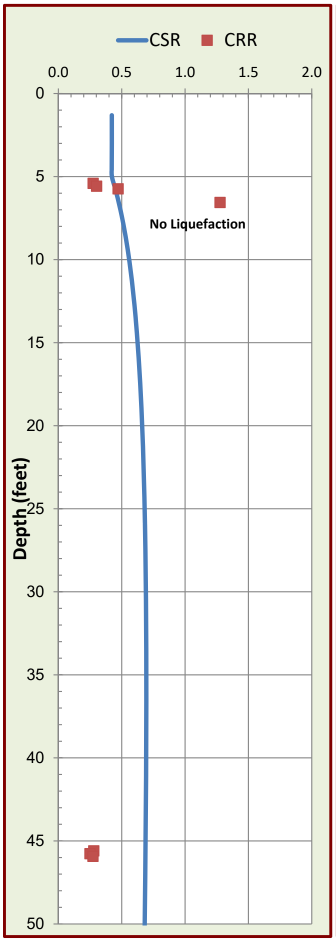
EXPECTED RANGE OF DISPLACEMENT

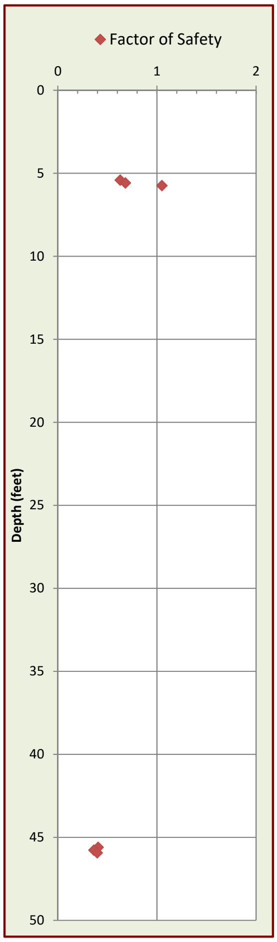
0.0 **to** **0.0** **feet**

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

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







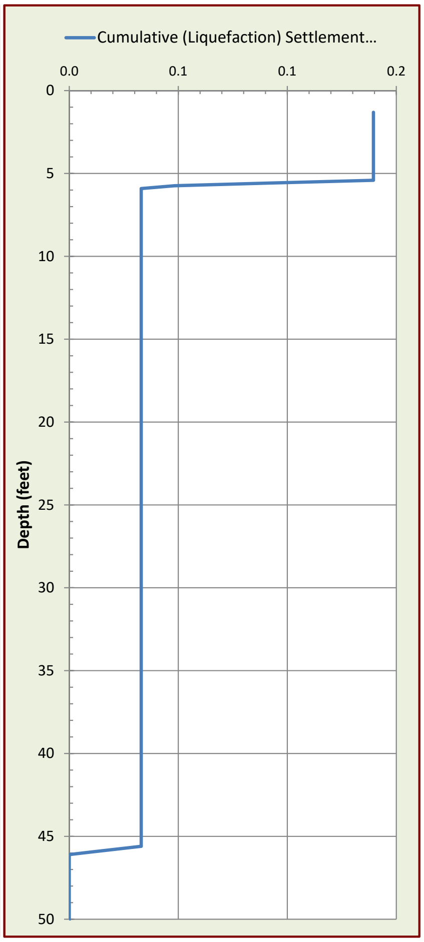


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FIGURE 4C

CPT NO. 3

PROJECT/CPT DATA

Project Title: **IPT - Winton Avenue**

Project No.: **855-7-1**

Project Manager: **NSD**

SEISMIC PARAMETERS

Controlling Fault: **Haward**

Earthquake Magnitude (Mw): **7.9**

PGA (Amax): **0.649** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet): **6**

Design Water Depth (feet): **5**

Ave. Unit Weight Above GW (pcf): **125**

Ave. Unit Weight Below GW (pcf): **120**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 5 **FEET**

0.07 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.17 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.2 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI^2 0.07 **L/H** 160.0

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

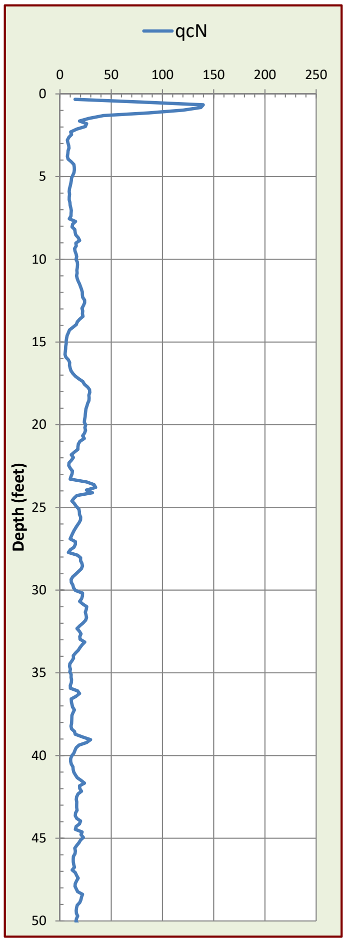
EXPECTED RANGE OF DISPLACEMENT

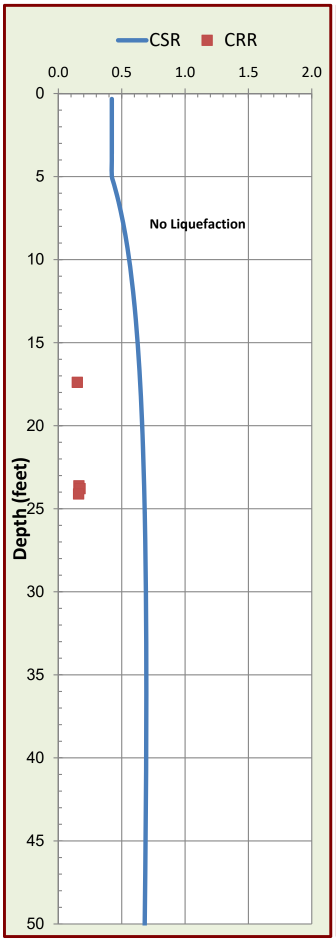
0.0 **to** **0.0** **feet**

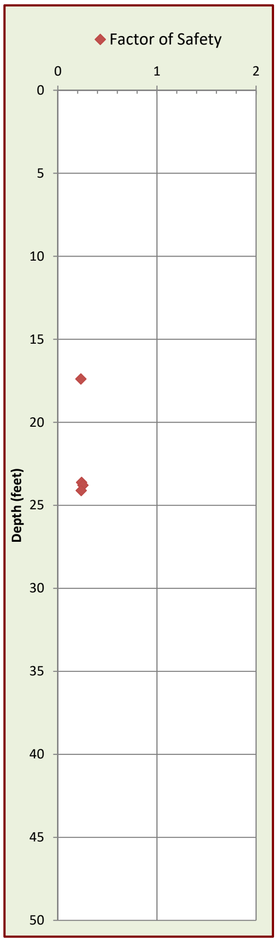
---

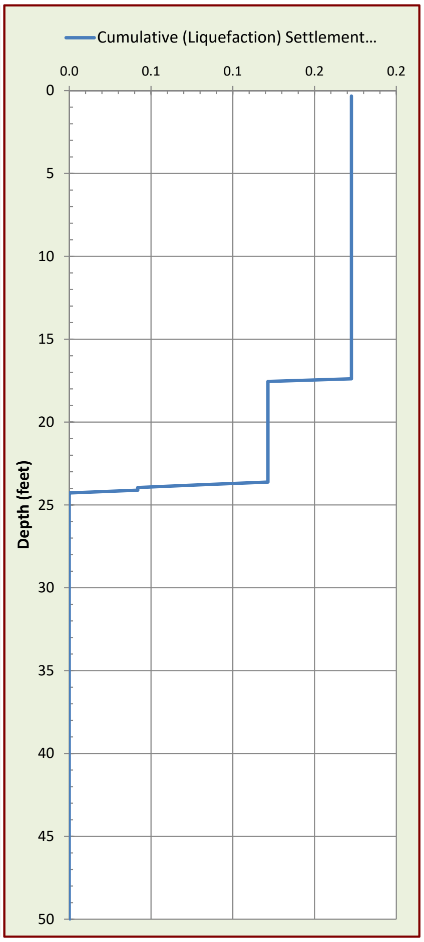
[1] Not Valid for L/H Values < 4 and > 40.

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









CORNERSTONE EARTH GROUP

FIGURE	**4D**
CPT NO.	**4**

PROJECT/CPT DATA

Project Title	**IPT - Winton Avenue**
Project No.	**855-7-1**
Project Manager	**NSD**

SEISMIC PARAMETERS

Controlling Fault	**Haward**	
Earthquake Magnitude (Mw)	**7.9**	
PGA (Amax)	**0.649**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**6**
Design Water Depth (feet)	**5**
Ave. Unit Weight Above GW (pcf)	**125**
Ave. Unit Weight Below GW (pcf)	**120**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 5 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.05 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.0 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI^2 0.05 **L/H** 160.0

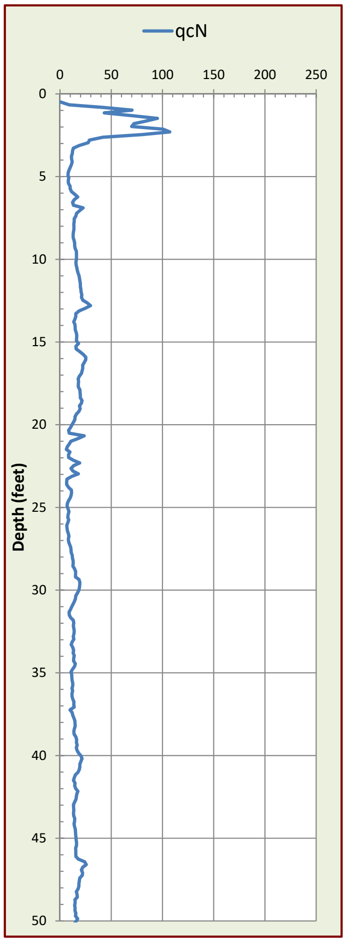
LDI^1**Corrected for Distance** 0.00 (4 < L/H < 40)

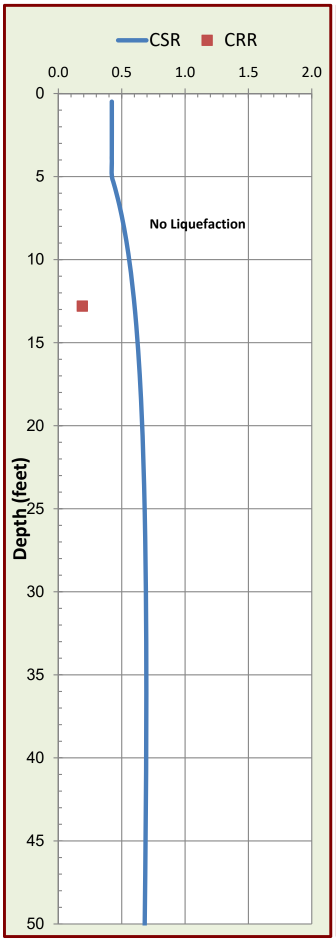
EXPECTED RANGE OF DISPLACEMENT

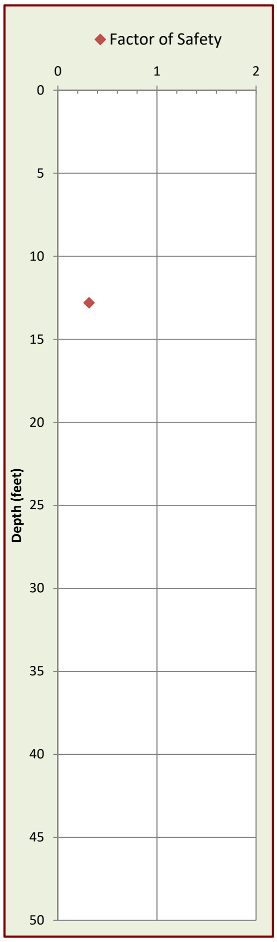
0.0 **to** **0.0** **feet**

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

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







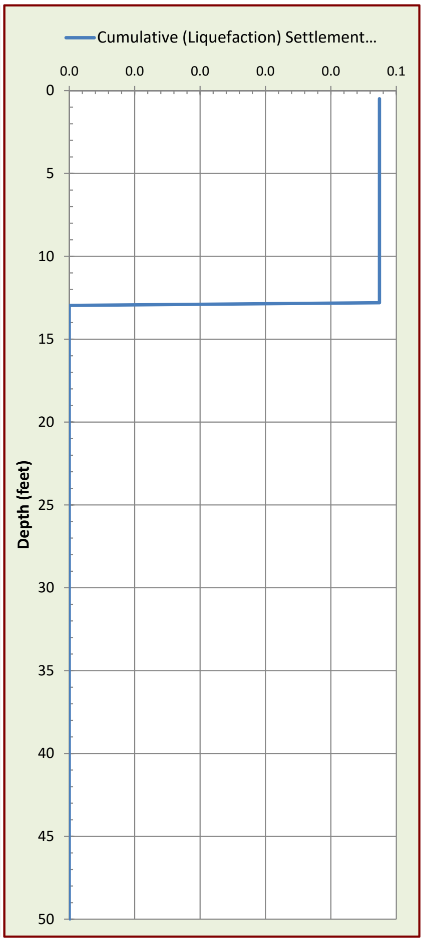


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FIGURE 4E

CPT NO. 5

PROJECT/CPT DATA

Project Title **IPT - Winton Avenue**

Project No. **855-7-1**

Project Manager **NSD**

SEISMIC PARAMETERS

Controlling Fault **Haward**

Earthquake Magnitude (Mw) **7.9**

PGA (Amax) **0.649** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet) **5**

Design Water Depth (feet) **5**

Ave. Unit Weight Above GW (pcf) **125**

Ave. Unit Weight Below GW (pcf) **120**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 5 **FEET**

0.02 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.08 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.1 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.05 **L/H** 160.0

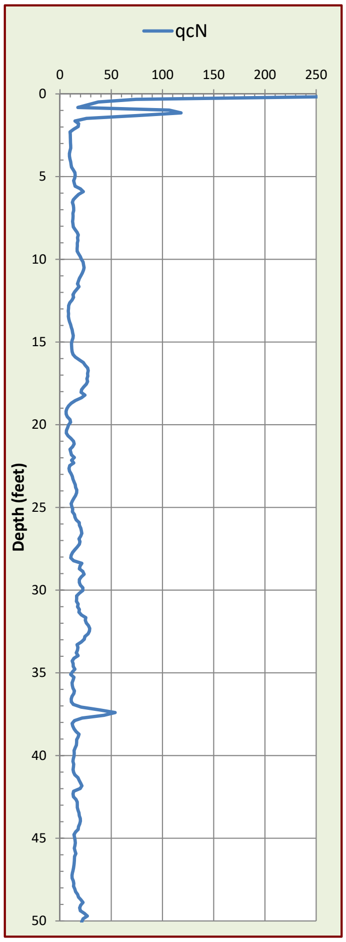
LDI[1]**Corrected for Distance** 0.01 (4 < L/H < 40)

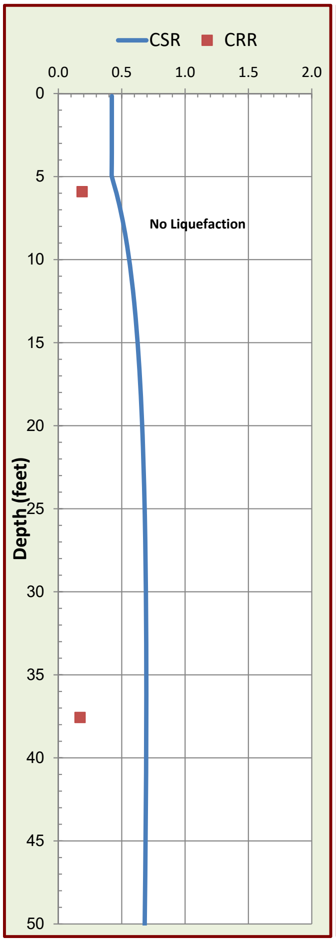
EXPECTED RANGE OF DISPLACEMENT

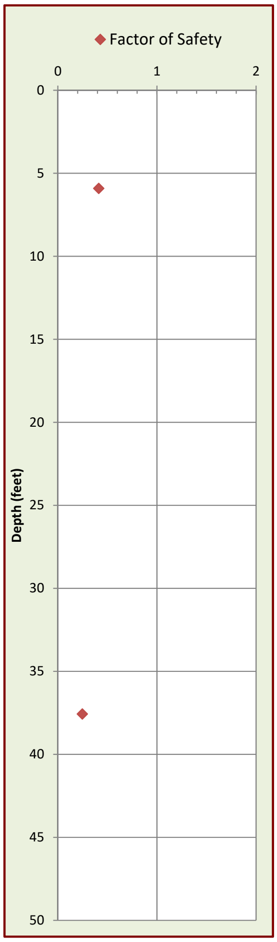
0.0 **to** **0.0** **feet**

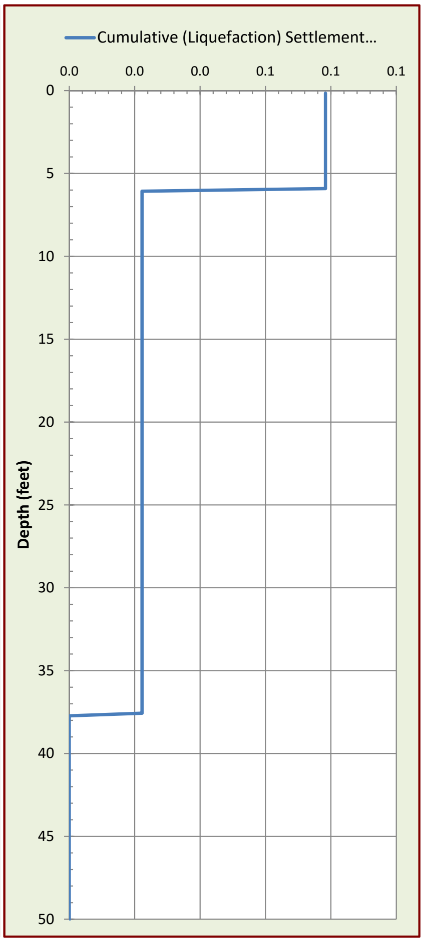
[1] Not Valid for L/H Values < 4 and > 40.

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









APPENDIX A: PRELIMINARY FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, hollow-stem auger drilling equipment and 20-ton truck-mounted Cone Penetration Test equipment. Two 8-inch-diameter exploratory borings were drilled on February 8, 2017, to depths of 30 to 50 feet. Five CPT soundings were also performed in accordance with ASTM D 5778-95 (revised, 2002) on February 3, 2017, to a depth of approximately 50½ to 100 feet. The approximate locations of exploratory borings and CPTs are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Boring logs, as well as a key to the classification of the soil, are included as part of this appendix. Boring and CPT locations were approximated using existing site boundaries and other site features as references. Boring and CPT elevations were not determined. The locations of the borings and CPTs should be considered accurate only to the degree implied by the method used.

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

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

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

Attached boring and CPT logs and related information depict subsurface conditions at the locations indicated and on the date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these boring and CPT locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition, any stratification lines on the logs represent the approximate boundary between soil types and the transition may be gradual.

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-10)

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

OTHER MATERIAL SYMBOLS

▨	Poorly-Graded Sand with Clay	░	Sand
▨	Clayey Sand	▥	Silt
▥	Sandy Silt	░	Well Graded Gravelly Sand
▩	Artificial/Undocumented Fill	▥	Gravelly Silt
░	Poorly-Graded Gravelly Sand	■	Asphalt
░	Topsoil	●	Boulders and Cobble
●	Well-Graded Gravel with Clay		
●	Well-Graded Gravel with Silt		

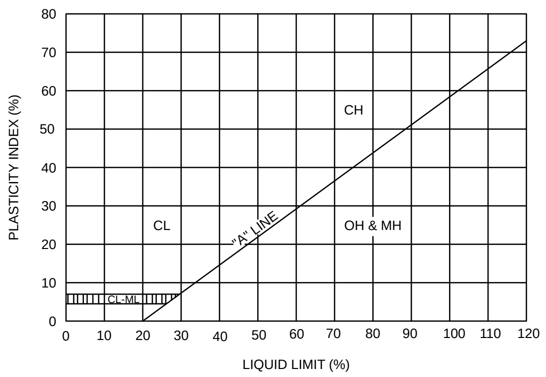
SAMPLER TYPES

☒	SPT	■	Shelby Tube
⧗	Modified California (2.5" I.D.)	○	No Recovery
▮	Rock Core	✋	Grab Sample

ADDITIONAL TESTS

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

PLASTICITY CHART



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

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

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

CORNERSTONE
EARTH GROUP

LEGEND TO SOIL DESCRIPTIONS

Figure Number
A-1

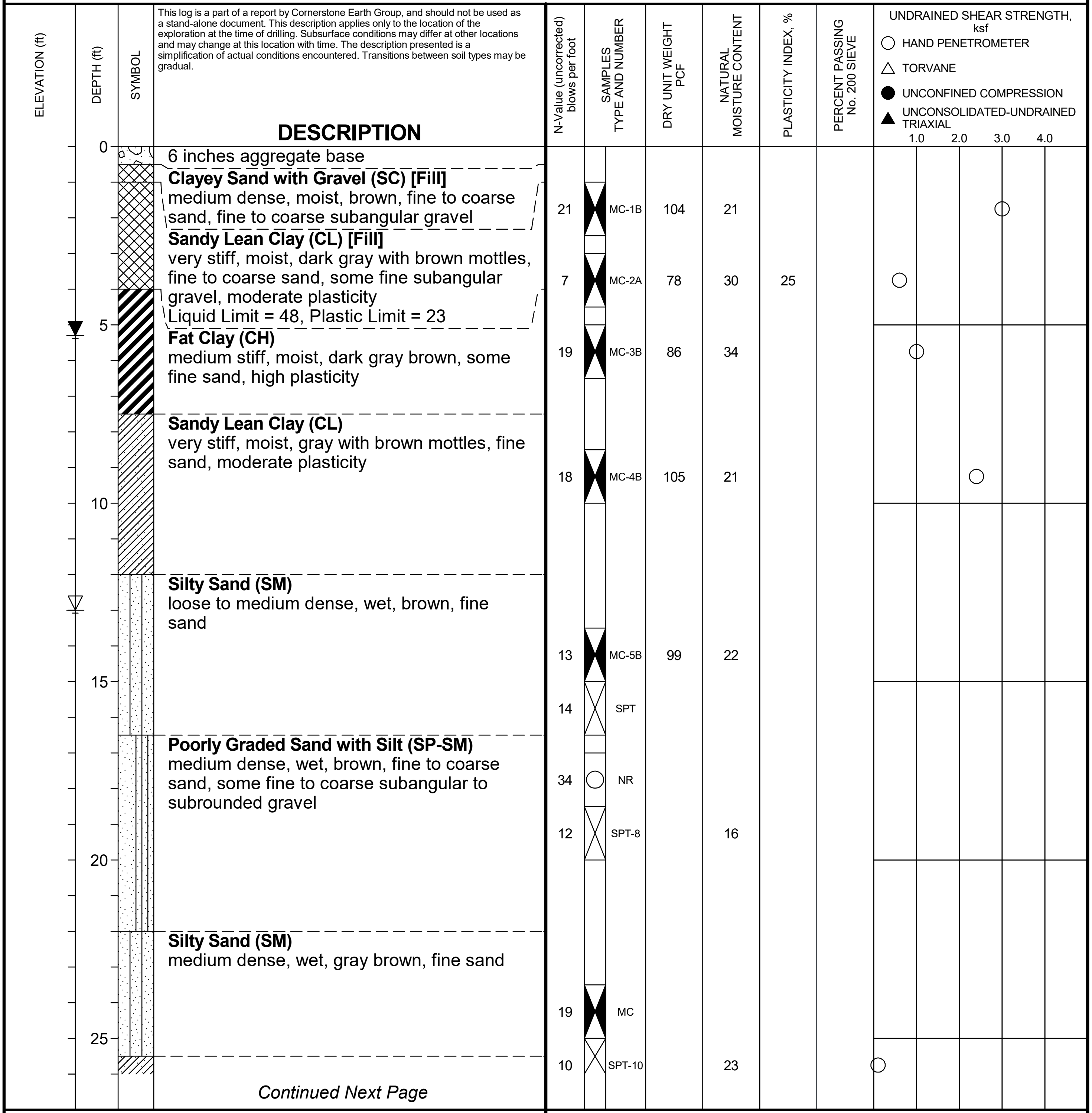
BORING NUMBER EB-1

CORNERSTONE EARTH GROUP

PROJECT NAME IPT Winton Ave
PROJECT NUMBER 855-7-1
PROJECT LOCATION Hayward, CA
GROUND ELEVATION
BORING DEPTH 50 ft.
LATITUDE 37.65360°
LONGITUDE -122.14239°
GROUND WATER LEVELS:
∇ **AT TIME OF DRILLING** 13 ft.
▼ **AT END OF DRILLING** 5.3 ft.

DATE STARTED 2/8/17
DATE COMPLETED 2/8/17
DRILLING CONTRACTOR Exploration Geoservices, Inc.
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
LOGGED BY DL
NOTES

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



ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
	0	6 inches aggregate base						
		Clayey Sand with Gravel (SC) [Fill] medium dense, moist, brown, fine to coarse sand, fine to coarse subangular gravel	21	MC-1B	104	21		
		Sandy Lean Clay (CL) [Fill] very stiff, moist, dark gray with brown mottles, fine to coarse sand, some fine subangular gravel, moderate plasticity Liquid Limit = 48, Plastic Limit = 23	7	MC-2A	78	30	25	
	5	**Fat Clay (CH)** medium stiff, moist, dark gray brown, some fine sand, high plasticity	19	MC-3B	86	34		
	10	**Sandy Lean Clay (CL)** very stiff, moist, gray with brown mottles, fine sand, moderate plasticity	18	MC-4B	105	21		
	15	**Silty Sand (SM)** loose to medium dense, wet, brown, fine sand	13 14	MC-5B SPT	99	22		
	20	**Poorly Graded Sand with Silt (SP-SM)** medium dense, wet, brown, fine to coarse sand, some fine to coarse subangular to subrounded gravel	34 12	NR SPT-8		 16		
	25	**Silty Sand (SM)** medium dense, wet, gray brown, fine sand	19	MC				
			10	SPT-10		23		

Continued Next Page

BORING NUMBER EB-1

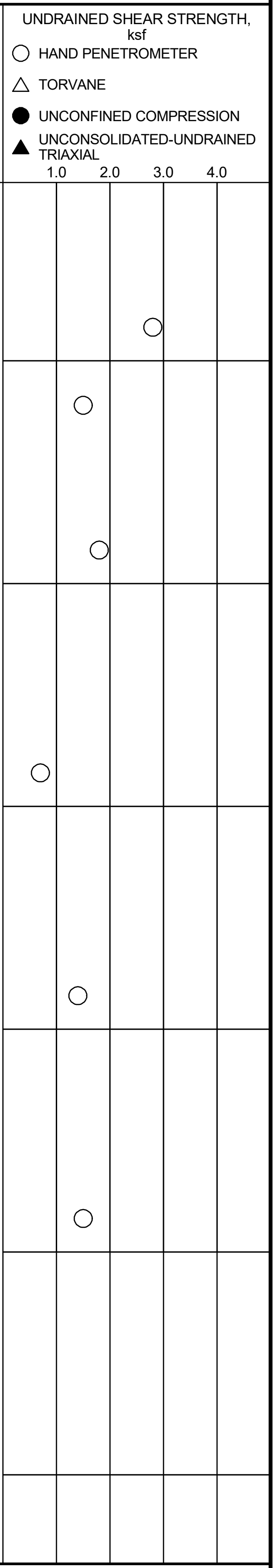
PAGE 2 OF 2

CORNERSTONE
EARTH GROUP

PROJECT NAME IPT Winton Ave
PROJECT NUMBER 855-7-1
PROJECT LOCATION Hayward, CA

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

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
			Sandy Lean Clay (CL) medium stiff, moist, gray brown, fine sand, low plasticity						
			Lean Clay with Sand (CL) very stiff to stiff, moist, gray with brown mottles, fine sand, moderate plasticity						
				26	MC-11B	107	20		
	30								
					NR				
				22	MC-13B	106	20		
	35								
			Lean Clay (CL) medium stiff, moist, bluish gray, some fine sand, moderate plasticity						
				22	MC-14B	100	24		
	40								
				9	SPT				
			Lean Clay with Sand (CL) stiff, moist, gray, fine sand, moderate plasticity						
				9	MC-16B	97	28		81
	45								
					ST				
			Sandy Lean Clay (CL) stiff, moist, gray brown, fine sand, low plasticity						
				32	NR				
	50		Bottom of Boring at 50.0 feet.						
	55								



CORNERSTONE EARTH GROUP

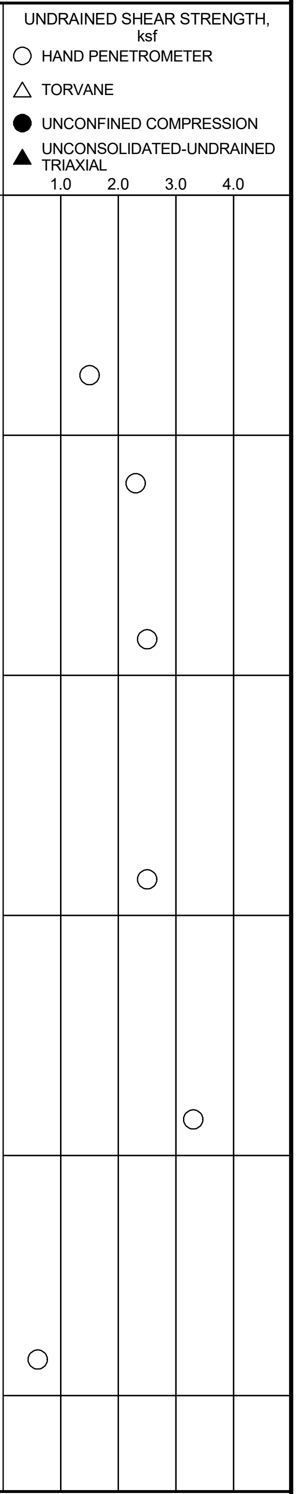
BORING NUMBER EB-2

PAGE 1 OF 2

PROJECT NAME IPT Winton Ave
PROJECT NUMBER 855-7-1
PROJECT LOCATION Hayward, CA
DATE STARTED 2/8/17 **DATE COMPLETED** 2/8/17
GROUND ELEVATION **BORING DEPTH** 30 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.651210° **LONGITUDE** -122.14309°
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY DL
▽ **AT TIME OF DRILLING** 13 ft.
NOTES
▼ **AT END OF DRILLING** 9 ft.

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

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
	0		**Clayey Sand with Gravel (SC) [Fill]** medium dense, moist, brown and dark brown mottled, fine to coarse sand, fine to coarse subangular gravel	23	MC-1B	111	17		
			Fat Clay (CH) stiff, moist, dark gray brown, some fine sand, high plasticity	10	MC-2B	89	30		
	5		**Lean Clay (CL)** very stiff, moist, gray, some fine sand, moderate plasticity		ST-3	100	23		
▼				15	MC-4B	105	22		
	10								
▽									
			some brown mottles	13	MC-5B	101	23		
	15								
				19	MC-6B	99	25		
	20				ST				
			Silty Sand (SM) medium dense, wet, brown, fine sand						
			Lean Clay with Sand (CL) medium stiff, moist, gray, fine sand, moderate plasticity	10	MC-8B	102	23		
	25								



Continued Next Page

BORING NUMBER EB-2

PROJECT NAME IPT Winton Ave

PROJECT NUMBER 855-7-1

PROJECT LOCATION Hayward, CA

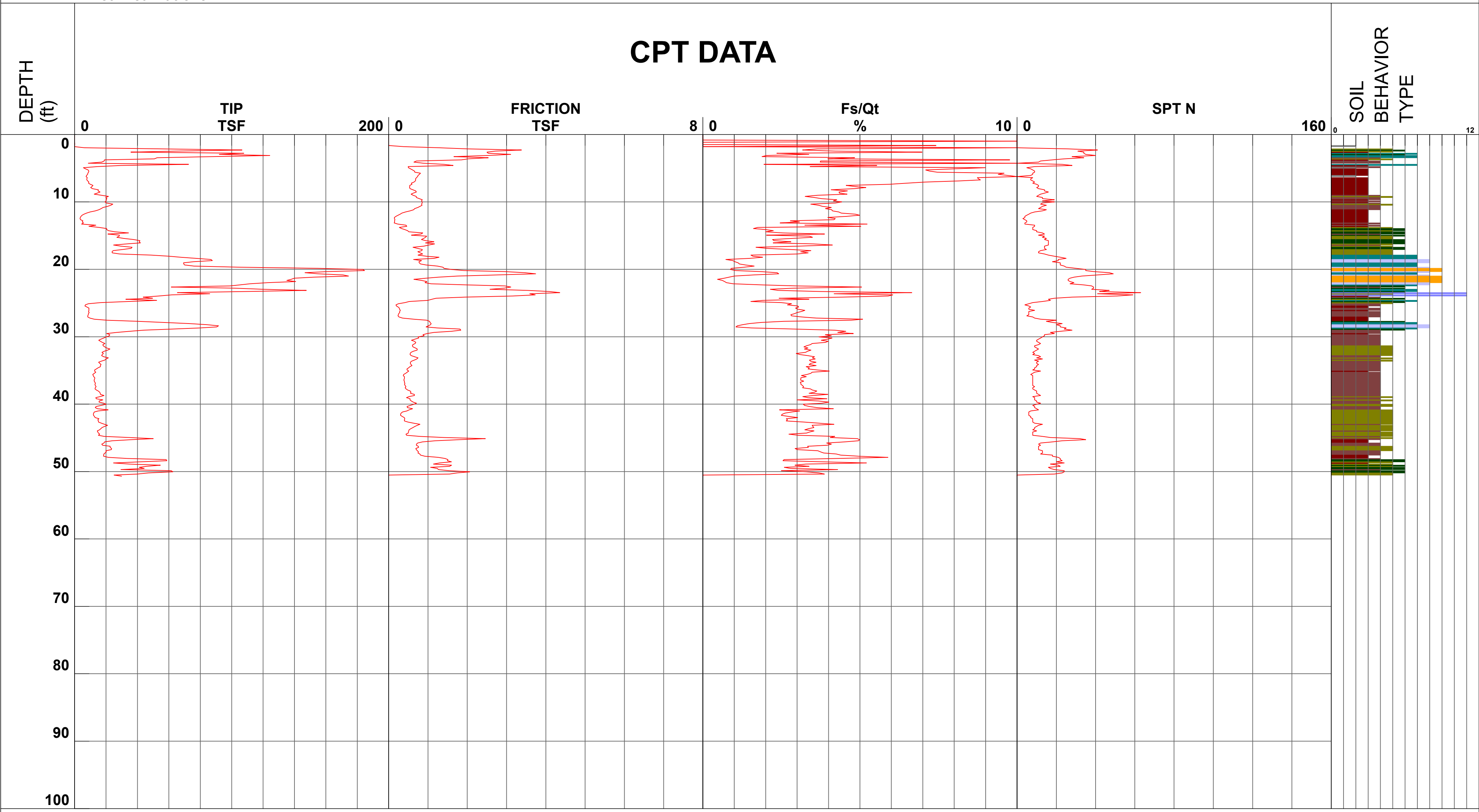
This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description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

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf ○ HAND PENETROMETER △ TORVANE ● UNCONFINED COMPRESSION ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL 1.0 2.0 3.0 4.0
		▨	**Lean Clay with Sand (CL)** stiff, moist, gray, fine sand, moderate plasticity	16	MC-9B	104	24			○
	30		Bottom of Boring at 30.0 feet.							
	35									
	40									
	45									
	50									
	55									

Cornerstone Earth Group

Project IPT Winton Avenue
Job Number 855-7-1
Hole Number CPT-01
EST GW Depth During Test 5.00 ft
Operator RB KK
Cone Number DDG1333
Date and Time 2/3/2017 2:44:57 PM
Filename SDF(076).cpt
GPS
Maximum Depth 50.69 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 10cm squared

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

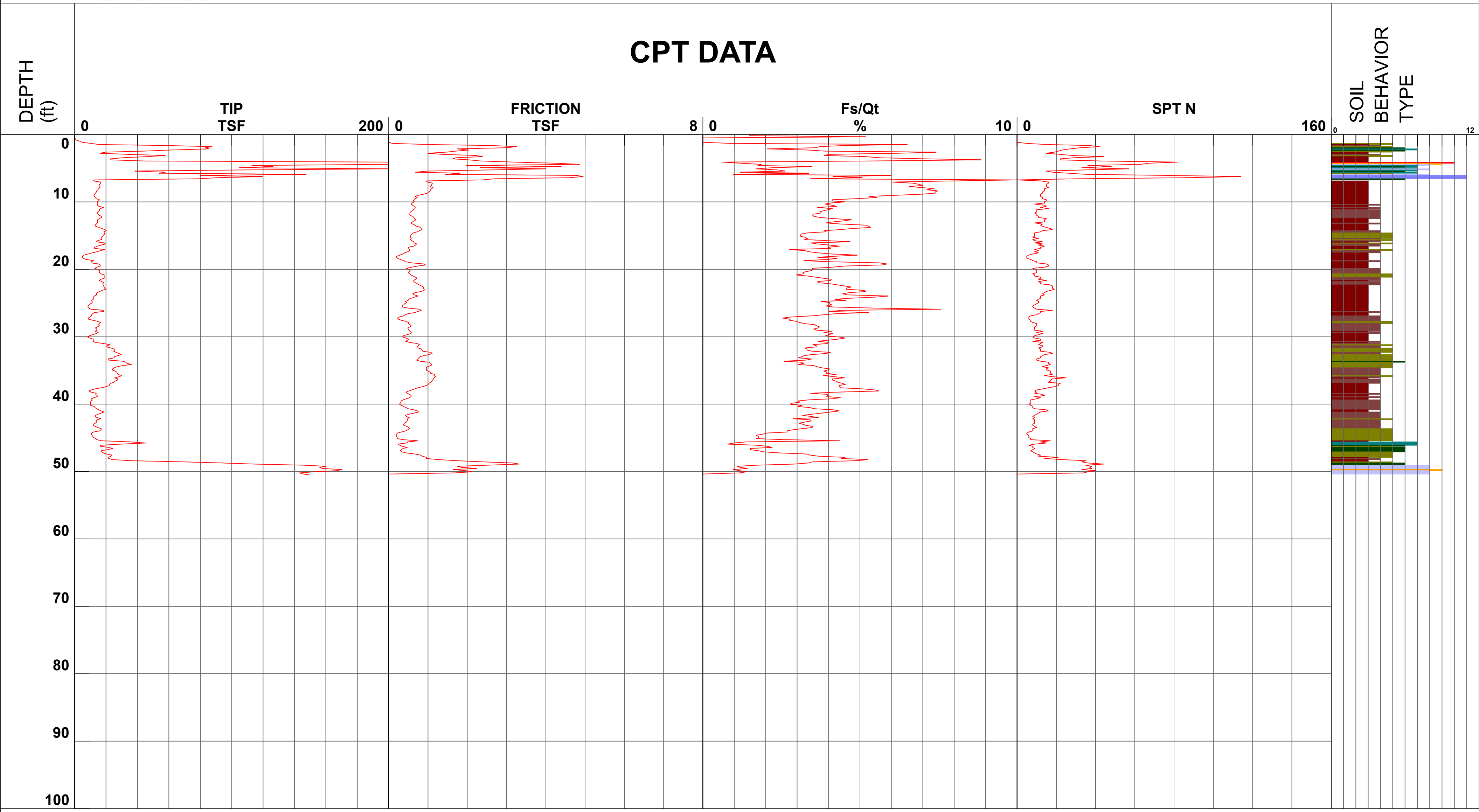
Cornerstone Earth Group

Middle Earth Geo Testing Inc.

Project	IPT Winton Avenue	**Operator**	RB KK	**Filename**	SDF(075).cpt
Job Number	855-7-1	**Cone Number**	DDG1333	**GPS**	
Hole Number	CPT-02	**Date and Time**	2/3/2017 1:54:43 PM	**Maximum Depth**	50.52 ft
EST GW Depth During Test		9.00 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

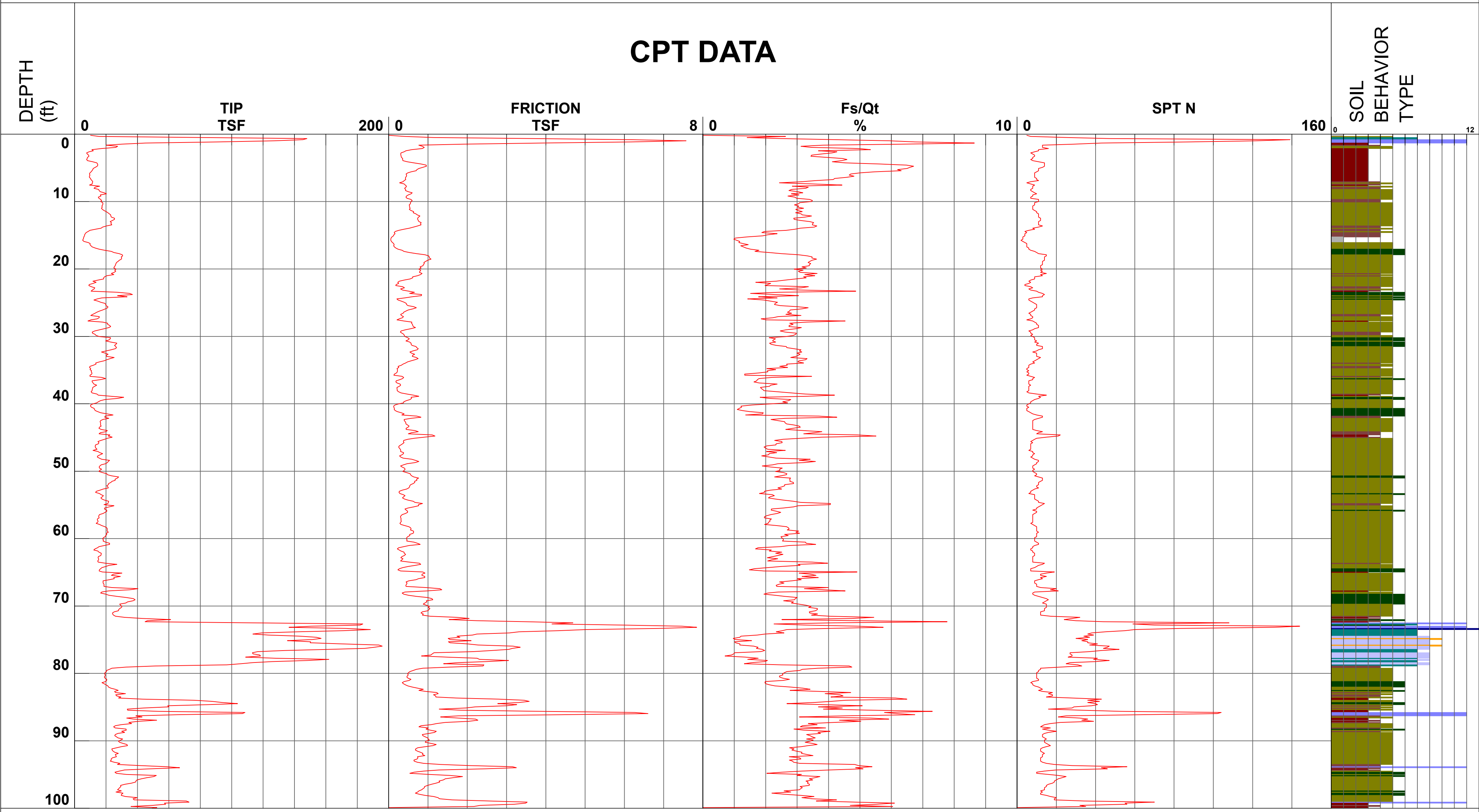
S*Soil behavior type and SPT based on data from UBC-1983

Middle Earth
Geo Testing Inc.

Cornerstone Earth Group

Project IPT Winton Avenue
Job Number 855-7-1
Hole Number CPT-03
EST GW Depth During Test 6.00 ft
Operator RB KK
Cone Number DDG1333
Date and Time 2/3/2017 10:28:08 AM
Filename SDF(072).cpt
GPS
Maximum Depth 100.06 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 10cm squared

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

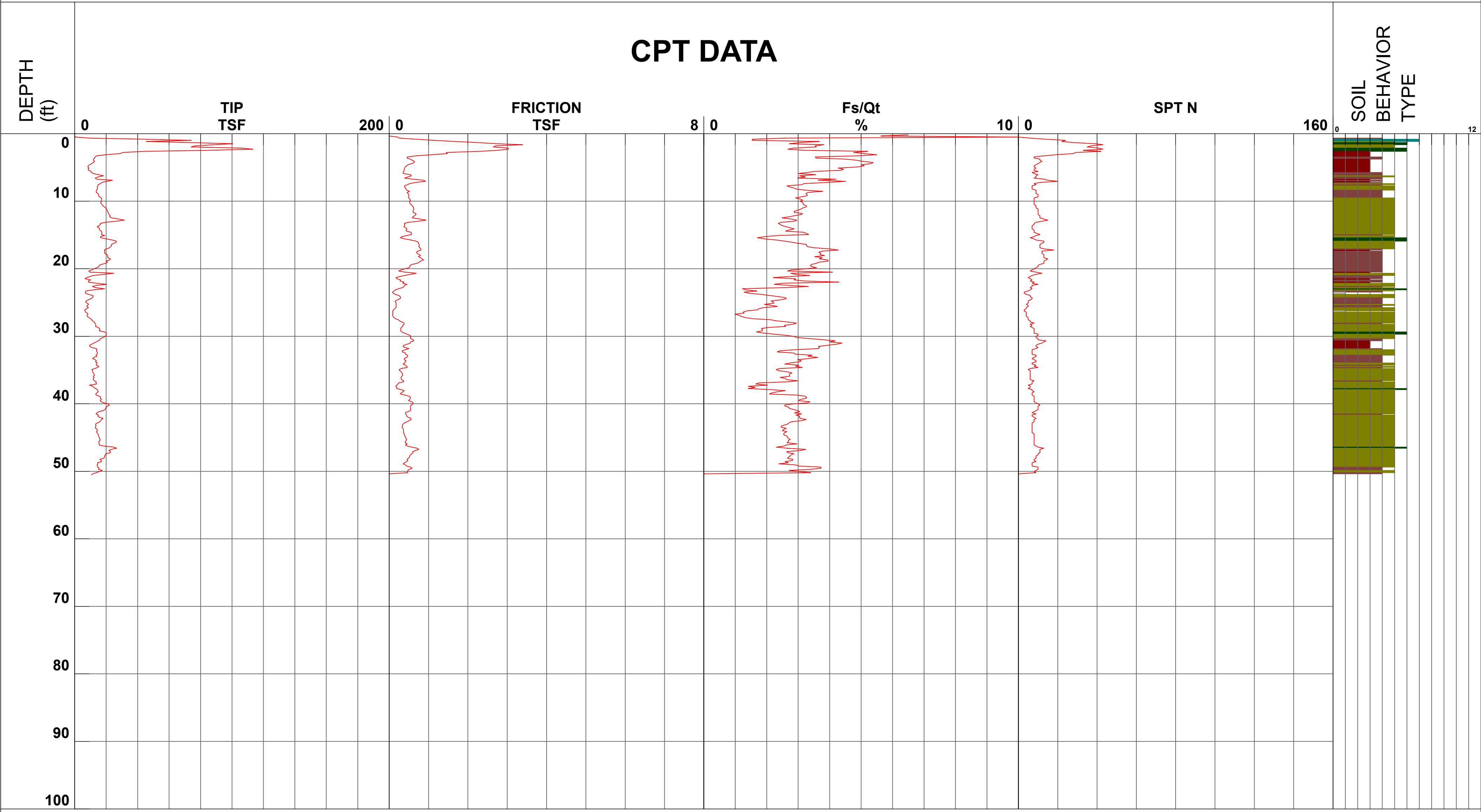
Middle Earth
Geo Testing Inc.

Cornerstone Earth Group

Project	IPT Winton Avenue	**Operator**	RB KK	**Filename**	SDF(069).cpt
Job Number	855-7-1	**Cone Number**	DDG1333	**GPS**	
Hole Number	CPT-04	**Date and Time**	2/3/2017 8:31:14 AM	**Maximum Depth**	50.52 ft
EST GW Depth During Test		5.00 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

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

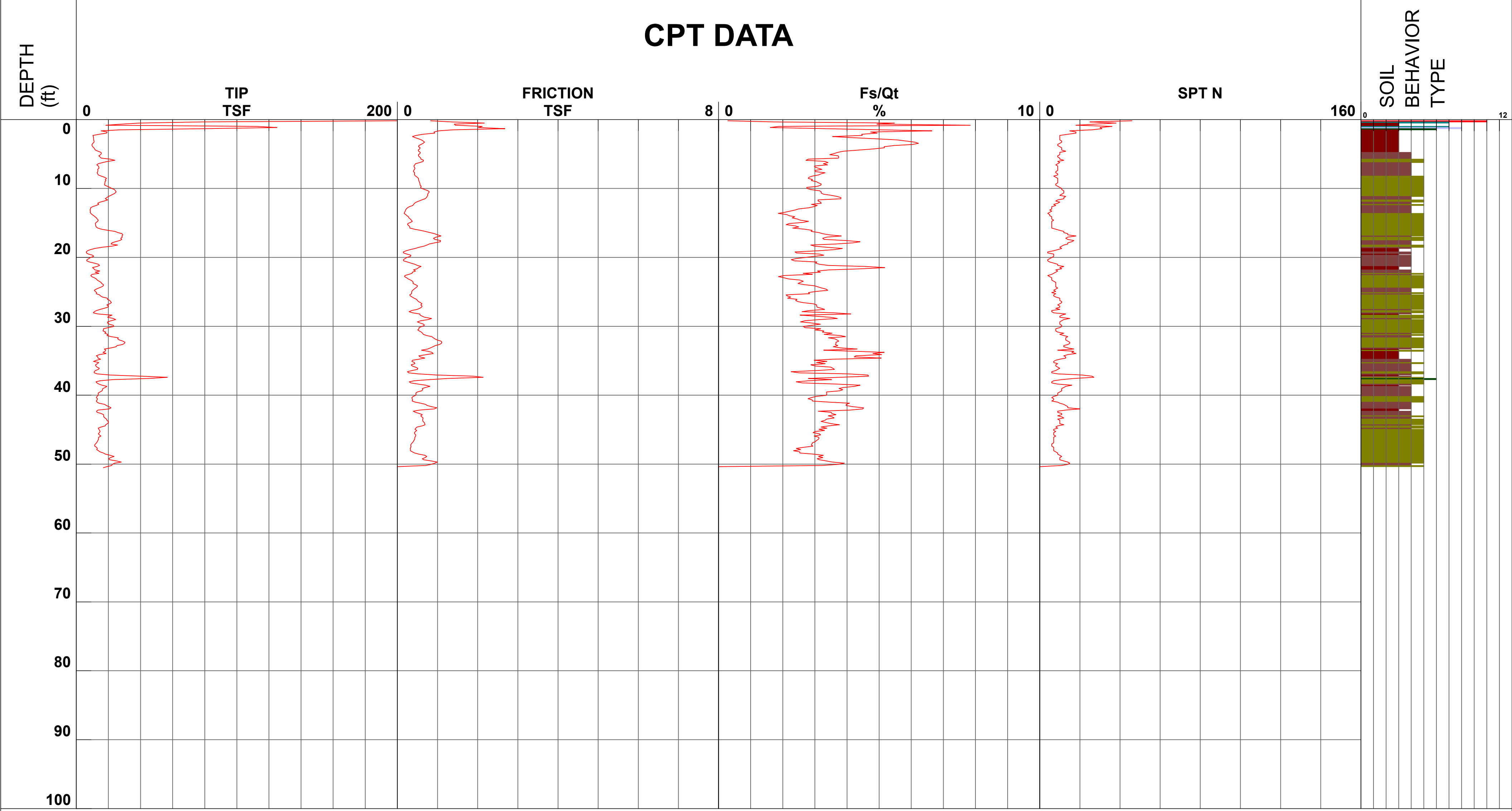
Middle Earth
Geo Testing Inc.

Cornerstone Earth Group

Project	IPT Winton Avenue	Operator	RB KK	Filename	SDF(073).cpt
Job Number	855-7-1	Cone Number	DDG1333	GPS	
Hole Number	CPT-05	Date and Time	2/3/2017 12:15:54 PM	Maximum Depth	50.52 ft
EST GW Depth During Test		5.00 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

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

APPENDIX B: PRELIMINARY LABORATORY TEST PROGRAM

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

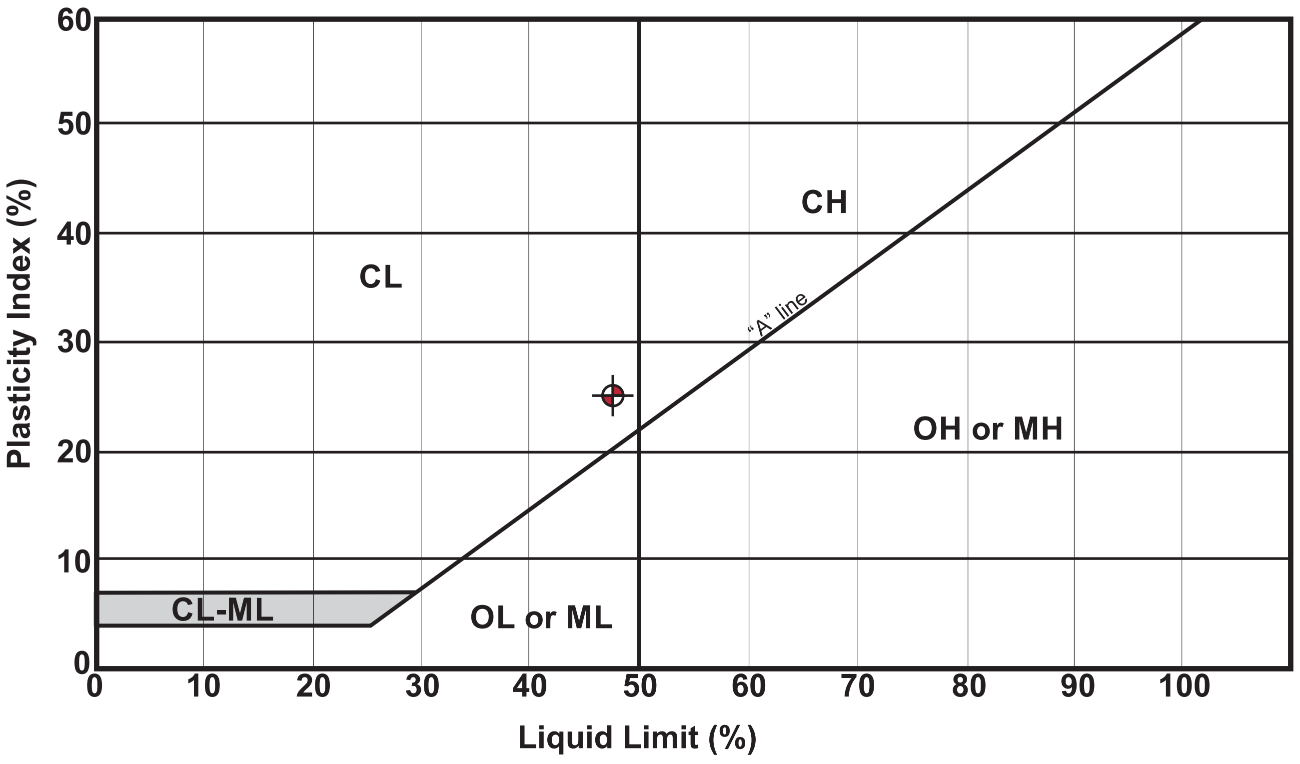
Moisture Content: The natural water content was determined (ASTM D2216) on 19 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry Densities: In place dry density determinations (ASTM D2937) were performed on 17 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Plasticity Index: One Plasticity Index determination (ASTM D4318) was performed on a sample of the subsurface soil to measure the range of water contents over which this material exhibits plasticity. The Plasticity Index was used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil expansion potential. Results of this test are shown on the boring log at the appropriate sample depth.

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

Plasticity Index (ASTM D4318) Testing Summary



Symbol	Boring No.	Depth (ft)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index	Passing No. 200 (%)	Group Name (USCS - ASTM D2487)
⊕	EB-1	3.5	30	48	23	25	—	Sandy Lean Clay (CL) [Fill]

CORNERSTONE
EARTH GROUP

Plasticity Index Testing Summary

IPT Winton Avenue
Hayward, CA

Project Number
855-7-1

Figure Number
Figure B1

Date
February 2017

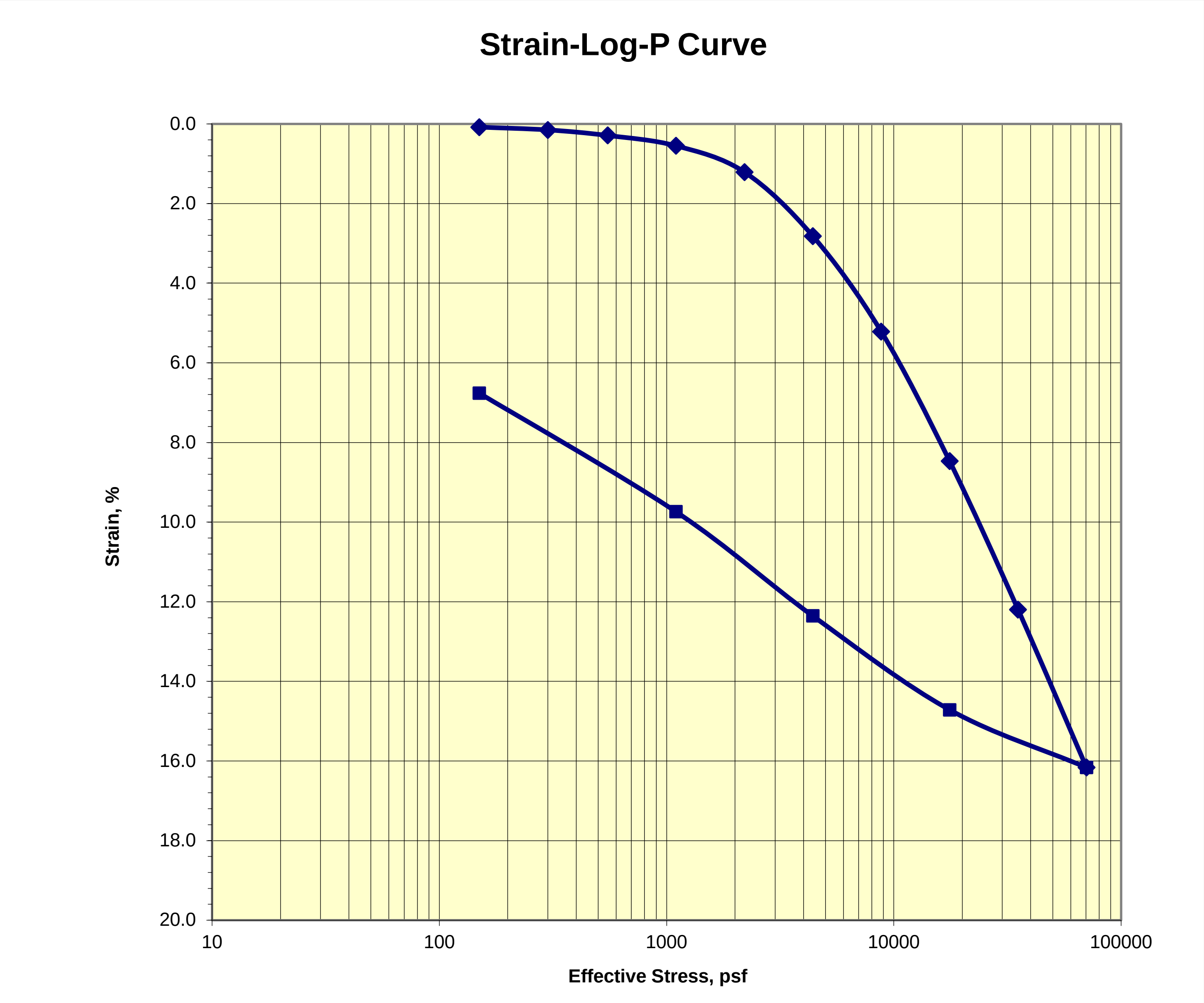
Drawn By
FLL

COOPER
TESTING LABORATORY

Consolidation Test

ASTM D2435

Job No.:	640-1089	**Boring:**	EB-2	**Run By:**	MD
Client:	Cornerstone Earth Group	**Sample:**	3	**Reduced:**	PJ
Project:	855-7-1	**Depth, ft.:**	5(Tip-4")	**Checked:**	PJ/DC
Soil Type:	Grayish Brown Sandy CLAY/ CLAY w/ Sand			**Date:**	2/23/2017



Assumed Gs 2.7	**Initial**	**Final**
Moisture %:	23.6	21.3
Dry Density, pcf:	101.2	106.9
Void Ratio:	0.666	0.576
% Saturation:	95.8	100.0

Remarks: