CORNERSTONE EARTH GROUP

TYPE OF SERVICES	Geotechnical Investigation
PROJECT NAME	LBA Logistics Center III
LOCATION	14800 W. Schulte Road Tracy, California
CLIENT	LBA Realty LLC
PROJECT NUMBER	750-2-1
DATE	February 19, 2020

CORNERSTONE
EARTH GROUP

Type of Services	**Geotechnical Investigation**
Project Name	**LBA Logistics Center III**
Location	**14800 W. Schulte Road** **Tracy, California**
Client	**LBA Realty LLC**
Client Address	**3347 Michelson Drive, Suite 200** **Irvine, California**
Project Number	**750-2-1**
Date	**February 19, 2020**

REGISTERED PROFESSIONAL ENGINEER
MAURA F. RUFFATTO
No. 84145
Exp. 09-30-21
CIVIL
STATE OF CALIFORNIA

Prepared by

Maura F. Ruffatto, P.E.
Project Engineer
Geotechnical Project Manager

Danh T. Tran, P.E.
Senior Principal Engineer
Quality Assurance Reviewer

1259 Oakmead Parkway | Sunnyvale, CA 94085
T 408 245 4600 | F 408 245 4620

1220 Oakland Boulevard, Suite 220 | Walnut Creek, CA 94596
T 925 988 9500 | F 925 988 9501

www.cornerstoneearth.com

CORNERSTONE EARTH GROUP

TABLE OF CONTENTS

Type of Services	**Geotechnical Investigation**
Project Name	**LBA Logistics Center III**
Location	**14800 West Schulte Road Tracy, California**

SECTION 1: INTRODUCTION

This geotechnical report was prepared for the sole use of LBA Realty LLC for the LBA Logistics Center III project in Tracy, California. The location of the site is shown on the Vicinity Map, Figure 1. For our use, we were provided with the following documents:

- A previous geotechnical report titled, "Geotechnical Investigation Report, Proposed Industrial Buildings, 14800 W. Schulte Road, Tracy, California," prepared by Technicon Engineering Services, Inc., dated August 10, 2018.

- A site plan titled "Schulte Road Development, 14800 W. Schulte Road Tracy, CA," Sheet A1-1P, prepared by RGA.

1.1 PROJECT DESCRIPTION

The currently planned project will consist of three warehouse/distribution center buildings on the approximately 38½-acre site. Buildings A, B, and C will range from about 166,000 to 280,000 square feet each. The buildings will be single-story with a maximum clear height of 45 feet. Office spaces will also be allocated for each building. Loading docks will be located along the interior of the long side of each building. At-grade trailer parking, access roads, and storm water management basins will occupy the remainder of the site. Appurtenant utilities, landscaping, and other improvements necessary for overall site development will also be constructed.

Structural loads are expected to be typical of this type of construction. Site grading with cuts and fills on the order of 10 to 20 feet are estimated based on observed site conditions.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposals dated November 6, 2019 and January 9, 2020, and consisted of a limited field program to evaluate engineering properties of the subsurface soils and reviewing field and laboratory programs performed by Technicon

Engineering Services, Inc. (Technicon, 2018) to evaluate physical and engineering properties of the subsurface soils, engineering analysis to prepare recommendations for site work and grading, building foundations, flatwork, retaining walls, and pavements, and preparation of this report. Brief descriptions of our exploration program is presented below.

1.3 EXPLORATION PROGRAM

Field exploration consisted of five Cone Penetration Tests (CPTs) advanced on January 30, 2020. The CPTs were advance to depths of about 50 to 100 feet. Seismic shear wave velocity measurements were collected from CPT-3.

The 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 explorations are shown on the Site Plan, Figure 2. Details regarding our field program are included in Appendix A.

1.4 EXPLORATION PROGRAM BY OTHERS

In addition to our explorations discussed above, we reviewed previous exploratory borings and CPT logs performed at the site by Technicon Engineering Services, Inc. (TES) during a previous geotechnical investigation. The previous field exploration by TES consisted of 11 borings drilled on June 19 and 20, 2018 with truck-mounted, hollow-stem auger drilling equipment and 5 percolation tests performed on June 19, 2018. The borings were drilled to depths of 11½ to 31½ feet; the percolation tests were performed at depths of 3 to 15 feet.

The approximate locations of previous explorations are shown on the Site Plan, Figure 2. Previous boring logs and percolation test results are provided in Appendix B.

1.5 LABORATORY TESTING PROGRAM BY OTHERS

Geotechnical laboratory test results from the previous investigation is included in Appendix C. Testing included moisture contents, dry densities, grain size analyses, Plasticity Index tests, Expansion Index tests, direct shear tests, a suite of corrosion tests, and R-value tests.

1.6 ENVIRONMENTAL SERVICES

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

SECTION 2: REGIONAL SETTING

2.1 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, geologists from the U.S. Geological

Survey have recently updated earlier estimates from their 2015 Uniform California Earthquake Rupture Forecast (Version 3) publication. The estimated probability of one or more magnitude 6.7 earthquakes (the size of the destructive 1994 Northridge earthquake) expected to occur somewhere in the San Francisco Bay Area has been revised (increased) to 72 percent for the period 2014 to 2043 (Aagaard et al., 2016). The faults in the region with the highest estimated probability of generating damaging earthquakes between 2014 and 2043 are the Hayward (33%), Rodgers Creek (33%), Calaveras (26%), and San Andreas Faults (22%). In this 30-year period, the probability of an earthquake of magnitude 6.7 or larger occurring is 22 percent along the San Andreas Fault and 33 percent for the Hayward or Rodgers Creek Faults.

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. Fault distances were determined using the program EZ Frisk (Risk Engineering, 2012) which uses the USGS 2008 fault model. It is noted that fault distances presented in Table 1 were determined from EZ Frisk and represent the rupture distance and may not be the distance to the surface expression of the fault that is shown on published geological maps and on-line resources such as Google Earth, etc. The seismic characteristics of some faults vary along its length so different segments of the same fault could be listed separately in the table.

Table 1: Approximate Fault Distances

Fault Name	Distance	
	(miles)	(kilometers)
Great Valley 7	4.4	7.1
Greenville	9.8	15.7

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 SURFACE DESCRIPTION

The site is currently occupied by various material stockpiles, drainage basins, and excavations. Site work has begun for the project and all previous structures have been demolished. Based on our observations, it appears subsurface improvements have been removed and areas of the site are significantly lower in elevation as a result of demolition. Two drainage basins are located along the northern edge of the property with depths of about 15 to 25 feet below adjacent grades. A temporary gravel road exists to the south of the basins and travels along the west and east property lines to the southern end of the side to form a rough loop around the property. The center of the site is roughly 10 to 20 feet lower in elevation than the adjacent gravel road and southern half of the site. Stockpiles of various sizes and heights (ranging from about 5 to 15 feet in height) are located within the lower central area. Two large stockpiles about 20 to 25 feet tall are located along the southern edge of the site. The stockpile in the southwest corner appears to be composed of soil and organic material and the stockpile in the southeast corner appears to be composed of aggregate base and/or rock salvaged during site demolition.

3.2 SUBSURFACE CONDITIONS

Below the ground surface, our CPTs generally encountered interbedded layers of medium dense sands and stiff to hard clays and silts to about 50 feet below the ground surface. Below a depth of 50 feet, our CPT-3 encountered primarily stiff to hard clays and silts to the maximum depth explored of 50 feet.

Based on the previous borings performed by Technicon, below the previous surface pavements or fill, where encountered, the site is generally underlain by low to moderately expansive clays with varying amounts of sand and silt with interbedded layers of silts with varying amounts of sand and sands and gravels with varying amounts of silt to the maximum boring depth of 31½ feet.

3.2.1 Plasticity/Expansion Potential

A Plasticity Index (PI) test performed by Technicon resulted in a PI of 25.5 and Expansion Index (EI) tests performed by Technicon resulted in EIs ranging from 82.8 to 83.5, indicating moderate expansion potential to wetting and drying cycles for the surficial soils.

3.2.2 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents within the upper 10 feet range from about 2 percent below to about 10 percent over the estimated laboratory optimum moisture.

3.3 GROUNDWATER

Groundwater was not encountered in previous explorations by Technicon. Groundwater was encountered in our CPT-3 at a depth of about 56 feet using a tape drop. The site is not currently mapped by CGS for historical high groundwater. Monitoring wells reported in the area by GeoTracker indicate seasonal high groundwater levels on the order of about 30 to 35 feet below existing grade should be expected. Based on the above information, we used a design groundwater depth of 25 feet for our analysis.

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

3.4 CORROSION SCREENING

Technicon (2018) tested three samples from RV-4 (bulk sample) at an unknown depth for soluble sulfates (Modified Caltrans 417), and chlorides (Modified Caltrans 417/422), and one sample from RV-4 (bulk sample) at an unknown depth for minimum resistivity and pH (Caltrans

California Test 643 – Method for Estimating the Service Life of Steel Culverts). The laboratory test results are summarized in Table 2.

Table 2A: Summary of Corrosion Test Results

Boring	Depth[5] (feet)	Soil pH[1]	Resistivity[1] (ohm-cm)	Chloride[2] (mg/kg)	Sulfate[3] (mg/kg)
RV-4	-	7.94	788	-	-
RV-4	-	-	-	16	57.1
RV-4	-	-	-	17.7	58.4
RV-4	-	-	-	17.7	58

Notes: [1]ASTM G51
[2] Cal 422 Modified
[3] Cal 417 Modified
[4]1 mg/kg = 0.0001 % by dry weight
[5]Depth not provided

Many factors can affect the corrosion potential of soil including moisture content, resistivity, permeability, and pH, as well as chloride and sulfate concentration. Typically, soil resistivity, which is a measurement of how easily electrical current flows through a medium (soil and/or water), is the most influential factor. In addition to soil resistivity, chloride and sulfate ion concentrations, and pH also contribute in affecting corrosion potential.

Based on the laboratory test results summarized in Table 2A and published correlations between resistivity and corrosion potential, the soils may be considered very severely corrosive to buried metallic improvements (Chaker and Palmer, 1989). We also recommend consideration be giving to running additional resistivity testing using ASTM G57 – 100% saturation methods to confirm the above data.

In accordance with the 2019 CBC Section 1904A.1, alternative cementitious materials for sulfate exposure shall be determined in accordance with ACI 318-14 Table 19.3.1.1, Table R19.3.1, and Table 19.3.2.1. Based on the laboratory sulfate test results, no cement type restriction is required, although, in our opinion, it is generally a good idea to include some sulfate resistance and to maintain a relatively low water-cement ratio. We have summarized applicable design values and parameters from ACI 318-14, Chapter 19 below in Table 2B. We recommend the structural engineer and a corrosion engineer be retained to confirm the information provided and for additional recommendations, as required.

Table 2B: ACI Sulfate Soil Corrosion Design Values and Parameters

Category	Water-Soluble Sulfate (SO4) in Soil (% by weight)	Sulfate (S) Class	Exposure Class	Cementitious Materials (2)
S, Sulfate	< 0.10	S0	F0	no type restriction

Notes: (1) above values and parameters are from on ACI 318-14, Table 19.3.1.1, Table R19.3.1, and Table 19.3.2.1
(2) cementitious materials are in accordance with ASTM C150, ASTM C595, and ASTM C1157

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 $(PGA)_M$ was estimated for analysis using a value equal to F_{PGA} x PGA, as allowed in the 2019 edition of the California Building Code. For our liquefaction analysis we used a PGA_M of 0.611g. We have assumed a site-specific analysis will not be required for this project; therefore, this is a code-based value of PGA_M.

4.3 LIQUEFACTION POTENTIAL

The site is not currently mapped by the State of California, and the liquefaction susceptibility maps by the Association of Bay Area Governments are currently unavailable online. However, we screened the site for liquefaction in the upper 50 feet during our site exploration by retrieving samples from the site, performing visual classification on sampled materials, evaluating CPT data, and performing various tests to further classify soil properties.

4.3.1 Background

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

4.3.2 Analysis

As discussed in the "Subsurface" section above, several sand layers were encountered below the design ground water depth of 25 feet. Following the liquefaction analysis framework in the 2008 monograph, *Soil Liquefaction During Earthquakes* (Idriss and Boulanger, 2008), incorporating updates in *CPT and SPT Based Liquefaction Triggering Procedures* (Boulanger and Idriss, 2014), and in accordance with CDMG Special Publication 117A guidelines (CDMG, 2008) for quantitative analysis, these layers were analyzed for liquefaction triggering and potential post-liquefaction settlement. These methods compare the ratio of the estimated cyclic shaking (Cyclic Stress Ratio - CSR) to the soil's estimated resistance to cyclic shaking (Cyclic Resistance Ratio - CRR), providing a factor of safety against liquefaction triggering. Factors of safety less than or equal to 1.3 are considered to be potentially liquefiable and capable of post-liquefaction re-consolidation (i.e. settlement).

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

The soil's CRR is estimated from the in-situ measurements from CPTs and laboratory testing on samples retrieved from our borings. SPT "N" values obtained from hollow-stem auger borings were not used in our analyses, as the "N" values obtained are less reliable in sands below 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.

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 post-liquefaction total settlement at the ground surface ranging up to about ⅓-inch based on the Yoshimine (2006) method. As discussed in SP 117A, differential movement for level ground sites over deep soil sites will be up to about two-thirds of the total settlement between independent foundation elements. In our opinion, differential settlements are anticipated to be on the order of ¼-inch, or less, between foundation elements, or about 50 to 60 feet.

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 25-foot thick layer of non-liquefiable cap is sufficient to prevent ground rupture; therefore the above total settlement estimates are reasonable.

4.4 LATERAL SPREADING

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

The potential for liquefaction is low and the potentially liquefiable layers are thin and well below the adjacent drainage basins bottoms; 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 soils encountered at the site were predominantly stiff to very stiff clays and medium dense to dense sands, in our opinion, the potential for significant differential seismic settlement affecting the proposed improvements is low.

4.6 FLOODING

Based on our internet search of the Federal Emergency Management Agency (FEMA) flood map public database, the site is located within Zone X, described as “Area of minimal flood hazard”. We recommend the project civil engineer be retained to confirm this information and verify the base flood elevation, if appropriate.

SECTION 5: CONCLUSIONS

5.1 SUMMARY

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

- Presence of moderately expansive soils
- Soil Corrosion Potential
- Variable on-site fill materials

5.1.1 Expansive Soils

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

5.1.2 Soil Corrosion Potential

As discussed, we performed a preliminary soil corrosion screening based on the results of analytical tests performed by Technicon (2018) on four samples at unknown depth. In general, the corrosion potential for buried concrete does not warrant the use of sulfate resistant concrete; however, the corrosion potential for buried metallic structures, such as metal pipes, is considered severely corrosive. As the preliminary soil corrosion screening was based on the results of limited sampling, consideration may be given to collecting and testing additional samples from the upper 5 feet for sulfates and pH to confirm the classification of corrosive to mortar coated steel and concrete.

5.1.3 Variable On-site Fill Materials

As previously discussed, stockpiles consisting of various materials are currently present at the site. We are in the process of completing our evaluation of organic contents for various material stockpiles onsite. Findings and recommendations for use as on-site fill will be provided in our forthcoming letter.

5.2 PLANS AND SPECIFICATIONS REVIEW

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

5.3 CONSTRUCTION OBSERVATION AND TESTING

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

SECTION 6: EARTHWORK

6.1 SITE DEMOLITION

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

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

6.1.1 Demolition of Existing Slabs, Foundations and Pavements

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

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

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

Existing foundations are typically mat-slabs, shallow footings, or piers/piles. If slab or shallow footings are encountered, they should be completely removed. If drilled piers are encountered, they should be cut off at an elevation at least 60-inches below proposed footings or the final subgrade elevation, whichever is deeper. The remainder of the drilled pier could remain in place. Foundation elements to remain in place should be surveyed and superimposed on the proposed development plans to determine the potential for conflicts or detrimental impacts to the planned construction. Following review, additional mitigation or planned foundation elements may need to be modified.

6.1.2 Abandonment of Existing Utilities

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

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

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

6.2 SITE CLEARING AND PREPARATION

6.2.1 Site Stripping

The site should be stripped of all surface vegetation, and surface and subsurface improvements to be removed within the proposed development area. Demolition of existing improvements is discussed in the prior paragraphs. A detailed discussion of removal of existing fills is provided later in this report. Surface vegetation and topsoil should be stripped to a sufficient depth to remove all material greater than 3 percent organic content by weight.

6.2.2 Tree and Shrub Removal

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

6.3 REMOVAL OF EXISTING FILLS

While Technicon did not indicate any fills were encountered in their borings aside for surface pavements and woodchips, there is high potential for undocumented fills to be present across the site. All fills should be completely removed from within building areas and to a lateral distance of at least 5 feet beyond the building footprint or to a lateral distance equal to fill depth below the perimeter footing, whichever is greater. Provided the fills meet the “Material for Fill” requirements below, the fills may be reused when backfilling the excavations. If materials are encountered that do not meet the requirements, such as debris, wood, trash, those materials should be screened out of the remaining material and be removed from the site. Backfill of excavations should be placed in lifts and compacted in accordance with the “Compaction” section below.

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

6.4 TEMPORARY CUT AND FILL SLOPES

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

Excavations performed during site demolition and fill removal should be sloped at 3:1 (horizontal:vertical) within the upper 5 feet below building subgrade. Excavations extending more than 5 feet below building subgrade and excavations in pavement and flatwork areas should be sloped in accordance with the OSHA soil classification.

6.5 SUBGRADE PREPARATION

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

6.6 SUBGRADE STABILIZATION MEASURES

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

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

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

6.6.1 Scarification and Drying

The subgrade may be scarified to a depth of 6 to 8 inches and allowed to dry to near optimum conditions, if sufficient dry weather is anticipated to allow sufficient drying. More than one round of scarification may be needed to break up the soil clods.

6.6.2 Removal and Replacement

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

6.6.3 Chemical Treatment

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

6.7 MATERIAL FOR FILL

6.7.1 Re-Use of On-site Soils

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

We are completing our evaluation of organic contents for various material stockpiles onsite. Findings and recommendations will be provided in our forthcoming letter.

6.7.2 Re-Use of On-Site Site Improvements

We understand that significant quantities of asphalt concrete (AC) grindings and aggregate base (AB) have been generated during previous site demolition. If the AC grindings are mixed with the underlying AB to meet Class 2 AB specifications, they may be reused within the new pavement and flatwork structural sections. AC/AB grindings may not be reused within the habitable building areas. Laboratory testing will be required to confirm the grindings meet project specifications.

6.7.3 Potential Import Sources

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

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

6.7.4 Non-Expansive Fill Using Lime Treatment

As discussed above, non-expansive fill should have a Plasticity Index (PI) of 15 or less. Due to the high clay content and PI of the on-site soil [and bedrock] materials, it is not likely that sufficient quantities of non-expansive fill would be generated from cut materials. As an alternative to importing non-expansive fill, chemical treatment can be considered to create non-expansive fill. If this option is considered, additional laboratory tests should be performed during initial site grading to further evaluate the optimum percentage of quicklime required.

6.8 COMPACTION REQUIREMENTS

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

Table 3: Compaction Requirements

Description	Material Description	Minimum Relative[1] Compaction (percent)	Moisture[2] Content (percent)
General Fill (within upper 5 feet)	On-Site Expansive Soils	87 – 92	>3
	Low Expansion Soils	90	>1
General Fill (below a depth of 5 feet)	On-Site Expansive Soils	95	>3
	Low Expansion Soils	95	>1
Trench Backfill	On-Site Expansive Soils	87 – 92	>3
Trench Backfill	Low Expansion Soils	90	>1
Trench Backfill (upper 6 inches of subgrade)	On-Site Low Expansion Soils	95	>1
Crushed Rock Fill	¾-inch Clean Crushed Rock	Consolidate In-Place	NA
Non-Expansive Fill	Imported Non-Expansive Fill	90	Optimum
Flatwork Subgrade	On-Site Expansive Soils	87 - 92	>3
Flatwork Subgrade	Low Expansion Soils	90	>1
Flatwork Aggregate Base	Class 2 Aggregate Base[3]	90	Optimum
Pavement Subgrade	On-Site Expansive Soils	87 - 92	>3
Pavement Subgrade	Low Expansion Soils	95	>1
Pavement Aggregate Base	Class 2 Aggregate Base[3]	95	Optimum
Asphalt Concrete	Asphalt Concrete	95 (Marshall)	NA

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

6.8.1 Construction Moisture Conditioning

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

6.9 TRENCH BACKFILL

Utility lines constructed within public right-of-way should be trenched, bedded and shaded, and backfilled in accordance with the local or governing jurisdictional requirements. Utility lines in private improvement areas should be constructed in accordance with the following requirements unless superseded by other governing requirements.

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

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

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

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

6.10 SITE DRAINAGE

6.10.1 Surface Drainage

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

6.11 LOW-IMPACT DEVELOPMENT (LID) IMPROVEMENTS

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

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

- The near-surface soils at the site are clayey, and categorized as Hydrologic Soil Group D, and is expected to have infiltration rates of less than 0.2 inches per hour. In our opinion, these clayey soils will significantly limit the infiltration of stormwater.
- Locally, seasonal high ground water is not mapped in the area, but is expected to be about 25 to 25 feet below grade, and therefore is not expected to be within 10 feet below the base of the infiltration measure.
- The site is not known, to our knowledge, to have pollutants with the potential for mobilization as a result of stormwater infiltration.
- In our opinion, infiltration locations within 10 feet of the buildings would create a geotechnical hazard.
- Infiltration devices should be located at least 100 feet away from septic tanks and underground storage tanks with hazardous materials, as well as any other potential underground sources of pollution.
- Locations where reduction of stormwater runoff may potentially impair beneficial uses of the receiving water, such as change of seasonality of ephemeral washes, as documented in a site-specific study (e.g., California Environmental Quality Act (CEQA) analysis) or watershed plan;
- Infiltration measures, devices, or facilities may conflict with the location of existing or proposed underground utilities or easements. Infiltration measures, devices, or facilities should not be placed on top of or very near to underground utilities such that they discharge to the utility trench, restrict access, or cause stability concerns.

- Local Water District policies or guidelines may limit locations where infiltration may occur, require greater separation from seasonal high groundwater, or require greater setbacks from potential sources of pollution.

6.11.1 Storm Water Treatment Design Considerations

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

6.11.1.1 GENERAL BIOSWALE DESIGN GUIDELINES

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

- Bioswales constructed within 3 feet of proposed buildings may be within the foundation zone of influence for perimeter wall loads. Therefore, where bioswales will parallel foundations and will extend below the “foundation plane of influence,” an imaginary 1:1 plane projected down from the bottom edge of the foundation, the foundation will need to be deepened so that the bottom edge of the bioswale filter material is above the foundation plane of influence.

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

6.11.1.2 BIOSWALE INFILTRATION MATERIAL

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

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

- If required, infiltration (percolation) testing should be performed on representative samples of potential bioswale materials prior to construction to check for general conformance with the specified infiltration rates.

- It should be noted that multiple laboratory tests may be required to evaluate the properties of the bioswale materials, including percolation, landscape suitability and possibly environmental analytical testing depending on the source of the material. We recommend that the landscape architect provide input on the required landscape suitability tests if bioswales are to be planted.

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

- If required by governing agencies, field infiltration testing should be specified on the grading and improvement plans. The appropriate infiltration test method, duration and frequency of testing should be specified in accordance with local requirements.

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

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

6.11.1.3 BIOSWALE CONSTRUCTION ADJACENT TO PAVEMENTS

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

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

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

6.12 LANDSCAPE CONSIDERATIONS

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

- Using drip irrigation
- Avoiding open planting within 3 feet of the building perimeter or near the top of existing slopes
- Regulating the amount of water distributed to lawns or planter areas by using irrigation timers
- Selecting landscaping that requires little or no watering, especially near foundations.

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

SECTION 7: FOUNDATIONS

7.1 SUMMARY OF RECOMMENDATIONS

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

7.2 SEISMIC DESIGN CRITERIA

We understand that the project structural design will be based on the 2019 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 at CPT-3 to a depth of 100 feet resulted in an average shear wave velocity of 986 feet per second (or 301 meters per second). Therefore, we have classified the site as Soil Classification D. The mapped spectral acceleration parameters S_S and S_1 were calculated using the SEAOC/OSHPD Seismic Design Maps on-line calculator (https://seismicmaps.org/), 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. At this time, we understand the project structural engineer (HAS & Associates, Inc.) will apply one of the exceptions listed in Chapter 16 of the CBC and Chapter 11 of ASCE 7-16, and a site-specific analysis will not be required. If an exception is not taken by the structural engineer, we should be contacted to perform a site-specific analysis and provide revised seismic design coefficients, as required.

Table 4: CBC Site Categorization and Site Coefficients

Classification/Coefficient	Design Value
Site Class	D
Site Latitude	37.71987°
Site Longitude	-121.49048°
0.2-second Period Mapped Spectral Acceleration[1], S_S	1.322g
1-second Period Mapped Spectral Acceleration[1], S_1	0.454g
Short-Period Site Coefficient – Fa	1.0
Long-Period Site Coefficient – Fv	null[2]
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - S_{MS}	1.322g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	null[2]
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	0.882g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	null[2]

[1]For Site Class B, 5 percent damped.
[2]See Section 11.4.8 of ASCE 7-16 for values and calculations.

7.3 SHALLOW FOUNDATIONS

7.3.1 Spread Footings

Spread footings should bear on natural, undisturbed soil or engineered fill, be at least 12 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 due to the presence of moderately [to highly] expansive soils, and is intended to embed the footing below the zone of significant seasonal moisture fluctuation, reducing the potential for differential movement.

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

7.3.2 Footing Settlement

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

Table 5: Assumed Structural Loading

Foundation Area	Range of Assumed Loads
Interior Isolated Column Footing	150 to 175 kips
Exterior Isolated Column Footing	50 to 75 kips
Perimeter Strip Footing	3 to 5 kips per lineal foot

Based on the above loading and the allowable bearing pressures presented above, we estimate that the total static footing settlement will be on the order of ½-inch, with less than ½-inch of post-construction differential settlement between adjacent foundation elements. In addition, we estimate that differential seismic movement will be on the order of ¼-inch, resulting in a total estimated differential footing movement on the order of ½-inch between foundation elements, assumed to be on the order of 50 to 60 feet. As our footing loads were assumed, we recommend we be retained to review the final footing layout and loading, and verify the settlement estimates above.

7.3.3 Lateral Loading

Lateral loads may be resisted by friction between the bottom of footing and the supporting subgrade, and also by passive pressures generated against footing sidewalls. An ultimate frictional resistance of 0.35 applied to the footing dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 450 pcf may be used in design. The structural engineer should apply an appropriate factor of safety (such as 1.5) to the ultimate values above. Where footings are adjacent to landscape areas without hardscape, the upper 12 inches of soil should be neglected when determining passive pressure capacity.

7.3.4 Spread Footing Construction Considerations

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

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

SECTION 8: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

8.1 OFFICE SLABS-ON-GRADE

As the Plasticity Index (PI) of the surficial soils ranges up to 25.5, the proposed slabs-on-grade should be supported on at least 12 inches of non-expansive fill (NEF) to reduce the potential for slab damage due to soil heave. The NEF layer should be constructed over subgrade prepared in accordance with the recommendations in the "Earthwork" section of this report. If moisture-sensitive floor coverings are planned, the recommendations in the "Interior Slabs Moisture Protection Considerations" section below may be incorporated in the project design if desired. If significant time elapses between initial subgrade preparation and slab-on-grade NEF construction, the subgrade should be proof-rolled to confirm subgrade stability, and if the soil has been allowed to dry out, the subgrade should be re-moisture conditioned to at least 3 percent over the optimum moisture content.

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

8.2 WAREHOUSE SLABS-ON-GRADE

Warehouse slabs-on-grade should be at least 6 inches thick, should have a minimum compressive strength of 3,500 psi, and should be designed for the specific warehouse loading (i.e. forklifts, rack loads, etc.). At this time, rack loading information, etc. was not available. During design of the slab, we should be consulted to provide subgrade modulus for design of the slab on anticipated rack loading, if needed. 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 4 inches of non-expansive fill (NEF) should underlie the upper granular base. As an alternative, the above recommended Class 2 aggregate base can also be increased to 10 inches to also account for required non-expansive fill. 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.

8.3 INTERIOR SLABS MOISTURE PROTECTION CONSIDERATIONS

The following general guidelines for concrete slab-on-grade construction where floor coverings are planned are presented for the consideration by the developer, design team, and contractor. These guidelines are based on information obtained from a variety of sources, including the American Concrete Institute (ACI) and are intended to reduce the potential for moisture-related problems causing floor covering failures, and may be supplemented as necessary based on project-specific requirements. The application of these guidelines or not will not affect the geotechnical aspects of the slab-on-grade performance.

- Place a minimum 10-mil vapor retarder conforming to ASTM E 1745, Class C requirements or better directly below the concrete slab; the vapor retarder should extend to the slab edges and be sealed at all seams and penetrations in accordance with manufacturer's recommendations and ASTM E 1643 requirements. A 4-inch-thick capillary break, consisting of crushed rock should be placed below the vapor retarder and consolidated in place with vibratory equipment. The mineral aggregate shall be of such size that the percentage composition by dry weight as determined by laboratory sieves will conform to the following gradation:

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

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

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

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

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

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

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

8.4 EXTERIOR FLATWORK

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. To help reduce the potential for uncontrolled shrinkage cracking, 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. Flatwork should be isolated from adjacent foundations or retaining walls except where limited sections of structural slabs are included to help span irregularities in retaining wall backfill at the transitions between at-grade and on-structure flatwork.

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 design R-value of 5. The design R-value was chosen based on the results of the laboratory testing performed by Technicon and engineering judgment considering the variable surface conditions.

Table 6: Asphalt Concrete Pavement Recommendations, Design R-value = 5

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	13.0	16.5
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
8.5	5.0	20.0	25.0
9.0	5.5	20.5	26.0
9.5	6.0	22.0	28.0
10.0	6.0	23.5	29.5
10.5	6.5	25.0	31.5
11.0	7.0	26.0	33.0

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

Table 7: Asphalt Concrete Pavement Recommendations (Lime-Treated Subgrade)

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base* (inches)	Total Pavement Section Thickness (inches)
4.0-4.5	2.5	4.0	6.5
5.0-5.5	3.0	4.0	7.0
6.0	3.5	4.0	7.5
6.5	4.0	4.0	8.0
7.0	4.0	4.5	8.5
7.5	4.5	5.0	9.5
8.0	5.0	5.0	10.0
8.5	5.0	6.5	11.5
9.0	5.5	6.5	12.0
9.5	6.0	7.0	13.0
10.0	6.0	8.0	14.0
10.5	6.5	8.5	15.0
11.0	7.0	8.5	15.5

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

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

9.2 PORTLAND CEMENT CONCRETE

The exterior Portland Cement Concrete (PCC) pavement recommendations tabulated below are based on methods presented in the Portland Cement Association (PCA) design manual (PCA, 1984). Recommendations for garage slabs-on-grade were provided in the "Concrete Slabs and Pedestrian Pavements" section above. 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 8: PCC Pavement Recommendations, Design R-value = 5

Allowable ADTT	Minimum PCC Thickness (inches)
13	5½
130	6

Table 9: PCC Pavement Recommendations (Lime-Treated Subgrade)

Allowable ADTT	Minimum PCC Thickness (inches)
13	5
150	5½

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 compacted as recommended in the "Earthwork" section, 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. Due to the expansive surficial soils present, we recommend that the construction and expansion joints be dowelled.

9.2.1 Stress Pads for Trash Enclosures

Pads where trash containers will be stored, and where garbage trucks will park while emptying trash containers, should be constructed on Portland Cement Concrete. We recommend that the trash enclosure pads and stress (landing) pads where garbage trucks will store, pick up, and empty trash be increased to a minimum PCC thickness of 7 inches. The compressive strength, underlayment, and construction details should be consistent with the above recommendations for PCC pavements.

9.3 PAVEMENT CUTOFF

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

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

SECTION 10: RETAINING WALLS

10.1 STATIC LATERAL EARTH PRESSURES

The structural design of any site retaining wall should include resistance to lateral earth pressures that develop from the soil behind the wall, any undrained water pressure, and surcharge loads acting behind the wall. Provided a drainage system is constructed behind the wall to prevent the build-up of hydrostatic pressures as discussed in the section below, we recommend that the walls with level backfill be designed for the following pressures:

Table 10: Recommended Lateral Earth Pressures

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

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

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

10.2 SEISMIC LATERAL EARTH PRESSURES

10.2.1 Site Walls

The 2019 CBC states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. At this time, we are not aware of any retaining walls for the project. However, minor landscaping walls (i.e. walls 6 feet or less in height) may be proposed. In our opinion, design of these walls for seismic lateral earth pressures in addition to static earth pressures is not warranted.

10.3 WALL DRAINAGE

10.3.1 At-Grade Site Walls

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

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

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

10.4 BACKFILL

Where surface improvements will be located over the retaining wall backfill, backfill placed behind the walls should be compacted to at least 95 percent relative compaction using light compaction equipment. Where no surface improvements are planned, backfill should be compacted to at least 90 percent. If heavy compaction equipment is used, the walls should be temporarily braced.

10.5 FOUNDATIONS

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

SECTION 11: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of LBA Realty LLC specifically to support the design of the LBA Logistics Center III Pre-Construction and Supplemental CPTs project in Tracy, California. The opinions, conclusions, and recommendations presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in Northern California at the time this report was prepared. No warranty, expressed or implied, is made or should be inferred.

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

LBA Realty LLC may have provided Cornerstone with plans, reports and other documents prepared by others. LBA Realty LLC 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 12: REFERENCES

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

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

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

California Building Code, 2019, Structural Engineering Design Provisions, Vol. 2.

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

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

Federal Emergency Management Administration (FEMA), 2009, FIRM County of San Joaquin, California, Community Panel #06077C0730F.

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

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

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

Portland Cement Association, 1984, Thickness Design for Concrete Highway and Street Pavements: report.

Pradell, D., 1988, Procedure to Evaluate Earthquake-Induced Settlements in Dry Sandy Soils, Journal of Geotechnical and Environmental Engineering, April 1998, p. 364 – 368 and Errata October 1998 p. 1048.

Risk Engineering Inc., EZ-FRISK Version 7.42 - A Computer Program for the Probabilistic Prediction of Peak Horizontal Acceleration: Digitized California Faults, PC Version.

Schwartz, D.P. 1994, New Knowledge of Northern California Earthquake Potential: in Proceedings of Seminar on New Developments in Earthquake Ground Motion Estimation and Implications for Engineering Design Practice, Applied Technology Council 35-1.

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

Seed, H.B. and I.M. Idriss, 1982, Ground Motions and Soil Liquefaction During Earthquakes: Earthquake Engineering Research Institute.

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

Southern California Earthquake Center (SCEC), 1999, Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction Hazards in California, March.

State of California Department of Transportation, 2015, Highway Design Manual, Fifth Edition, December 31, 2015.

Structural Engineers Association of California (SEAOC), 2019, Seismic Design Maps, web-based application available at https://seismicmaps.org/

Tokimatsu, K., and Seed, H. Bolton, 1987, Evaluation of Settlements in Sands due to Earthquake Shaking, ASCE Journal of Geotechnical Engineering, Vol. 113, August 1987, pp. 861-878.

Townley, S.D. and M.W. Allen, 1939, Descriptive Catalog of Earthquakes of the Pacific Coast of the United States, 1769 to 1928: Bulletin of the Seismological Society of America, Vol. 29, No. 1, pp. 1247-1255.

Witter, R.C., Knudsen, K.L., Sowers, J.M., Wentworth, C.M., Koehler, R.D., Randolph, C.E., Brooks, S, K. and Gans, K.D., 2006, Maps of Quaternary deposits and liquefaction susceptibility in the central San Francisco Bay region, California: U.S. Geological Survey, Open-File Report OF-2006-1037, scale 1:200000.

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

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

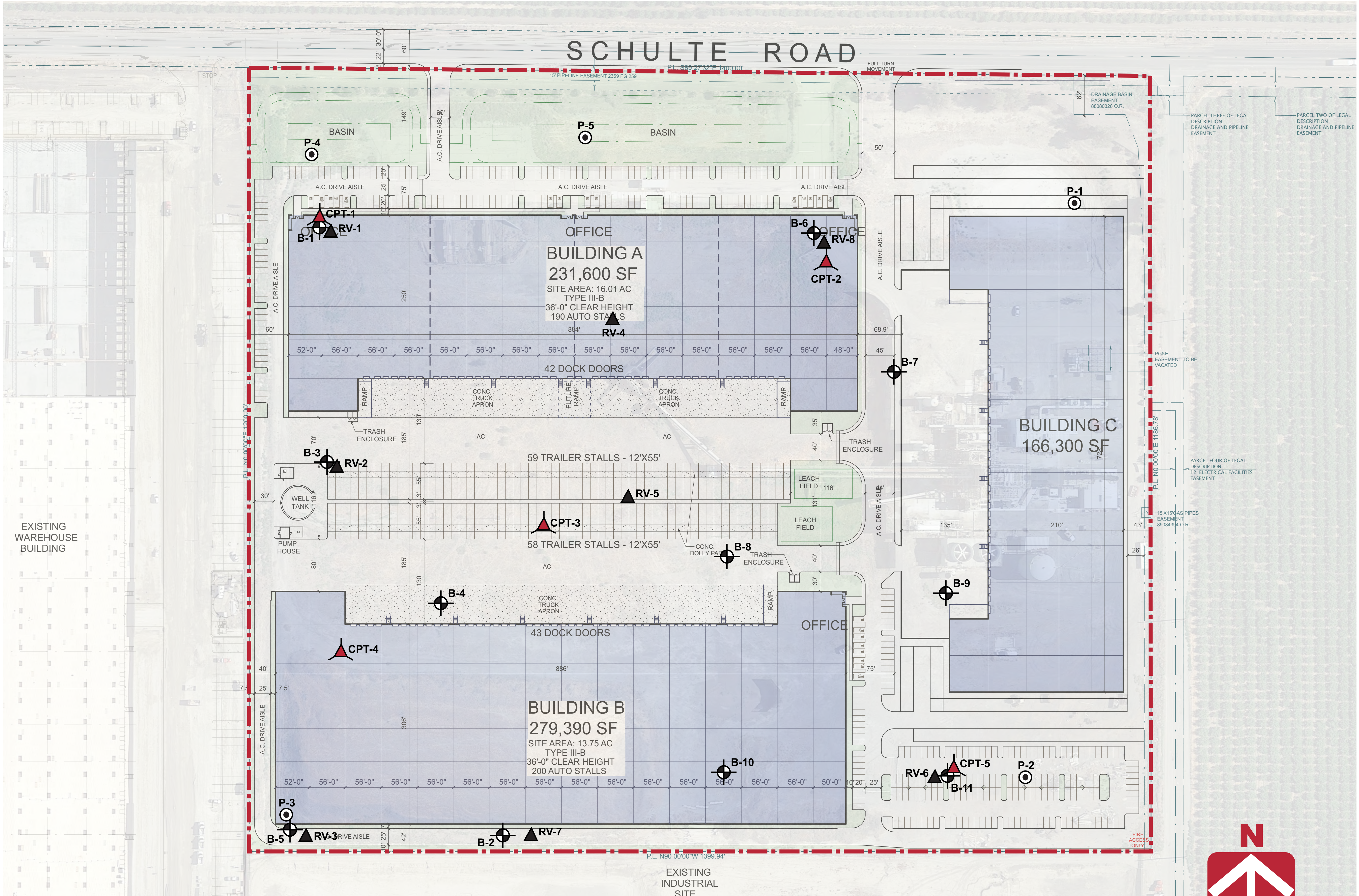
Youd, T.L. and Idriss, I.M., et al, 1997, Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils: National Center for Earthquake Engineering Research, Technical Report NCEER - 97-0022, January 5, 6, 1996.

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

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

CORNERSTONE EARTH GROUP

Vicinity Map	Project Number 750-2-1
14800 West Schulte Road Tracy, CA	Figure Number Figure 1
	Date February 2020 · Drawn By RRN



Legend

CPT-1 ▲ Approximate location of cone penetration test (CPT) (Cornerstone, current investigation)	RV-8 ▲ Approximate location of R-Value (RV) (Technicon Engineering Services, Inc., 2018)
B-11 ⊕ Approximate location of soil boring (B) (Technicon Engineering Services, Inc., 2018)	P-5 ⊙ Approximate location of percolation test (P) (Technicon Engineering Services, Inc., 2018)

N

0 150 300

APPROXIMATE SCALE (FEET)

Base by Google Earth, dated 06/28/2018
Overlay by RGA Architectural Design, Site Plan - A1-1P, dated 8/30/2019

Project Number 750-2-1

Figure Number Figure 2

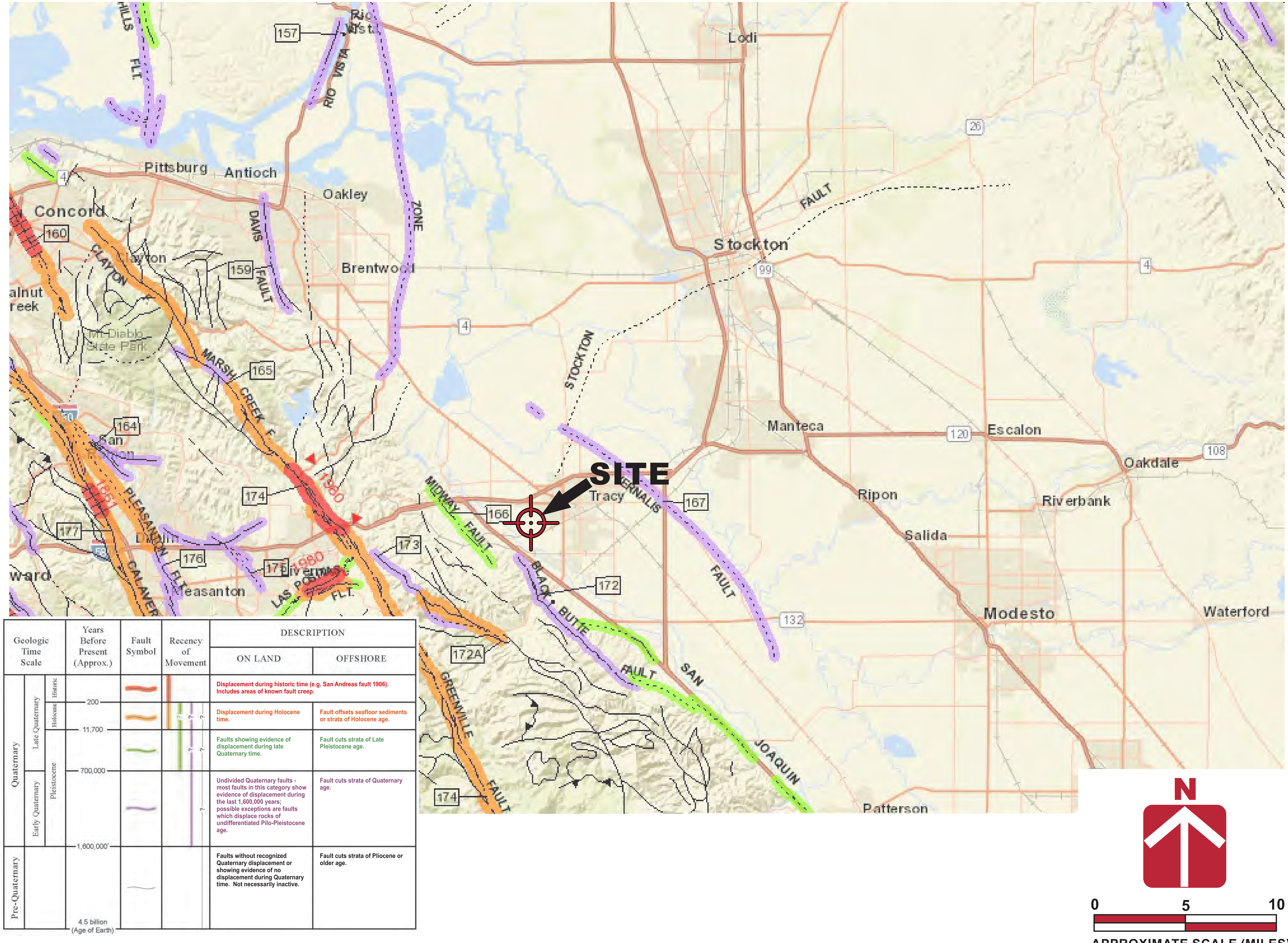
Drawn By RRN

Date February 2020

Site Plan

14800 West Schulte Road
Tracy, CA

CORNERSTONE EARTH GROUP



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION ON LAND	DESCRIPTION OFFSHORE
Quaternary	Late Quaternary	Historic				Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
			200				
		Holocene				Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
			11,700				
		Pleistocene				Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
			700,000				
	Early Quaternary					Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Pilo-Pleistocene age.	Fault cuts strata of Quaternary age.
			1,600,000				
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)

Project Number 750-2-1

Figure Number Figure 3

Drawn By RRN

Date February 2020

Regional Fault Map

14800 West Schulte Road
Tracy, CA

CORNERSTONE EARTH GROUP

CORNERSTONE EARTH GROUP

FIGURE 4A

CPT NO. 1

PROJECT/CPT DATA

Project Title	**LBA Logistitics Center III**
Project No.	**750-2-1**
Project Manager	**MFR**

SEISMIC PARAMETERS

Controlling Fault	**Great Valley**	
Earthquake Magnitude (Mw)	**6.92**	
PGA (Amax)	**0.611**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**56**
Design Water Depth (feet)	**25**
Ave. Unit Weight Above GW (pcf)	**120**
Ave. Unit Weight Below GW (pcf)	**125**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 25 **FEET**

0.01 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.04 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.1 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.00 **L/H** 34.0

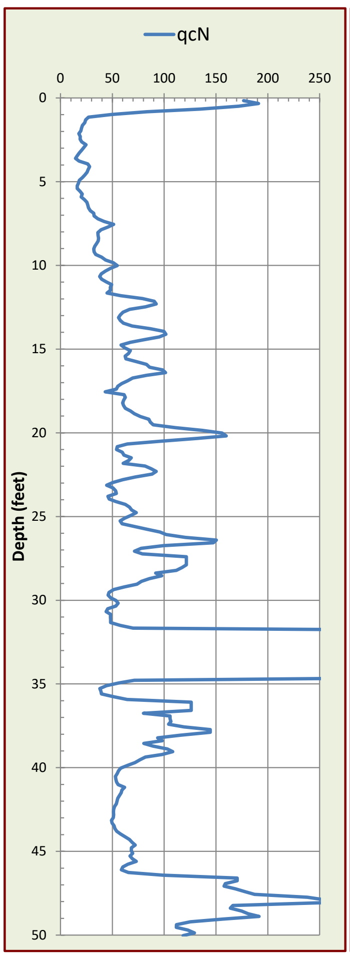
LDI[1]**Corrected for Distance** 0.00 (4 < L/H < 40)

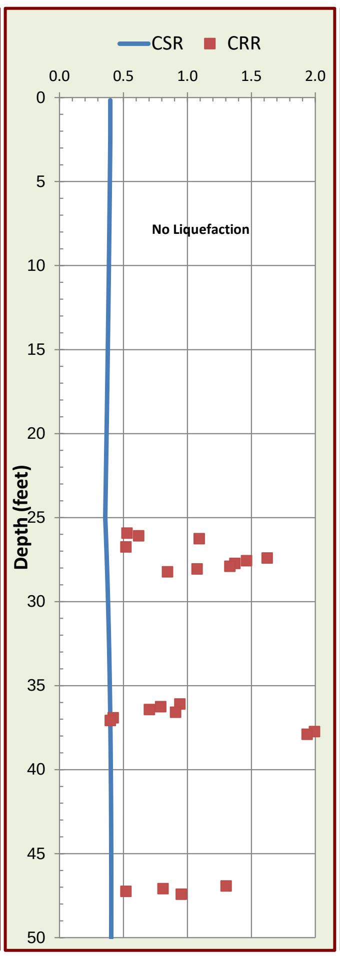
EXPECTED RANGE OF DISPLACEMENT

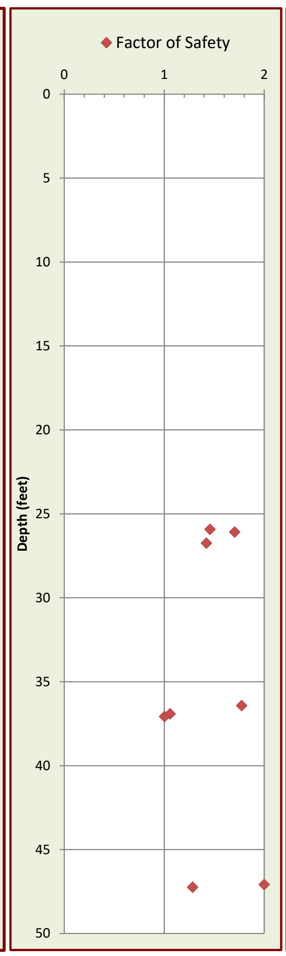
0.0 **to** **0.0** **feet**

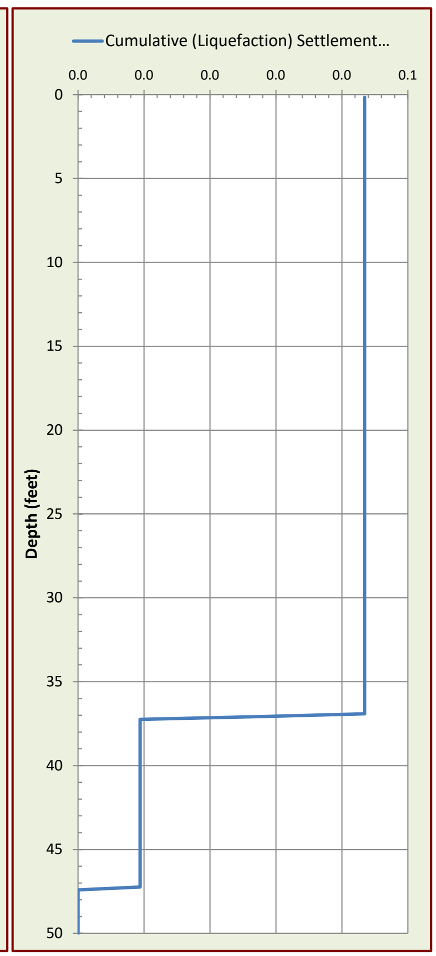
[1]Not Valid for L/H Values < 4 and > 40.

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









CORNERSTONE EARTH GROUP

FIGURE 4B

CPT NO. 2

PROJECT/CPT DATA

Project Title	**LBA Logistitics Center III**
Project No.	**750-2-1**
Project Manager	**MFR**

SEISMIC PARAMETERS

Controlling Fault	**Great Valley**	
Earthquake Magnitude (Mw)	**6.92**	
PGA (Amax)	**0.611**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**56**
Design Water Depth (feet)	**25**
Ave. Unit Weight Above GW (pcf)	**120**
Ave. Unit Weight Below GW (pcf)	**125**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM	25	**FEET**
	0.01	**(Inches)**
LIQUEFACTION SETTLEMENT FROM	50	**FEET**
	0.06	**(Inches)**
TOTAL SEISMIC SETTLEMENT	0.1	**INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.00 **L/H** 4.0

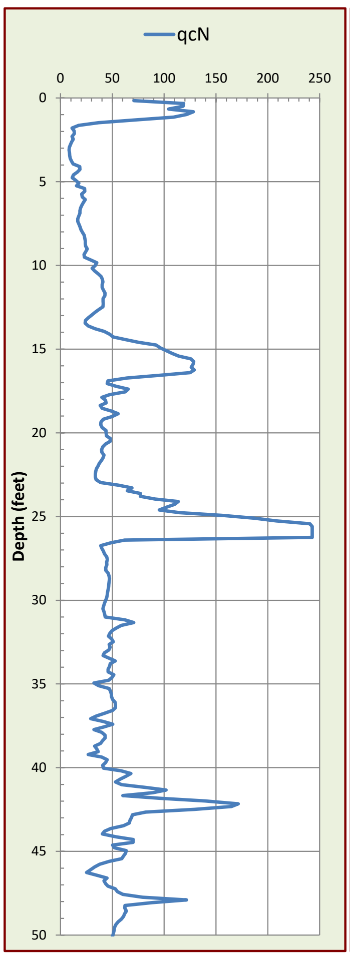
LDI[1]**Corrected for Distance** 0.00 (4 < L/H < 40)

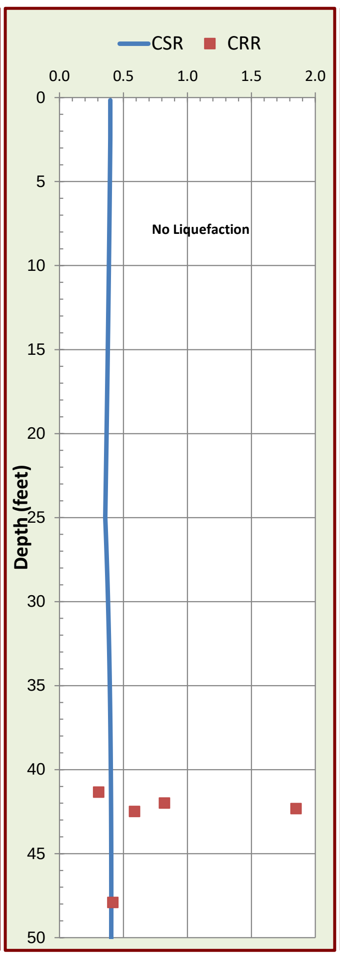
EXPECTED RANGE OF DISPLACEMENT

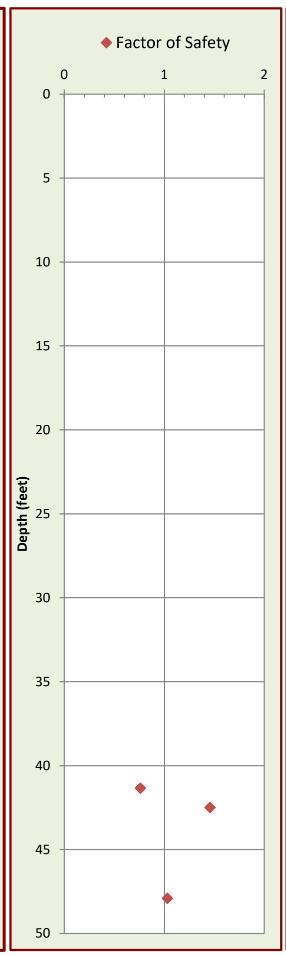
0.0 **to** **0.0** **feet**

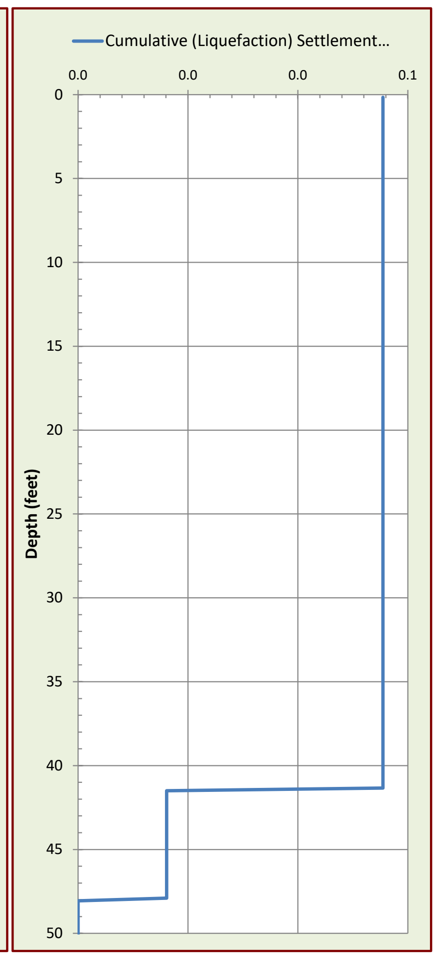
[1]Not Valid for L/H Values < 4 and > 40.

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









CORNERSTONE EARTH GROUP

FIGURE 4C

CPT NO. 3

PROJECT/CPT DATA

Project Title	**LBA Logistitics Center III**
Project No.	**750-2-1**
Project Manager	**MFR**

SEISMIC PARAMETERS

Controlling Fault	**Great Valley**	
Earthquake Magnitude (Mw)	**6.92**	
PGA (Amax)	**0.611**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**56**
Design Water Depth (feet)	**25**
Ave. Unit Weight Above GW (pcf)	**120**
Ave. Unit Weight Below GW (pcf)	**125**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 25 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.08 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.1 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

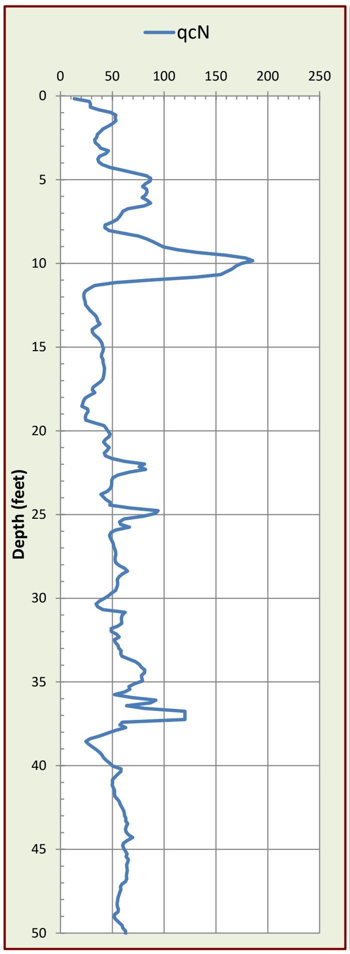
LDI[2] 0.05 **L/H** 32.0

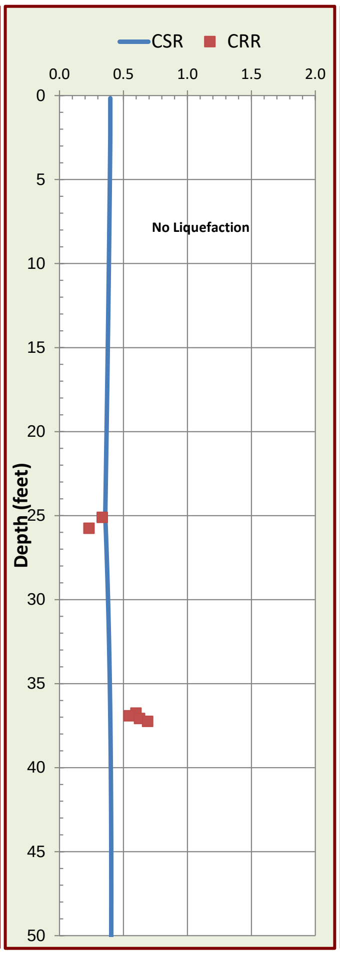
LDI[1] Corrected for Distance 0.02 (4 < L/H < 40)

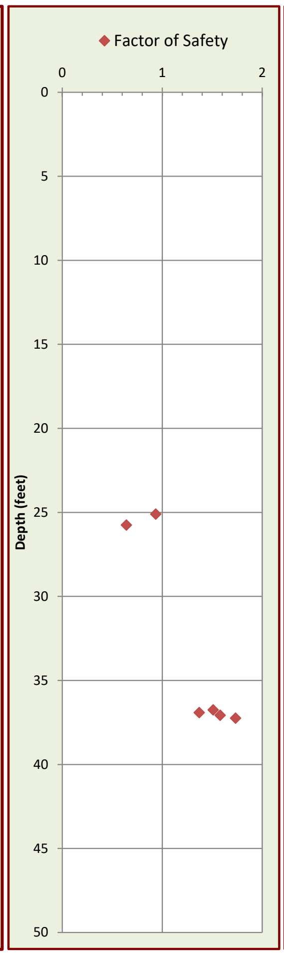
EXPECTED RANGE OF DISPLACEMENT

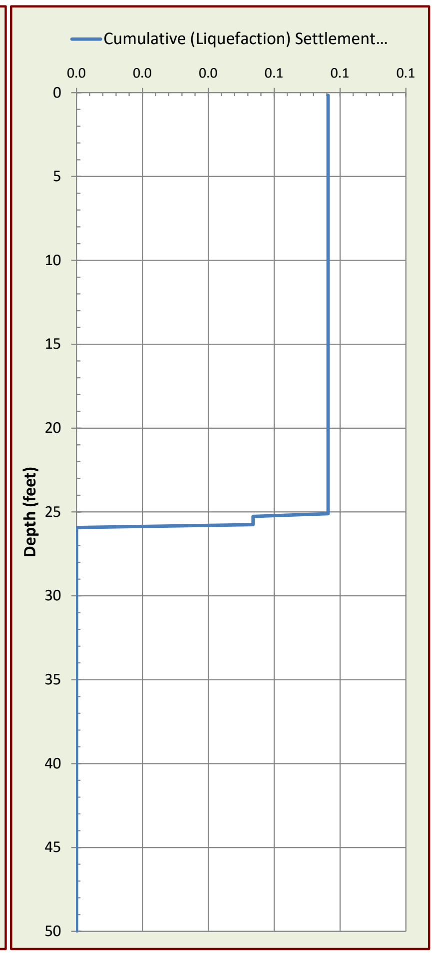
0.0 **to** 0.0 **feet**

[1]Not Valid for L/H Values < 4 and > 40.
[2]LDI Values Only Summed to 2H Below Grade.









CORNERSTONE EARTH GROUP

FIGURE 4D

CPT NO. 4

PROJECT/CPT DATA

Project Title	**LBA Logistitics Center III**
Project No.	**750-2-1**
Project Manager	**MFR**

SEISMIC PARAMETERS

Controlling Fault	**Great Valley**	
Earthquake Magnitude (Mw)	**6.92**	
PGA (Amax)	**0.611**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**56**
Design Water Depth (feet)	**25**
Ave. Unit Weight Above GW (pcf)	**120**
Ave. Unit Weight Below GW (pcf)	**125**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 25 **FEET**

0.00 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.00 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.0 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.01 **L/H** 50.0

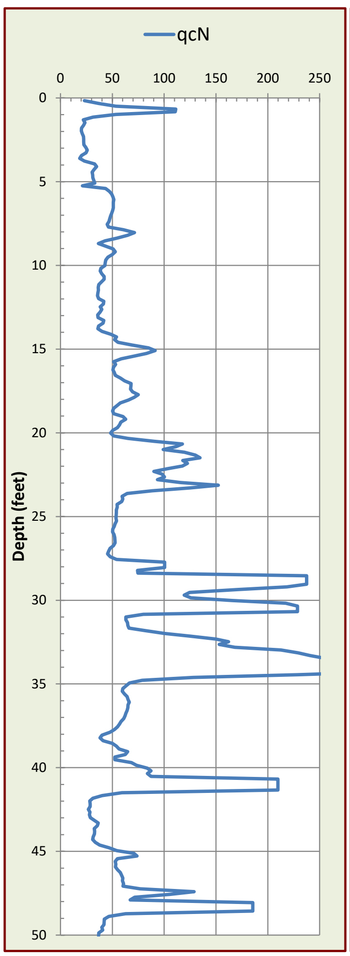
LDI[1] **Corrected for Distance** 0.00 (4 < L/H < 40)

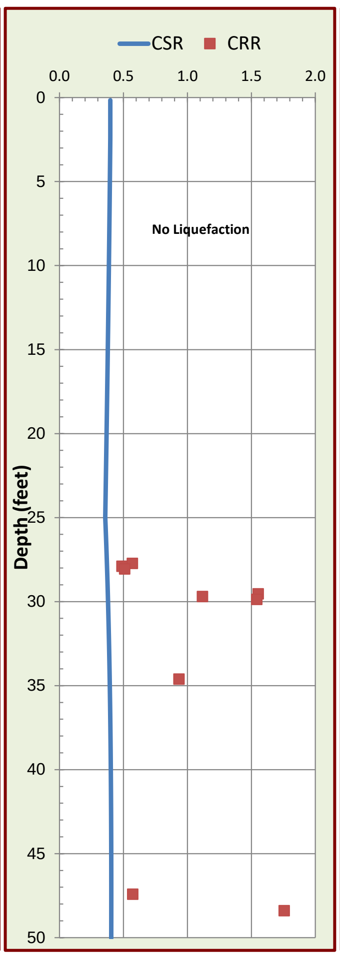
EXPECTED RANGE OF DISPLACEMENT

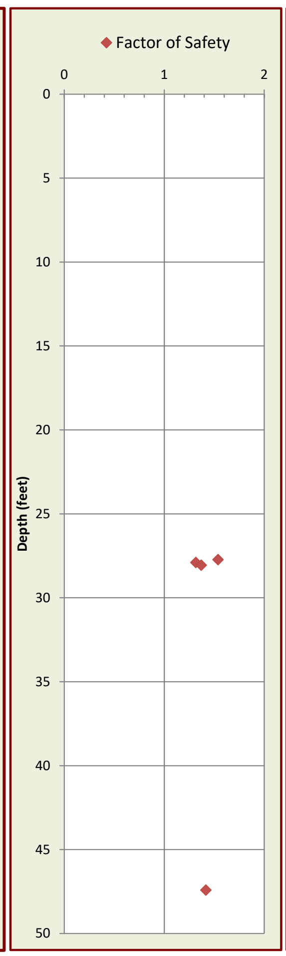
0.0 **to** **0.0** **feet**

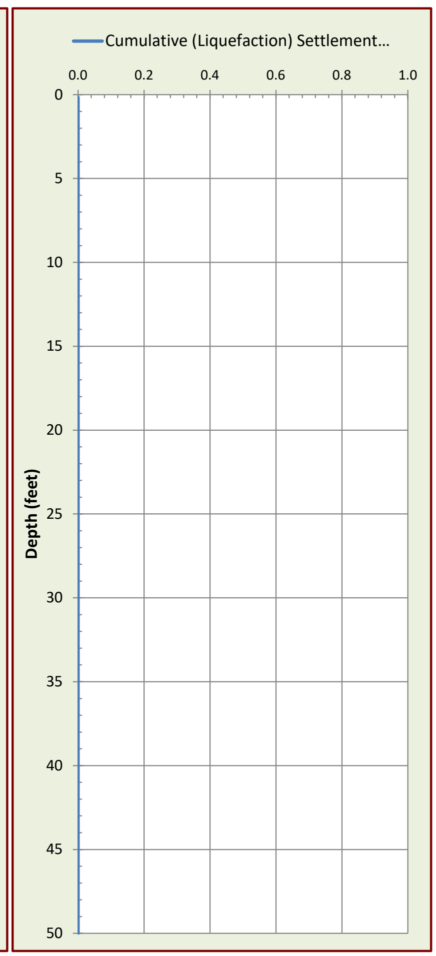
[1]Not Valid for L/H Values < 4 and > 40.

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









CORNERSTONE EARTH GROUP

FIGURE 4E

CPT NO. 5

PROJECT/CPT DATA

Project Title	LBA Logistitics Center III
Project No.	750-2-1
Project Manager	MFR

SEISMIC PARAMETERS

Controlling Fault	Great Valley	
Earthquake Magnitude (Mw)	6.92	
PGA (Amax)	0.611	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	56
Design Water Depth (feet)	25
Ave. Unit Weight Above GW (pcf)	120
Ave. Unit Weight Below GW (pcf)	125

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 25 FEET

0.00 (Inches)

LIQUEFACTION SETTLEMENT FROM 50 FEET

0.30 (Inches)

TOTAL SEISMIC SETTLEMENT 0.3 INCHES

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.09 L/H 40.0

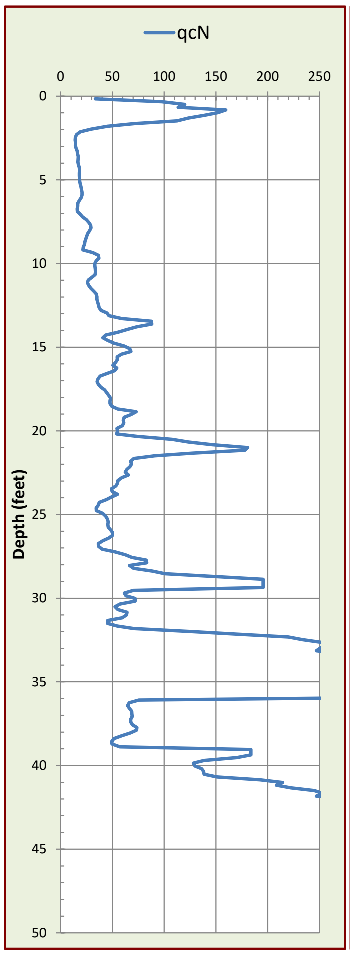
LDI[1] Corrected for Distance 0.03 (4 < L/H < 40)

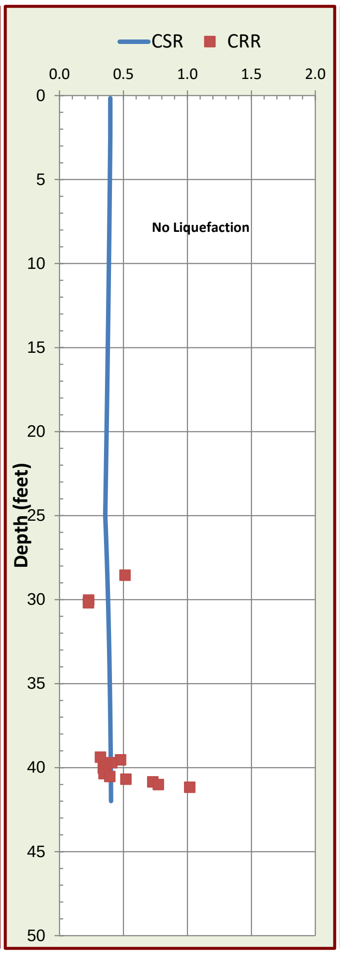
EXPECTED RANGE OF DISPLACEMENT

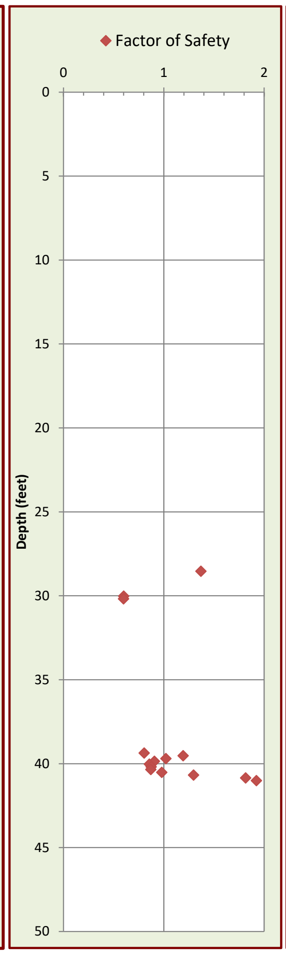
0.0 to 0.1 feet

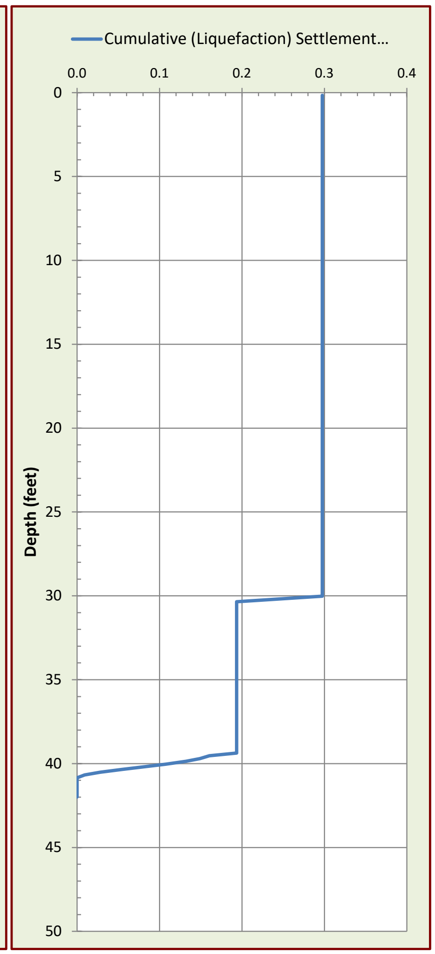
[1]Not Valid for L/H Values < 4 and > 40.

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









APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using 20-ton truck-mounted Cone Penetration Test equipment. Five CPT soundings were performed in accordance with ASTM D 5778-95 (revised, 2002) on January 30, 2020, to depths ranging from about 50 to about 100 feet. The approximate locations of CPTs are shown on the Site Plan, Figure 2.

CPT locations were approximated using existing site boundaries, a hand held GPS unit, and other site features as references. CPT elevations were not determined. The locations of the CPTs should be considered accurate only to the degree implied by the method used.

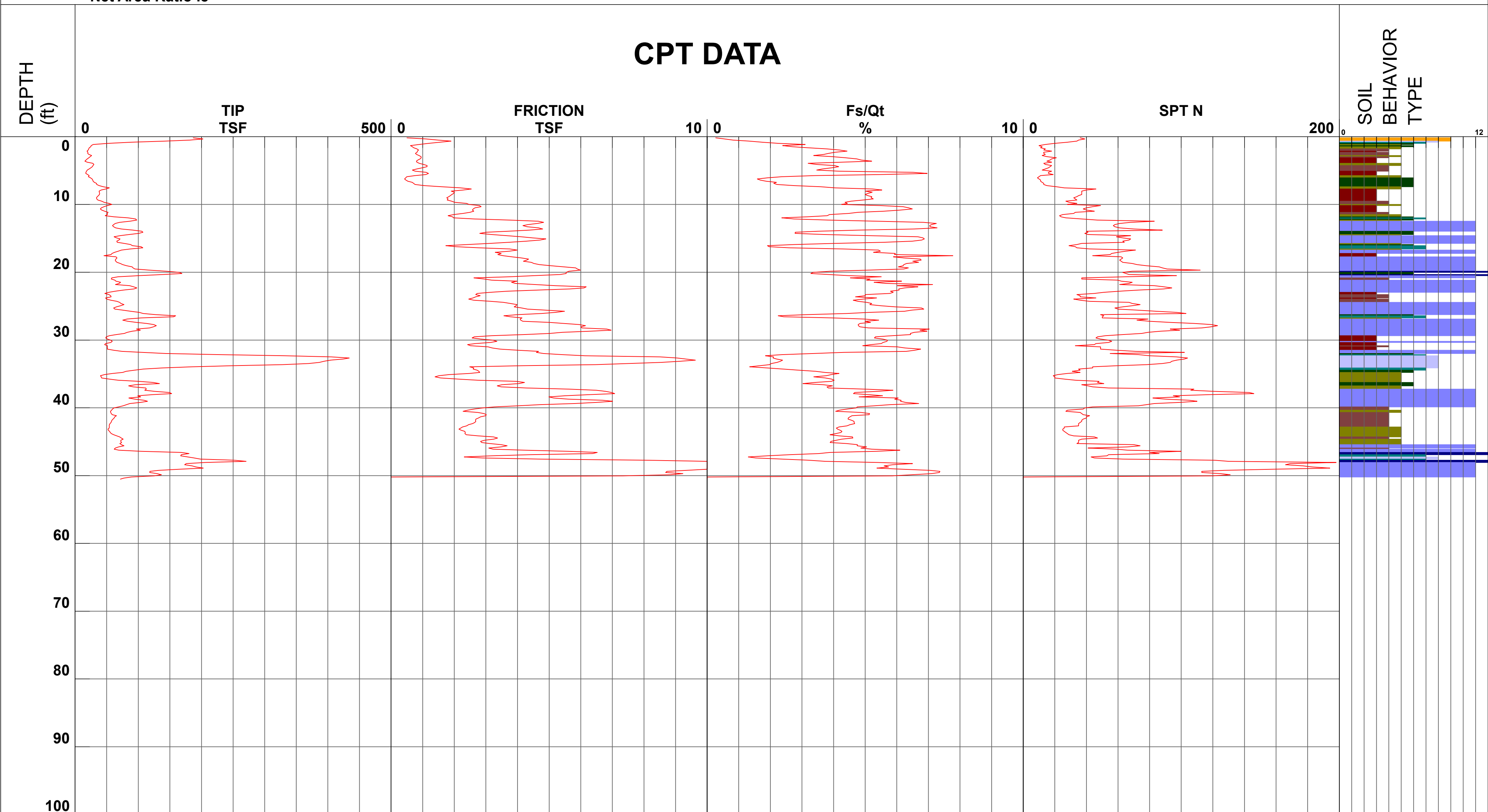
The CPT involved advancing an instrumented cone-tipped probe into the ground while simultaneously recording the resistance at the cone tip (q_c) and along the friction sleeve (f_s) at approximately 5-centimeter intervals. Based on the tip resistance and tip to sleeve ratio (R_f), the CPT classified the 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.

Attached 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 CPT locations. The passage of time may result in altered subsurface conditions due to environmental changes.

Cornerstone Earth Group

Project	14800 W. Schulte Road	Operator	BH-AJ	Filename	SDF(427).cpt
Job Number	750-2-1	Cone Number	DDG1496	GPS	
Hole Number	CPT-01	Date and Time	1/30/2020 4:07:05 PM	Maximum Depth	50.52 ft
EST GW Depth During Test		56.00 ft			

Net Area Ratio .8



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

Cone Size 15cm squared

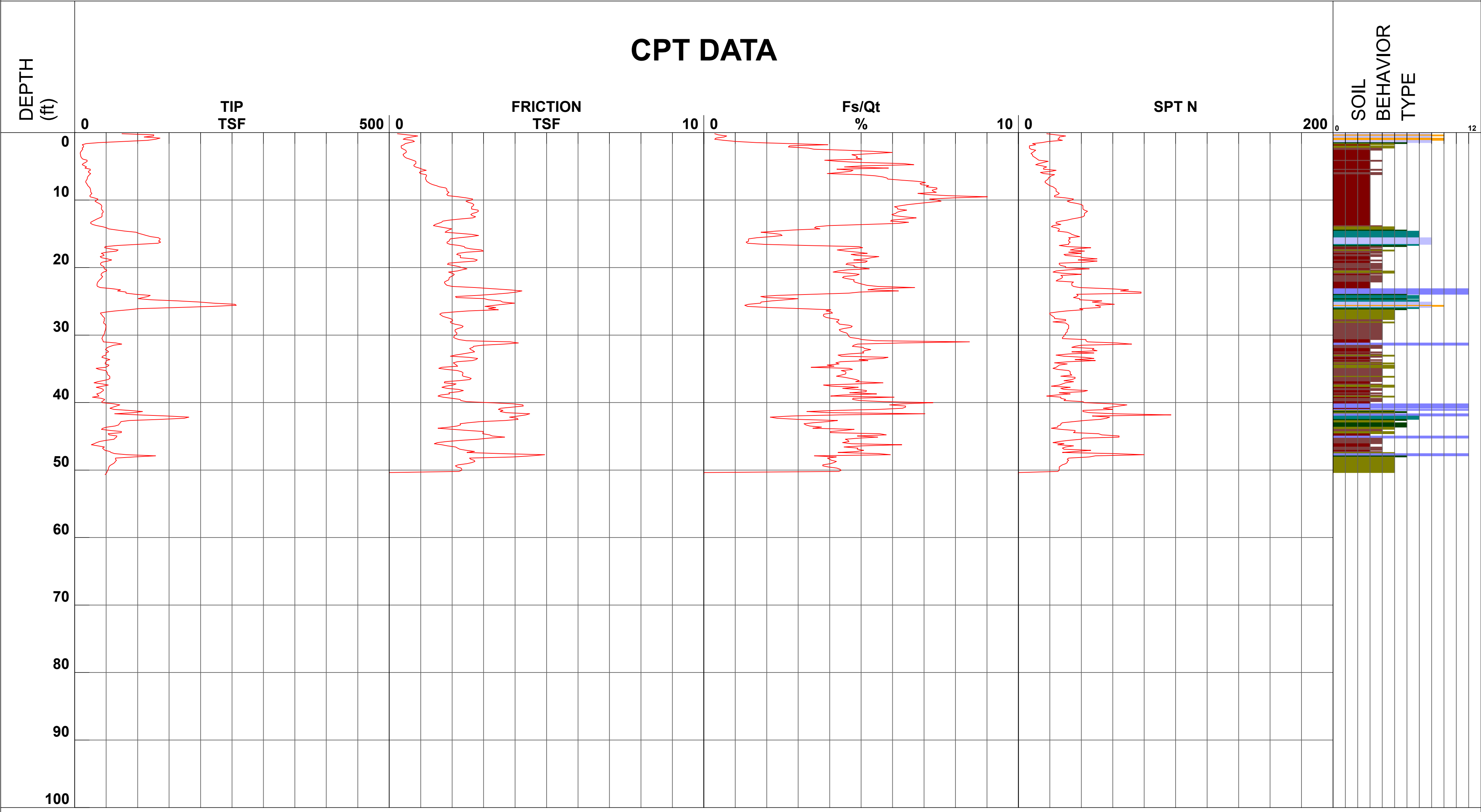
S*Soil behavior type and SPT based on data from UBC-1983

Cornerstone Earth Group

Project	14800 W Schulte Road	Operator	JM-AJ	Filename	SDF(334).cpt
Job Number	750-2-1	Cone Number	DDG1489	GPS	
Hole Number	CPT-02	Date and Time	1/30/2020 4:48:49 PM	Maximum Depth	50.69 ft
EST GW Depth During Test		56.00 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 15cm squared

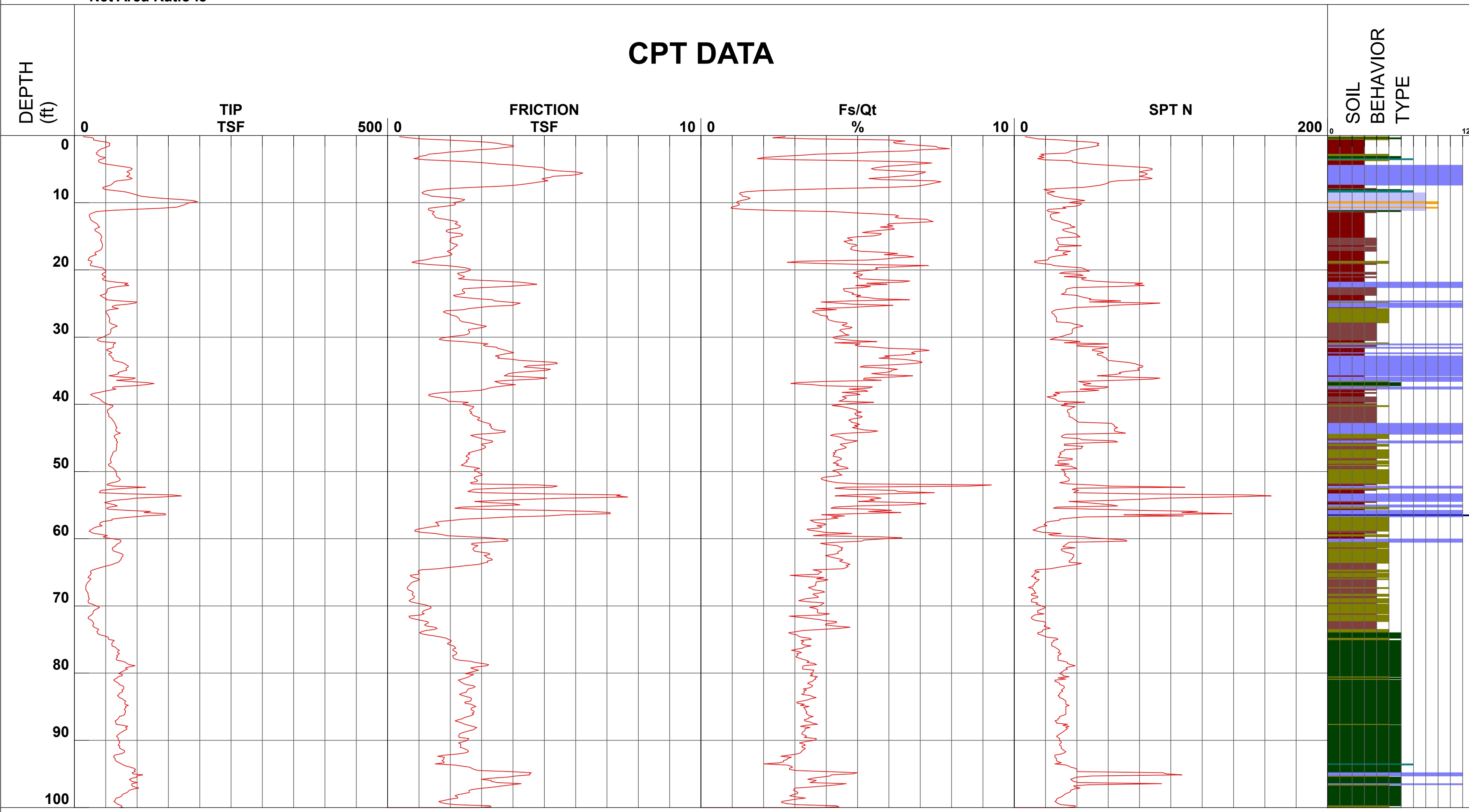
S*Soil behavior type and SPT based on data from UBC-1983

Cornerstone Earth Group

Project	14800 W. Schulte Road	Operator	BH-AJ	Filename	SDF(424).cpt
Job Number	750-2-1	Cone Number	DDG1496	GPS	
Hole Number	CPT-03	Date and Time	1/30/2020 10:18:02 AM	Maximum Depth	100.56 ft
EST GW Depth During Test		56.00 ft			

Net Area Ratio .8

CPT DATA

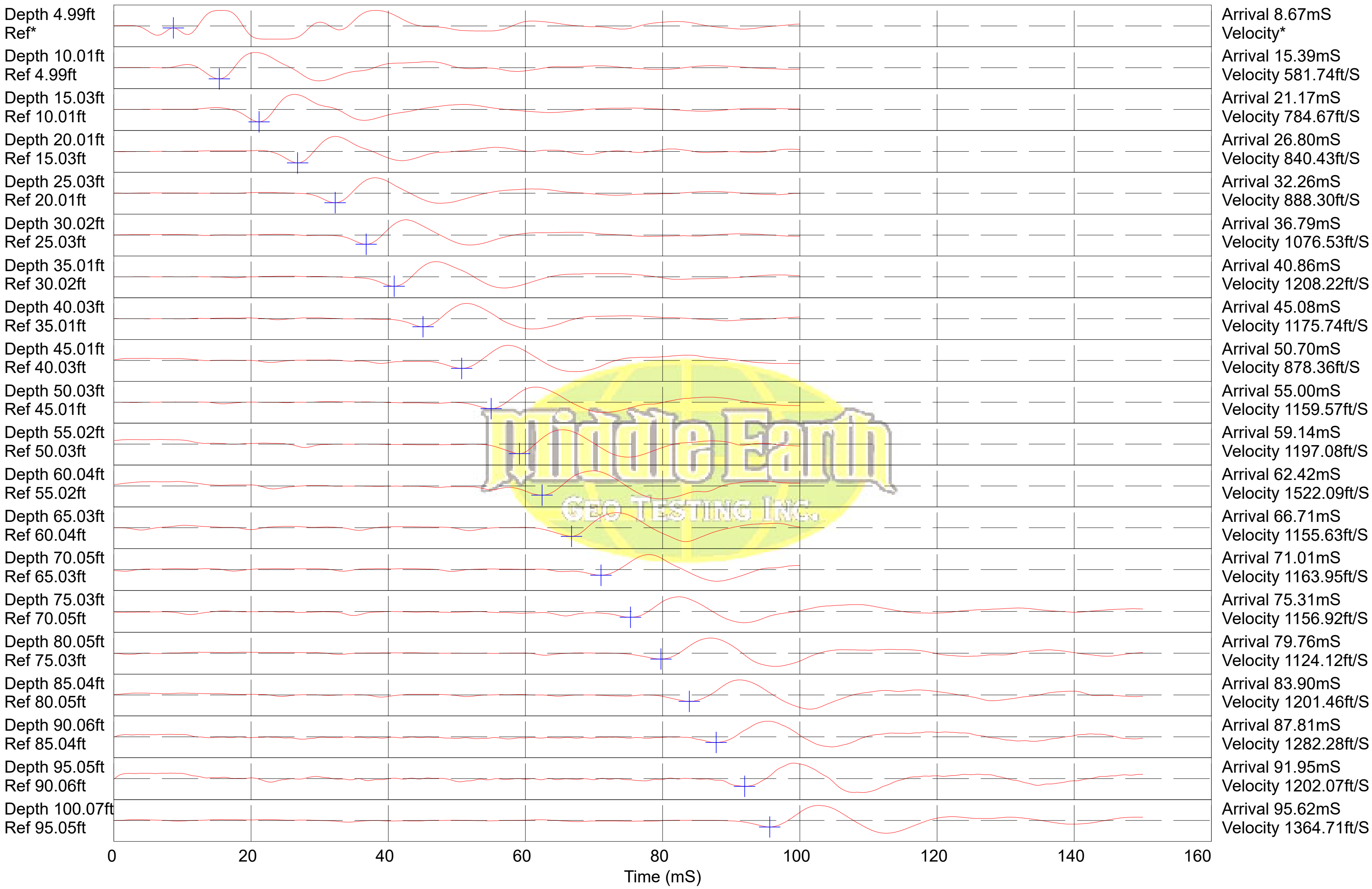


1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 15cm squared

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

CPT-03 Cornerstone Earth Group 14800 W. Schulte Road



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

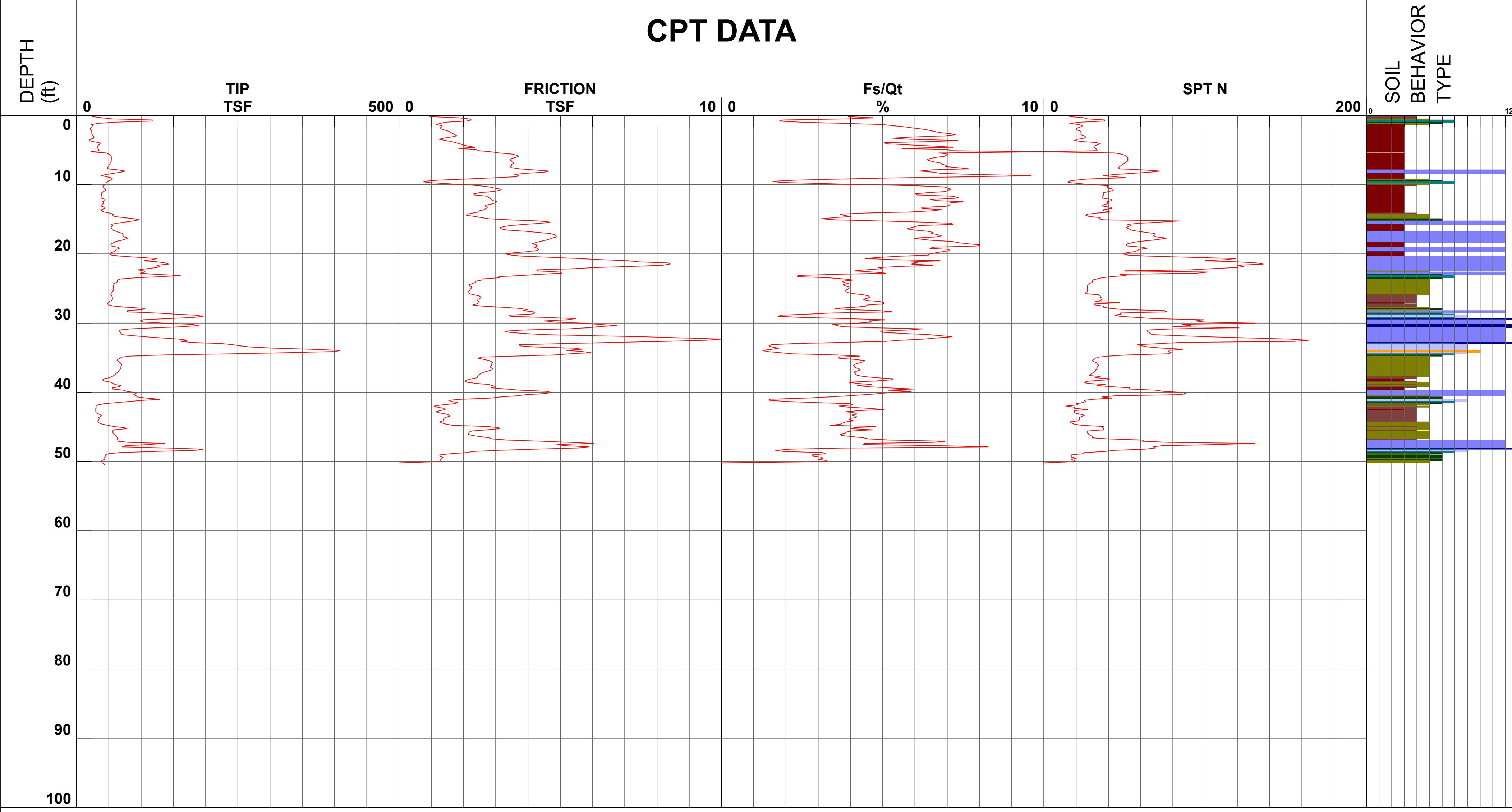
COMMENT:

Cornerstone Earth Group

Project	14800 W. Schulte Road	Operator	BH-AJ	Filename	SDF(426).cpt
Job Number	750-2-1	Cone Number	DDG1496	GPS	
Hole Number	CPT-04	Date and Time	1/30/2020 2:32:25 PM	Maximum Depth	50.52 ft
EST GW Depth During Test		56.00 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 (*)

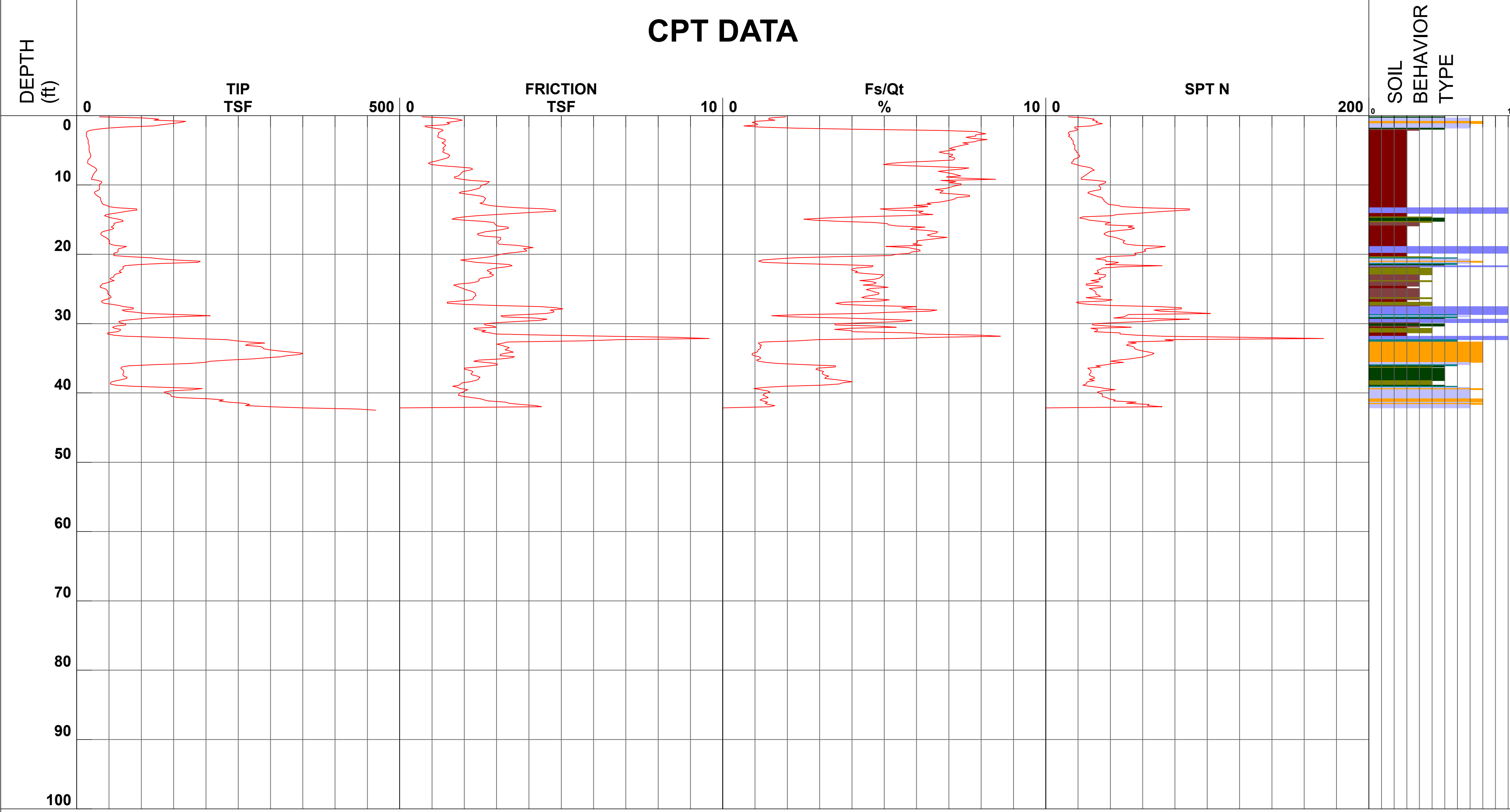
Cone Size 15cm squared

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

Cornerstone Earth Group

Project	14800 W. Schulte Road	Operator	BH-AJ	Filename	SDF(425).cpt
Job Number	750-2-1	Cone Number	DDG1496	GPS	
Hole Number	CPT-05	Date and Time	1/30/2020 12:56:00 PM	Maximum Depth	42.49 ft
EST GW Depth During Test		56.00 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

APPENDIX B: FIELD INVESTIGATION BY TECHNICON (2018)

TECHNICON Enineering Services, INC.
4539 N Brawley Ave #108
Fresno, CA 93722
Telephone: 559-276-9311

KEY TO SYMBOLS

PROJECT NAME Proposed Industrial Building

DATE OF EXPLORATION 6/16/2018

PROJECT LOCATION 14800 W Schulte Road

PROJECT NUMBER 180431

LITHOLOGIC SYMBOLS (Unified Soil Classification System)

		FILL
	SW	WELL GRADED SAND
	SP	POORLY GRADED SAND
	SM	SILTY SAND
	SC	CLAYEY SAND
	PT	PEAT
	OL	LOW PLASTICITY ORGANIC SILT
	OH	HIGH PLASTICITY ORGANIC SILT
	ML	LOW PLASTICITY SILT
	MH	HIGH PLASTICITY SILT
	GW	WELL GRADED GRAVEL
	GP	POORLY GRADED GRAVEL
	GM	SILTY GRAVEL
	GC	CLAYEY GRAVEL
	CL	LOW PLASTICITY CLAY
	CH	HIGH PLASTICITY CLAY

SAMPLER SYMBOLS

STANDARD PENETRATION TEST

CALIFORNIA SAMPLER

MODIFIED CALIFORNIA SAMPLER

SHELBY TUBE SAMPLER

ROCK CORE BARREL

BULK SAMPLE

Water Level at Time of Drilling

Water Level at End of Drilling

Water Level After 24 Hours

Assumed stratum line

Observed stratum line

Note 1: The degree of saturation shown on the boring logs is based on an assumed specific gravity of 2.65. The actual degree of saturation may vary.

Note 2: The stratum lines shown on the logs represent the approximate boundary between soil types; the actual in-situ transition may be gradual.

ABBREVIATIONS

LL - LIQUID LIMIT (%)
PI - PLASTIC INDEX (%)
W - MOISTURE CONTENT (%)
DD - DRY DENSITY (PCF)
S - DEGREE OF SATURATION (%)
NP - NON PLASTIC
-200 PERCENT PASSING NO. 200 SIEVE
PP - POCKET PENETROMETER (TSF)

TV - TORVANE
PID - PHOTOIONIZATION DETECTOR
UC - UNCONFINED COMPRESSION
ppm - PARTS PER MILLION

TECHNICON Eningeering Services, INC.
4539 N Brawley Ave #108
Fresno, CA 93722
Telephone: 559-276-9311

BORING B 1

PROJECT NAME Proposed Industrial Building
PROJECT NUMBER 180431
PROJECT LOCATION 14800 W Schulte Road
SURFACE DESCRIPTION Wood Chips
DATE STARTED 6/16/18 **COMPLETED** 6/19/18
GROUND ELEVATION
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc.
GROUND WATER LEVEL No groundwater encountered.
DRILL RIG TYPE CME 55
BORING DEPTH 18 ft
DRILLING METHOD 7.5-inch Hollow Stem Auger
LOGGED BY K. Weatherford **CHECKED BY** S. Alvarez

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
0				Wood Chips				
				CLAY (CL) - stiff, dark brown, moist, high plasticity				
	CAL	9-7-12 (19)			103.7	21.6	S = 96 %	
5	SPT	4-4-4 (8)						
				Silty CLAY (CL-ML) - medium stiff, light tannish brown, moist, with fine sand				
10	CAL	5-5-14 (19)		Stiff	104.2	12.0	S = 54 %	
15	SPT	1-1-1 (2)		Very soft				
	CAL	9-17-26 (43)		Hard				

NOTES:
1. Bottom of boring at 18.0 feet.
2. No groundwater encountered.
3. Boring backfilled with soil cuttings 6/19/18.

TECHNICON Eningeering Services, INC.
4539 N Brawley Ave #108
Fresno, CA 93722
Telephone: 559-276-9311

BORING B 2

PROJECT NAME Proposed Industrial Building
PROJECT NUMBER 180431
PROJECT LOCATION 14800 W Schulte Road
SURFACE DESCRIPTION Flat, Moderate Vegetation
DATE STARTED 6/20/18 **COMPLETED** 6/20/18
GROUND ELEVATION
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc.
GROUND WATER LEVEL No groundwater encountered.
DRILL RIG TYPE CME 55
BORING DEPTH 21.5 ft
DRILLING METHOD 7.5-inch Hollow Stem Auger
LOGGED BY K. Weatherford **CHECKED BY** S. Alvarez

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
0	CAL	4-10-15 (25)		**Sandy CLAY (CL)** - very stiff, dark brown, moist, medium plasticity, with fine to medium sand	108.6	12.5	S = 63 %	
4				**Silty CLAY (CL-ML)** - very stiff, light brown, moist, with fine sand				
5	CAL	13-16-14 (30)			94.7	10.2	S = 36 %	
9				**Sandy SILT (ML)** - medium stiff, light brown, moist, with fine sand				
10	SPT	3-3-4 (7)						
14				**Sandy GRAVEL (GP)** - very dense, light brown, moist, medium to coarse grained				
15	CAL	12-33-50 (83)						
18				**Silty CLAY (CL-ML)** - very stiff, light brown, moist, with fine sand				
20	SPT	8-10-11 (21)						

NOTES:
1. Bottom of boring at 21.5 feet.
2. No groundwater encountered.
3. Boring backfilled with soil cuttings 6/20/18.

TECHNICON Eningeering Services, INC.
4539 N Brawley Ave #108
Fresno, CA 93722
Telephone: 559-276-9311

BORING B 3

PROJECT NAME Proposed Industrial Building
PROJECT NUMBER 180431
PROJECT LOCATION 14800 W Schulte Road
SURFACE DESCRIPTION Wood Chips
DATE STARTED 6/19/18 **COMPLETED** 6/19/18
GROUND ELEVATION
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc.
GROUND WATER LEVEL No groundwater encountered.
DRILL RIG TYPE CME 55
BORING DEPTH 16.5 ft
DRILLING METHOD 7.5-inch Hollow Stem Auger
LOGGED BY K. Weatherford **CHECKED BY** S. Alvarez

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
0				Wood Chips				
	CAL	15-18-17 (35)		**CLAY (CL)** - very stiff, dark brown, moist, high plasticity	110.0	14.5	S = 77 %	
5	CAL	9-13-20 (33)			105.5	14.7	S = 69 %	
10	SPT	19-14-14 (28)		**Silty CLAY (CL-ML)** - very stiff, light brown, moist, with fine sand				
15	CAL	31-16-18 (34)						

NOTES:
1. Bottom of boring at 16.5 feet.
2. No groundwater encountered.
3. Boring backfilled with soil cuttings 6/19/18.

TECHNICON Eningeering Services, INC.
4539 N Brawley Ave #108
Fresno, CA 93722
Telephone: 559-276-9311

BORING B 4

PROJECT NAME Proposed Industrial Building
PROJECT NUMBER 180431
PROJECT LOCATION 14800 W Schulte Road
SURFACE DESCRIPTION Wood Chips
DATE STARTED 6/19/18 **COMPLETED** 6/19/18
GROUND ELEVATION
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc.
GROUND WATER LEVEL No groundwater encountered.
DRILL RIG TYPE CME 55
BORING DEPTH 31.5 ft
DRILLING METHOD 7.5-inch Hollow Stem Auger
LOGGED BY K. Weatherford **CHECKED BY** S. Alvarez

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
0				Wood Chips				
				CLAY (CL) - stiff, black, moist, medium plasticity, with fine sand				
	CAL	11-9-9 (18)			108.6	17.7	S = 90 %	
5				**Silty CLAY (CL-ML)** - stiff, light brown, moist, with fine sand, moderate cementation				
	SPT	3-4-9 (13)						
10	CAL	13-23-33 (56)		Hard, strong cementation	106.3	11.1	S = 53 %	
15	SPT	8-11-13 (24)		Very stiff				
20	CAL	10-19-24 (43)		Hard, light brown and white				
25	SPT	5-5-7 (12)		Medium stiff				
30	SPT	13-15-17 (32)		Hard				

NOTES:
1. Bottom of boring at 31.5 feet.
2. No groundwater encountered.
3. Boring backfilled with soil cuttings 6/19/18.

TECHNICON Eningeering Services, INC.
4539 N Brawley Ave #108
Fresno, CA 93722
Telephone: 559-276-9311

BORING B 5

PROJECT NAME Proposed Industrial Building
PROJECT NUMBER 180431
PROJECT LOCATION 14800 W Schulte Road
SURFACE DESCRIPTION Flat, Moderate Vegetation
DATE STARTED 6/20/18 **COMPLETED** 6/20/18
GROUND ELEVATION
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc.
GROUND WATER LEVEL No groundwater encountered.
DRILL RIG TYPE CME 55
BORING DEPTH 21.5 ft
DRILLING METHOD 7.5-inch Hollow Stem Auger
LOGGED BY K. Weatherford **CHECKED BY** S. Alvarez

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
0				**Silty CLAY (CL-ML)** - very stiff, light brown and white, moist, with fine to medium sand, moderate cementation				
	CAL	5-13-15 (28)			114.4	13.0	S = 77 %	
5	SPT	8-10-12 (22)		Very stiff				
10	CAL	8-20-33 (53)		Hard	99.8	18.3	S = 74 %	
15	SPT	6-10-14 (24)		Very stiff, light brown, with fine sand				
20	CAL	7-17-26 (43)		Hard				

NOTES:
1. Bottom of boring at 21.5 feet.
2. No groundwater encountered.
3. Boring backfilled with soil cuttings 6/20/18.

TECHNICON Eningeering Services, INC.
4539 N Brawley Ave #108
Fresno, CA 93722
Telephone: 559-276-9311

BORING B 6

PROJECT NAME Proposed Industrial Building
PROJECT NUMBER 180431
PROJECT LOCATION 14800 W Schulte Road
SURFACE DESCRIPTION Paved Road
DATE STARTED 6/20/18 **COMPLETED** 6/20/18
GROUND ELEVATION
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc.
GROUND WATER LEVEL No groundwater encountered.
DRILL RIG TYPE CME 45
BORING DEPTH 16.5 ft
DRILLING METHOD 7.5-inch Hollow Stem Auger
LOGGED BY K. Weatherford **CHECKED BY** S. Alvarez

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
0				Asphalt				
				Silty CLAY (CL-ML) - medium stiff, light brown, moist, with fine sand				
	CAL	2-3-5 (8)			97.0	25.9	S = 97 %	
5				**CLAY (CL)** - stiff, dark brown, moist, high plasticity				
	SPT	2-4-8 (12)			109.6	16.1	S = 84 %	
10	CAL	5-9-16 (25)		Very stiff	102.7	20.2	S = 88 %	
15	SPT	5-8-6 (14)		**Silty SAND (SM)** - medium dense, light brown, moist, fine to medium grained				

NOTES:
1. Bottom of boring at 16.5 feet.
2. No groundwater encountered.
3. Boring backfilled with soil cuttings 6/20/18.

TECHNICON Eningeering Services, INC.
4539 N Brawley Ave #108
Fresno, CA 93722
Telephone: 559-276-9311

BORING B 7

PROJECT NAME Proposed Industrial Building
PROJECT NUMBER 180431
PROJECT LOCATION 14800 W Schulte Road
SURFACE DESCRIPTION Flat, Clear soil
DATE STARTED 6/20/18 **COMPLETED** 6/20/18
GROUND ELEVATION
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc.
GROUND WATER LEVEL No groundwater encountered.
DRILL RIG TYPE CME 45
BORING DEPTH 16.5 ft
DRILLING METHOD 7.5-inch Hollow Stem Auger
LOGGED BY K. Weatherford **CHECKED BY** S. Alvarez

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
0				Asphalt				
				Silty CLAY (CL-ML) - stiff, light brown, moist, with fine sand				
	CAL	4-6-7 (13)			102.5	21.6	S = 93 %	
				CLAY (CL) - stiff, dark brown, moist, high plasticity				
5	SPT	4-6-7 (13)						
10	CAL	4-7-12 (19)			98.0	22.9	S = 88 %	
				Sandy SILT (ML) - stiff, light brown, moist, with fine sand				
15	SPT	6-6-7 (13)						

NOTES:
1. Bottom of boring at 16.5 feet.
2. No groundwater encountered.
3. Boring backfilled with soil cuttings 6/20/18.

TECHNICON Eningeering Services, INC.
4539 N Brawley Ave #108
Fresno, CA 93722
Telephone: 559-276-9311

BORING B 8

PROJECT NAME Proposed Industrial Building
PROJECT NUMBER 180431
PROJECT LOCATION 14800 W Schulte Road
SURFACE DESCRIPTION Wood Chips
DATE STARTED 6/20/18 **COMPLETED** 6/20/18
GROUND ELEVATION
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc.
GROUND WATER LEVEL No groundwater encountered.
DRILL RIG TYPE CME 45
BORING DEPTH 11.5 ft
DRILLING METHOD 7.5-inch Hollow Stem Auger
LOGGED BY K. Weatherford **CHECKED BY** S. Alvarez

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
0				Wood Chips				
				CLAY (CL) - stiff, dark brown, moist, high plasticity				
	CAL	5-6-9 (15)			108.1	22.4	S = 112 %	
5				**Silty CLAY (CL-ML)** - stiff, light brown and white, moist, with fine sand				
	SPT	4-8-9 (17)						
10								
	CAL	9-16-27 (43)		Hard, moderate cementation	104.1	19.8	S = 89 %	

NOTES:
1. Bottom of boring at 11.5 feet.
2. No groundwater encountered.
3. Boring backfilled with soil cuttings 6/20/18.

TECHNICON Eningeering Services, INC.
4539 N Brawley Ave #108
Fresno, CA 93722
Telephone: 559-276-9311

BORING B 9

PROJECT NAME Proposed Industrial Building
PROJECT NUMBER 180431
PROJECT LOCATION 14800 W Schulte Road
SURFACE DESCRIPTION Flat, Clear soil
DATE STARTED 6/20/18 **COMPLETED** 6/20/18
GROUND ELEVATION
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc.
GROUND WATER LEVEL No groundwater encountered.
DRILL RIG TYPE CME 55
BORING DEPTH 16.5 ft
DRILLING METHOD 7.5-inch Hollow Stem Auger
LOGGED BY K. Weatherford **CHECKED BY** S. Alvarez

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
0	CAL	50		**Sandy GRAVEL (GP)** - very dense, black, moist, medium to coarse grained	107.3	16.5	S = 81 %	
				CLAY (CL) - stiff, dark brown, moist, high plasticity				
5	SPT	4-5-7 (12)						
10				**Silty CLAY (CL-ML)** - hard, light brown, moist, with fine sand				
	CAL	10-11-34 (45)			106.0	20.1	S = 95 %	
15	SPT	5-8-10 (18)		Very stiff				

NOTES:
1. Bottom of boring at 16.5 feet.
2. No groundwater encountered.
3. Boring backfilled with soil cuttings 6/20/18.

TECHNICON Eningeering Services, INC.
4539 N Brawley Ave #108
Fresno, CA 93722
Telephone: 559-276-9311

BORING B10

PROJECT NAME Proposed Industrial Building
PROJECT NUMBER 180431
PROJECT LOCATION 14800 W Schulte Road
SURFACE DESCRIPTION Burned Wood Chips
DATE STARTED 6/20/18 **COMPLETED** 6/20/18
GROUND ELEVATION
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc.
GROUND WATER LEVEL No groundwater encountered.
DRILL RIG TYPE CME 55
BORING DEPTH 21.5 ft
DRILLING METHOD 7.5-inch Hollow Stem Auger
LOGGED BY K. Weatherford **CHECKED BY** S. Alvarez

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
0	CAL	18-21-36 (57)	Burned Wood Chips				
			Asphalt				
			Burned Wood Chips				
5	CAL	8-12-17 (29)					
			Sandy CLAY (CL) - very stiff, light brown, moist, with fine sand	112.1	15.2	S = 84 %	
10			**Silty CLAY (CL-ML)** - very stiff, light brown, moist, with fine sand				
	SPT	5-7-11 (18)					
15	CAL	4-12-20 (32)					
20	SPT	12-11-9 (20)	**Sandy SILT (ML)** - very stiff, light brown, moist, with fine sand				

NOTES:
1. Bottom of boring at 21.5 feet.
2. No groundwater encountered.
3. Boring backfilled with soil cuttings 6/20/18.

BORING B11

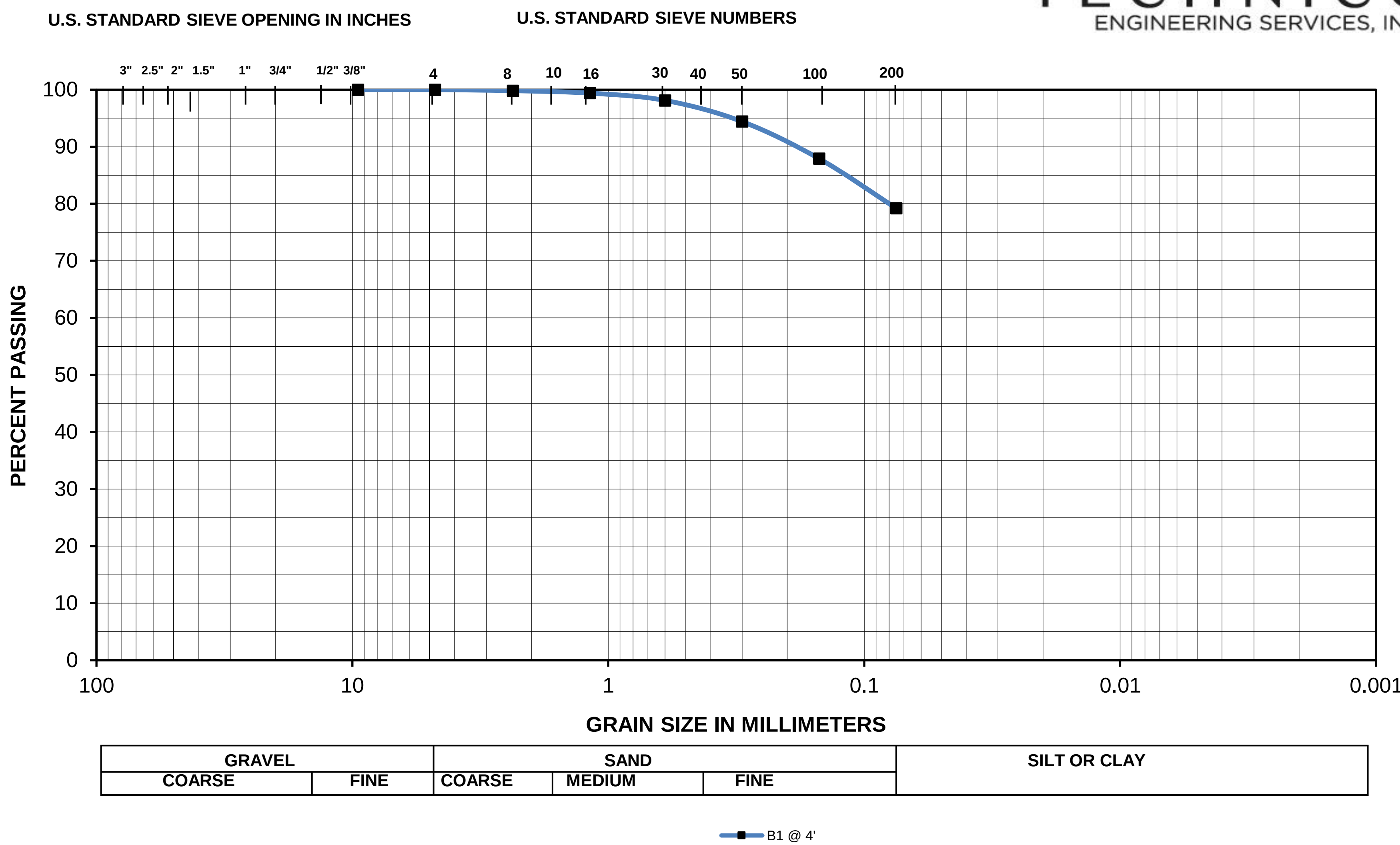
TECHNICON Eningeering Services, INC.
4539 N Brawley Ave #108
Fresno, CA 93722
Telephone: 559-276-9311

PROJECT NAME Proposed Industrial Building
PROJECT NUMBER 180431
PROJECT LOCATION 14800 W Schulte Road
SURFACE DESCRIPTION Asphalt
DATE STARTED 6/20/18 **COMPLETED** 6/20/18
GROUND ELEVATION
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc.
GROUND WATER LEVEL No groundwater encountered.
DRILL RIG TYPE CME 55
BORING DEPTH 16.5 ft
DRILLING METHOD 7.5-inch Hollow Stem Auger
LOGGED BY K. Weatherford **CHECKED BY** S. Alvarez

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
0				Asphalt				
	CAL	15-10-16 (26)		**CLAY (CL)** - very stiff, dark brown, moist, high plasticity	82.5	20.6	S = 54 %	
5	SPT	3-3-5 (8)		**Silty CLAY (CL-ML)** - stiff, light brown, moist, with fine sand				
10	CAL	7-12-17 (29)		Very stiff	103.0	21.4	S = 93 %	
15	SPT	5-12-8 (20)		**Sandy SILT (ML)** - very stiff, light brown, moist, with fine sand				

NOTES:
1. Bottom of boring at 16.5 feet.
2. No groundwater encountered.
3. Boring backfilled with soil cuttings 6/20/18.

APPENDIX C: LABORATORY TEST PROGRAM BY TECHNICON (2018)



Sample No.	Classification	% Gravel	% Sand	% Fines	% Moist.	LL	PL	PI	Project	Industrial Building
B1 @ 4'	Sandy CLAY (CL)	0	20.8	79.2	21.6					Tracy, CA
									TES No.	180431
									Date	6/22/2018

TECHNICON
ENGINEERING SERVICES, INC.

Sieve Analysis for Coarse and Fine Aggregate
ASTM C 136

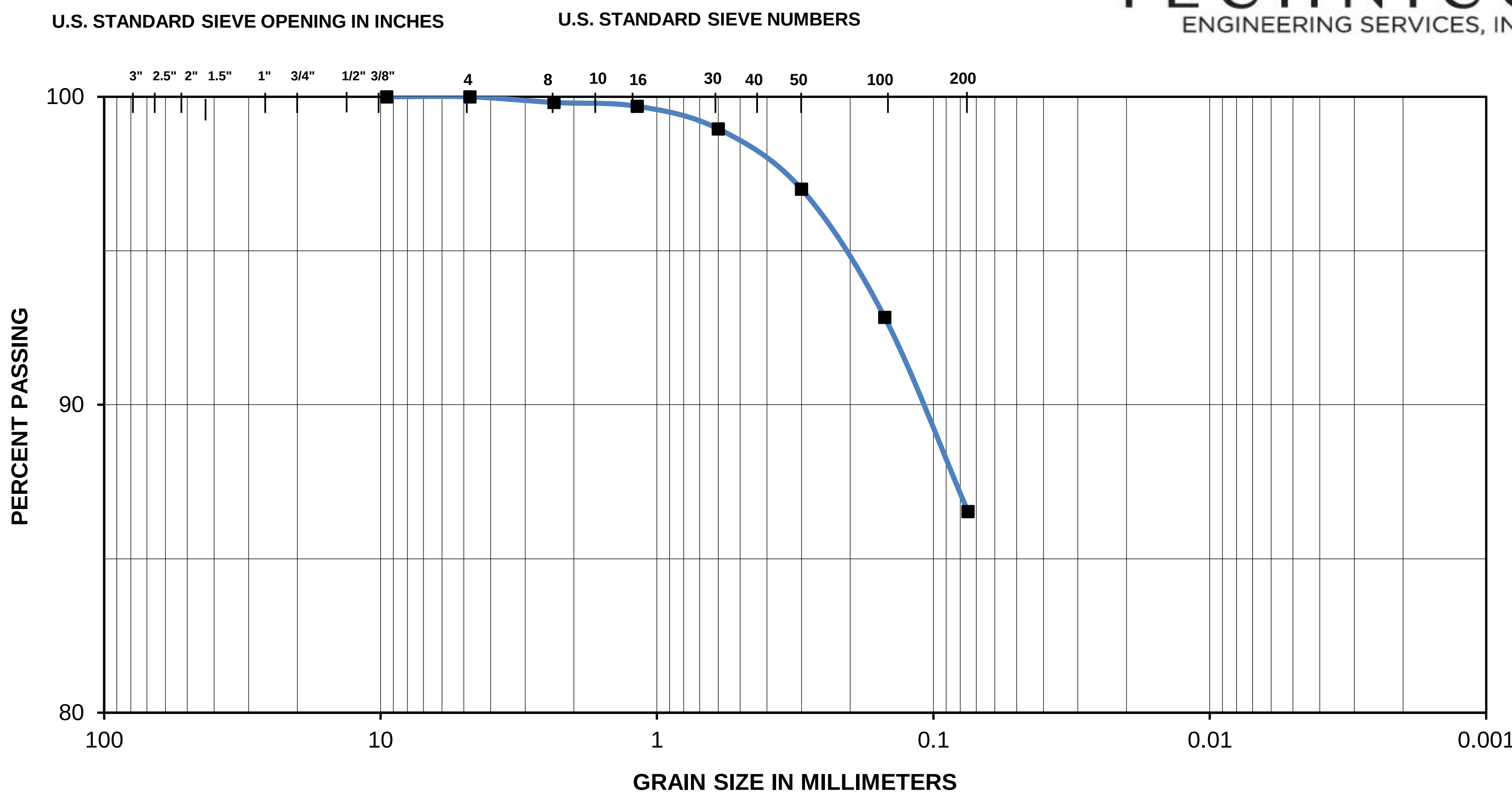
Project	Industrial Building	Technician	WJ
	Tracy, CA	Date	6/22/2018
TES No.	180431	Sample No.	B1 @ 4'
Lab No.		Remarks	Sandy CLAY (CL)

	Weight (lbs. or grams)	Maximum Sieve Size	Minimum Weight of Test Specimen, lbs. (kg)
Total Dry Sample + Tare Wt.		Sand	1.0 (0.5)
Tare Weight		3/8"	2.0 (1.0)
Total Dry Sample Wt.	164.5	1/2"	4.0 (2.0)
Initial Weight Fine Aggregate Before Wash		3/4"	11.0 (5.0)
		1"	22.0 (10.0)
Final Weight Fine Aggregate After Wash		1 1/2"	33.0 (15.0)
	36.32	2"	44.0 (20.0)

Sieve Size	Cumulative Weight Retained	Individual % Retained	Cumulative % Retained	Cumulative % Passing	Specs.
3 in.		0.0	0.0	100.0	
2 1/2 in.		0.0	0.0	100.0	
2 in.		0.0	0.0	100.0	
1 1/2 in.		0.0	0.0	100.0	
1 in.		0.0	0.0	100.0	
3/4 in.		0.0	0.0	100.0	
1/2 in.		0.0	0.0	100.0	
3/8 in.		0.0	0.0	100.0	
#4	0.0	0.0	0.0	100.0	
#8	0.3	0.2	0.2	99.8	
#16	1.0	0.4	0.6	99.4	
#30	3.1	1.3	1.9	98.1	
#50	9.2	3.7	5.6	94.4	
#100	19.9	6.5	12.1	87.9	
#200	34.2	8.7	20.8	79.2	
Pan					

4539 N. Brawley Avenue, #108, Fresno, CA 93722
Phone (559) 276-9311 Fax (559) 276-9344

TECHNICON
ENGINEERING SERVICES, INC.



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

B8 @ 3'

Sample No.	Classification	% Gravel	% Sand	% Fines	% Moist.	LL	PL	PI	Project	Industrial Building
B8 @ 3'	Sandy CLAY (CL)	0	13.5	86.5	22.4					Tracy, CA
									TES No.	180431
									Date	6/22/2018

TECHNICON
ENGINEERING SERVICES, INC.

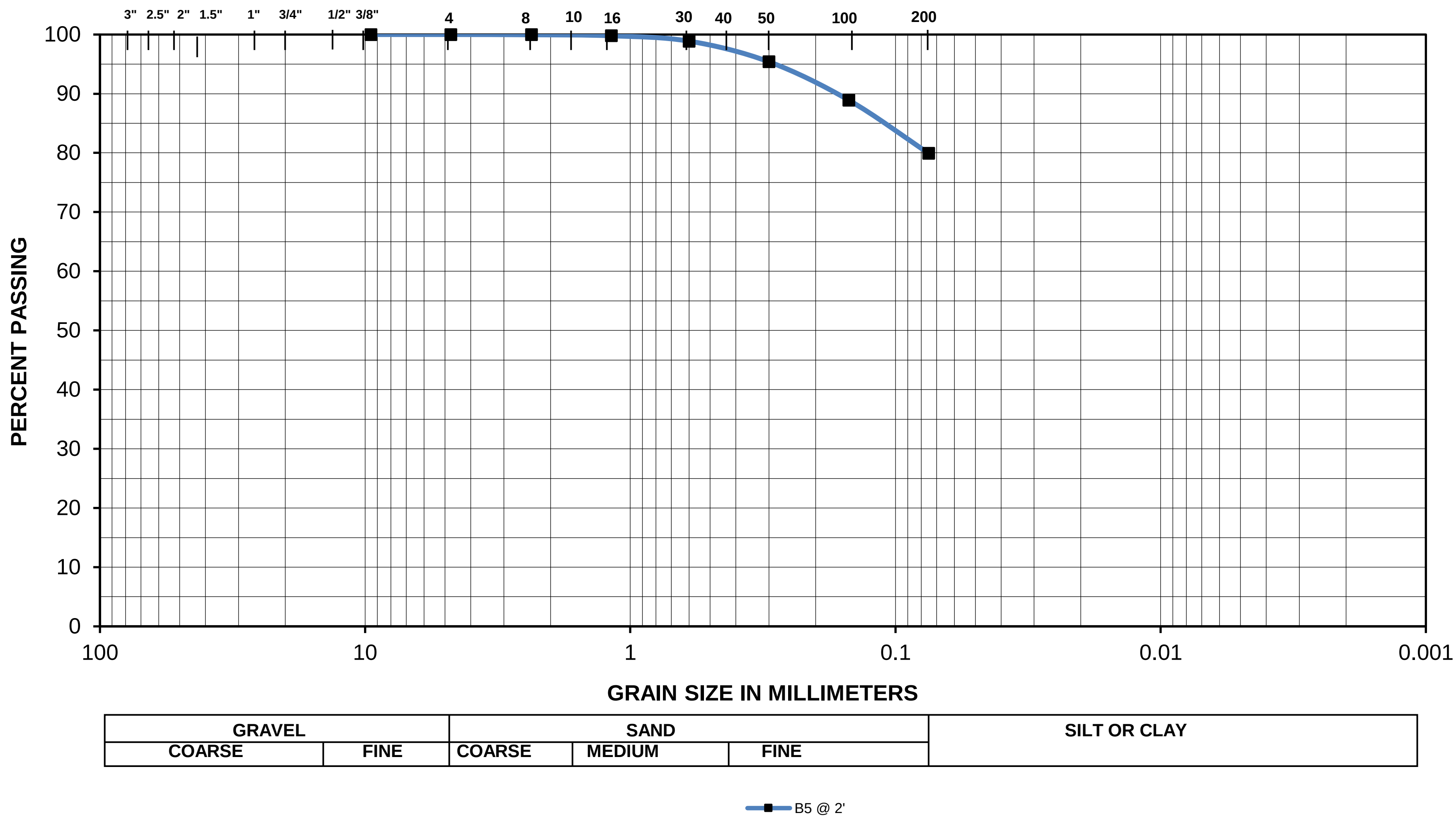
Sieve Analysis for Coarse and Fine Aggregate
ASTM C 136

Project	Industrial Building	Technician	WJ
	Tracy, CA	Date	6/22/2018
TES No.	180431	Sample No.	B8 @ 3'
Lab No.		Remarks	Sandy CLAY (CL)

	Weight (lbs. or grams)	Maximum Sieve Size	Minimum Weight of Test Specimen, lbs. (kg)
Total Dry Sample + Tare Wt.		Sand	1.0 (0.5)
Tare Weight		3/8"	2.0 (1.0)
Total Dry Sample Wt.	163.4	1/2"	4.0 (2.0)
Initial Weight Fine Aggregate Before Wash		3/4"	11.0 (5.0)
		1"	22.0 (10.0)
Final Weight Fine Aggregate After Wash	23.51	1 1/2"	33.0 (15.0)
		2"	44.0 (20.0)

Sieve Size	Cumulative Weight Retained	Individual % Retained	Cumulative % Retained	Cumulative % Passing	Specs.
3 in.		0.0	0.0	100.0	
2 1/2 in.		0.0	0.0	100.0	
2 in.		0.0	0.0	100.0	
1 1/2 in.		0.0	0.0	100.0	
1 in.		0.0	0.0	100.0	
3/4 in.		0.0	0.0	100.0	
1/2 in.		0.0	0.0	100.0	
3/8 in.		0.0	0.0	100.0	
#4	0.0	0.0	0.0	100.0	
#8	0.3	0.2	0.2	99.8	
#16	0.5	0.1	0.3	99.7	
#30	1.7	0.7	1.0	99.0	
#50	4.9	2.0	3.0	97.0	
#100	11.7	4.2	7.2	92.8	
#200	22.0	6.3	13.5	86.5	
Pan					

4539 N. Brawley Avenue, #108, Fresno, CA 93722
Phone (559) 276-9311 Fax (559) 276-9344



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Classification	% Gravel	% Sand	% Fines	% Moist.	LL	PL	PI	Project	Industrial Building
B5 @ 2'	Sandy CLAY (CL)	0	20.1	79.9	13.0					Tracy, CA
									TES No.	180431
									Date	6/22/2018

TECHNICON
ENGINEERING SERVICES, INC.

Sieve Analysis for Coarse and Fine Aggregate
ASTM C 136

Project	Industrial Building	Technician	WJ
	Tracy, CA	Date	6/22/2018
TES No.	180431	Sample No.	B5 @ 2'
Lab No.		Remarks	Sandy CLAY (CL)

	Weight (lbs. or grams)	Maximum Sieve Size	Minimum Weight of Test Specimen, lbs. (kg)
Total Dry Sample + Tare Wt.		Sand	1.0 (0.5)
Tare Weight		3/8"	2.0 (1.0)
Total Dry Sample Wt.	177.0	1/2"	4.0 (2.0)
Initial Weight Fine Aggregate Before Wash		3/4"	11.0 (5.0)
		1"	22.0 (10.0)
Final Weight Fine Aggregate After Wash		1 1/2"	33.0 (15.0)
	38	2"	44.0 (20.0)

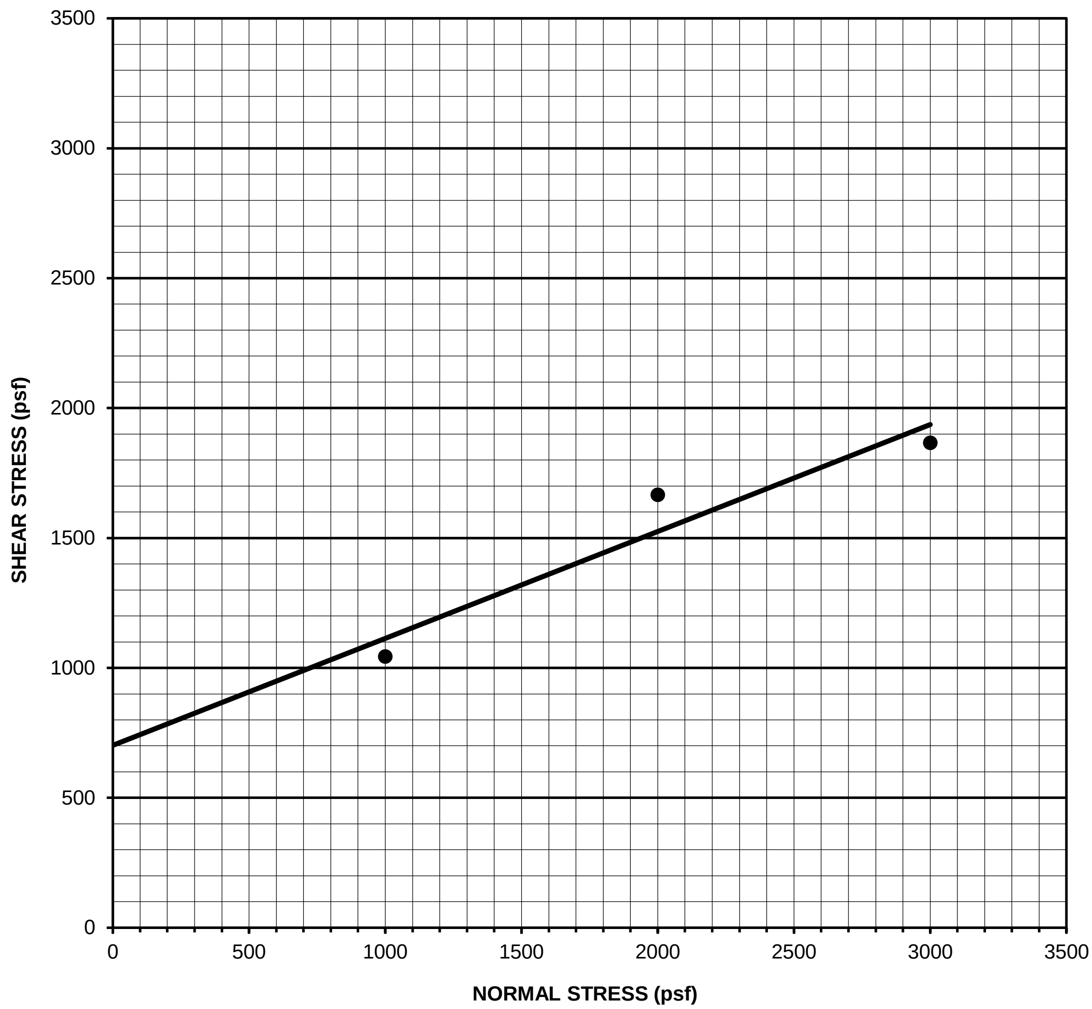
Sieve Size	Cumulative Weight Retained	Individual % Retained	Cumulative % Retained	Cumulative % Passing	Specs.
3 in.		0.0	0.0	100.0	
2 1/2 in.		0.0	0.0	100.0	
2 in.		0.0	0.0	100.0	
1 1/2 in.		0.0	0.0	100.0	
1 in.		0.0	0.0	100.0	
3/4 in.		0.0	0.0	100.0	
1/2 in.		0.0	0.0	100.0	
3/8 in.		0.0	0.0	100.0	
#4	0.0	0.0	0.0	100.0	
#8	0.1	0.0	0.0	100.0	
#16	0.4	0.2	0.2	99.8	
#30	2.0	0.9	1.1	98.9	
#50	8.2	3.5	4.6	95.4	
#100	19.6	6.5	11.1	88.9	
#200	35.5	9.0	20.1	79.9	
Pan					

4539 N. Brawley Avenue, #108, Fresno, CA 93722
Phone (559) 276-9311 Fax (559) 276-9344

TECHNICON
ENGINEERING SERVICES, INC.

Direct Shear Test

ASTM D3080



Project	Industrial Building
TES No.	180431
Sample Date	6/20/2018
Sample No.	B5 @ 2'
Description	Sandy CLAY (CL)

Cohesion (psf)	**700**
Internal Friction Angle (ϕ)	**22**

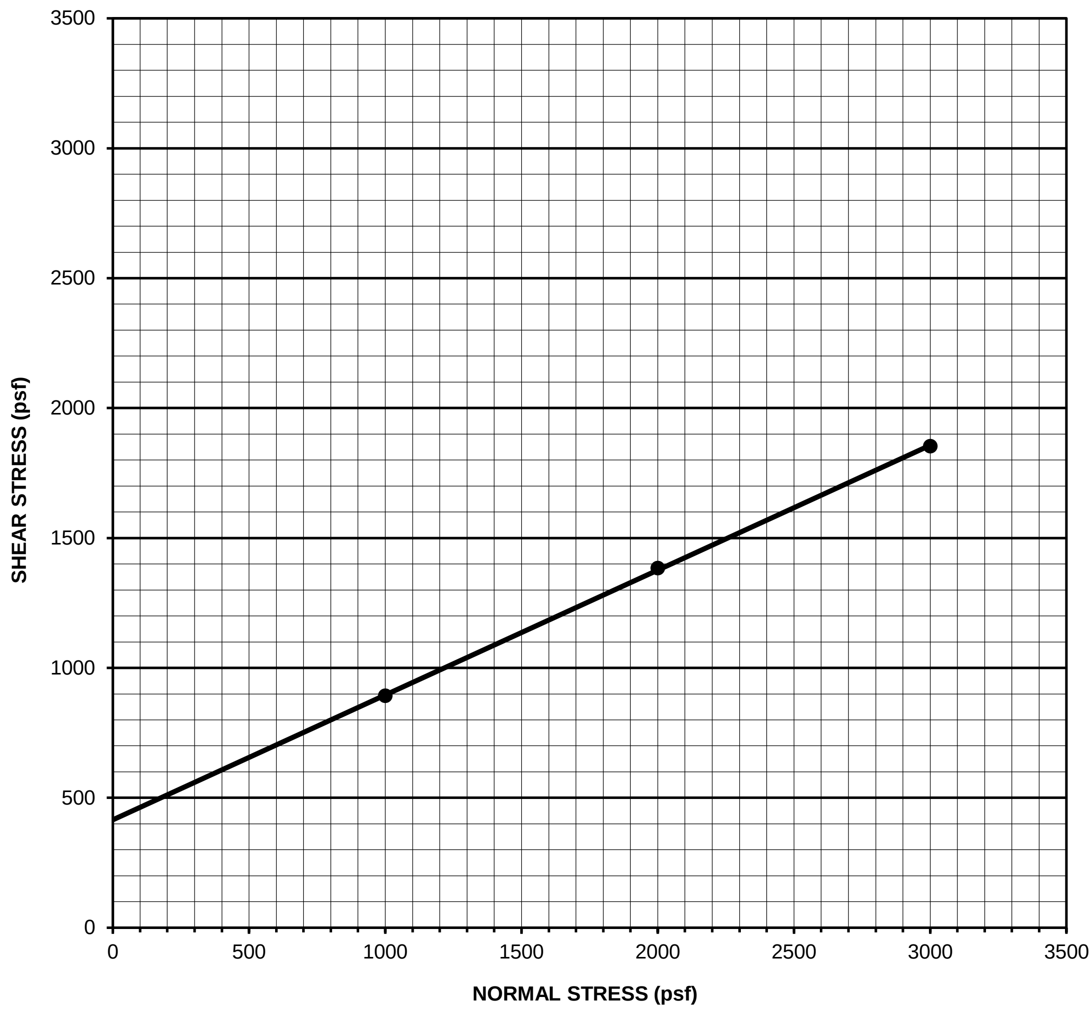
Specimen	A	B	C	D	E
Dry Density (pcf)	114.2	114.2	114.2	---	---
Initial Water Content (%)	13.0	13.0	13.0	---	---
Final Water Content (%)	24.0	20.6	21.9	---	---
Normal Stress (pcf)	1000	2000	3000	---	---
Maximum Shear (pcf)	1043	1666	1866	---	---

Engineering Materials Laboratory
4539 N. Brawley #108, Fresno, CA 93722
559-276-9311

TECHNICON
ENGINEERING SERVICES, INC.

Direct Shear Test

ASTM D3080



Project	Industrial Building
TES No.	180431
Sample Date	6/20/2018
Sample No.	B8 @ 3'
Description	Sandy CLAY (CL)

Cohesion (psf)	**420**
Internal Friction Angle (ϕ)	**26**

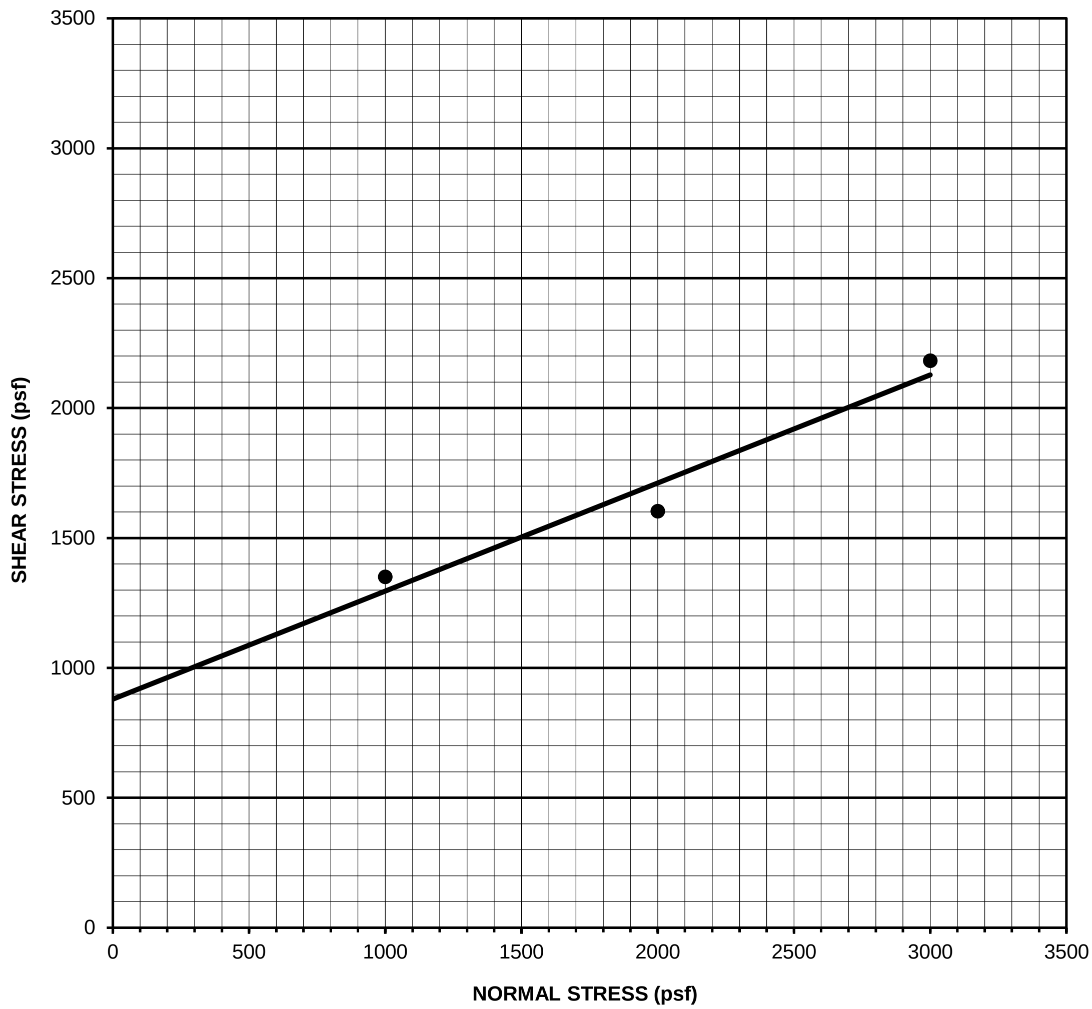
Specimen	A	B	C	D	E
Dry Density (pcf)	108.1	108.1	108.1	---	---
Initial Water Content (%)	22.4	22.4	22.4	---	---
Final Water Content (%)	28.9	25.5	26.5	---	---
Normal Stress (pcf)	1000	2000	3000	---	---
Maximum Shear (pcf)	892	1384	1853	---	---

Engineering Materials Laboratory
4539 N. Brawley #108, Fresno, CA 93722
559-276-9311

TECHNICON
ENGINEERING SERVICES, INC.

Direct Shear Test

ASTM D3080



Project	Industrial Building
TES No.	180431
Sample Date	6/20/2018
Sample No.	B1 @ 4'
Description	Sandy CLAY (CL)

Cohesion (psf)	**880**
Internal Friction Angle (ϕ)	**23**

Specimen	A	B	C	D	E
Dry Density (pcf)	103.7	103.7	103.7	---	---
Initial Water Content (%)	21.6	21.6	21.6	---	---
Final Water Content (%)	25.5	24.2	22.8	---	---
Normal Stress (pcf)	1000	2000	3000	---	---
Maximum Shear (pcf)	1350	1603	2182	---	---

Engineering Materials Laboratory
4539 N. Brawley #108, Fresno, CA 93722
559-276-9311

TECHNICON
ENGINEERING SERVICES, INC.

Method for Estimating the Service Life of Steel Culverts
Caltrans California Test 643

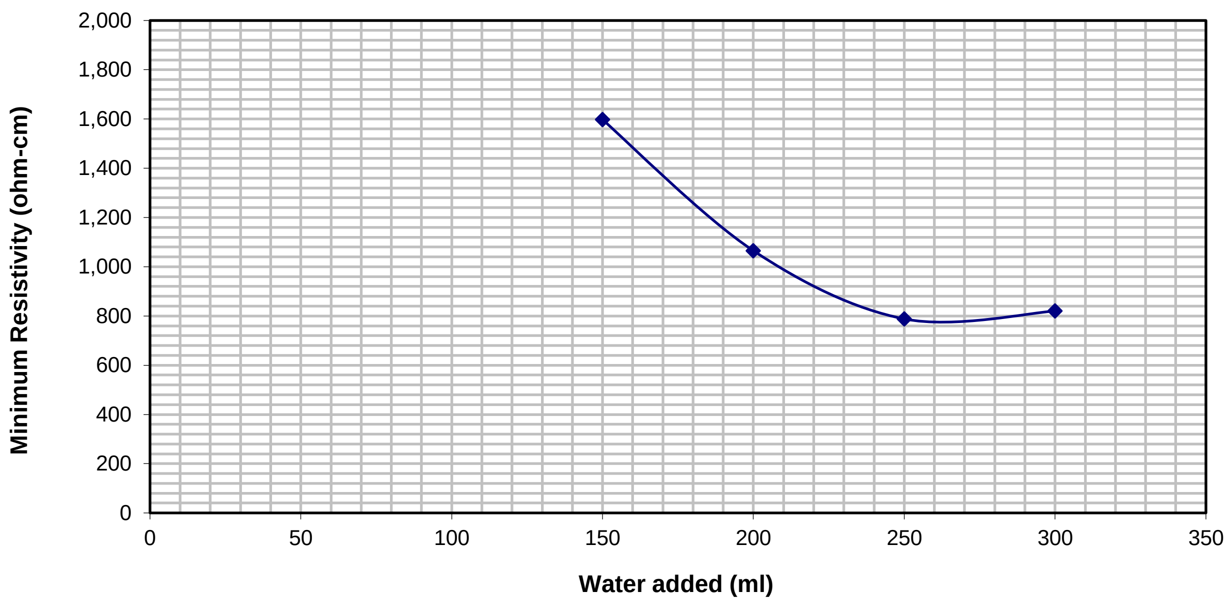
Project Name	Industrial Building	Sample Location	RV-4
Project Number	180431	Test Date	6/26/2018
Sample Date	6/19/2018	Tested By	K. Weatherford
Sampled By	K. Weatherford	Material Description	Sandy CLAY (CL)

Sample Condition	**As Received**	**Minimum Resistivity**					
Water Added (ml)	0	150	200	250	300		
Resistance (ohm)	1,000,000	1,500	1,000	740	770		
Resistivity (ohm-cm)	1,065,000	1,598	1,065	788	820		

Minimum Resistivity (ohm-cm)	**788**	**Field Resistivity (ohm-cm)**

pH = 7.94 **EC =**

Box Constant=1.065



Years to perforation*	22

* Caltrans California Test 643 - Method for Estimating the Service Life of Steel Culverts

TECHNICON
ENGINEERING SERVICES, INC.

Chemical Analysis
SO_4 - Modified Caltrans 417 & CL - Modified Caltrans 417/422

Project	Industrial Building	Technician	K.Weatherford
	Tracy, CA	Date	3/22/2017
TES No.	180431	Remarks	Sandy CLAY (CL)

Sample Location	Soluble Sulfate SO_4-S		Soluble Chloride Cl	
RV-4	57.1	mg/Kg	16	mg/Kg
RV-4	58.4	mg/Kg	17.7	mg/Kg
RV-4	58	mg/Kg	17.7	mg/Kg
Average	**57.83**	**mg/Kg**	**17.13**	**mg/Kg**

4539 N. Brawley Avenue, #108, Fresno, CA 93722
Phone (559) 276-9311 Fax (559) 276-9344

TECHNICON
ENGINEERING SERVICES, INC.
*Construction Testing & Inspection * Geotechnical & Environmental Engineering*

Expansion Index Test
UBC Standard 29-2 / ASTM D4829

Project: Industrial Building
Tracy, CA
TES No. 180431
Lab No.

Technician: WJ
Date: 7/2/2018
Sample No. RV-7
Remarks: Sandy CLAY (CL)

Water Added (ml)	Dry Back			
Wt. Of Soil + Mold (g)	730.1			
Wt. of Mold (g)	367.0			
Wt. of Soil (g)	363.1			
Wt. of Soil (lb)	0.800			
Wet Density (pcf)	109.5			
Moisture Sample, wet (g)	200.0			
Moisture Sample, dry (g)	173.4			
Moisture Content (%)	15.3			
Dry Density (pcf)	94.9			
Specific Gravity	2.7			
Degree of Saturation (%)	**53.5**			

Time	Dial Reading
7/2/18 1:00	0.0000
7/2/18 2:00	0.0684
7/3/18 1:00	0.0798
FINAL	0.0798

Expansion meas. = 0.0798
Exp. Index meas. = 79.8
Exp. Index 50 = 82.8

EXPANSION INDEX = 82.8

Expansion Potential Table	
Expansion Index	Potential Expansion
0-20	Very Low
21-50	Low
51-90	**Medium**
91-130	High
>130	Very High

Engineering Materials Laboratory
4539 N. Brawley Avenue, #108, Fresno, CA 93722
Phone (559) 276-9311 Fax (559) 276-9344

TECHNICON
ENGINEERING SERVICES, INC.
*Construction Testing & Inspection * Geotechnical & Environmental Engineering*

Expansion Index Test
UBC Standard 29-2 / ASTM D4829

Project: Industrial Building
Tracy, CA
TES No. 180431
Lab No.

Technician: WJ
Date: 7/2/2018
Sample No. RV-7
Remarks: Sandy CLAY (CL)

Water Added (ml)	Dry Back			
Wt. Of Soil + Mold (g)	740.2			
Wt. of Mold (g)	368.6			
Wt. of Soil (g)	371.6			
Wt. of Soil (lb)	0.819			
Wet Density (pcf)	112.1			
Moisture Sample, wet (g)	200.0			
Moisture Sample, dry (g)	173.6			
Moisture Content (%)	15.2			
Dry Density (pcf)	97.3			
Specific Gravity	2.7			
Degree of Saturation (%)	**56.0**			

Time	Dial Reading
7/2/18 1:00	0.0000
7/2/18 2:00	0.0702
7/3/18 1:00	0.0782
FINAL	0.0782

Expansion meas. = 0.0782
Exp. Index meas. = 78.2
Exp. Index 50 = 83.5

EXPANSION INDEX = 83.5

Expansion Potential Table

Expansion Index	Potential Expansion
0-20	Very Low
21-50	Low
51-90	**Medium**
91-130	High
>130	Very High

Engineering Materials Laboratory
4539 N. Brawley Avenue, #108, Fresno, CA 93722
Phone (559) 276-9311 Fax (559) 276-9344

TECHNICON
ENGINEERING SERVICES, INC.

Determination of Atterberg Limits

ASTM D 4318, CTM 204

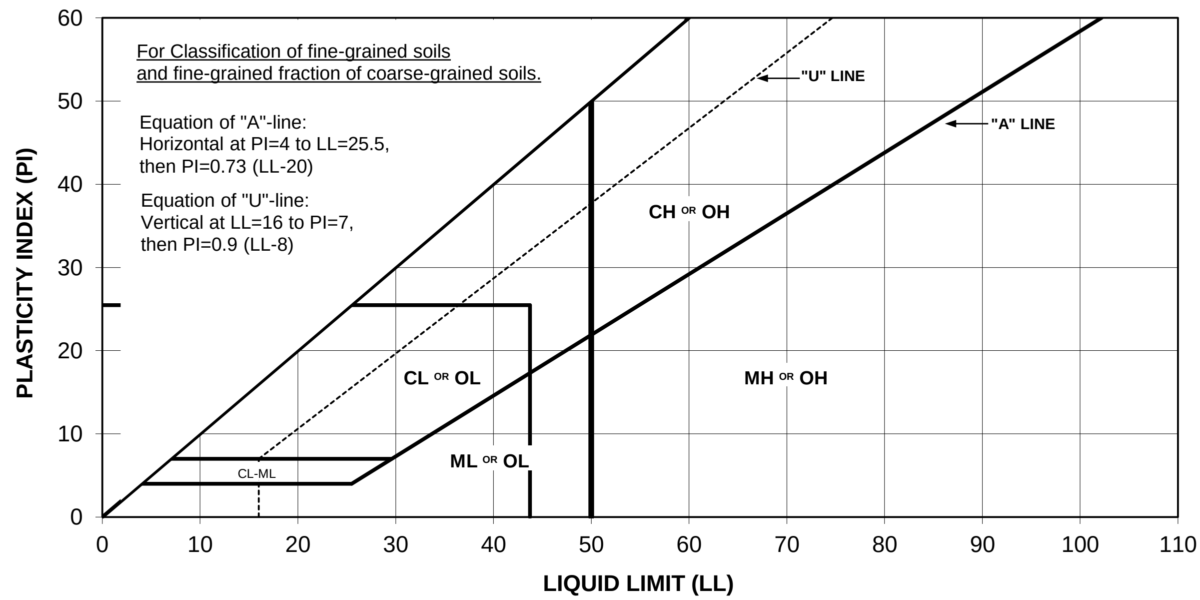
Project Name	Industrial Building	Project No.	180431
Sample Location	B6 @ 5'	Tested By	WJ
Soil Classification	Sandy CLAY (CL)	Date	7/5/18

		PLASTIC LIMIT				LIQUID LIMIT			
A	Tes No.	1	2	3	No. of Blows	17	26	32	
B	Tare No.								
C	Mass of Pan + Dry Soil, g	23.53	23.24	23.97		33.24	26.04	26.29	
D	Mass of Pan + Wet Soil, g	24.08	23.79	24.63		35.10	28.21	28.78	
E	Mass of Pan, g	20.52	20.20	20.39		29.23	20.99	20.34	
F	Mass of Water, g	0.55	0.55	0.66		1.86	2.17	2.49	
G	Mass of Dry Soil, g	3.01	3.04	3.58		4.01	5.05	5.95	
H	Moisture Content, %	18.27	18.09	18.44		46.38	42.97	41.85	
I	Average Moisture Content, % (PL)		18.27						

Liquid Limit: Read from graph	**43.7**
Plastic Limit: Line I	**18.3**
Plasticity Index: PI = LL - PL	**25.5**

FLOW CURVE

ASTM D2487



TECHNICON
ENGINEERING SERVICES, INC.

Organic Content of Soils

ASTM D 2974

Project Number	: 180431
Project Name	: Tracy Industrial Building
Date	: 6/26/2018
Sample # / Location	: RV-6
Technician	: KW

Sample/Lot/Boring #		
Mass of Wet Sample + Tare, 0.01g		200.0
Mass of Wet Sample, 0.01g (100-300g)		200.0
Mass of Tare, g		0.0
Mass of Dry Sample + Tare, 0.01g		166.75
Mass of Dry Sample, g		166.8
Mass of Water, g		33.3
Moisture Content, %		19.9%
Mass of Dry Porcelain Dish, 0.01g	A	116.75
Mass of Soil (50g) + Dish Before Incineration, 0.01g	B	166.75
Mass of Soil + Dish After Incineration, 0.01g	C	163.33
Organic Content, 0.01% (B-C)/(B-A)		6.84%

TECHNICON
ENGINEERING SERVICES, INC.

Organic Content of Soils

ASTM D 2974

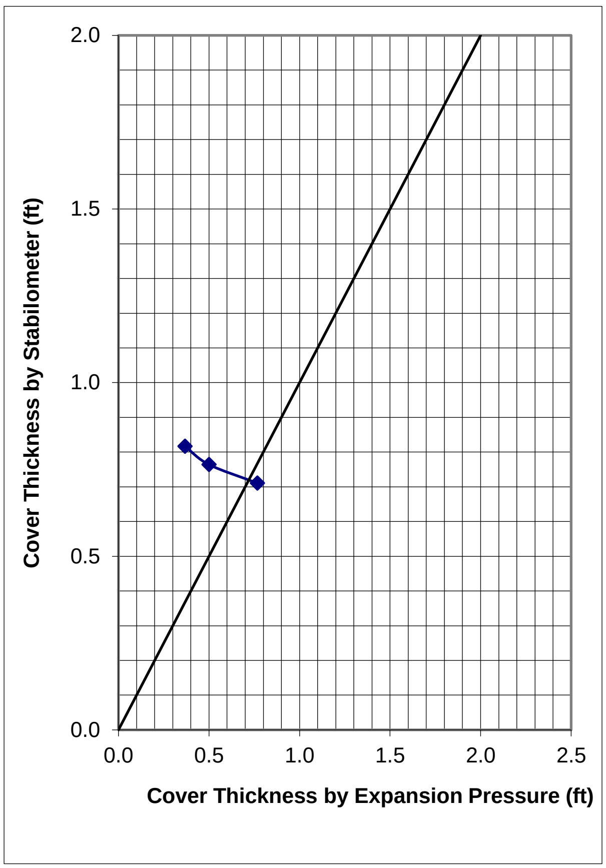
Project Number	: 180431
Project Name	: Tracy Industrial Building
Date	: 6/26/2018
Sample # / Location	: RV-4
Technician	: KW

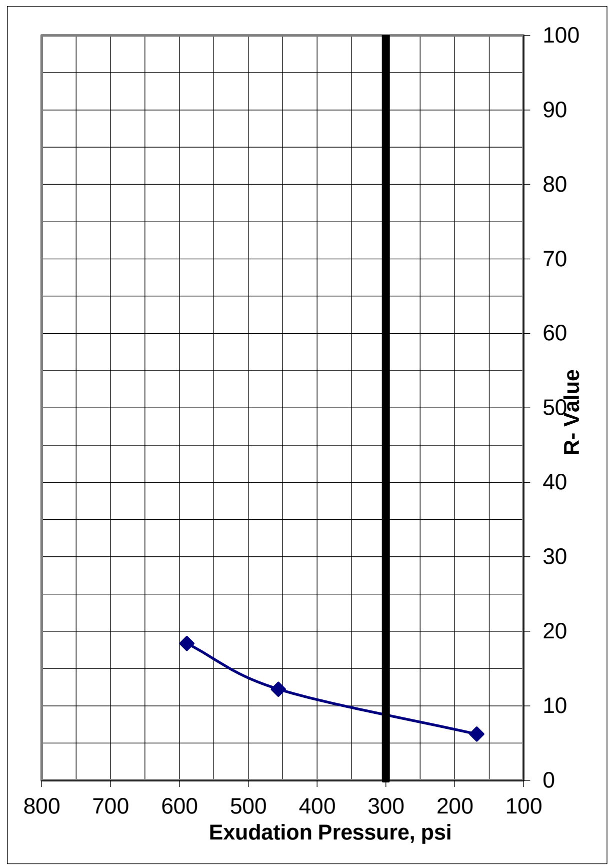
Sample/Lot/Boring #		
Mass of Wet Sample + Tare, 0.01g		189.1
Mass of Wet Sample, 0.01g (100-300g)		189.1
Mass of Tare, g		0.0
Mass of Dry Sample + Tare, 0.01g		164.39
Mass of Dry Sample, g		164.4
Mass of Water, g		24.7
Moisture Content, %		15.0%
Mass of Dry Porcelain Dish, 0.01g	A	114.39
Mass of Soil (50g) + Dish Before Incineration, 0.01g	B	164.39
Mass of Soil + Dish After Incineration, 0.01g	C	161.18
Organic Content, 0.01% (B-C)/(B-A)		6.42%

Resistance *R* - Value and Expansion Pressure of Compacted Soils

ASTM D2844-94, Cal 301

Project Name	Industrial Building, Tracy	Lab ID Number	18-231
Project Number	180431	Sample Location	RV-1
Sample Date	6/19/18	Tested By	FM
Sampled By	KW	Date Tested	7/5/2018
Material Description	Sandy CLAY (CL)		





Specimen	1	2	3
Exudation Pressure, psi	589	456	168
Moisture at Test, %	17.8	19.6	22.9
Dry Density, pcf	104.8	101.1	95.1
Expansion Pressure, psf	100	65	48
Thickness by Stabilometer, ft.	0.7	0.8	0.8
Thickness by Expansion Pressure, ft.	0.8	0.5	0.4
R-Value by Stabilometer	18	12	6
R-Value by Expansion Pressure (TI=4.5)	NA		
R-Value at 300 psi Exudation Pressure	8		

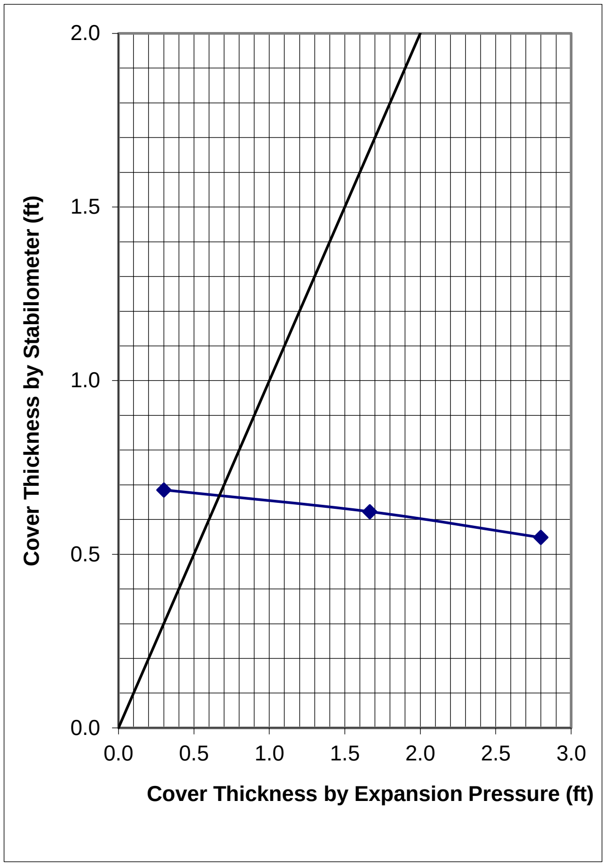
Controlling R-Value	**8**

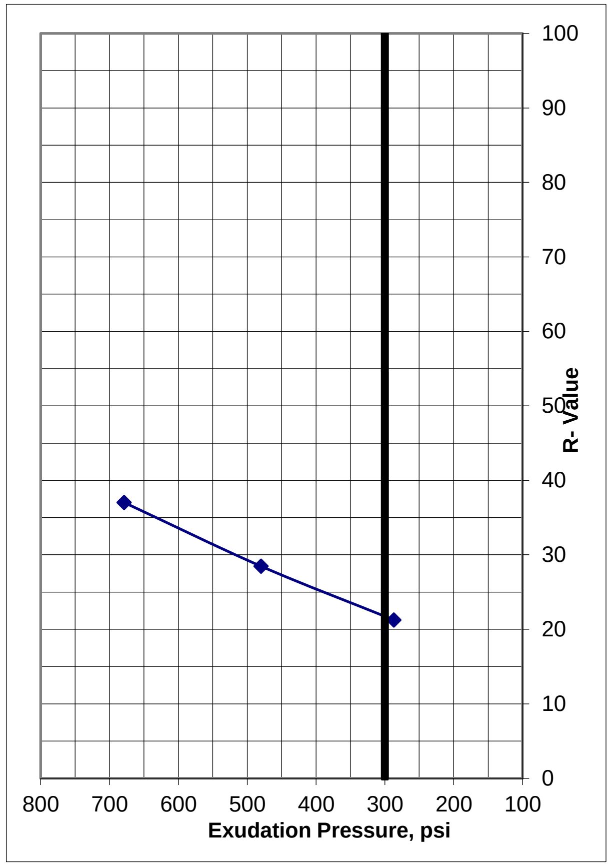
TECHNICON
ENGINEERING SERVICES, INC.

Resistance *R* - Value and Expansion Pressure of Compacted Soils

ASTM D2844-94, Cal 301

Project Name	Industrial Building, Tracy	Lab ID Number	18-231
Project Number	180431	Sample Location	RV-2
Sample Date	6/19/18	Tested By	FM
Sampled By	KW	Date Tested	6/29/2018
Material Description	Sandy CLAY (CL)		





Specimen	1	2	3
Exudation Pressure, psi	679	480	287
Moisture at Test, %	24.2	27.1	30.5
Dry Density, pcf	85.4	80.1	77.9
Expansion Pressure, psf	364	217	39
Thickness by Stabilometer, ft.	0.5	0.6	0.7
Thickness by Expansion Pressure, ft.	2.8	1.7	0.3
R-Value by Stabilometer	37	28	21
R-Value by Expansion Pressure (TI=4.5)	21		
R-Value at 300 psi Exudation Pressure	22		

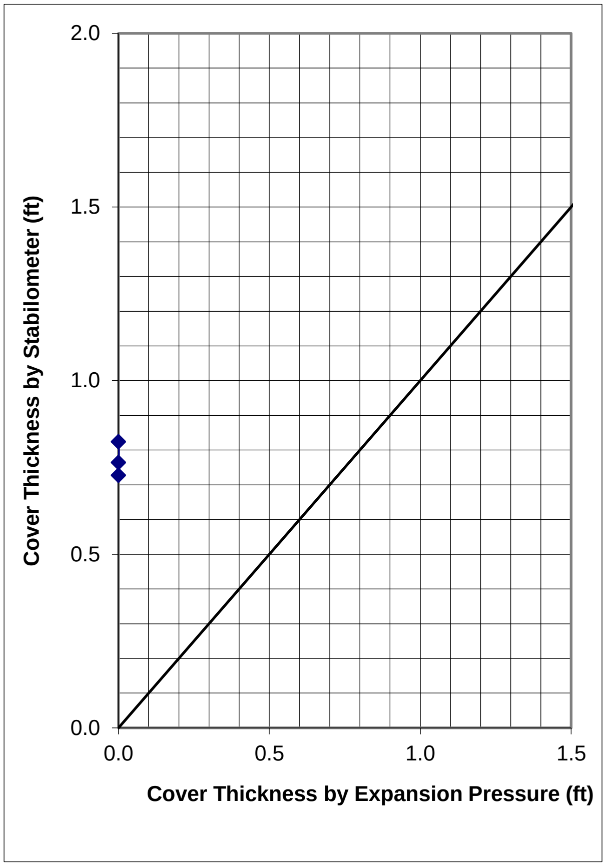
Controlling R-Value	**21**

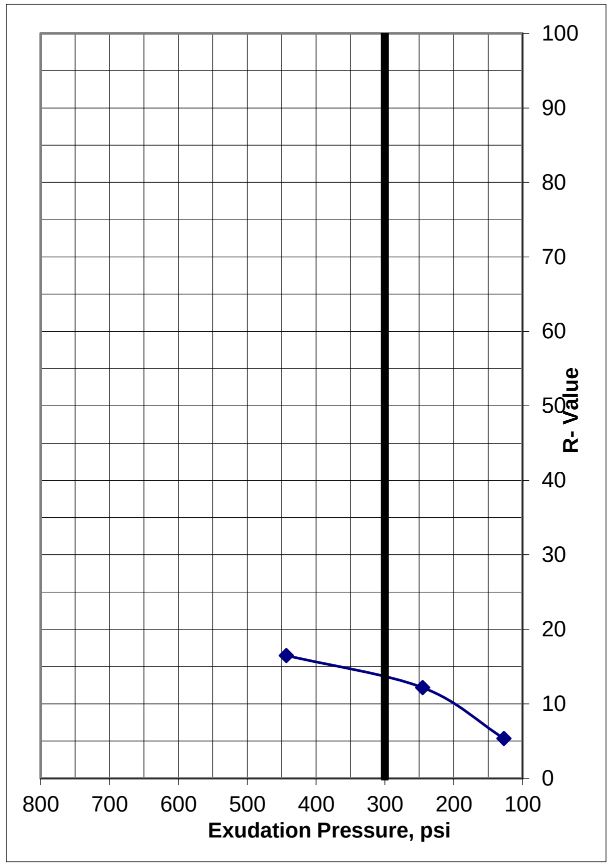
Engineering Materials Laboratory
4539 N. Brawley #108, Fresno, CA, 93722
WWW.TECHNICON.NET

Resistance *R* - Value and Expansion Pressure of Compacted Soils

ASTM D2844-94, Cal 301

Project Name	Industrial Building, Tracy	Lab ID Number	18-231
Project Number	180431	Sample Location	RV-3
Sample Date	6/19/18	Tested By	FM
Sampled By	KW	Date Tested	6/29/2018
Material Description	Sandy CLAY (CL)		





Specimen	1	2	3
Exudation Pressure, psi	443	245	127
Moisture at Test, %	14.3	15.9	19.1
Dry Density, pcf	113.8	110.4	105.6
Expansion Pressure, psf	0	0	0
Thickness by Stabilometer, ft.	0.7	0.8	0.8
Thickness by Expansion Pressure, ft.	0.0	0.0	0.0
R-Value by Stabilometer	16	12	5
R-Value by Expansion Pressure (TI=4.5)	NA		
R-Value at 300 psi Exudation Pressure	14		

Controlling R-Value	**14**

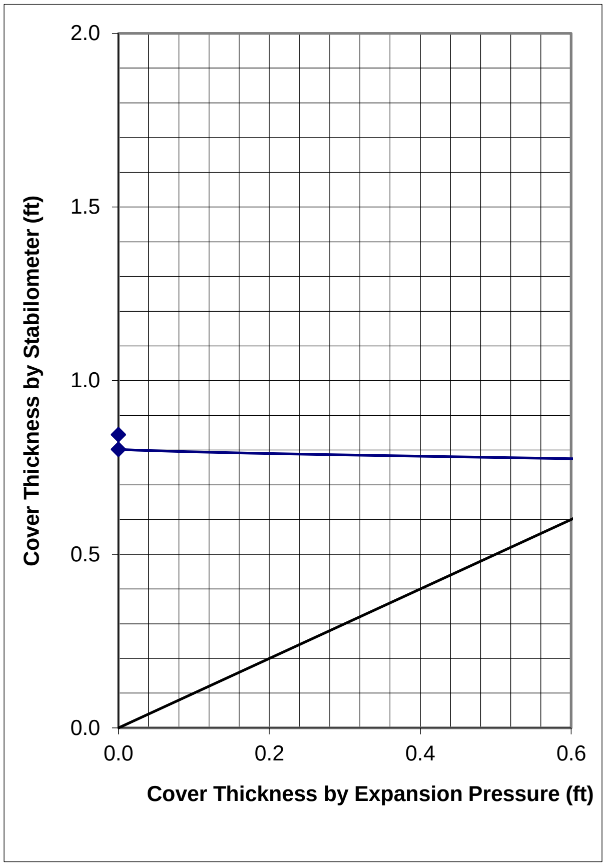
Engineering Materials Laboratory
4539 N. Brawley #108, Fresno, CA, 93722
WWW.TECHNICON.NET

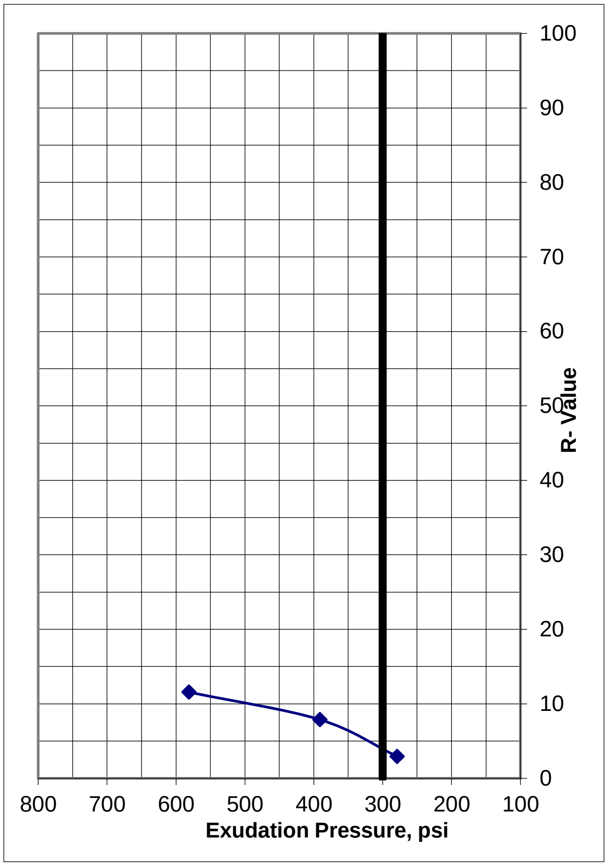
TECHNICON ENGINEERING SERVICES, INC.

Resistance *R* - Value and Expansion Pressure of Compacted Soils

ASTM D2844-94, Cal 301

Project Name	Industrial Building, Tracy	Lab ID Number	18-231
Project Number	180431	Sample Location	RV-4
Sample Date	6/19/18	Tested By	FM
Sampled By	KW	Date Tested	6/27/2018
Material Description	Sandy CLAY (CL)		





Specimen	1	2	3
Exudation Pressure, psi	581	391	279
Moisture at Test, %	20.3	22.4	25.4
Dry Density, pcf	98.7	97.7	91.8
Expansion Pressure, psf	95	0	0
Thickness by Stabilometer, ft.	0.8	0.8	0.8
Thickness by Expansion Pressure, ft.	0.7	0.0	0.0
R-Value by Stabilometer	12	8	3
R-Value by Expansion Pressure (TI=4.5)	NA		
R-Value at 300 psi Exudation Pressure	5		

Controlling R-Value	**5**

Engineering Materials Laboratory

4539 N. Brawley #108, Fresno, CA, 93722

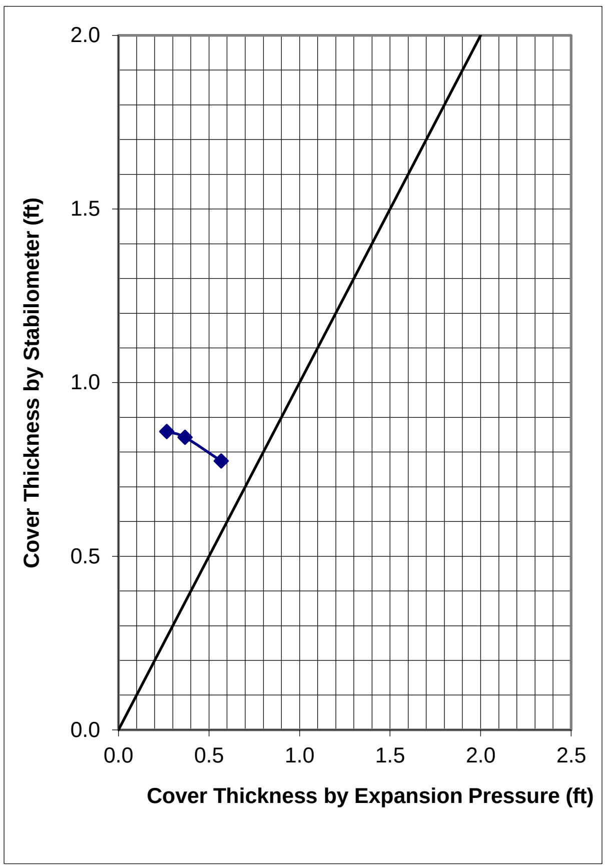
WWW.TECHNICON.NET

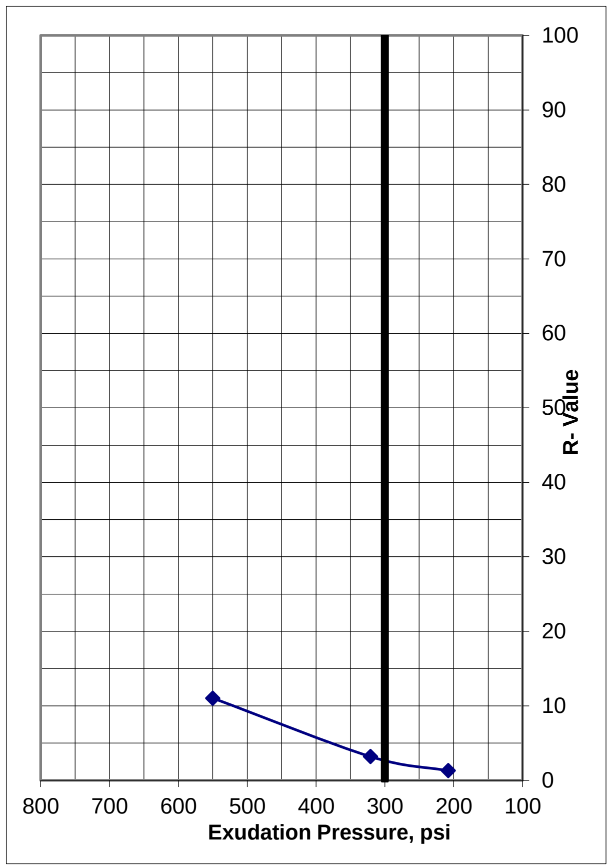
TECHNICON
ENGINEERING SERVICES, INC.

Resistance *R* - Value and Expansion Pressure of Compacted Soils

ASTM D2844-94, Cal 301

Project Name	Industrial Building, Tracy	Lab ID Number	18-231
Project Number	180431	Sample Location	RV-5
Sample Date	6/19/18	Tested By	FM
Sampled By	KW	Date Tested	7/9/2018
Material Description	Sandy CLAY (CL)		





Specimen	1	2	3
Exudation Pressure, psi	550	321	208
Moisture at Test, %	21.5	25.7	30.1
Dry Density, pcf	98.8	92.5	83.4
Expansion Pressure, psf	74	48	35
Thickness by Stabilometer, ft.	0.8	0.8	0.9
Thickness by Expansion Pressure, ft.	0.6	0.4	0.3
R-Value by Stabilometer	11	3	1
R-Value by Expansion Pressure (TI=4.5)	NA		
R-Value at 300 psi Exudation Pressure	5		

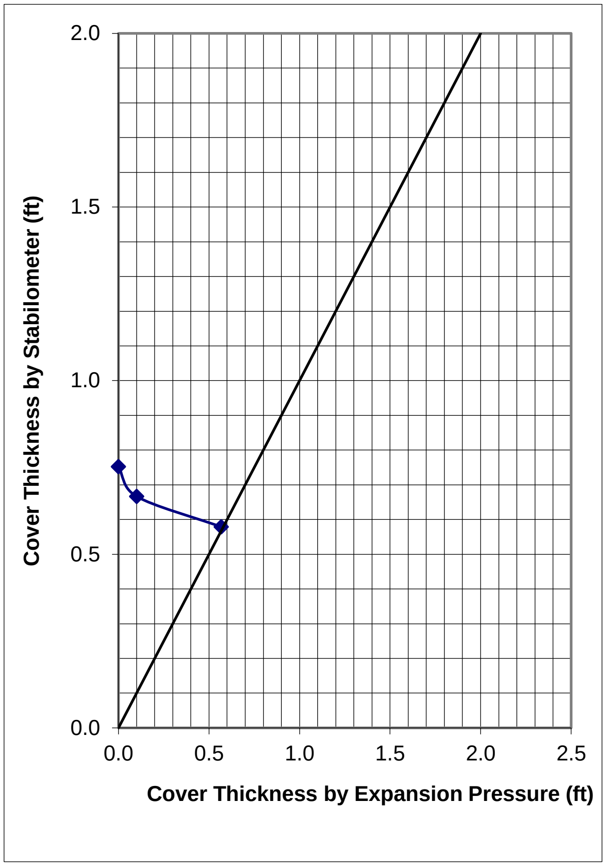
Controlling R-Value	**5**

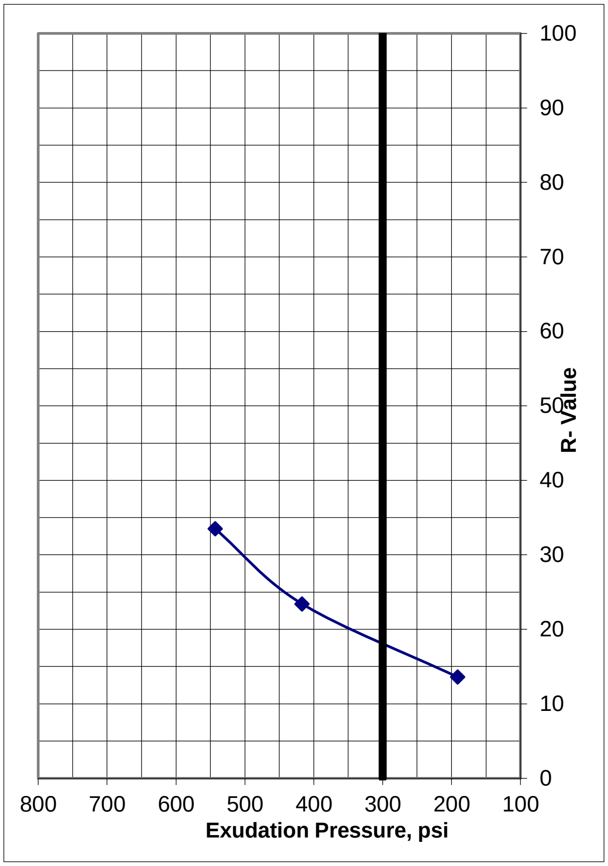
Engineering Materials Laboratory
4539 N. Brawley #108, Fresno, CA, 93722
WWW.TECHNICON.NET

TECHNICON
ENGINEERING SERVICES, INC.

Resistance *R* - Value and Expansion Pressure of Compacted Soils
ASTM D2844-94, Cal 301

Project Name	Industrial Building, Tracy	Lab ID Number	18-231
Project Number	180431	Sample Location	RV-6
Sample Date	6/19/18	Tested By	FM
Sampled By	KW	Date Tested	6/27/2018
Material Description	Sandy CLAY (CL)		





Specimen	1	2	3
Exudation Pressure, psi	543	417	191
Moisture at Test, %	15.5	17.3	19.4
Dry Density, pcf	107.5	105.9	101.3
Expansion Pressure, psf	74	13	0
Thickness by Stabilometer, ft.	0.6	0.7	0.8
Thickness by Expansion Pressure, ft.	0.6	0.1	0.0
R-Value by Stabilometer	33	23	14
R-Value by Expansion Pressure (TI=4.5)	NA		
R-Value at 300 psi Exudation Pressure	17		

Controlling R-Value	**17**

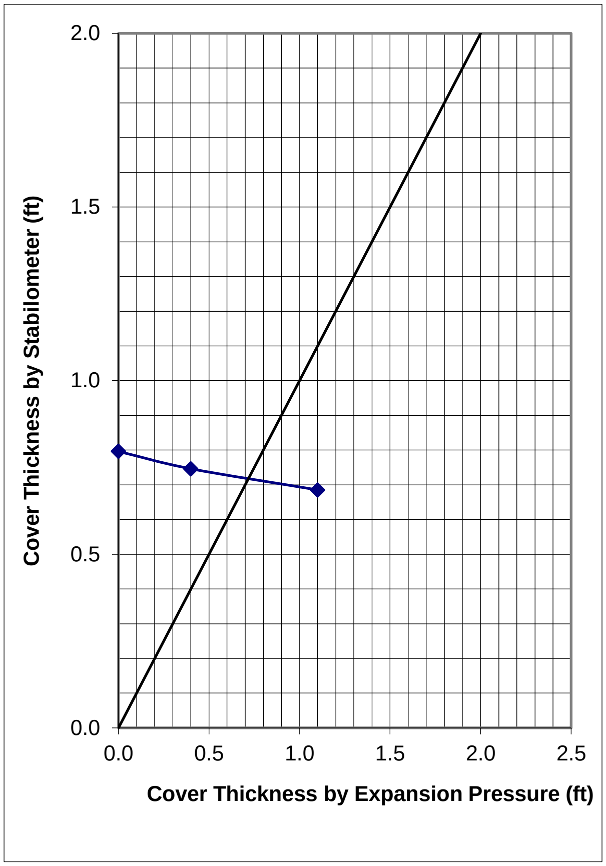
Engineering Materials Laboratory
4539 N. Brawley #108, Fresno, CA, 93722
WWW.TECHNICON.NET

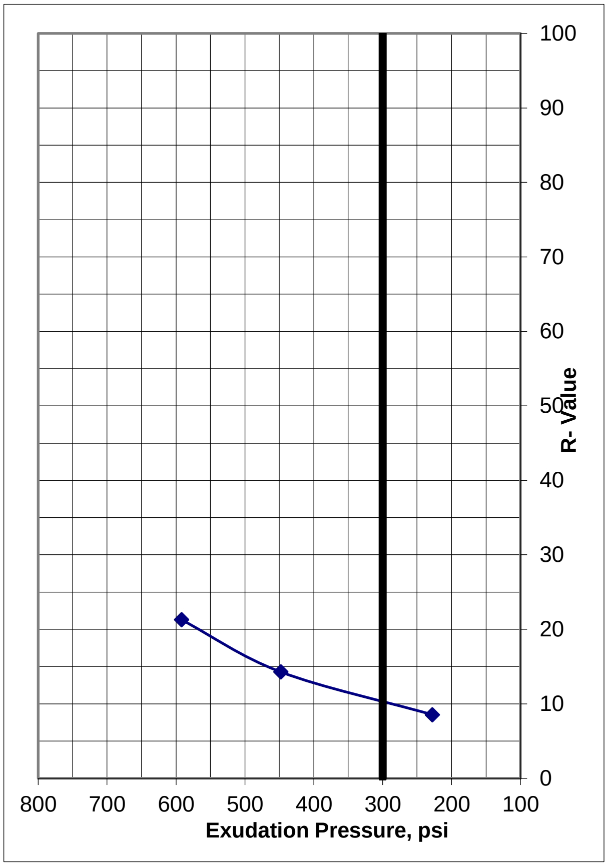
TECHNICON
ENGINEERING SERVICES, INC.

Resistance *R* - Value and Expansion Pressure of Compacted Soils

ASTM D2844-94, Cal 301

Project Name	Industrial Building, Tracy	Lab ID Number	18-231
Project Number	180431	Sample Location	RV-7
Sample Date	6/19/18	Tested By	FM
Sampled By	KW	Date Tested	7/5/2018
Material Description	Sandy CLAY (CL)		





Specimen	1	2	3
Exudation Pressure, psi	592	448	228
Moisture at Test, %	19.2	20.8	23.6
Dry Density, pcf	99.9	97.7	92.5
Expansion Pressure, psf	143	52	0
Thickness by Stabilometer, ft.	0.7	0.7	0.8
Thickness by Expansion Pressure, ft.	1.1	0.4	0.0
R-Value by Stabilometer	21	14	9
R-Value by Expansion Pressure (TI=4.5)	NA		
R-Value at 300 psi Exudation Pressure	10		

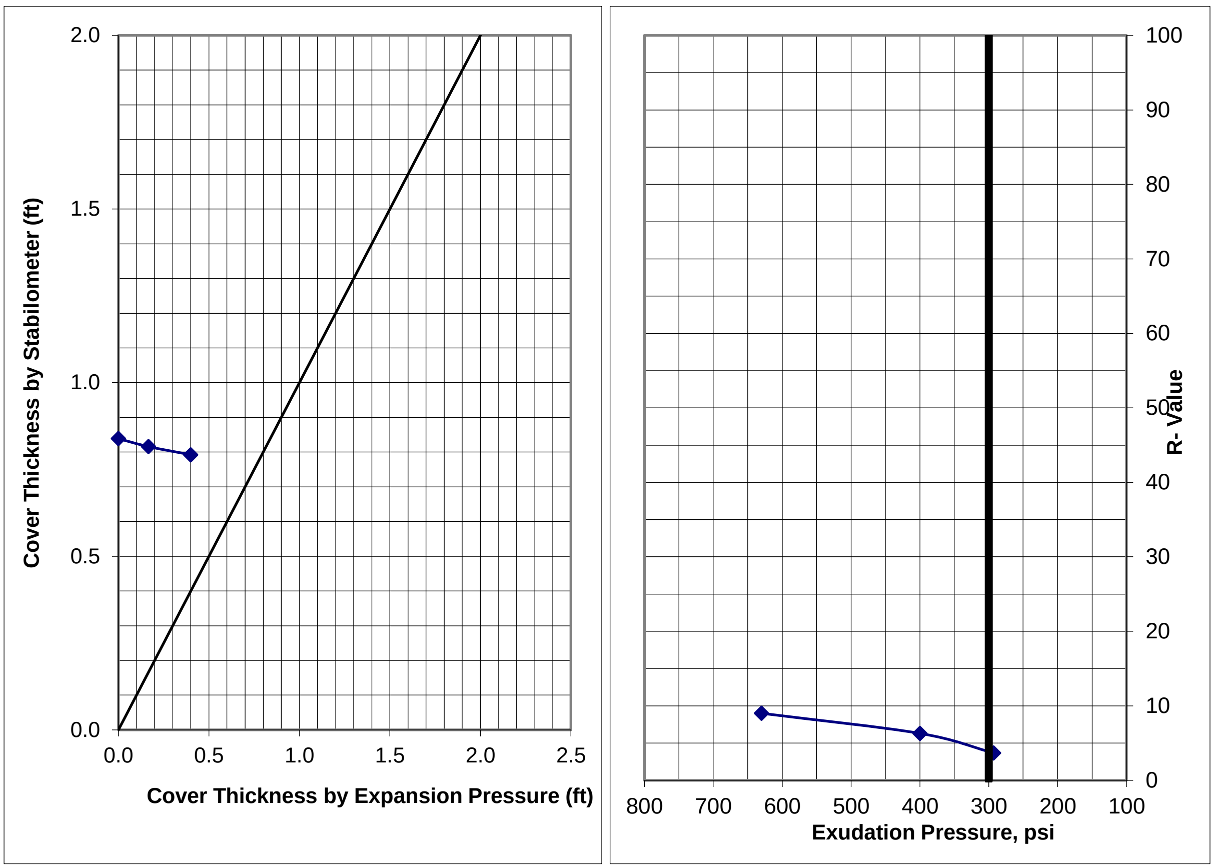
Controlling R-Value	**10**

TECHNICON
ENGINEERING SERVICES, INC.

Resistance *R* - Value and Expansion Pressure of Compacted Soils

ASTM D2844-94, Cal 301

Project Name	Industrial Building, Tracy	Lab ID Number	18-231
Project Number	180431	Sample Location	RV-8
Sample Date	6/19/18	Tested By	FM
Sampled By	KW	Date Tested	7/5/2018
Material Description	Sandy CLAY (CL)		



Specimen	1	2	3
Exudation Pressure, psi	630	400	293
Moisture at Test, %	16.9	19.3	21.9
Dry Density, pcf	108.3	103.1	97.4
Expansion Pressure, psf	52	22	0
Thickness by Stabilometer, ft.	0.8	0.8	0.8
Thickness by Expansion Pressure, ft.	0.4	0.2	0.0
R-Value by Stabilometer	9	6	4
R-Value by Expansion Pressure (TI=4.5)	NA		
R-Value at 300 psi Exudation Pressure	5		

Controlling R-Value	**5**

Engineering Materials Laboratory
4539 N. Brawley #108, Fresno, CA, 93722
WWW.TECHNICON.NET