DOCKETED

Docket Number:	22-SPPE-02
Project Title:	San Jose Data Center 04
TN #:	245955
Document Title:	San Jose Data Center 04 - SPPE Application - Appendices D and E
Description:	N/A
Filer:	Scott Galati
Organization:	DayZenLLC
Submitter Role:	Applicant Representative
Submission Date:	9/9/2022 2:50:56 PM
Docketed Date:	9/9/2022

Geotechnical Engineering Report

Bay AZ1 Site (Northtown)
San Jose, California
December 15, 2020
Terracon Project No. ND205079

Prepared for:
Burns & McDonnell Engineering Company, Inc.
Columbus, Ohio

Prepared by:
Terracon Consultants, Inc.
Concord, California

December 15, 2020

Burns & McDonnell Engineering Company, Inc.
530 W. Spring Street, Suite 200
Columbus, Ohio 43215

Attn: Ms. Brandi Sauter
P: (614) 499 1009
E: blsauter@burnsmcd.com

Re: Geotechnical Engineering Report
Bay AZ1 Site (Northtown)
Orchard Parkway
San Jose, California
Terracon Project No. ND205079

Dear Ms. Sauter:

We have completed the Geotechnical Engineering services for the above referenced project. This study was performed in general accordance with Terracon Proposal No. PND205079 dated November 13, 2020. This report presents the findings of the subsurface exploration and provides geotechnical recommendations concerning earthwork and the design and construction of foundations, pavements, and floor slabs for the proposed project.

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report or if we may be of further service, please contact us.

Sincerely,
Terracon Consultants, Inc.

REGISTERED PROFESSIONAL ENGINEER
NOAH T. SMITH
No. 2950
12/15/2020
GEOTECHNICAL
STATE OF CALIFORNIA

Noah T. Smith, P.E., G.E.
Principal

Ryan Feist
Assistant Service Line Director

SME Review: Brett E. Bradfield

REPORT TOPICS

Note: This report was originally delivered in a web-based format. **Orange Bold** text in the report indicates a referenced section heading. The PDF version also includes hyperlinks which direct the reader to that section and clicking on the ***GeoReport*** logo will bring you back to this page. For more interactive features, please view your project online at client.terracon.com.

ATTACHMENTS

Note: Refer to each individual Attachment for a listing of contents.

REPORT SUMMARY

Topic [1]	Overview Statement [2]
Project Description	The project site is currently undeveloped. The project will consist of the construction of (2) five-story data center buildings. Development will include a substation, a water tank, stormwater retention facilities, and access roads and parking areas. ■ Columns: 400 to 500 kips (assumed) ■ Walls: 7 to 8 kips per linear foot (assumed)
Geotechnical Characterization	Subgrade soils encountered in our borings and CPTs generally consisted of 15 to 30 feet of lean to fat clay with variable amounts of sand underlain by about 5 to 20 feet of sand with variable amounts of clay. The sand was followed by lean to fat clay to the maximum depth explored of 100½ feet below the existing ground surface (bgs) with a 15 to 25-foot-thick layer of sand encountered at a depth of approximately 45 feet bgs. Groundwater was encountered in the borings at depths ranging from 5 to 29 feet bgs and in the CPT soundings a depth of 4 to 10 feet bgs at the time they were performed.
Earthwork	Cuts up to 15 feet and up to 10 feet of fill will be placed across the property to elevate the site above the design flood risk elevation. The near surface soils have moderate to high plasticity and are sensitive to moisture variation. The upper 18 inches of subgrade below soil supported floor slabs and exterior hardscapes should consist of chemically treated soil or Low Volume Change (LVC) structural fill. Grading should be conducted in accordance with the **Earthwork** section of this report.
Foundations	We estimate potential liquefaction settlement up to 2 inches across the site and the placement of fill will trigger up to 6 inches of settlement depending on the final fill thickness and location. In order to mitigate the effects of the total and associated differential settlements on the proposed buildings, substation and water tank from liquefaction and fill placement, we have provided the following two options for foundation support for the proposed improvements. Option 1 – **Shallow Foundations** over Ground Improvement Option 2 – **Deep Foundations** consisting of either auger cast piles or driven piles Recommendations for the design of each option are provided in the **Foundations** section of this report.
Retaining Structures	Building elevator pits. **Lateral Earth Pressures** have been provided for use in design.
Pavements	Pavement sections are provided for both rigid and flexible pavements. Alternate pavement sections utilizing chemical treatment are provided.
General Comments	This section contains important information about the limitations of this geotechnical engineering report.

1. If the reader is reviewing this report as a pdf, the topics above can be used to access the appropriate section of the report by simply clicking on the topic itself.
2. This summary is for convenience only. It should be used in conjunction with the entire report for design purposes.

Geotechnical Engineering Report

Bay AZ1 Site (Northtown)
Orchard Parkway
San Jose, California
Terracon Project No. ND205079
December 15, 2020

INTRODUCTION

This report presents the results of our subsurface exploration and geotechnical engineering services performed for the proposed data center facility to be located at Orchard Parkway in San Jose, California. The purpose of these services is to provide information and geotechnical engineering recommendations relative to:

- Subsurface soil conditions
- Groundwater conditions
- Site preparation and earthwork
- Seismic site classification per 2019 CBC
- Foundation design and construction
- Floor slab design and construction
- Liquefaction analysis
- Percolation testing results
- Thermal resistivity
- Field electrical resistivity
- Soil corrosivity

The geotechnical engineering Scope of Services for this project included the advancement of twenty-three test borings to depths ranging from approximately 5 to 51½ feet below existing site grades (bgs). Additionally, eleven Cone Penetration Test (CPT) soundings were advanced to depths varying from 50½ and 100½ feet bgs. Geophysical surveys consisting of four field electrical resistivity tests and a Multichannel Analysis of Surface Waves survey were performed on the site. Four percolation tests were also performed at a depth of 5 feet bgs.

Maps showing the site and exploration locations are shown in the **Site Location** and **Exploration Plan** sections, respectively. The results of the laboratory testing performed on soil samples obtained from the site during the field exploration are included on the boring logs and as separate graphs in the **Exploration Results** section.

SITE CONDITIONS

The following description of site conditions was derived from our site visit in association with the field exploration and our review of publicly available geologic and topographic maps.

Item	Description
Parcel Information	The proposed project is located on Orchard Parkway in San Jose, California. The project parcel encompasses approximately 22 acres. 37.3781°N 121.9332°W (approximate) See **Site Location**
Existing Improvements	The project site currently consists of undeveloped land.
Current Ground Cover	Earthen and grasses.
Existing Topography (per the Conceptual Grading Plan prepared by HMH, dated 11/4/2020)	The site is relatively flat. However, the property varies in elevation from about 48.32 feet to about 26.5 feet above Mean Sea Level (MSL) due to the presence of a mound near the northwest edge of the property.
Geology	Geologic maps indicate the subsurface conditions at the site consist of Holocene Age alluvial gravel, sand, and clay and including alluvial fan and levee deposits.[1] The subgrade soils encountered in our borings/CPTs were generally consistent with the mapped geology.

We also collected photographs at the time of our field exploration program. Representative photos are provided in our **Photography Log**.

PROJECT DESCRIPTION

Our initial understanding of the project was provided in our proposal and was discussed during project planning. A period of collaboration has transpired since the project was initiated, and our final understanding of the project conditions is as follows:

Item	Description
Information Provided	The following documents were provided to Terracon for review by Burns & McDonnell: ■ Site Due Diligence and Master Planning Scope of Services: Bay Site AZ1 site, San Jose, CA, prepared by Microsoft, undated. ■ Conceptual Grading Plan, Topo Map prepared by HMH, Inc. dated 11/4/2020. ■ Master plan Site Exhibit prepared by HMH, Inc. dated 11/6/2020.
Project Description	The project will consist of the construction of a data center facility that includes (2) data center buildings, a substation, a water tank, stormwater retention facilities, access roads, and parking areas.

[1] *Dibblee, T.W., and Minch, J.A., 2005, Geologic map of the Milpitas quadrangle, Alameda & Santa Clara Counties, California: Dibblee Geological Foundation, Dibblee Foundation Map DF-153, scale 1:24,000*

Item	Description
Proposed Structures	■ The data center buildings will be five-story ballard structures with footprints of about 90,000 square feet (sf) each. ■ The substation will be about 90,00 sf in size ■ It is proposed the water tank will have a diameter of about 150 feet and will be 30 feet tall.
Building Construction	We anticipate construction of data center buildings will consist of concrete and steel frame with slab-on-grade or structural slab floors.
Maximum Loads (assumed)	■ Columns: 400 to 500 kips ■ Walls: 7 to 8 kips-per-linear foot (klf) ■ Slabs: 250 pounds-per-square foot (psf) uniform loading
Finished Floor Elevation (per Conceptual Grading Plan prepared by HMH, dated 11/4/2020)	■ Data Center Buildings - 37.0 feet above Mean Sea Level ■ Substation – 30.0 feet above Mean Sea Level
Grading	We understand the site will be elevated up 10 feet to raise the proposed improvements above the design flood risk elevation. We anticipate site grading may consist of fills up to 10 feet and cuts will be made along the northwestern portion of the site to depths up of about 15 feet below current grade. We do not anticipate ant cut or fill slopes at the site.
Below-Grade Structures	Construction will include below-grade elevator pits up to 5 feet deep for the data center buildings. We have assumed the elevator pit walls will consist of cantilevered concrete construction.
Free-Standing Retaining Walls	None anticipated.

GEOTECHNICAL CHARACTERIZATION

We have developed a general characterization of the subsurface conditions based upon our review of the subsurface exploration, laboratory data, geologic setting and our understanding of the project. This characterization, termed GeoModel, forms the basis of our geotechnical calculations and evaluation of site preparation and foundation options. Conditions encountered at each exploration point are indicated on the individual logs. The individual logs can be found in the **Exploration Results** section and the GeoModel can be found in the **Figures** section of this report.

As part of our analyses, we identified the following model layers within the subsurface profile. For a more detailed view of the model layer depths at each boring location, refer to the GeoModel.

Model Layer	Layer Name	General Description
1	**Fat Clay**	Medium to very stiff fat clay with variable amounts of sand and gravel.
2	**Lean Clay**	Soft to very stiff lean clay with variable amounts of sand.

Model Layer	Layer Name	General Description
3	**Sand**	Very loose to dense sand with variable amounts of clay, silt, and gravel.
4	**Silt**	Medium to very stiff sandy silt.

Groundwater Conditions

The boreholes were observed while drilling and after completion for the presence and level of groundwater. Pore pressure dissipation tests were also performed in the CPTs to help determine approximate groundwater levels. The water levels observed in the boreholes and CPTs can be found on the boring/CPT logs in **Exploration Results** and are summarized below.

Boring/CPT Number	Approximate Depth to Groundwater[1] (feet)	
	While Drilling/Testing	After Drilling
B1	10	10
B2	20	5
B3, B6, B7, B8, B13	20	20
B4	23	23
B5	20	29
B9	10	14
B11	14	14
B12	10	13
B14	25	27
CPT-1, CPT-2, CPT-6, CPT-10, CPT-11	10	--
CPT-3, CPT-7, CPT-8, CPT-9	8	--
CPT-4	9	--
CPT-5	4	--

1. Below ground surface

Groundwater was not encountered in borings B10, P1 through P5, and I1 through I4. Since the borings were backfilled relatively soon after completion, the water levels summarized above for the borings are not stable groundwater levels. Due to the low permeability of soils encountered in the borings, a relatively long period may be necessary for a groundwater level to develop and stabilize in a borehole. Long term observations in piezometers or observation wells sealed from the influence of surface water are often required to define groundwater levels in materials of this type.

Groundwater level fluctuations occur due to seasonal variations in the amount of rainfall, runoff and other factors not evident at the time the borings and CPTs were performed. Therefore, groundwater levels during construction or at other times in the life of the structures may be different than the levels indicated on the boring/CPT logs. The possibility of groundwater level fluctuations should be considered when developing the design and construction plans for the project.

GEOTECHNICAL OVERVIEW

The subject site has several geotechnical considerations that will affect the construction and performance of the proposed improvements that are discussed in this report. The primary geotechnical considerations that have been identified at the subject site that will affect development of the site are the following:

- Excessive settlement considerations
- Moderately to highly plastic clay considerations

Excessive Settlement Considerations

We anticipate liquefaction settlement up to 2 inches total settlement and ¾ inches over 100 feet may occur across the site during a seismic event.

As indicated in the **Geotechnical Characterization** section of this report, the subgrade soils encountered in our borings and CPTs generally consisted of 15 to 30 feet of lean to fat clay with variable amounts of sand underlain by about 5 to 20 feet of sand with variable amounts of clay. The sand was followed by lean to fat clay to the maximum depth explored of 100½ feet below the existing ground surface (bgs) with a 15 to 25-foot-thick layer of sand encountered at a depth of approximately 45 feet bgs. The subsurface clay soils are compressible. Laboratory testing and CPT data indicated the subsurface clay layers are over-consolidated.

We understand site grades may be elevated up to 10 feet in the area of the data center buildings and water tank and up to 3 feet in the area of the substation to accommodate development and raise the site grades above the design flood risk elevation. Placement of the fill will trigger consolidation settlement of the clay soils. A settlement analysis was performed to estimate the anticipated settlement generated under the weight of the fill. The analysis was performed using the results of laboratory testing, CPT data, and our experience. Calculations were based on the average soil layer thicknesses of the soil lithology encountered in our borings and CPTs at each of the Building 1, Building 2, and substation areas. The results of our settlement analysis are presented in the following table.

Settlement from New Fill Placement		
Additional Fill Height (feet)	**Fill Load (psf)** [1]	**Estimated Total Settlement** [2] **(inches)**
Building 1 Area		
1	120	1
2	240	2
3	360	2½
4	480	3
5	600	3½
6	720	4
7	840	4½
8	960	5
9	1080	5½
10	1200	6
Building 2 Area		
1	120	½
2	240	1½
3	360	2
4	480	2½
5	600	3
6	720	3½
7	840	4
8	960	4½
9	1080	5
10	1200	5½
Substation Area		
1	120	1
2	240	2
3	360	2½

1. Based on a normal weight engineered fill having a unit weight of 120 pcf.
2. Does not include settlement due to liquefaction or structural loads.

Based on our analysis, we anticipate 75 percent of the settlement due to fill placement may occur in the upper 60 feet of soil at Building 1, in the upper 40 feet of soil at Building 2, and in the upper 50 feet at the substation. We also anticipate any settlement occurring in the sand with gravel layers due to fill placement will occur shortly after placement of the fill; however, the portion of settlement occurring in the clay soil layers could be prolonged over about 6 months in the area of Building 1, about 4 to 5 months in the area of Building 2, and about 4 months in the area of the substation for approximately 90 percent of the settlement to occur. **We recommend the placement of the fill occur early in the construction process or well in advance of mobilizing for other construction to allow for consolidation settlement from the fill to complete prior to foundation construction.** The time for consolidation settlement to occur could be reduced by installing open graded aggregate piers into the native clay subsoils as part of **Ground Improvement** or by installing wick drains. The fill should be surveyed periodically (at least monthly) after placement to monitor settlement and confirm when settlement due to the fill has completed and foundation construction can begin.

In order to mitigate the effects of total and differential settlements on the proposed improvements from structural loads and liquefaction, the improvements should be supported by **Shallow Foundations** supported on subgrade mitigated by **Ground Improvement** methods, or by **Deep Foundations** provided settlement due to fill placement is complete prior to foundation construction. Deep foundation elements would need to be designed with consideration of negative side friction if installation is taking place while consolidation settlement is on-going.

We understand a water tank with a proposed diameter of 150 feet and a height of 30 feet may be constructed at the site. The location of the water tank was not known at the time this report was published. A water tank of this size may influence may impose stresses and generate settlements to a soil lithology deeper than was explored as part of the field investigation associated with this report. Once the location and size of the water tank are confirmed, we recommend a supplemental field exploration be conducted in the tank footprint to a depth sufficient to perform a settlement analysis on the soils influenced by the size and loading of the tank.

In areas not improved by **Ground Improvement** methods, the anticipated differential movement should be considered when planning development. Special design details and long-term maintenance should be considered for underground utility lines; site development such as hardscape, entrances, and pavements adjacent to structures supported by **Deep Foundations** or **Ground Improvement**; and site drainage.

The **General Comments** section provides an understanding of the report limitations.

Moderately to Highly Plastic Clay Considerations

As indicated in the **Geotechnical Characterization** section of this report, the near surface native soils predominantly consist of stiff to very stiff moderate to high plasticity clay with variable amounts of sand which could become unstable with typical earthwork and construction traffic, especially after precipitation events. Effective drainage should be completed early in the construction sequence and maintained after construction to avoid potential issues. If possible, the grading should be performed during the warmer and drier times of the year. If grading is performed during the winter months or periods of normally high precipitation, an increased risk for possible undercutting and replacement of unstable subgrade will persist. Additional site preparation recommendations, including subgrade improvement and fill placement, are provided in the **Earthwork** section.

The native clay soils encountered in our borings/CPTs exhibit the potential for shrink-swell movements with changes in moisture content from seasonal or manmade conditions. Additional areas of localized moderately to highly plastic clays are likely present in the building areas where borings/CPTs were not performed. In order to help mitigate the effects of moderate to high volume change clay, building floors slabs and exterior hardscapes should be underlain by at least 18 inches chemically treated soil or Low Volume Change (LVC) structural fill. If building floors will consist of structural floors spanning between foundation systems, floor slabs could be underlain by minimum 3-inch-thick void forms as an alternative to chemically treated soil or LVC structural fill to help mitigate the effects of uplift pressures from the moderate to high volume change clays.

Using chemically treated soil or LVC structural fill for additional support of soil supported slabs-on-grade as recommended in this report may not eliminate all future subgrade volume change and resultant slab movements. However, the procedures outlined herein should help to reduce the potential for subgrade volume change in the native clay soils. Details regarding chemically treated soil and LVC structural fill are provided in **Earthwork**. Even if these procedures are followed, some movement and cracking in the buildings should be anticipated. The severity of cracking and other (cosmetic) damage such as uneven floor slabs will likely increase if any modification of the site results in excessive wetting or drying of the expansive soils.

We anticipate the proposed improvements will be supported by **Deep Foundations**, or **Shallow Foundations** supported by subgrade soils mitigated with **Ground Improvement.** If **Ground Improvement** is performed, **Shallow Foundations** should have a minimum embedment of 24 inches below lowest adjacent grade.

EARTHWORK

We understand the site will be elevated up 10 feet in order to raise the proposed improvements above the design flood plain. We anticipate site grading may consist of cuts up to 15 feet along the northern edge of the site and 3 feet along the eastern edge of the site and fills up to 10 feet in the data center building areas and up to 3 feet in the substation area. If greater cuts or fills are required, Terracon should be contacted to determine if additional earthwork recommendations are warranted, particularly with regard to potential ground settlement. Earthwork will include clearing and grubbing, fill placement, and excavations. The following sections provide recommendations for use in the preparation of specifications for the work. Recommendations include critical quality criteria as necessary to render the site in the state considered in our geotechnical engineering evaluation for foundations, floor slabs, and pavements.

Site Preparation

The property is undeveloped. Prior to placing fill, existing vegetation and root mat, debris and any other deleterious material should be removed. Complete stripping of the topsoil should be performed in proposed building, hardscape, and parking/driveway areas.

Subgrade Preparation

After clearing, any required cuts should be made. The presence of a high volume of organic material may warrant additional cuts or over-excavation at the time of grading operations.

Once cuts and over-excavation operations are completed, the resulting subgrade should be proof-rolled with an adequately loaded vehicle such as a fully loaded tandem axle dump truck. The proof-rolling should be performed under the direction of the Geotechnical Engineer. Areas excessively deflecting under the proof-roll should be delineated and subsequently addressed by the Geotechnical Engineer. Such areas should either be removed or modified by stabilizing as noted in the following section **Soil Stabilization**. Excessively wet or dry material should either be removed, or moisture conditioned and recompacted. Exposed surfaces should be free or mounds and depressions which could prevent uniform compaction.

Excavated material may be stockpiled for use as fill provided it is cleaned of vegetation, debris, and any other deleterious material and meets the criteria for engineered fill specified in the ***Fill Material Types*** section of this report.

Once proof rolling has been performed, and prior to placing any fill, the subgrade soil should be scarified and compacted. The depth of scarification of subgrade soils and moisture conditioning of the subgrade is highly dependent on the time of year of construction and the site conditions that exist immediately prior to construction. If construction occurs during the winter or spring, when the subgrade soils are typically already in a moist condition, scarification and compaction may only be 12 inches. If construction occurs during the summer or fall when the subgrade soils have been allowed to dry out deeper, the depth of scarification and moisture conditioning may be as much as 18 inches. A representative from Terracon should be present to observe the exposed subgrade and specify the depth of scarification and moisture conditioning required.

Following scarification and compaction of the subgrade, any required fill may be placed and compacted in accordance with the ***Fill Material Types*** and ***Fill Compaction Requirements*** sections of this report.

Due to the moderate to high volume change soils present across the site, the near surface soils at finished grades could be chemically treated to create a stable working surface during construction, especially during the rainy season.

The moisture content and compaction of subgrade soils, and especially the lime treated soil sections, shall be maintained until foundation, slab, and pavement construction. Care should be taken to also prevent wetting or drying of the bearing materials during construction.

Soil Stabilization

Methods of subgrade improvement, as described below, could include scarification, moisture conditioning and recompaction, and removal of unstable materials and replacement with granular fill (with or without geosynthetics) and/or chemical treatment with lime and/or cement. The appropriate method of improvement, if required, would be dependent on factors such as schedule, weather, the size of the area to be stabilized, and the nature of the instability. More detailed recommendations can be provided during construction as the need for subgrade stabilization occurs. Performing site grading operations during warm seasons and dry periods would help to reduce the amount of subgrade stabilization required.

If the exposed subgrade is unstable during proof rolling operations, it could be stabilized using one of the methods outlined below.

- **Scarification and Compaction** – It may be feasible to scarify, dry, and compact the exposed soils. The success of this procedure would depend primarily upon favorable weather and sufficient time to dry the soils. Stable subgrades likely would not be achievable if the thickness of the unstable soil is greater than about 1 foot or if construction is performed during a period of wet or cool weather when drying is difficult.

- **Aggregate Base** – The use of Caltrans Class II aggregate base is the most common procedure to improve subgrade stability. Typical undercut depths would be expected to range from about 6 to 18 inches below finished subgrade elevation with this procedure. The use of high modulus geosynthetics (i.e., engineering fabric or geogrid) could also be considered after underground work such as utility construction is completed. Prior to placing the fabric or geogrid, we recommend that all below-grade construction, such as utility line installation, be completed to avoid damaging the fabric or geogrid. Equipment should not be operated above the fabric or geogrid until one full lift of aggregate base is placed above it. The maximum particle size of granular material placed over geotextile fabric or geogrid should meet the manufacturer’s specifications.

- **Chemical Treatment** – Chemical treatment involves treating the unstable or pavement subgrade soils with a certain percentage of high calcium quicklime or Portland cement. Usually 4.0 to 5.5 percent based on the dry unit weight of the soil, for a depth of 12 to 18 inches. For estimating purposes, we recommend using 5 percent lime and a soil unit weight of 110 pounds per cubic foot. For 12-inch and 18-inch treatment depths, this results in an estimated minimum spread rate of 5.5 and 8.3 pounds per square foot lime/cement, respectively. The actual amount of lime/cement to be used should be determined by Terracon and by laboratory testing **at least three weeks prior** to the start of grading operations. Chemical treatment is performed after rough grading is completed.

Further evaluation of the need and recommendations for subgrade stabilization can be provided during construction as the geotechnical conditions are exposed.

Fill Material Types

Fill required to achieve design grade should be classified as structural fill and general fill. Structural fill is material used below, or within 5 feet of structures, pavements or constructed slopes. General fill is material used to achieve grade outside of these areas. Earthen materials used for structural and general fill should meet the following material property requirements:

Fill Type[1]	USCS Classification	Acceptable Location for Placement
Lean Clay/Clayey Sand	CL, SC (LL<40)	All structural and general fill locations and elevations, except as LVC material unless material explicitly meets LVC requirements.
Moderate Plasticity Material[2]	CL (50>LL≥40 or 30>PI≥25)	All general fill locations and elevations.
Well-graded Granular[3]	GM, GC, SM, SW	All structural and general fill locations and elevations.
Low Volume Change (LVC) Material[4]	CL, SC (LL<30 & PI<10) or Well-graded Granular Material [3]	Within 1½ feet below bottom of floor elevations. All structural and general fill locations and elevations.
On-site Soils[5]	SM, SW, CL, SC	As noted above.
	SP	All general fill locations and elevations and may be used as structural fil provided material is blended with other onsite soils.
	CH, ML	May be used as general fill.

Fill Type[1]	USCS Classification	Acceptable Location for Placement

1. Compacted structural fill should consist of approved materials that are free of organic matter and debris. A sample of each material type should be submitted to Terracon for evaluation at least two weeks prior to grading.
2. Delineation of moderate to highly plastic clays should be performed in the field by a qualified geotechnical engineer or their representative and could require additional laboratory testing.
3. Caltrans Class II aggregate base may be used for this material.
4. Low plasticity cohesive soil or granular soil having low plasticity fines. Material should be approved by the geotechnical engineer.
5. This material should be removed and recompacted if used as a structural fill as described in section **Fill Compaction Requirements**. Existing fill soils to be re-used as structural or general fill should be processed to remove any vegetation or debris present in the material prior to re-use.

For all import material, the contractor shall submit current verified reports from a recognized analytical laboratory indicating that the import has a "not applicable" (Class S0) potential for sulfate attack based upon current ACI criteria and is "mildly corrosive" to ferrous metal and copper. The reports shall be accompanied by a written statement from the contractor that the laboratory test results are representative of all import material that will be brought to the project.

Fill Compaction Requirements

Structural and general fill should meet the following compaction requirements.

Item	Structural Fill	General Fill
Maximum Lift Thickness[2]	8 inches or less in loose thickness when heavy, self-propelled compaction equipment is used 4 to 6 inches in loose thickness when hand-guided equipment (i.e. jumping jack or plate compactor) is used	Same as Structural fill
Minimum Compaction Requirements [1, 3]	95% of max: Upper 12 inches of subgrade in pavement areas, for aggregate base and chemically treated soil, below slabs and foundations, and where fill thicknesses will exceed 5 feet. 90% of max: All other locations	90% of max.
Water Content Range [1]	Low plasticity cohesive: +1% to +3% above optimum High plasticity cohesive: +2% to +4% above optimum Granular: -2% to +2% of optimum	As required to achieve min. compaction requirements[4]

1. Maximum density and optimum water content as determined by the Modified Proctor test (ASTM D 1557).
2. Reduced lift thicknesses are recommended in confined areas (e.g., utility trenches, foundation excavations, and foundation backfill) and when hand-operated compaction equipment is used.
3. We recommend that engineered fill be tested for moisture content and compaction during placement. Should the results of the in-place density tests indicate the specified moisture or compaction limits have not been met, the area represented by the test should be reworked and retested as required until the specified moisture and compaction requirements are achieved. This procedure is intended for soils with 30 percent

Item	Structural Fill	General Fill

or less material larger than ¾ inch. Accordingly, we recommend full time proof roll observation be performed instead of moisture density testing for materials containing more than 30 percent aggregate retained on the ¾-inch sieve.

4. Specifically, moisture levels should be maintained low enough to allow for satisfactory compaction to be achieved without the cohesionless fill material pumping when proof rolled.

Utility Trench Backfill

Due to the anticipated settlements associated with fill placement across the site, special design may be required for utilities in areas not mitigated with **Ground Improvement**. Utility design should account for the anticipated settlements provided in **Geotechnical Overview.** It is recommended utilities and piping be designed with flexible connections and/or other means to accommodate such soil movement to preclude damage between areas mitigated and not mitigated with **Ground Improvement** or between non-mitigated areas and structures supported by **Deep Foundations**. Utility and drain lines designed for gravity flow should consider steeper gradients to account for anticipated settlements, especially where such lines enter buildings supported by **Ground Improvement** methods or **Deep Foundations**.

All trench excavations should be made with sufficient working space to permit construction including backfill placement and compaction. If utility trenches are backfilled with relatively clean granular material, they should be capped with at least 18 inches of cementitious flowable fill or cohesive fill in non-pavement areas to reduce the infiltration and conveyance of surface water through the trench backfill. Attempts should also be made to limit the amount of fines migration into the clean granular material. Fines migration into clean granular fill may result in unanticipated localized settlements over a period of time. To help limit the amount of fines migration, Terracon recommends the use of a geotextile fabric that is designed to prevent fines migration in areas of contact between clean granular material and fine-grained soils. Terracon also recommends that clean granular fill be tracked or tamped in place where possible in order to limit the amount of future densification which may cause localized settlements over time.

Utility trenches are a common source of water infiltration and migration. Utility trenches penetrating beneath buildings should be effectively sealed to restrict water intrusion and flow through the trenches, which could migrate below the buildings. The trench should provide an effective trench plug that extends at least 5 feet from the face of the building exterior. The plug material should consist of cementitious flowable fill or low permeability clay. The trench plug material should be placed to surround the utility line. If used, the clay trench plug material should be placed and compacted to comply with the water content and compaction recommendations for structural fill stated previously in this report.

If chemical (lime) treatment of subgrade soils occurs before utility construction, Controlled Low Strength Material (CLSM) or sand/cement slurry should be used as backfill material to cap utility trenches in all areas where trenches have cut through the treated subgrade. The thickness of the CLSM or slurry should be at least the thickness or depth of chemically treated subgrade. Such areas trenched through chemically treated soil should not be backfilled with aggregate base, native soil, or chemically treated soil.

Post construction trenching through geogrid may be accomplished with conventional trenching equipment. Repairs to the trenched section shall be accomplished using a full structural replacement of the displaced materials or with a repaired section that is identical to the original section. If the trench section is repaired to match the original, the trench backfill must be compacted to the same or higher density and the geogrid must be over-lapped a minimum 3-inches at the proper geogrid elevation.

Grading and Drainage

All grades must provide effective drainage away from the improvements during and after construction and should be maintained throughout the life of the structures. Water retained next to the structures can result in soil movements greater than those discussed in this report. Greater movements can result in unacceptable differential floor slab and/or foundation movements, cracked slabs and walls, and roof leaks. The roof should have gutters/drains with downspouts that discharge into a site drainage system or at least 10 feet from the buildings.

Exposed ground should be sloped and maintained at a minimum 5 percent away from the improvements for at least 10 feet beyond the perimeter of the foundations. If a minimum 5 percent slope cannot be achieved due to site grades, a minimum 2½ percent slope could be used provided pavement or hardscape surrounds and extends to the improvements or a subdrain could be installed around the perimeter of the foundations that carries water away from the improvements. Locally, flatter grades may be necessary to transition ADA access requirements for flatwork. After building construction and landscaping, final grades should be verified to document effective drainage has been achieved. Grades around the structure should also be periodically inspected and adjusted as necessary as part of the structure's maintenance program. Where paving or flatwork abuts the structure a maintenance program should be established to effectively seal and maintain joints and prevent surface water infiltration.

Planters and bio-swales located within 10 feet of structures should be self-contained or lined with an impermeable membrane to prevent water from accessing buildings subgrade soils. Sprinkler mains and spray heads should be located a minimum of 5 feet away from the building lines.

Trees or other vegetation whose root systems have the ability to remove excessive moisture from the subgrade and foundation soils should not be planted next to the structures. Trees and shrubbery should be kept away from the exterior of the structures a distance at least equal to their expected mature height.

Implementation of adequate drainage for this project can affect the surrounding developments. Consequently, in addition to designing and constructing drainage for this project, the effects of site drainage should be taken into consideration for the planned structures on this property, the undeveloped portions of this property, and surrounding sites. Extra care should be taken to ensure irrigation and drainage from adjacent areas do not drain onto the project site or saturate the construction area.

Earthwork Construction Considerations

Excavations for the proposed structures are anticipated to be accomplished with conventional construction equipment. Upon completion of filling and grading, care should be taken to maintain the subgrade moisture content prior to construction of foundations, slabs and pavements. Construction traffic over the completed subgrade should be avoided to the extent practical. The site should also be graded to prevent ponding of surface water on the prepared subgrades or in excavations. If the subgrade should become desiccated, saturated, or disturbed, the affected material should be removed, or these materials should be scarified, moisture conditioned, and recompacted prior to slab or pavement construction.

We recommend that the earthwork portion of this project be completed during extended periods of dry weather if possible. If earthwork is completed during the wet season (typically November through April) it may be necessary to take extra precautionary measures to protect subgrade soils. Wet season earthwork operations may require additional mitigation measures beyond that which would be expected during the drier summer and fall months. This could include ground stabilization utilizing chemical treatment of the subgrade, diversion of surface runoff around exposed soils, and draining of ponded water on the site. Once subgrades are established, it may be necessary to protect the exposed subgrade soils from construction traffic.

<u>As a minimum, excavations should be performed in accordance with OSHA 29 CFR, Part 1926, Subpart P, “Excavations” and its appendices, and in accordance with any applicable local, and/or state regulations.</u>

Construction site safety is the sole responsibility of the contractor who controls the means, methods, and sequencing of construction operations. Under no circumstances shall the information provided herein be interpreted to mean Terracon is assuming responsibility for construction site safety, or the contractor's activities; such responsibility shall neither be implied nor inferred.

Construction Observation and Testing

The earthwork efforts should be monitored under the direction of the Geotechnical Engineer. Monitoring should include documentation of adequate removal of vegetation and top soil, proof-rolling and mitigation of areas delineated by the proof-roll to require mitigation.

Each lift of compacted fill should be tested, evaluated, and reworked as necessary until approved by the Geotechnical Engineer prior to placement of additional lifts. Each lift of fill should be tested for density and water content at a frequency of at least one test for every 5,000 square feet (sf) of compacted fill in the building and pavement areas. One density and water content test should be performed per lift for every 50 linear feet of compacted utility trench backfill.

In areas of foundation excavations, the bearing subgrade should be evaluated under the direction of the Geotechnical Engineer. In the event that unanticipated conditions are encountered, the Geotechnical Engineer should prescribe mitigation options.

In addition to the documentation of the essential parameters necessary for construction, the continuation of the Geotechnical Engineer into the construction phase of the project provides the continuity to maintain the Geotechnical Engineer's evaluation of subsurface conditions, including assessing variations and associated design changes.

SEISMIC CONSIDERATIONS

Due to the subsurface conditions encountered our borings and CPTs, the potential for liquefaction at the site, and the proposed construction, a Site-Specific Ground Motion Study was performed in order to develop appropriate data for use in the seismic design of the proposed improvements. The Site-Specific Ground Motion Study included both a ground motion hazard analysis and a site response analysis. The results of the study are presented in a Report of Site-Specific Ground Motion Study included in the **Supporting Information** section of this report.

LIQUEFACTION

Liquefaction is a mode of ground failure that results from the generation of high pore water pressures during earthquake ground shaking, causing loss of shear strength. Liquefaction is typically a hazard where loose sandy soils or low plasticity fine grained soils exist below groundwater. The California Geological Survey (CGS) has designated certain areas within California as potential liquefaction hazard zones. These are areas considered at a risk of liquefaction-related ground failure during a seismic event, based upon mapped surficial deposits and the presence of a relatively shallow water table. The project site is located within both a CGS liquefaction hazard zone and a Santa Clara County liquefaction hazard zone. Therefore, a liquefaction analysis was performed to determine the liquefaction induced settlement at the site.

Our liquefaction hazard evaluation was performed in general compliance with the California Geological Survey (CGS) Special Publication 117A (2008); Southern California Earthquake Center "Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Liquefaction Hazards in California," 1999 report; and the

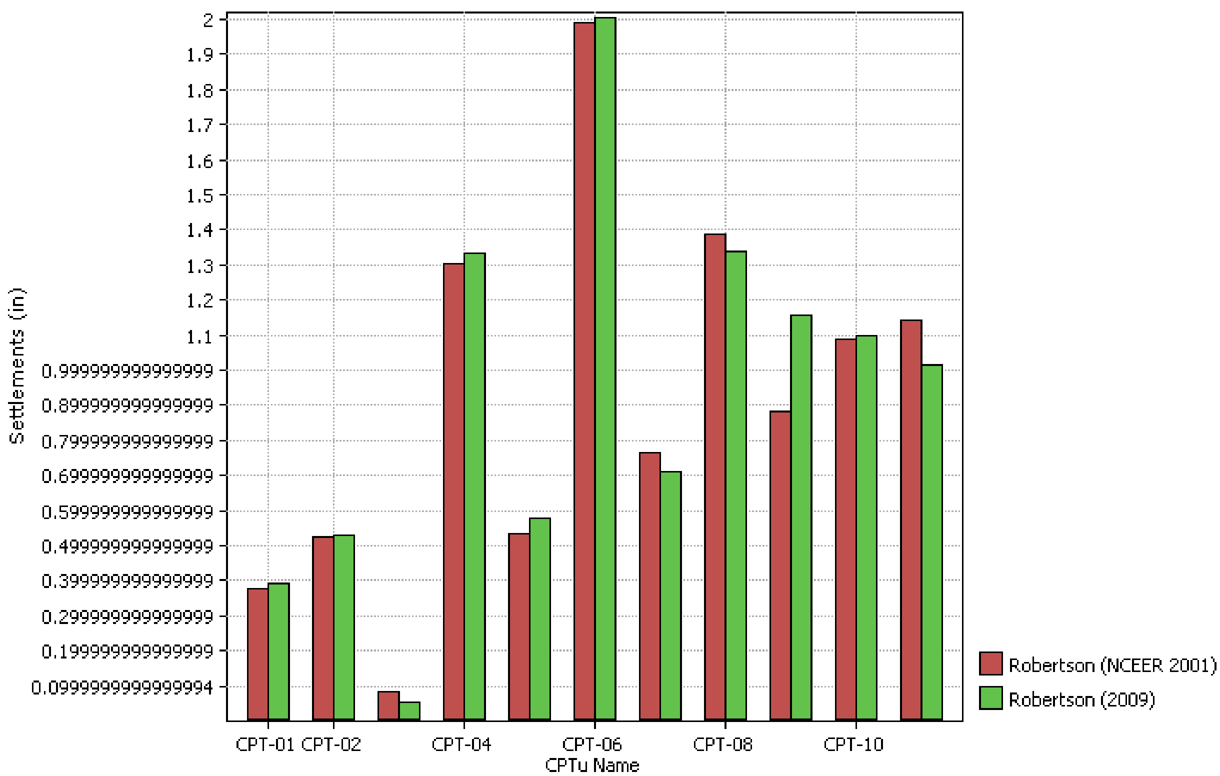
Seismic Hazard Zone Report for the Milpitas 7.5-Minute Quadrangle, Alameda and Santa Clara Counties, California (SHZR 051).

As recommended in the above reports, we performed a screening analysis to determine if there is a potential for liquefaction to occur at the site. We evaluated the soils encountered in our borings advanced to a maximum depth of approximately 51½ feet below the existing ground surface (bgs) and eleven cone penetration tests (CPTs) which were advanced to depths varying from 50½ to 100½ feet bgs. We evaluated these soils based on soil classification, corrected SPT blow counts, water content, Atterberg limits, groundwater elevation, shear strength, peak ground acceleration, and CPT data. In our screening investigation we looked at the Atterberg limits for fine-grained soils in the upper 60 feet of our soil borings. The Atterberg limits for these fine-grained soils exhibited liquid limits ranging from 21 to 77 and plasticity indexes ranging from 3 to 54. We also calculated the ratio of the in-situ moisture content to the liquid limit. This data was then compared to the criteria by Idriss and Boulanger (2006) and Bray and Sancio (2006) for potential liquefaction or cyclic softening of fine-grained soils. These soils classify as "clay-like" and non-liquefiable by Idriss/Boulanger with exception of the sandy silt layer encountered at a depth of 26 feet bgs in Boring B6 which classified as "sand-like". Based on the plasticity index and the ratio of the in-situ moisture content to the liquid limit of the fine-grained soils, we believe the fine grained soils at the site have a low susceptibility to cyclic softening/liquefaction by Bray/Sancio with exception of the sandy silt layer encountered at a depth of 26 feet bgs in Boring B6 that has a high susceptibility to liquefaction and the lean clay with sand layer encountered at a depth of 15 feet bgs in Boring B3. The sandy lean clay layer encountered at a depth of 30 feet bgs in Boring B8 classifies as having a moderate susceptibility to liquefaction. However, CPT data indicates the sandy lean clay layer is over-consolidated. Subsequently, we believe the liquefaction susceptibility of this layer is low. Given these two fine-grained layers with liquefaction susceptibility and the presence of sand layers in our borings and CPTs we determined there is a potential for liquefaction to occur and further evaluation was warranted. We therefore performed a quantitative evaluation of the potential for liquefaction to occur and the effects if liquefaction were to occur on this project.

A ground motion hazard analysis and a site response analysis were performed for this site and the proposed development in accordance with ASCE 7-16. The results of these analyses are presented in report provided in the **Supporting Information** section of this report. Based on the results of these analyses, a Peak Ground Acceleration (PGA) of 0.512 and an earthquake magnitude of 7.0 for the project site was used in our evaluation. Seismic Hazard Zone Report 051 for the Milpitas Quadrangle indicates the historic high groundwater depth for the site varies between 5 and 10 feet bgs. Groundwater was encountered at depths varying from 5 to 29 feet bgs in our borings and at depth varying from 4 to 10 feet bgs in our CPTs at the time they were performed. As a result, a groundwater depth of 4 feet was utilized in our evaluation.

A liquefaction analyses was performed in general accordance with California Geologic Survey Special Publication 117. The liquefaction study utilized the software "CLiq" by GeoLogismiki

Geotechnical Software. These analyses were based on the soil data obtained from the eleven CPT soundings supplemented by laboratory data performed on samples obtained from our borings. Analyses were performed on data obtained from CPT-01 through CPT-011. CPT calculations were assessed using the Robertson (NCEER 2001) and Robertson (2009) methods. A factor of safety of 1.3 was used against liquefaction. The liquefaction potential analyses were calculated from a depth of 4 to 60 feet bgs. A summary of the results of our analyses using the Robertson (2009) method has been attached to this report. The following table summarizes the vertical settlements calculated using the noted methods.



Based on the analysis, liquefiable layers most susceptible to liquefaction were encountered between the depths of approximately 10 to 35 feet bgs and 46 to 60 feet bgs across the site. Based on our review of the calculations, the anticipated potential total liquefaction-induced settlement across the site varies from negligible to about 2 inches. We estimate the differential liquefaction-induced settlement may be about 3/4 inches over 100 feet across the site.

Due to the cohesive nature and thickness of non-liquefiable soils across the surface of the site as well as the lithology consisting predominantly of clayey soils, we believe the probability for liquefaction to manifest at the surface is low. Furthermore, our review of the Seismic Hazard Zone Report 051 for the Milpitas Quadrangle reported no historic evidence of surface manifestations of liquefaction or ground failures at the site. The only historic ground failure noted in the report in the area of the site was an occurrence of ground settlement approximately ½ mile north of the site.

With regards to the potential for lateral spreading, we note that the site and surrounding area is relatively level. The Guadalupe River borders the site to the west. Based on aerial photographs and relative topographic information, the bottom of the Guadalupe River appears to be about 6 feet lower in elevation than the project site. Given the relative flatness of the local topography and that identified liquefiable soils at the site were located at least 10 feet below the existing ground surface, it is our opinion that the potential for lateral spreading to affect this site is low.

FOUNDATIONS

We understand site grades may be elevated up to 10 feet in the area of the data center buildings and water tank and up to 3 feet in the area of the substation to accommodate development and raise the site grades above the design flood risk elevation. The placement of the fill will trigger consolidation settlement of the subgrade soils up to an estimated 5 to 6 inches. The foundation recommendations presented in this report have assumed fill placement will occur early in the construction process to allow for consolidation settlement from the fill to complete prior to foundation construction, and that the settlement period would be monitored.

In order to mitigate the effects of total and differential settlements on the proposed improvements from structural loads and liquefaction, we have provided the following two options for foundation support for the proposed improvements.

Option 1 – Shallow Foundations over Ground Improvement:

The improvements may be supported by **Shallow Foundations** provided **Ground Improvement** is performed to mitigate the anticipated settlement from **Liquefaction** and structural loads. For this option, **Ground Improvement** should be performed to stiffen the upper compressible clay layers, and help mitigate the potential liquefaction of the underlying sands. All footings should extend at least 24 inches below lowest adjacent grade. The building floor slabs should be underlain by minimum 18 inches of chemically treated material or LVC structural fill to help mitigate the effects of moderate to high volume change clays.

Option 2 – Deep Foundations:

If Ground Improvement will not be performed, the proposed improvements should be supported by **Deep Foundations** to help protect the improvements against the estimated total and differential settlements due to structural loads and **Liquefaction**. The **Deep Foundations** may consist of auger cast piles (ACP) or driven piles and should extend through the potentially liquefiable sand layers and derive their support from the subgrade soils below a depth of 60 feet below the existing ground surface (bgs). For this option, **Deep Foundations** will be required to account for a drag load imposed on the foundation from settlement due to liquefaction and building floor slabs should be designed as structural slabs that span between foundation elements and rely on the foundations for support.

GROUND IMPROVEMENT

As a cost-effective alternative to supporting the improvements on **Deep Foundations**, ground improvement may be utilized to help mitigate the anticipated excessive settlement due to structural loads and the potential liquefaction of the underlying sands. Ground improvement methods such as aggregate piers and drilled displacement columns (DDC) are proprietary systems designed by licensed contractors who could provide further information regarding support options. Considering the various methods available for ground improvement, it is our opinion aggregate piers or drilled displacement columns (DDCs) would be suitable options for ground improvement at this site. Aggregate piers are often one of the most cost-effective methods for ground improvement and the spoils generated from installing aggregate piers would provide the added benefit of producing material that could be used as fill in raising portions of the site outside of the developed building area to design elevations. The installation of open graded aggregate piers prior to fill placement on the building site could also help reduce the time for consolidation settlement to occur due to fill placement. DDCs would help increase the bearing capacity of the subgrade soils while reducing settlement and help mitigate potential liquefaction settlement below the improvements by transferring the tank loads to deeper, more competent soils and improving the soil around the DDCs. However, if the Contractor or Structural Engineer have worked with a different ground improvement method that has proven successful to mitigate the hazards present at this site with similar subgrade soil conditions, Terracon could consider such options if desired.

Aggregate Piers

As a way to mitigate excessive settlement below the proposed improvements due to the presence of compressible clays and potential liquefaction, the subgrade soils could be improved with aggregate piers installed on a grid pattern. This option would eliminate the need for **Deep Foundations**, and would allow for the use of **Shallow Foundations** over the aggregate pier-reinforced subgrade. Aggregate pier systems are typically installed after clearing and grubbing and could be installed prior or after fill construction. If aggregate piers are installed prior to fill construction, spoils generated from the pier installation could be used as fill material across the site. The aggregate pier system will serve to stiffen the compressible clay and may also serve as gravel conduits for the dissipation of pore-water pressure depending on the aggregate grading, thereby shortening the time required for consolidation settlements. Aggregate piers can also be used to densify the potential liquefiable cohesionless soils.

Aggregate piers are typically constructed by advancing a drill or mandrel to design depths, then building a bottom bulb of clean, open-graded stone. The pier is built on top of the bottom bulb, using graded aggregate placed in thin lifts (12 to 24 inches compacted thickness). We anticipate piers would extend to depths varying from 25 to 35 feet below the existing ground surface at this site depending on the soils targeted for improvement. The result is a reinforced zone of soils directly under the foundations, which allows for the design and construction of foundations for relatively higher bearing pressures and with lower anticipated settlements. Aggregate piers can also be installed where differential movement is a concern between underground utility lines; site development such as hardscape, entrances, and pavements adjacent to structures supported by **Deep Foundations** or **Ground Improvement**; and site drainage.

We anticipate foundations supported over aggregate piers installed following fill placement may be designed using an allowable bearing pressure of 2,500 to 3,000 pound per square foot (psf) for dead plus live loads. However, the final design allowable bearing pressure should be confirmed by the design-build contractor installing the aggregate piers and coordinated with the structural engineer. If aggregate piers are installed prior to fill placement, the allowable bearing pressure provided in the **Shallow Foundations** section of this report should be used and should be accounted for in the design of the aggregate pier system. The aggregate pier ground improvement system for this project should meet the following design criteria:

Bearing Capacity Factor of Safety = 2.0
Global Stability (static) = 1.3
Global Stability (dynamic) = 1.1
Post-construction Settlement:
< ½-inch if piers will be installed prior to fill placement
<1 inch if piers will be installed after fill placement
Post-construction Differential Settlement:
< ¼ inch / 40 feet if piers will be installed prior to fill placement
< ½ inch / 40 feet if piers will be installed after fill placement

Aggregate pier systems should be designed and constructed by a specialty ground improvement contractor. Since this would be specialty work, we recommend consideration of using a design-build process if this alternative is selected. The contractor should provide detailed design calculations sealed by a professional engineer licensed in the State of California. Terracon should be afforded the opportunity to review the design and calculations.

Drilled Displacement Columns

DDCs would help increase the bearing capacity of the subgrade soils while reducing settlement and help mitigate potential liquefaction settlement below the improvements by transferring their loads to deeper, more competent soils and improving the soil around the DDCs. We have assumed DDCs would be installed after fill placement and consolidation settlement due to the fill is complete. DDCs are constructed by using a displacement auger to create a soil shaft filled with Controlled Low Strength Material (CLSM) injected under pressure as the displacement auger is withdrawn. DDCs generate a minimal amount of soil cuttings compared to other foundation and ground improvement methods. The diameter of DDCs typically vary between 18 to 36 inches and based on foundation load requirements the strength of CLSM can typically vary between 100 to 500 psi at 28 days.

We anticipate foundations supported on DDCs may be designed using an allowable bearing pressure of 3,000 to 3,500 pounds per square foot (psf) for dead plus live loads. However, the final design allowable bearing pressure should be confirmed by the design-build contractor installing the DDCs and coordinated with the structural engineer. The DDC ground improvement system for this project should meet the following design criteria:

Bearing Capacity Factor of Safety = 2.0
Global Stability (static) = 1.3
Global Stability (dynamic) = 1.1
Post-construction Settlement: < 1-inch
Post-construction Differential Settlement:< ½ -inch / 40 feet

DDC capacities and settlement based on anticipated loading should be evaluated by a design-build contractor. Terracon should be afforded the opportunity to review the bearing capacity and settlement evaluation.

DDCs should extend to depths varying from at least 40 to 50 feet below the existing ground surface at this site depending on the soils targeted for improvement. We anticipate DDCs would extend at least 50 feet below finished pad grade after fill placement. Minimum DDC lengths shall be confirmed by Terracon during construction. A 12 to 24-inch cushion of Caltrans Class II aggregate base be should installed between the top of the DDCs and the bottom of foundations and slabs.

Design and installation of DDCs shall be performed by a qualified design-build contractor. At least two load tests should be performed in the footprint of each improvement to confirm DDC capacities prior to installing production columns. Axial compression load tests shall be performed according to ASTM D1143. Only columns meeting the load testing criteria shall be used as final production columns. A design and installation package including a quality control plan for DDC installation should be prepared by the design-build contractor and submitted to Terracon for review and approval prior to construction. The package should also include information regarding load test such as proposed test location, set-up, and testing parameters. Terracon should be present on-site during load test and production to observe installation and testing of the DDCs.

SHALLOW FOUNDATIONS

If foundation support **Option 1** is utilized and the site has been prepared in accordance with the requirements noted in **Earthwork**, the improvements can be supported on shallow foundations**.** The following design parameters are applicable for shallow foundations and have prepared with the assumption that fill placement will occur early in the construction process to allow for consolidation settlement from the fill to complete prior to foundation construction.

Design Parameters – Compressive Loads

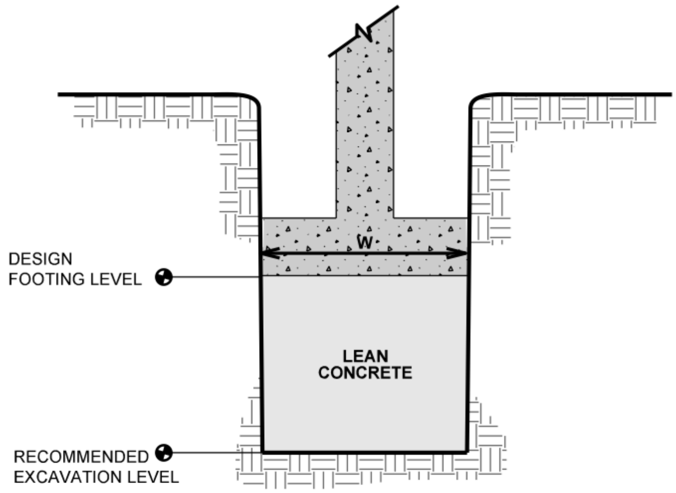
Item	Description
Maximum Net Allowable Bearing Pressure [1, 2]	■ 1,500 psf for footings over aggregate piers installed prior to fill placement (assumes footings will bear on 10 feet of structural fill placed over native soils mitigated with ground improvement). ■ Per ground improvement contractor design if ground improvement is installed directly below shallow foundation elements following fill placement.
Required Bearing Stratum [3]	■ Native soil or compacted structural fill improved by ground improvement methods ■ If Ground improvement is performed prior to fill placement: Compacted structural fill
Minimum Footing Width	Columns: 2 feet Continuous: 1½ feet
Maximum Footing Width	Columns: 15 feet Continuous: 8 feet
Passive Resistance [4,8] **(equivalent fluid pressure)**	300 pcf
Coefficient of Sliding Friction [5,8]	0.30
Minimum Embedment below Finished Grade [6]	24 inches
Estimated Total Settlement from Structural Loads [2]	Less than about 1 inch
Estimated Differential Settlement [2, 7]	About 1/2 of total settlement

Item	Description
1.	The maximum net allowable bearing pressure is the pressure in excess of the minimum surrounding overburden pressure at the footing base elevation. This bearing pressure can be increased by 1/3 for transient loads unless those loads have been factored to account for transient conditions. An appropriate factor of safety has been applied. Values assume that exterior grades are no steeper than 20% within 10 feet of structure.
2.	Values provided are for maximum loads noted in **Project Description**. **The settlement estimates are for static conditions and are inclusive of settlement within the ground improvement system, but do not include settlement associated with liquefaction.**
3.	Unsuitable or soft soils should be over-excavated and replaced per the recommendations presented in the **Earthwork**.
4.	Use of passive earth pressures require the sides of the excavation for the spread footing foundation to be nearly vertical and the concrete placed neat against these vertical faces or that the footing forms be removed and compacted structural fill be placed against the vertical footing face.
5.	Can be used to compute sliding resistance where foundations are placed on suitable soil/materials. Should be neglected for foundations subject to net uplift conditions.
6.	Embedment necessary to minimize the effects of seasonal water content variations. For sloping ground, maintain depth below the lowest adjacent exterior grade within 5 horizontal feet of the structure.
7.	Differential settlements are as measured over a span of 40 feet.
8.	Passive pressure and sliding friction may be combined to resist sliding provided the passive pressure is reduced by 50 percent.

Foundation Construction Considerations

As noted in **Earthwork**, the footing excavations should be evaluated under the direction of the Geotechnical Engineer. The base of all foundation excavations should be free of water and loose soil, prior to placing concrete. Concrete should be placed soon after excavating to reduce bearing soil disturbance. Care should be taken to prevent wetting or drying of the bearing materials during construction. Excessively wet or dry material or any loose/disturbed material in the bottom of the footing excavations should be removed/reconditioned before foundation concrete is placed.

If unsuitable bearing soils are encountered at the base of the planned footing excavation, the excavation should be extended deeper to the ground improvement elements or to suitable soils based on the support option being used, and the footings could bear directly on these soils at the lower level or on lean concrete backfill placed in the excavations. This is illustrated on the sketch below.



LEAN CONCRETE BACKFILL

NOTE: EXCAVATIONS ARE SHOWN VERTICAL; HOWEVER, THE SIDEWALLS SHOULD BE SLOPED AS NECESSARY FOR SAFETY

To ensure foundations have adequate support, special care should be taken when footings are located adjacent to trenches. The bottom of such footings should be at least 1 foot below an imaginary plane with an inclination of 1.5 horizontal to 1.0 vertical extending upward from the nearest edge of adjacent trenches.

DEEP FOUNDATIONS

If foundation support **Option 2** is utilized, the proposed improvements may be supported by **Deep Foundations** consisting of either auger cast piles or driven piles. Recommendations for the design and construction of auger cast piles and driven piles based on the in-situ soil conditions are provided in the following paragraphs with the assumption that fill placement will occur early in the construction process to allow for consolidation settlement from the fill to complete prior to foundation construction, and that the settlement period will be monitored.

Auger Cast Pile Design Parameters

The proposed improvements may be supported by a deep foundation system consisting of auger cast piles (ACP). Due to relatively high groundwater encountered in our borings and CPTs, ACPs would likely prove to be easier construction that a traditional drilled pier foundation.

ACP foundations are installed by advancing a hollow-stem auger to a predetermined depth in the ground, and then pumping high-strength flowable cement grout into the hole through the bottom of the hollow auger as the auger is slowly withdrawn. The grout is pumped under relatively high pressure and a positive head of grout is maintained above the base of the auger during auger extraction. After the auger is completely removed, reinforcing steel is then placed. Full scale, on-site load tests are customarily performed on auger cast piles to verify the desired capacity is achievable prior to construction.

ACPs should extend through soils susceptible to liquefaction to a minimum depth of 70 feet below existing site grade into underlying firm soil. Long-term settlement of ACIP foundations designed and constructed in accordance with the recommendations presented in this report should be about ½ inch or less in addition to elastic shortening. Once the pile loads and layout are determined, Terracon should review the design and update anticipated settlements as appropriate.

ACPs penetrating firm native soils below soils susceptible to liquefaction will use a combination of end bearing and skin friction in developing their load carrying capacity. The pile design should include a dynamic drag load due to negative skin friction to a depth of 60 feet bgs as result of liquefaction settlement. The ACPs should be initially proportioned using an allowable bearing pressure provided in the table below. Additional load carrying capacity can be gained by utilizing an allowable skin friction for that portion of the pile embedded in the soils below a depth of 60 feet bgs. This skin friction value may be used for compressive or tensile loads. Deeper penetrations may be required to develop additional skin friction and/or uplift resistance to accommodate structural loads. ACPs should be tied together using a structural slab or grade beams so that they act as a unit.

Design parameters for an ACP foundation are presented below. Due to the variability in the soil layers encountered in our borings/CPTs across the site we have provided tables for the each of the areas at Building 1, Building 2, and the substation for use in design.

ACP Design Summary	
Description	**Recommendations**
Minimum Embedment Depth	70 feet below existing grade
Skin Friction/End Bearing Considerations	Skin friction and end bearing should only be relied on for capacity below a depth of 60 feet from existing grade for seismic design that considers liquefaction.
Down Drag Load	<u>Due to the potential for liquefaction of the sand layers, ACP design should incorporate a dynamic drag load of 190, 194, or 191 kips per pile for the Building 1, Building 2, and the substation areas, respectively. These drag loads do not include down drag due to the placement of fill across the site. The drag loads are based on the piles being 24-inch diameter piles. Terracon should be contacted to provide revised drag loads if piles will have a different dimension.</u>

ACP Design Summary	
Description	**Recommendations**
	The project Structural Engineer should confirm combined drag and design loads do not exceed the structural capacity of the pile.
Minimum center to center spacing to develop full skin friction	3 times the diameter of the ACP.
Groups of 3 or more piles spaced closer than 3 pile diameters	Should be evaluated on a case by case basis by Terracon. Alternative installation sequences may be needed to allow for a minimum of 48 hours concrete curing time, before installation of adjacent shafts.
Minimum Pile Diameter	24 inches
Settlement	Less than 1/2 inch

Auger Cast Pile Axial Design Parameters [1]			
Approximate Depth from Finished Pad Elevation [8] (feet)	**Stratigraphy [2]**	**Skin Friction (psf) [3]**	**End Bearing Pressure (ksf) [4]**
Building 1			
0 to 10[7]	FILL	200[5]	0[5]
10 to 18	Clay	650[5]	3.5[5]
18 to 27	Clay	390[5]	1[5]
27 to 36	Sand	830[5]	6[5]
36 to 59	Clay	350[5]	3.5[5]
59 to 70	Sand	460[5]	12.5[5]
70 to 110[6]	Clay	650	8.5
Building 2			
0 to 10[7]	FILL	200[5]	0[5]
10 to 20	Clay	425[5]	3[5]
20 to 27	Clay	450[5]	1[5]
27 to 34	Sand	630[5]	19.5[5]

Auger Cast Pile Axial Design Parameters [1]			
Approximate Depth from Finished Pad Elevation [8] (feet)	**Stratigraphy [2]**	**Skin Friction (psf) [3]**	**End Bearing Pressure (ksf) [4]**
34 to 57	Clay	470[5]	4[5]
57 to 70	Sand	475[5]	18[5]
70 to 82	Sand	365	18
82 to 87[6]	Clay	610	8.5
Substation			
0 to 3[7]	FILL	200[5]	0[5]
3 to 11	Clay	615[5]	5.5[5]
11 to 22	Clay	600[5]	3.5[5]
22 to 30	Sand	520[5]	19.5[5]
30 to 52	Clay	445[5]	5[5]
52 to 63	Sand	540[5]	18[5]
63 to 75	Sand	375	18
75 to 85[6]	Clay	610	8.5

1. Design capacities are dependent upon the method of installation, and quality control parameters. The values provided are estimates and should be verified when installation protocol have been finalized.
2. See **Geotechnical Characterization** for more details on Stratigraphy
3. Applicable for compressive loading only. Reduce to 2/3 of values shown for uplift loading. Effective weight of pile can be added to uplift load capacity.
4. Piles should extend 10 feet into the bearing stratum for end bearing to be considered.
5. For seismic design, skin friction and end bearing should be neglected for pile capacity in this layer due to dynamic down drag from potential liquefaction in the sand layers. Anticipated down drag forces for 24-inch diameter ACP included in **ACP Design Summary** table.
6. Our current scope of work included extending CPTs to a maximum depth of about 100 feet, 77 feet, and 62 feet below existing site grades at Building 1, Building 2, and the substation location respectively. We have assumed similar soils extend below the maximum explored depths. Deeper borings/CPTs shall be utilized for confirmation if needed.
7. For static design, neglect the upper 5 feet of the pile for skin friction.
8. Table assumes 10 feet of fill will be placed at building 1 and building 2 pile locations and 3 feet of fill will be placed at the substation location. Depths may be adjusted accordingly if fill thicknesses will be less.

Ultimate load capacity and settlement of the ACP foundations should be verified using load test procedures as described in ASTM D1143. Final design ACP capacities and settlements should be based upon field load testing data. The allowable axial and uplift capacities are based on a minimum factor of safety of 2.5 for skin friction and 3 for end bearing. However, ultimate capacity should be determined by the load test (ASTM D1143).

The ACPs may be subject to uplift as a result of wind and seismic loading. The piles must contain sufficient continuous vertical reinforcing and embedment depth to resist the net tensile load.

Auger Cast Pile Lateral Loading

The following table lists input values for use in LPILE analyses. LPILE will estimate values of k_h and ε_{50} based on strength; however, non-default values of k_h should be used where provided. Since deflection or a service limit criterion will most likely control lateral capacity design, no safety/resistance factor is included with the parameters. Due to the variability in the soils encountered in our boring/CPTs we have provided three tables for use in design based on soil profiles encountered within the areas of building1, building 2, and the substation.

Building 1- Lateral Load Analyses						
Estimated Engineering Properties of Soils						
Top Depth / **Bottom Depth**	**Effective Unit Weight (pcf)**	**L-PILE/ GROUP Soil Type**	**Internal Angle of Friction (Degrees)**	**Cohesion (psf)**	**Coeff. of Static Subgrade Reaction K_s (pci)**	ε_{50}
0[3]	120	Stiff Clay without free water	--	1,500	--	0.007
10			--	1,500	--	0.007
10	115	Stiff Clay without free water	--	3,600	--	0.005
18			--	3,600	--	0.005
18	48	Stiff Clay without free water	--	1,000	--	0.008
27			--	1,000	--	0.008
27	58	Liquefied Sand	--	--	--	--
36			--	--	--	--
36	51	Stiff Clay without free water	--	1,400	--	0.007
59			--	1,400	--	0.007
59	63	Liquefied Sand	--	--	--	--
70			--	--	--	--
70	53	Stiff Clay without free water	--	3,000	--	0.005
110			--	3,000	--	0.005
Building 2 - Lateral Load Analyses						
Estimated Engineering Properties of Soils						

Top Depth / Bottom Depth	Effective Unit Weight (pcf)	L-PILE/ GROUP Soil Type	Internal Angle of Friction (Degrees)	Cohesion (psf)	Coeff. of Static Subgrade Reaction K_s (pci)	ε_{50}
0[3]	120	Stiff Clay without free water	--	1,500	--	0.007
10			--	1,500	--	0.007
10	115	Stiff Clay without free water	--	2,000	--	0.006
20			--	2,000	--	0.006
20	53	Stiff Clay without free water	--	1,400	--	0.007
27			--	1,400	--	0.007
27	56	Liquefied Sand	--	--	--	--
34			--	--	--	--
34	48	Stiff Clay without free water	--	1,800	--	0.007
57			--	1,800	--	0.007
57	68	Liquefied Sand	--	--	--	--
70			--	--	--	--
70	68	Reese Sand	40	--		--
82			40	--		--
82	58	Stiff Clay without free water	--	3,000	--	0.005
87			--	3,000	--	0.005

Substation - Lateral Load Analyses

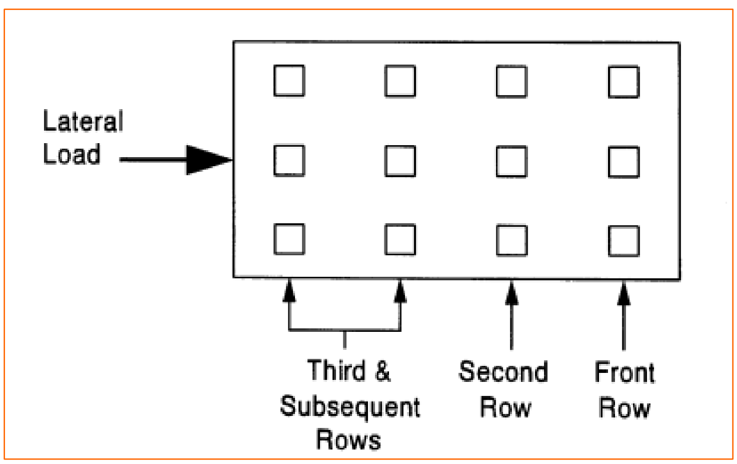
Estimated Engineering Properties of Soils

Top Depth / Bottom Depth	Effective Unit Weight (pcf)	L-PILE/ GROUP Soil Type	Internal Angle of Friction (Degrees)	Cohesion (psf)	Coeff. of Static Subgrade Reaction K_s (pci)	ε_{50}
0[3]	120	Stiff Clay without free water	--	1,500	--	0.007
3			--	1,500	--	0.007
3	120	Stiff Clay without free water	--	5,000	--	0.004
11			--	5,000	--	0.004
11	53	Stiff Clay without free water	--	2,000	--	0.006
22			--	2,000	--	0.006
22	58	Liquefied Sand	--	--	--	--
30			--	--	--	--
30	53	Stiff Clay without free water		1,800	--	0.007
52				1,800	--	0.007
52	63	Liquefied Sand	--	--	--	--
63			--	--	--	--
63	63	Reese Sand	36	--	125	--

Substation - Lateral Load Analyses						
75			36	--	125	--
75	58	Stiff Clay without free water		3,000	--	0.005
85				3,000	--	0.005

1. See **Subsurface Profile** in **Geotechnical Characterization** for more details on Stratigraphy.
2. Parameters assume groundwater is located at a depth deeper than 18, 20, and 11 feet below the top of the proposed fill at Building 1, Building 2, and the substation respectively.
3. Table assumes 10 feet of fill will be placed at Building 1 and Building 2 pile locations and 3 feet of fill will be placed at the substation location. Depths may be adjusted accordingly if fill thicknesses will be less. The upper 3 feet of the drilled pier should be neglected from design.
4. Our current scope of work included extending CPTs to a maximum depth of about 100 feet, 77 feet, and 62 feet below existing site grades at Building 1, Building 2, and the substation location respectively. We have assumed similar soils extend below the maximum explored depths. Deeper borings/CPTs shall be utilized for confirmation if needed.

When piles are used in groups, the lateral capacities of the piles in the second, third, and subsequent rows of the group should be reduced as compared to the capacity of a single, independent pile. Guidance for applying p-multiplier factors to the p values in the p-y curves for each row of pile foundations within a pile group are as follows:



- Front row: $P_m = 0.8$;
- Second row: $P_m = 0.4$
- Third and subsequent row: $P_m = 0.3$.

For the case of a single row of piles supporting a laterally loaded grade beam, group action for lateral resistance of piles would need to be considered when spacing is less than three pile diameters (measured center-to-center). However, spacing closer than 3D (where D is the diameter of the pile) is not recommended, due to potential for the installation of a new pile disturbing an adjacent installed pile, likely resulting in axial capacity reduction.

The load capacities provided herein are based on the stresses induced in the supporting soil strata. The structural capacity of the piles should be checked to assure they can safely accommodate the combined stresses induced by axial and lateral forces. The response of the ACPr foundations to lateral loads is dependent upon the soil/structure interaction as well as the pile's diameter, length, stiffness and "fixed head" or "free head" condition. The load-carrying capacity of piles may be increased by increasing the diameter and/or length.

Auger Cast Pile Construction Considerations

ACPs shall be made by rotating a continuous flight, hollow-shaft auger into the ground to design depth. High strength grout shall then be injected through the auger shaft in such a way as to exert positive upward grout pressure on the auger flights and positive lateral earth pressure shaft walls, as the auger is being withdrawn. Grout used for the ACPs should have a flow rate of 10 to 25 seconds when a ¾-inch opening (Modified Corps. of Engineers) flow cone is used.

Piles should be tested within the proposed improvements prior to commencement of foundation construction using the ASTM D1143 axial pile load test procedure. The test locations should be approved by Terracon. After completion of the standard loading cycle of the pile load test, the pile should be reloaded until failure or at least three times the design load, whichever comes first. The test pile loaded to failure should not become part of the permanent foundation system. Load testing should be performed by the auger cast pile contractor in the presence of a representative from Terracon. Cost of the load test performed by the piling contractor should be included in the base bid price.

ACP installations should be performed with equipment suitable to perform this work, by a contractor with experience with subsurface conditions similar to those encountered at this site.

We recommend that Terracon be retained to observe and document the ACP construction. Terracon should document the pile diameter, drilling elevation, tip elevation, elevation of butt, quantity of grout placed, reinforcement steel, plumbness, and the minimum penetration into the bearing soils. Significant deviations from the specified or anticipated conditions should be reported to the owner's representative, the structural engineer and the geotechnical engineer.

Driven Pile Design Parameters

Axial Loading: We anticipate driven pre-cast concrete piles could be used to support the proposed improvements at the site. Use of pre-cast concrete piles is common in this area. Compressive axial loads on pile foundations are resisted by both side skin friction along the pile and by end bearing at the base of the pile, while uplift loads are resisted solely by side friction along the pile and by the weight of the pile. Driven piles should be spaced at least three pile widths apart (center-to-center) if side friction is used for compressive loads.

Lateral Loading: The proposed improvements will be subjected to lateral loading. The lateral resistance of a pile can be estimated using L-Pile analysis.

The lateral load design L-Pile input parameters and axial load design parameters are provided in the following sections. Due to the variability in the thickness of the soil layers encountered in our borings/CPTs we have provided tables for the Building 1, Building 2, and substation areas for use in design.

Driven Pile Axial Design Parameters

Axial Design	
Description	**Recommendations**
Pile Type (assumed)	Pre-cast concrete pile
Pile Dimension (assumed)	16 inches square
Minimum Pile Embedment for Axial Design	70 feet below the existing ground surface
Total Estimated Settlement	Less than ½ inch in addition to elastic shortening

The following tables can be used for design of pile axial and uplift capacities. Skin friction and end bearing values were calculated using CivilTech AllPile V7.21h software for a single 16-inch square pre-cast concrete pile. If a different pile type or size will be used, Terracon should be consulted to provide revised design parameters. The allowable axial and uplift capacities are based on a minimum factor of safety of 2.5 for skin friction and 3 for end bearing.

Driven Pre-Cast Concrete Pile Design Summary [1]			
Approximate Depth[8] (feet)	**Stratigraphy [2]**	**Skin Friction (psf) [3]**	**End Bearing Pressure (ksf) [4]**
Building 1			
0 to 10[7]	FILL	620[5]	0[5]
10 to 18	Clay	1,400[5]	5[5]
18 to 27	Clay	400[5]	7.5[5]
27 to 36	Sand	620[5]	5.5[5]
36 to 48	Clay	560[5]	3.5[5]
48 to 59	Clay	550[5]	11[5]
59 to 70	Sand	700[5]	9.5[5]

Driven Pre-Cast Concrete Pile Design Summary [1]			
Approximate Depth[8] (feet)	**Stratigraphy [2]**	**Skin Friction (psf) [3]**	**End Bearing Pressure (ksf) [4]**
70 to 110[6]	Clay	1,200	9
Building 2			
0 to 10[7]	FILL	620[5]	0[5]
10 to 20	Clay	790[5]	5.5[5]
20 to 27	Clay	560[5]	7.5[5]
27 to 34	Sand	665[5]	6.5[5]
34 to 44	Clay	710[5]	5[5]
44 to 57	Clay	720[5]	7.5[5]
57 to 70	Sand	750[5]	16.5[5]
70 to 82	Sand	750	10.5
82 to 87[6]	Clay	1,200	8.5
Substation			
0 to 3[7]	FILL	200[5]	0[5]
3 to 11	Clay	1,490[5]	7.5[5]
11 to 22	Clay	790[5]	6[5]
22 to 30	Sand	520[5]	6.5[5]
30 to 45	Clay	720[5]	5[5]
45 to 52	Clay	700[5]	6.5[5]
52 to 63	Sand	525[5]	16.5[5]
63 to 75	Sand	580	9
75 to 85[6]	Clay	1,200	9

Driven Pre-Cast Concrete Pile Design Summary [1]			
Approximate Depth[8] (feet)	Stratigraphy [2]	Skin Friction (psf) [3]	End Bearing Pressure (ksf) [4]

1. Design capacities are dependent upon the method of installation, and quality control parameters. The values provided are estimates and should be verified when installation protocol have been finalized.
2. See **Geotechnical Characterization** for more details on Stratigraphy
3. Applicable for compressive loading only. Reduce to 2/3 of values shown for uplift loading. Effective weight of pile can be added to uplift load capacity.
4. Piles should extend 10 feet into the bearing stratum for end bearing to be considered.
5. For seismic design, skin friction and end bearing should be neglected for pile capacity in this layer due to dynamic down drag from potential liquefaction in the sand layers.
6. Our current scope of work included extending CPTs to a maximum depth of about 100 feet, 77 feet, and 62 feet below existing site grades at Building 1, Building 2, and the substation location respectively. We have assumed similar soils extend below the maximum explored depths. Deeper borings/CPTs shall be utilized for confirmation if needed.
7. For static design, neglect the upper 5 feet of the pile for skin friction.
8. Table assumes 10 feet of fill will be placed at Building 1 and Building 2 pile locations and 3 feet of fill will be placed at the substation location. Depths may be adjusted accordingly if fill thicknesses will be less.

Due to the potential for liquefaction of the sand layers, pile design should incorporate a dynamic drag load of 249, 260, or 279 kips per pile for the Building 1, Building 2, or the substation areas respectively. These drag loads do not include down drag due to the placement of fill across the site. The drag loads are based on the piles being 16-inch square piles. Terracon should be contacted to provide revised drag loads if piles will have a different shape or dimension. The project Structural Engineer should confirm combined drag and design loads do not exceed the structural capacity of the pile.

If desired, the drag loads could be reduced by the following methods:

- Pre-drilling oversized holes prior to pile driving and filling the resulting annular space with bentonite slurry.
- Providing a casing sleeve around the piles to separate the piles from direct contact with liquefiable soils.
- Coating the piles with bitumen to allow slippage.

Driven Pile Lateral Loading Design Parameters

The lateral design of driven piles may utilize the LPILE input parameters provided in the ***Auger Cast Pile Lateral Loading Design Parameters*** section of this report.

Driven Pile Construction Considerations

Soils that are pre-drilled should not be relied on for lateral support. We recommend the bore hole be no larger in diameter than the smallest dimension of the pile.

The contractor should select a driving hammer and cushion combination which can install the selected piling without overstressing the pile material. The hammer should have a rated energy in foot-pounds at least equal to 15 percent of the design compressive load capacity in pounds. The contractor should submit the pile driving plan and the pile hammer-cushion combination to the engineer for evaluation of the driving stresses in advance of pile installation. During driving a maximum of 10 blows per inch is recommended to reduce the potential of damage to the piles.

If practical refusal is experienced above the design embedment elevation, the pile may be on an obstruction and a replacement pile should be driven adjacent to the original pile. If this occurs, the situation should be evaluated by Terracon during the pile driving operations. The contractor should be prepared to cut or splice piles, as necessary. Splicing of piles should be in accordance with specifications provided by the project Structural Engineer.

Pile driving conditions, hammer efficiency, and stress on the pile during driving could be better evaluated during installation using a Pile Driving Analyzer (PDA). A Terracon representative should observe pile driving operations. Each pile should be observed and checked for buckling, crimping and alignment in addition to recording penetration resistance, depth of embedment, and general pile driving operations.

Vibrations during pile driving can cause settlement of fill materials and can adversely affects improvements on adjacent sites. Potential settlement of the fill materials across the site following pile driving should be planned and accounted for. The condition of improvements on adjacent sites should be documented prior to pile installation and should be monitored during construction. Pile driving should be stopped, and Terracon contacted if movement or cracking of existing improvements are observed. Monitoring vibration levels during pile driving should be considered. Although vibrations from pile driving may be below levels that will cause structural damage, they may be felt by occupants of the adjacent buildings.

Some ground heave may be experienced as a result of pile driving at each site. Therefore, it is recommended that the top elevations of the initial piles driven be surveyed. If any heave is noted after the driving of subsequent piles, the piles should be re-driven to their original top elevation. This problem can be particularly acute in pile groups.

The pile driving process should be performed under the direction of the Geotechnical Engineer. The Geotechnical Engineer should document the pile installation process including soil and groundwater conditions encountered, consistency with expected conditions, and details of the installed pile.

Indicator Piles

The subsurface profile across the site has significant variations. Geophysical testing could be utilized to help establish the subsurface soil profile between soil borings and CPTs. Variations in the required pile lengths should be anticipated and planned for. In order to help establish final pile driving criteria, we recommend installing indicator piles. The number of and locations of the indicator piles required will be dependent on the layout of the piles and the site conditions at the time of construction. Terracon should review the final foundation plans and recommend the locations and quantity of indicator piles. Indicator piles should be at least 5 feet longer than anticipated pile lengths to confirm field pile capacities. The indicator piles should be driven with the same equipment as planned for use during production pile driving. Indicator piles may be used as production piles provided the piles meet minimum lengths and no structural damage occurs to the pile during installation.

Pile load testing is recommended to further optimize the proposed pile foundation design. The contractor typically is responsible for the supplying the required equipment and materials and conducting the testing program. Pile load testing should be reviewed and monitored by Terracon and the project Structural Engineer.

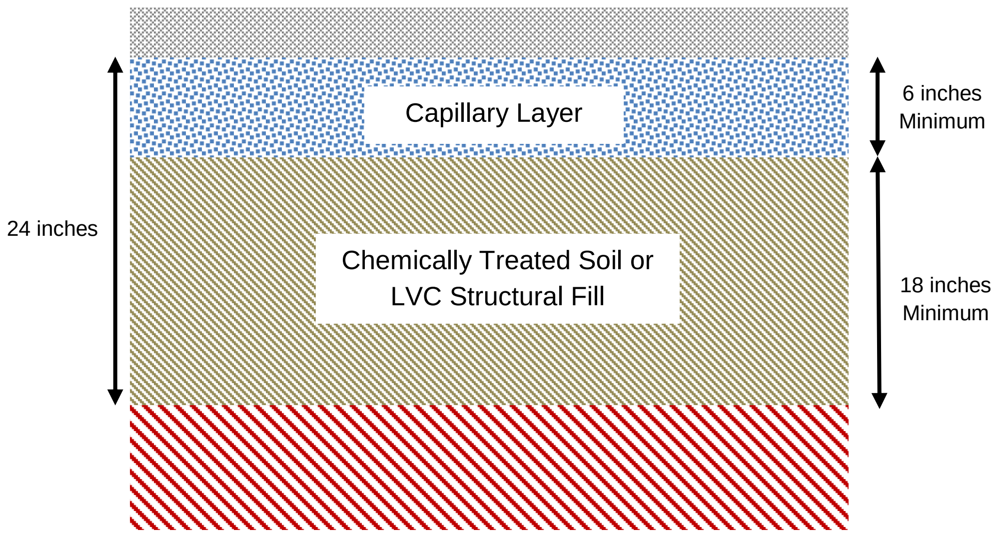
FLOOR SLABS

The surficial soils predominantly consist of moderately to highly plastic lean and fay clay with variable amounts of sand exhibiting the potential for volume change with changes in moisture. Changes in water content could cause the subgrade soils to shrink and swell damaging the slabs. Special measures should be taken to protect the floor slabs from swelling pressures of the surficial clays.

If foundation **Option 1** is selected and the improvements are supported by **Shallow Foundations** over soils mitigated by **Ground Improvement**, floor slabs may derive support from compacted subgrade soils. Floor slabs that will consist of soil supported slabs-on-grade should be underlain by a minimum 18 inches chemically treated soil or Low Volume Change (LVC) structural fill. Using 18 inches of chemically treated soil or LVC structural fill as recommended in this report may not eliminate all future subgrade volume change and resultant slab movements. However, the procedures outlined herein should help to reduce the potential for subgrade volume change. Chemically treated soil and LVC structural fill should meet the specifications and be placed and compacted as recommended in **Earthwork** section of this report. Due to the potential for significant moisture fluctuations of subgrade material beneath slabs supported at-grade, the Geotechnical Engineer should evaluate the material within 12 inches of the bottom of the chemically treated soil or LVC structural fill immediately prior to placement of additional fill or slabs. Soils below the specified water contents within this zone should be moisture conditioned or replaced with structural fill as stated in our **Earthwork** section.

If foundation **Option 2** is selected and the improvements are supported by **Deep Foundations**, building floor slabs should be designed as structural slabs that span between foundation elements and rely on the foundations for support. Such structural floor slabs could be underlain by minimum 3-inch-thick void forms as an alternative to chemically treated soil or LVC structural fill to help mitigate the effects of uplift pressures from the moderate to high volume change clays on the floor slabs.

Concrete Slab-on-Grade



Design parameters for soil supported floor slabs assume the requirements for **Earthwork** have been followed. Specific attention should be given to positive drainage away from the structures.

Floor Slab Design Parameters

Item	Description
Floor Slab Support Course [1]	Minimum 6 inches of Caltrans Class II aggregate base.
Bearing Support [2]	At least 18 inches of chemically treated soil or LVC material over firm native soil or compacted structural fill.
Estimated Modulus of Subgrade Reaction [3]	k_1 = 50 psi/in – Substation footprint k_1 = 100 psi/in – Building footprints

Item	Description
	$K_{(BxB)} = K_1 \left(\frac{B+1}{2B}\right)^2$ $K_{(BxL)} = \frac{K_{(BxB)}\left(1 + 0.5 * \left(\frac{B}{L}\right)\right)}{1.5}$ Where: k_1 = coefficient of subgrade reaction of foundations measuring 1 ft. x 1ft. $K_{(BxB)}$ = coefficient of subgrade modulus for a square foundation having dimensions BxB. $K_{(BxL)}$ = coefficient of subgrade modulus for a rectangular foundation having dimensions BxL.

1. This material is in addition to the chemically treated or LVC material. May be omitted if LVC material consists of Caltrans Class II aggregate base.
2. Floor slabs should be structurally independent of building foundations or walls to reduce the possibility of floor slab cracking caused by differential movements between the slab and foundation.
3. Modulus of subgrade reaction is an estimated value based upon our experience with the subgrade condition, the requirements noted in **Earthwork**, and the floor slab support as noted in this table.

The use of a vapor retarder should be considered beneath concrete slabs on grade covered with wood, tile, carpet, or other moisture sensitive or impervious coverings, or when the slab will support equipment sensitive to moisture. When conditions warrant the use of a vapor retarder, the slab designer should refer to ACI 302 and/or ACI 360 for procedures and cautions regarding the use and placement of a vapor retarder.

Saw-cut control joints should be placed in the slab to help control the location and extent of cracking. For additional recommendations refer to the ACI Design Manual. Joints or cracks should be sealed with a water-proof, non-extruding compressible compound specifically recommended for heavy duty concrete pavement and wet environments.

Where floor slabs are tied to perimeter walls or turn-down slabs to meet structural or other construction objectives, our experience indicates differential movement between the walls and slabs will likely be observed in adjacent slab expansion joints or floor slab cracks beyond the length of the structural dowels. The Structural Engineer should account for potential differential settlement through use of sufficient control joints, appropriate reinforcing or other means.

Floor Slab Construction Considerations

Finished subgrade, within and for at least 10 feet beyond floor slabs, should be protected from traffic, rutting, or other disturbance and maintained in a relatively moist condition until floor slabs are constructed. If the subgrade should become damaged or desiccated prior to construction of floor slabs, the affected material should be removed, and structural fill should be added to replace the resulting excavation. Final conditioning of the finished subgrade should be performed immediately prior to placement of the floor slab support course.

The Geotechnical Engineer should approve the condition of the floor slab subgrades immediately prior to placement of the floor slab support course, reinforcing steel, and concrete. Attention should be paid to high traffic areas that were rutted and disturbed earlier, and to areas where backfilled trenches are located.

Exterior Hardscape

In order to help protect exterior hardscape against the swell pressure of moderately to highly plastic clays, we recommend the hardscapes be underlain by 18 inches of chemically treated soil or LVC structural fill.

Exterior hardscape, exterior architectural features, and utilities may experience some movement due to the volume change of the subgrade soils. To reduce the potential for damage caused by movement, we recommend:

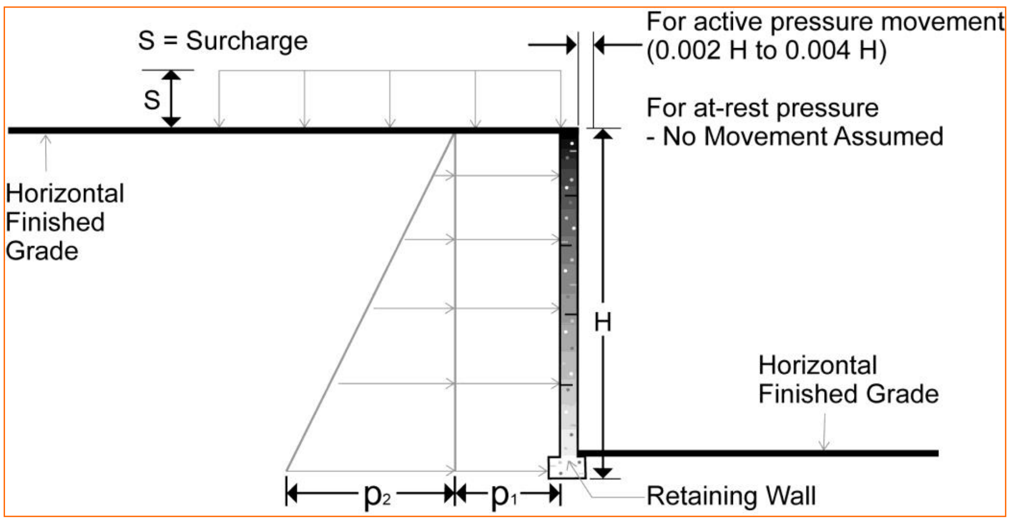
- Minimizing moisture increases in the subgrade soils and backfill;
- Controlling moisture-density during placement of fill;
- Using designs which allow vertical movement between the exterior features and adjoining structural elements;
- Placing effective control joints on relatively close centers.
- Ensure clay subgrade soils are in a moist condition prior to slab construction.
- Reinforce exterior slabs and flatwork with a minimum No. 4 bars at 12 inches on center.

LATERAL EARTH PRESSURES

Design Parameters

We understand retaining walls are not planned for this site. However, below-grade walls will be required to facilitate construction of building elevator pits. Construction of below grade walls for building elevator pits may consist of cantilevered concrete retaining walls. The lateral earth pressure recommendations given in the following paragraphs are applicable to the design of retaining walls subject to slight rotation and rigid retaining or below grade walls, such as cantilever or gravity type concrete walls. These recommendations are not applicable to the design of modular block - geogrid reinforced backfill walls. Recommendations covering these types of wall systems are beyond the scope of services for this assignment. However, we would be pleased to develop recommendations for the design of such wall systems upon request.

Structures with unbalanced backfill levels on opposite sides should be designed for earth pressures at least equal to values indicated in the following table. Earth pressures will be influenced by structural design of the walls, conditions of wall restraint, methods of construction and/or compaction and the strength of the materials being restrained. Two wall restraint conditions are shown in the diagram below. Active earth pressure is commonly used for design of free-standing cantilever retaining walls and assumes wall movement. The "at-rest" condition assumes no wall movement and is commonly used for basement walls, loading dock walls, or other walls restrained at the top. The recommended design lateral earth pressures do not include a factor of safety and do not provide for possible hydrostatic pressure on the walls (unless stated).



Lateral Earth Pressure Design Parameters				
Earth Pressure Condition [1]	Coefficient for Backfill Type [2]	Surcharge Pressure [3, 4, 5] p_1 (psf)	Effective Fluid Pressures (psf) [2, 4, 5]	
			Unsaturated [6]	Saturated [6]
Active (Ka)	Structural granular fill - 0.31	(0.31)S	(40)H	(80)H
	On-site clay soils – 0.70	(0.70)S	(85)H	(110)H
At-Rest (Ko)	Structural granular fill - 0.47	(0.47)S	(55)H	(90)H
	On-site clay soils - 0.83	(0.83)S	(100)H	(120)H
Passive (Kp)	Structural granular fill - 3.25	---	(375)H	(250)H
	On-site clay soils 1.42	---	(175)H	(40)H

1. For active earth pressure, wall must rotate about base, with top lateral movements 0.002 H to 0.004 H, where H is wall height. For passive earth pressure, wall must move horizontally to mobilize resistance.

Lateral Earth Pressure Design Parameters				
Earth Pressure Condition [1]	Coefficient for Backfill Type[2]	Surcharge Pressure [3, 4, 5] p_1 (psf)	Effective Fluid Pressures (psf) [2, 4, 5]	
			Unsaturated[6]	Saturated[6]

2. Uniform, horizontal backfill, compacted to at least 90 percent of the ASTM D 1557 maximum dry density, rendering a maximum unit weight of 120 pcf.
3. Uniform surcharge, where S is surcharge pressure. The project structural engineer should provide any surcharge loading.
4. Loading from heavy compaction equipment is not included.
5. No safety factor is included in these values.
6. To achieve "Unsaturated" conditions, follow guidelines in **Subsurface Drainage for Below-Grade Walls** below. "Saturated" conditions are recommended when drainage behind walls is not incorporated into the design.

Backfill placed against structures should consist of granular soils or low plasticity cohesive soils. For the granular values to be valid, the granular backfill must extend out and up from the base of the wall at an angle of at least 45 and 60 degrees from vertical for the active/at-rest and passive cases, respectively.

Total lateral earth pressure acting on retaining or below grade walls during a seismic event will likely include the active or at-rest static force and dynamic increment. The active dynamic increment should be applied to the wall as resultant force acting at 0.6H height from the base of the wall and the at-rest dynamic increment should be applies to the wall as resultant force acting at 0.63H height from the base of the wall. Such increments should be added to the static earth pressures. A dynamic lateral earth resultant force of $15H^2$ (in units of pounds per linear foot (plf), where H (in units of feet) is the height of the soil behind the wall[1] should be used in design.

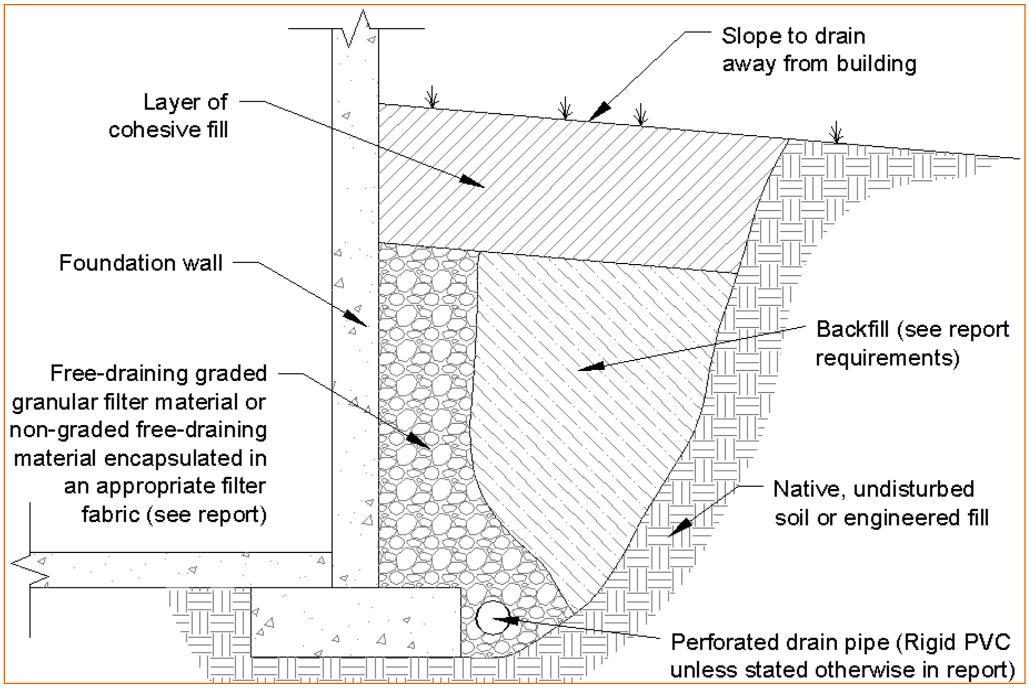
Heavy equipment should not operate within a distance closer than the exposed height of retaining or below grade walls to prevent lateral pressures more than those provided. Compaction of each lift adjacent to wall should be accomplished with hand-operated tampers for other lightweight compactors. Over-compaction may cause excessive lateral earth pressures which could result in wall movement.

Subsurface Drainage for Below-Grade Walls

A perforated rigid plastic drain line installed behind the base of walls and extends below adjacent grade is recommended to prevent hydrostatic loading on the walls. The invert of a drain line around a below-grade building area or exterior retaining wall should be placed near foundation bearing level. The drain line should be sloped to provide positive gravity drainage to daylight or to a sump pit and pump. The drain line should be surrounded by clean, free-draining granular

[1] *Seed & Whitman (1970)*

material having less than 5% passing the No. 200 sieve, such as No. 57 aggregate. The free-draining aggregate should be encapsulated in a filter fabric. The granular fill should extend to within 12 inches of final grade, where it should be capped with compacted cohesive fill to reduce infiltration of surface water into the drain system.



As an alternative to free-draining granular fill, a pre-fabricated composite drain may be used. A pre-fabricated composite drain is a plastic drainage core which is covered with filter fabric to prevent soil intrusion and is fastened to the wall prior to placing backfill.

PERCOLATION TESTING AND STORM WATER INFILTRATION

We performed four percolation tests within the proposed site development for use by the project civil engineers in the design on the storm water retention system. Four borings, I1, I2, P3, and I4, were drilled to depths ranging from approximately 3½ to 5 feet bgs. The approximate locations of the test holes are shown on the **Exploration Plan**.

After drilling the test holes, we placed approximately 2 inches of gravel in the bottom of each hole, then placed a slotted PVC pipe in each hole, and filled the annual space around the pipe with gravel. The test holes were filled with water and left to saturate for a minimum 24 hours. We then filled the shallow holes with water to depths ranging from 1½ to 3½ feet and measured the drop-in water surface over a period varying from approximately 3 to 8 hours depending on the hole, refilling the holes as necessary to maintain the desired head.

The measured percolation and infiltration rates are summarized below.

Perc Test #	Depth (ft)	Avg. Head (ft)	Perc. Rate (min/inch)	Perc. Rate (inch/hr)	Infiltration Rate (inch/hr)
I1	3½	1.5	31.3	1.92	0.237
I2	4½	0.84	0	0	0
P3	5	1.26	500	0.12	0.01
I4	5	0.60	750	0.08	0.02

Since we used test borings to perform percolation testing, we have used the Porchet formula (aka Inverse Borehole Formula) to calculate the test infiltration rate which takes into account sidewall area of the bore hole. Since our test was performed using clean water, the storm water runoff may likely contain materials such as silt, leaves, oil residues, and other matter that may reduce the percolation characteristics of the soil. We therefore recommend that a filtration system be implemented into the design and installed. An appropriate safety factor should be applied to the measured infiltration rates by the designer for use in design and be based on the amount of filtration designed into the system, at a minimum a Safety Factor of 2 shall be utilized. The values above are clear water rates and do not have a safety factor applied. In addition, we recommend a regular maintenance program be implemented to monitor the storm drainage/filtration system prior to the beginning of each wet weather season.

We have provided the following considerations for the design and construction of the retention/detention facilities. Planned retention/detention facilities should be located no closer than 10 feet to structural site improvements.

The long-term infiltration rates will depend on many factors, and can vary or be reduced if the following conditions are present:

- Fill placement,
- Groundwater table,
- Variability of site soils,
- Fine layering of soils, or
- Maintenance and pre-treatment (filtration) of the influent are not performed regularly

Given the fine-grained nature of the near surface soils encountered on this site and the slow infiltration rates measured, it is our opinion that the native soils may not be very conductive for bioretention system design. We understand the site needs to handle the on-site storm water. Additional storage capacity of the proposed system may be needed.

<u>Fill Placement:</u> As indicated in the report, we anticipate earthwork required to develop the site may consist of cuts up to 15 feet and fills up to 10 feet. It is unknown whether final grades will

consist of native material or imported fill. As a result, the percolation tests performed may not be representative of the final soil conditions depending on the blend of soils utilized as structural fill and native soils exposed where large cuts are made. Additional percolation testing may be warranted following rough grading to confirm the values utilized in design are appropriate.

Subsurface Soil Variation: Variations in subsurface soil conditions and the presence of fine layering can affect the infiltration rate of the receptor soils. As shown in the **Geomodel** section of this report, fine grained alluvial soils (clay) were encountered in most of our borings to depths ranging from about 15 to 30 feet bgs. Fine-grained soils were underlain by sands with variable amounts of clay and gravel. Due to variation in thickness of the upper surface fine grained soils, infiltration rates may vary across the site.

Groundwater: Based on the measured depth to groundwater ranging from approximately 4 to 25 feet bgs in our borings/CPTs at the time of our field exploration, we anticipate that there could be difficulty in maintaining adequate separation (greater than 5 feet) between the bottom of the system and groundwater depending on the final grades across the site.

Construction Considerations: The infiltration rate of the receptor soils will be reduced in the event that fine sediment, organic materials, and/or oil residue are allowed to accumulate in the retention facilities. The use of a filtration system is highly recommended as well as a maintenance program.

Operation of heavy equipment during construction may densify the receptor soils below the infiltration facility. The soils exposed in the bottom of the infiltration facility should not be compacted and should remain in their native condition. This may require scarification of the soils prior to construction.

Maintenance of Facilities: Satisfactory long-term performance of an infiltration facility will require some degree of maintenance. Accumulations of sediment, organic materials, or other material that serve to reduce their permeability of the receptor soils should be removed from the filtration system on a regular basis so as not to enter the retention system. The filtration system shall have a rigorous maintenance program, debris from the filtration maintenance should be disposed of at an approved facility in accordance with applicable regulation.

FIELD ELECTRICAL RESISTIVITY

In-situ electrical resistivity testing was performed during our field explorations in general accordance with ASTM G57, utilizing the Wenner Four-Electrode Method. Each test consisted of two mutually perpendicular arrays approximately oriented in the north-south and east-west directions. Within a test array, potential electrodes are created on a transverse line between the current electrodes. An equal "A" spacing between electrodes is maintained. In-situ electrical resistivity measurements were taken at four test locations. Individual in-situ electrical resistivity values at various "A" spacings along each array are summarized in the table provided in the results. A Geophysical Survey Report (Project No. NS205134, Dated November 12, 2020) presenting the field test results and approximate test locations is presented in **Exploration Results** of this report.

As indicated, grading is anticipated to consist of cuts up to 15 feet and fills up to 10 feet to develop the site. Subsequently, field resistivity testing was conducted in areas that may experience these cuts and fills. As a result, the field resistivity test results for the existing subgrade conditions could vary from actual conditions following rough grading. Therefore, additional testing may be warranted following rough grading to obtain final field resistivity values.

THERMAL RESISTIVITY

We understand that new underground utilities will be installed as part of part of the proposed data center construction. Terracon collected bulk samples in borings B5, B8, B9, B10, B11, P1, P5, and I4 for testing to be completed on each sample. Laboratory compaction characteristics testing was performed on the samples in accordance with ASTM D1557. In accordance with IEEE Standard 442-2017, thermal resistivity testing was performed on the samples at 90 percent of maximum dry density as determined by ASTM D1557. The test results are presented in **Exploration Results** section of this report.

As indicated, grading is anticipated to consist of cuts up to 15 feet and fills up to 10 feet to develop the site. Thermal conductivity was performed on the native subsurface soils. Therefore, additional testing may be warranted following rough grading to obtain final thermal conductivity values. Terracon can collect fill samples and perform additional thermal resistivity tests if desired.

CORROSIVITY

The table below lists the results of laboratory soluble sulfate, soluble chloride, electrical resistivity, and pH testing. The values may be used to estimate potential corrosive characteristics of the on-site soils with respect to contact with the various underground materials which will be used for project construction.

Corrosivity Test Results Summary

Boring	Sample Depth (feet)	Soil Description	Soluble Sulfate (%)	Soluble Chloride (%)	Electrical Resistivity (Ω-cm)	pH
B2	1 to 2½	Silty Sand	0.0133	0.0038	2,959	7.81
B3	20 to 21½	Sand	0.0182	0.0060	1,455	8.04
B5	5 to 6½	Lean Clay	0.0122	0.0113	644	7.55

Corrosivity Test Results Summary						
Boring	**Sample Depth (feet)**	**Soil Description**	**Soluble Sulfate (%)**	**Soluble Chloride (%)**	**Electrical Resistivity (Ω-cm)**	**pH**
B6	2½ to 4	Lean Clay	0.0103	0.0055	1,067	7.24
B13	5 to 6½	Lean Clay	0.0212	0.0045	1,213	7.44
B14	1 to 2½	Fat Clay	0.0115	0.0053	1,164	7.76

These test results are provided to assist in determining the type and degree of corrosion protection that may be required for the project. We recommend that a certified corrosion engineer determine the need for corrosion protection and design appropriate protective measures.

Resistivity

The resistivity values indicate the samples tested exhibit a moderate to very high corrosive potential to buried metal pipes at the respective borings where the samples were collected. Evaluation of the test results is based upon the guidelines of J.F. Palmer, "Soil Resistivity Measurements and Analysis", Materials Performance, Volume 13, January 1974. The following table outlines the guidelines for soil resistivity for corrosion potential.

Corrosion Potential of Soil on Steel	
Soil Resistivity (ohm-cm)	**Corrosion Potential**
0 to 1,000	Very High
1,000 to 2,000	High
2,000 to 5,000	Moderate
> 5,000	Mild

Sulfates

The sulfate test results indicate that the soil from collected from the noted borings classifies as Class S0 according to Table 19.3.1.1 of ACI 318-14. This indicates that the sulfate severity is negligible when considering corrosion to concrete. Based on the sulfate content test results, ACI 318-14, Section 19.3 does not specify a required type of cement or a maximum water-cement ratio for concrete for sulfate Class S0. For further information, see ACI 318-14, Section 19.3.

Laboratory pH

Data suggests the soil pH should not be the dominant soil variable affecting soil corrosion if the soil has a pH in the 5 to 8 range. The pH of the samples collected and tested from Borings B2, B5, B6, B13, and B14 were within the recommended range, and should therefore not need to be considered when determining soil corrosion potential. However, the sample collected and tested from Boring B3 was slightly above the recommended range and should therefore be considered when determining soil corrosion potential.

GENERAL COMMENTS

Our analysis and opinions are based upon our understanding of the project, the geotechnical conditions in the area, and the data obtained from our site exploration. Natural variations will occur between exploration point locations or due to the modifying effects of construction or weather. The nature and extent of such variations may not become evident until during or after construction. Terracon should be retained as the Geotechnical Engineer, where noted in this report, to provide observation and testing services during pertinent construction phases. If variations appear, we can provide further evaluation and supplemental recommendations. If variations are noted in the absence of our observation and testing services on-site, we should be immediately notified so that we can provide evaluation and supplemental recommendations.

Our Scope of Services does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

Our services and any correspondence or collaboration through this system are intended for the sole benefit and exclusive use of our client for specific application to the project discussed and are accomplished in accordance with generally accepted geotechnical engineering practices with no third-party beneficiaries intended. Any third-party access to services or correspondence is solely for information purposes to support the services provided by Terracon to our client. Reliance upon the services and any work product is limited to our client, and is not intended for third parties. Any use or reliance of the provided information by third parties is done solely at their own risk. No warranties, either express or implied, are intended or made.

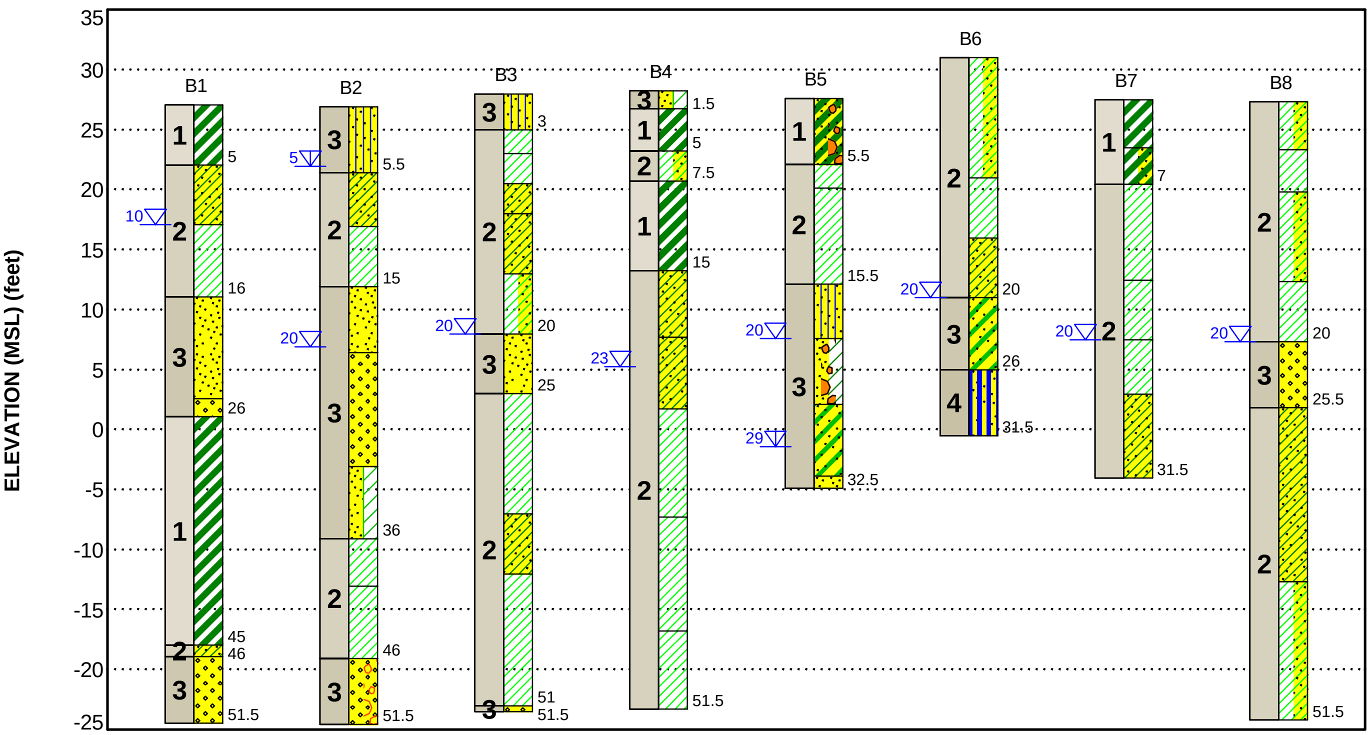
Site characteristics as provided are for design purposes and not to estimate excavation cost. Any use of our report in that regard is done at the sole risk of the excavating cost estimator as there may be variations on the site that are not apparent in the data that could significantly impact excavation cost. Any parties charged with estimating excavation costs should seek their own site characterization for specific purposes to obtain the specific level of detail necessary for costing. Site safety, and cost estimating including, excavation support, and dewatering requirements/design are the responsibility of others. If changes in the nature, design, or location of the project are planned, our conclusions and recommendations shall not be considered valid unless we review the changes and either verify or modify our conclusions in writing. This report should not be used after 3 years without written authorization from Terracon.

FIGURES

Contents:

GEOMODEL

Bay AZ1 Site (Northtown) ■ San Jose, CA
Terracon Project No. ND205079



This is not a cross section. This is intended to display the Geotechnical Model only. See individual logs for more detailed conditions.

Model Layer	Layer Name	General Description
1	Fat Clay	Medium to very stiff fat clay with variable amounts of sand and gravel.
2	Lean Clay	Soft to very stiff lean clay with variable amounts of sand.
3	Sand	Very loose to dense sand with variable amounts of clay, silt, and gravel.
4	Silt	Medium to very stiff sandy silt.

LEGEND

- Fat Clay
- Sandy Lean Clay
- Lean Clay
- Poorly-graded Sand
- Well-graded Sand
- Silty Sand
- Poorly-graded Sand with Clay
- Well-graded Sand with Gravel
- Lean Clay with Sand
- Sandy Fat Clay with Gravel
- Poorly-graded Sand with Clay and Gravel
- Clayey Sand
- Sandy Silt
- Fat Clay with Sand

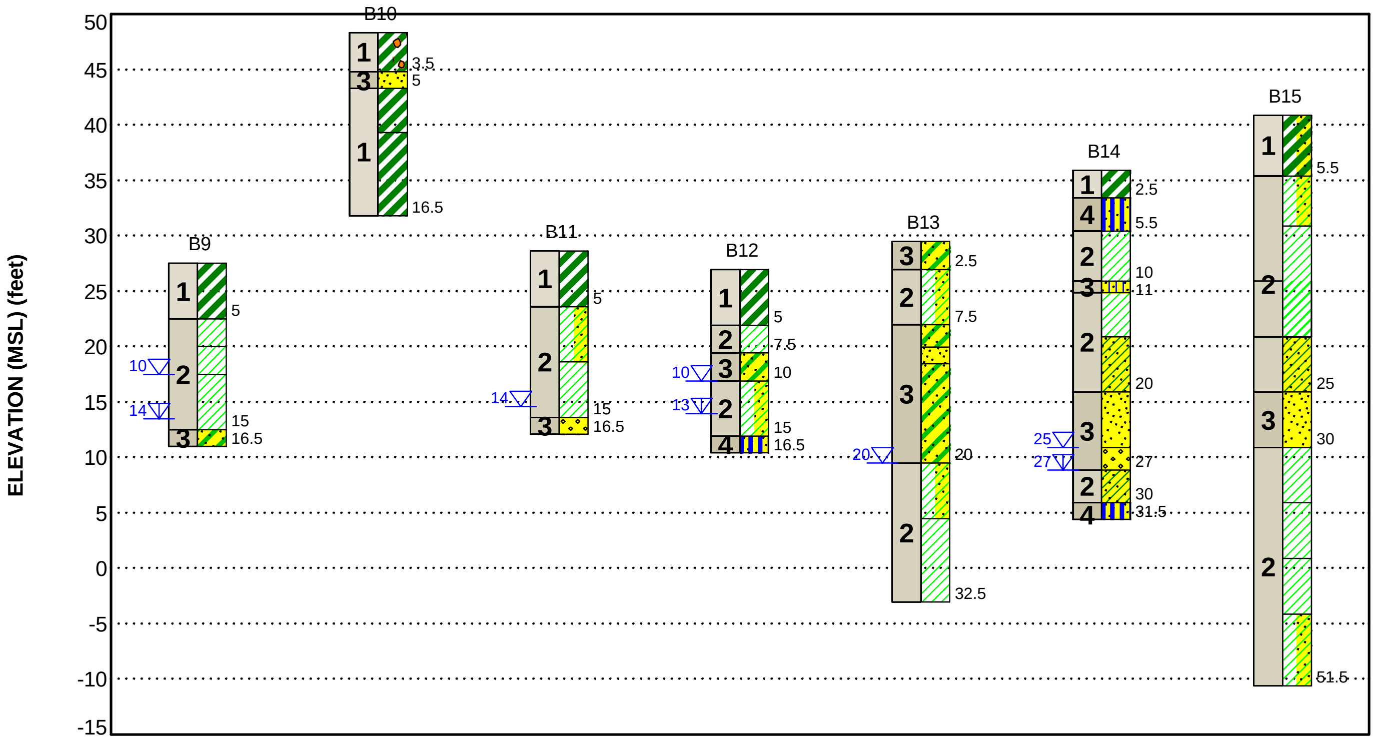
First Water Observation
Second Water Observation

Groundwater levels are temporal. The levels shown are representative of the date and time of our exploration. Significant changes are possible over time.
Water levels shown are as measured during and/or after drilling. In some cases, boring advancement methods mask the presence/absence of groundwater. See individual logs for details.

NOTES:
Layering shown on this figure has been developed by the geotechnical engineer for purposes of modeling the subsurface conditions as required for the subsequent geotechnical engineering for this project.
Numbers adjacent to soil column indicate depth below ground surface.

GEOMODEL

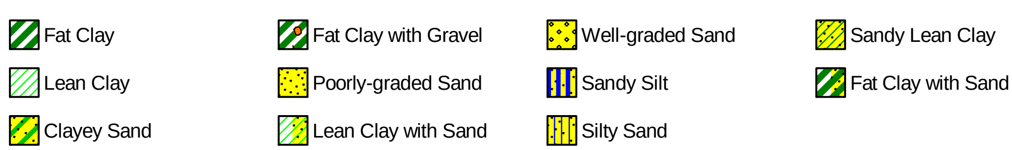
Bay AZ1 Site (Northtown) ■ San Jose, CA
Terracon Project No. ND205079



This is not a cross section. This is intended to display the Geotechnical Model only. See individual logs for more detailed conditions.

Model Layer	Layer Name	General Description
1	Fat Clay	Medium to very stiff fat clay with variable amounts of sand and gravel.
2	Lean Clay	Soft to very stiff lean clay with variable amounts of sand.
3	Sand	Very loose to dense sand with variable amounts of clay, silt, and gravel.
4	Silt	Medium to very stiff sandy silt.

LEGEND



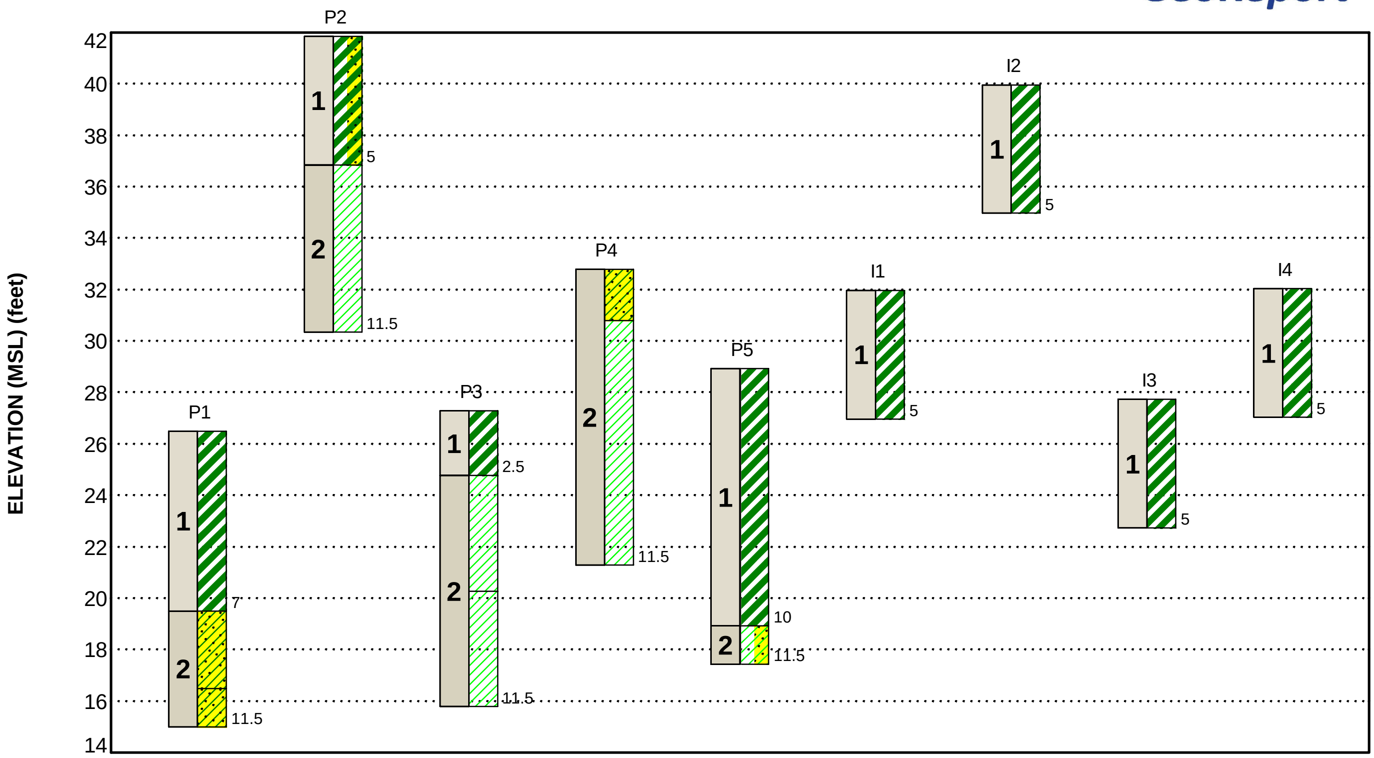
First Water Observation
Second Water Observation

Groundwater levels are temporal. The levels shown are representative of the date and time of our exploration. Significant changes are possible over time.
Water levels shown are as measured during and/or after drilling. In some cases, boring advancement methods mask the presence/absence of groundwater. See individual logs for details.

NOTES:
Layering shown on this figure has been developed by the geotechnical engineer for purposes of modeling the subsurface conditions as required for the subsequent geotechnical engineering for this project.
Numbers adjacent to soil column indicate depth below ground surface.

GEOMODEL

Bay AZ1 Site (Northtown) ■ San Jose, CA
Terracon Project No. ND205079



This is not a cross section. This is intended to display the Geotechnical Model only. See individual logs for more detailed conditions.

Model Layer	Layer Name	General Description
1	Fat Clay	Medium to very stiff fat clay with variable amounts of sand and gravel.
2	Lean Clay	Soft to very stiff lean clay with variable amounts of sand.
3	Sand	Very loose to dense sand with variable amounts of clay, silt, and gravel.
4	Silt	Medium to very stiff sandy silt.

LEGEND

Fat Clay

Lean Clay

Sandy Lean Clay

Lean Clay with Sand

Fat Clay with Sand

First Water Observation

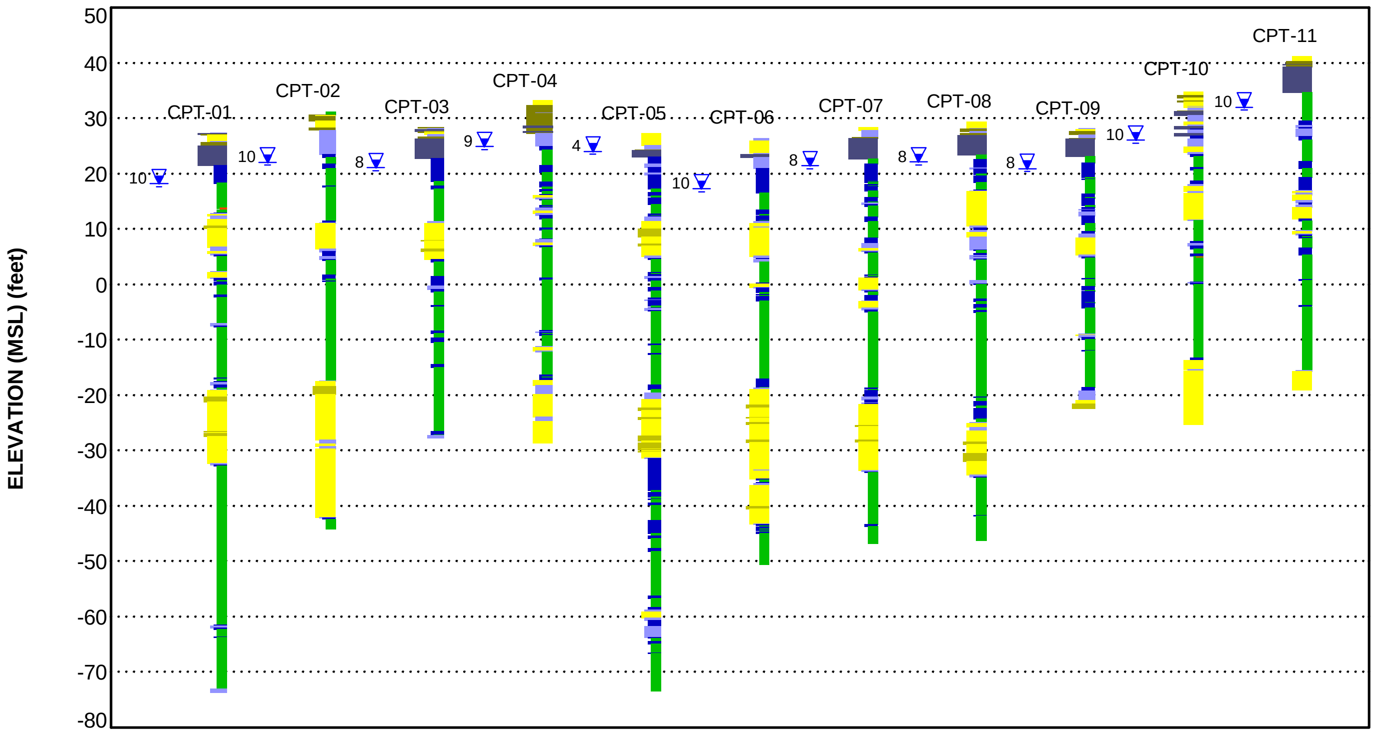
Second Water Observation

Groundwater levels are temporal. The levels shown are representative of the date and time of our exploration. Significant changes are possible over time.
Water levels shown are as measured during and/or after drilling. In some cases, boring advancement methods mask the presence/absence of groundwater. See individual logs for details.

NOTES:
Layering shown on this figure has been developed by the geotechnical engineer for purposes of modeling the subsurface conditions as required for the subsequent geotechnical engineering for this project.
Numbers adjacent to soil column indicate depth below ground surface.

GEOMODEL

Bay AZ1 Site (Northtown) ■ San Jose, CA
Terracon Project No. ND205079



This is not a cross section. This is intended to display the Geotechnical Model only. See individual logs for more detailed conditions.

Model Layer	Layer Name	General Description
1	Fat Clay	Medium to very stiff fat clay with variable amounts of sand and gravel.
2	Lean Clay	Soft to very stiff lean clay with variable amounts of sand.
3	Sand	Very loose to dense sand with variable amounts of clay, silt, and gravel.
4	Silt	Medium to very stiff sandy silt.

LEGEND

Soil Behavior Type (SBT)

- 1 Sensitive, fine grained
- 2 Organic soils - clay
- 3 Clay - silty clay to clay
- 4 Silt mixtures - clayey silt to silty clay
- 5 Sand mixtures - silty sand to sandy silt
- 6 Sands - clean sand to silty sand
- 7 Gravelly sand to dense sand
- 8 Very stiff sand to clayey sand
- 9 Very stiff fine grained

CPT Assumed Water Depth

First Water Observation
Second Water Observation

Groundwater levels are temporal. The levels shown are representative of the date and time of our exploration. Significant changes are possible over time.
Water levels shown are as measured during and/or after drilling. In some cases, boring advancement methods mask the presence/absence of groundwater. See individual logs for details.

NOTES:
Layering shown on this figure has been developed by the geotechnical engineer for purposes of modeling the subsurface conditions as required for the subsequent geotechnical engineering for this project.
Numbers adjacent to soil column indicate depth below ground surface.

ATTACHMENTS

EXPLORATION AND TESTING PROCEDURES

Field Exploration

Number of Borings	Boring Depth (feet)	Planned Location
6	51½	Planned building and substation footprints
5	31½	Planned substation footprints
4	16½	Planned building footprints
4	11½	Planned parking and drive areas
2 CPT[1]	100 - 100½	Planned building footprints
4 CPT[1]	75½ - 77	Planned building footprint and substation footprints
3 CPT[1]	60½ - 62	Planned substation footprints
2 CPT[1]	50½ - 56	Planned substation footprints
4[2]	5	Planned bio-retention facility

1. Cone penetration test.
2. Percolation test boreholes

Boring/CPT Layout/Elevations: The boring/CPT layout was performed by Terracon. The boring/CPT locations and elevations were surveyed and provided by HMH Engineers.

Subsurface Exploration Procedures: We advanced borings with a truck-mounted drill rigs using continuous flight, hollow stem augers. Two to four samples were obtained in the upper 10 feet of each boring and at intervals of 5 feet thereafter. Additionally, bulk samples were collected in upper 5 feet of borings B5, B8 through B12, P1 through P5, and I4. Soil sampling was performed using split-barrel sampling and thin-walled tube sampling. In the split-barrel sampling procedure, a standard 2-inch outer diameter split-barrel sampling spoon or 2.5-inch outer diameter Modified California split-barrel sampling spoon was driven into the ground by a 140-pound automatic hammer falling a distance of 30 inches. The number of blows required to advance the sampling spoon the last 12 inches of a normal 18-inch penetration is recorded as the Standard Penetration Test (SPT) resistance value. Tube-lined, split-barrel sampling procedures are similar to standard split spoon sampling procedure; however, blow counts are not equivalent to the SPT blow counts. The values provided on our boring logs are uncorrected. In the thin-walled tube sampling, a standard 30-inch long Shelby Tube with 3-inch outer diameter was directly driven into the ground where undisturbed soil samples were collected. Additionally, we observed and recorded groundwater levels during drilling and sampling. Per the requirements of the local health department and for safety purposes, all borings were backfilled with grout after their completion.

For the cone penetrometer testing, the CPT rig hydraulically pushes an instrumented cone through the soil while nearly continuous readings are recorded to a portable computer. The cone is equipped with electronic load cells to measure tip resistance and sleeve resistance and a pressure transducer to measure the generated ambient pore pressure. The face of the cone has an apex angle of 60° and an area of 15 cm^2. Digital Data representing the tip resistance, friction resistance, pore water pressure, and probe inclination angle are recorded about every 2 centimeters while advancing through the ground at a rate between 1½ and 2½ centimeters per second. These measurements are correlated to various soil properties used for geotechnical design. No soil samples are gathered through this subsurface investigation technique. CPT testing was conducted in general accordance with ASTM D5778 "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils."

The sampling depths, penetration distances, and other sampling information were recorded on the field boring logs. The samples were placed in appropriate containers and taken to a Terracon soil laboratory for testing and classification by a geotechnical engineer. Our exploration team prepared field boring logs as part of the drilling operations. These field logs include visual classifications of the materials encountered during drilling and our interpretation of the subsurface conditions between samples. Final boring logs were prepared from the field logs. The final boring logs represent the geotechnical engineer's interpretation of the field logs and include modifications based on observations and tests of the samples in our laboratory.

Laboratory Testing

The project engineer reviewed the field data and assigned laboratory tests to understand the engineering properties of the various soil strata, as necessary, for this project. Procedural standards noted below are for reference to methodology in general. In some cases, variations to methods were applied because of local practice or professional judgment. Standards noted below include reference to other, related standards. Such references are not necessarily applicable to describe the specific test performed.

- ASTM D2216 Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
- ASTM D4318 Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils
- ASTM D1140 Standard Test Method for Determining the Amount of Material Finer than No. 200 Sieve by Soil Washing
- ASTM D2166/D2166M Standard Test Method for Unconfined Compressive Strength of Cohesive Soil
- ASTM D2435/D2435M Standard Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading
- ASTM D3080 Standard Test Method for Direct Shear Test of Soils Under Consolidation Drained Conditions

- ASTM G162 – 99 Standard Practice for Conducting and Evaluating Laboratory Corrosion Tests in Soils
- ASTM D5334 Standard Test Method for Determination of Thermal Conductivity of Soil and Soft Rock by Thermal Needle Probe Procedure
- AASHTO T 99 Standard Test Method for Moisture-Density Relations of Soil
- ASTM D1883 Standard Test Method for California Bearing Ratio (CBR) of Laboratory-Compacted Soils

The laboratory testing program often included examination of soil samples by an engineer. Based on the material's texture and plasticity, we described and classified the soil samples in accordance with the Unified Soil Classification System.

PHOTOGRAPHY LOG

Photo 1 – Approximately 10 feet northwest of boring B3 facing southeast

Photo 2 – Near boring B15 facing northwest

Photo 3 – Near boring B6 facing northeast

Photo 4 – Between boring B1 and B2 facing southwest

SITE LOCATION AND EXPLORATION PLANS

Contents:

Note: All attachments are one page unless noted above.

SITE LOCATION

Bay AZ1 Site (Northtown) ▪ San Jose, California
December 15, 2020 ▪ Terracon Project No. ND205079

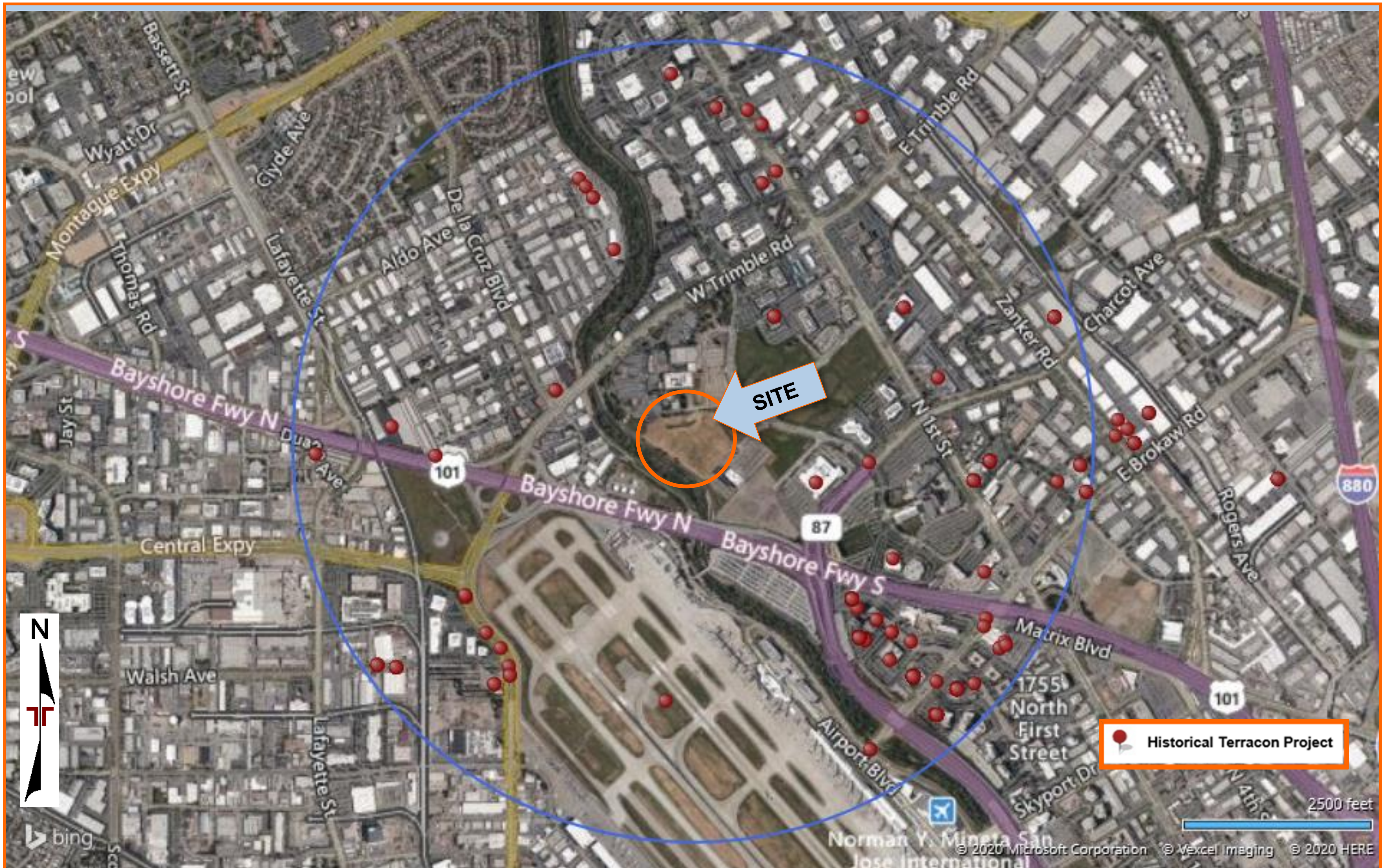


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS

EXPLORATION PLAN

Bay AZ1 Site (Northtown) ■ San Jose, California

December 15, 2020 ■ Terracon Project No. ND205079

Terracon GeoReport

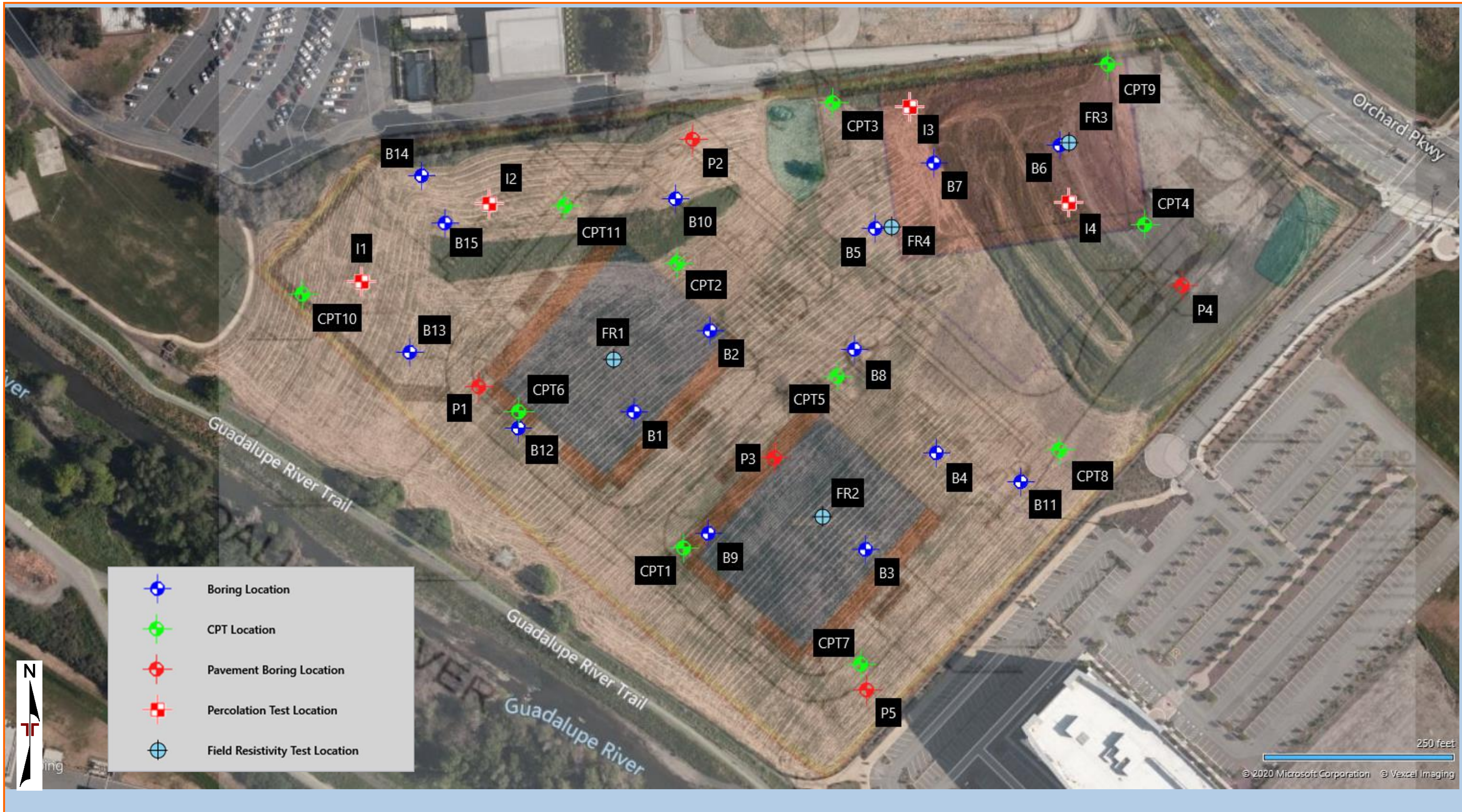


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS

EXPLORATION RESULTS

Contents:

Note: All attachments are one page unless noted above.

BORING LOG NO. B1

PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc Columbus, OH

SITE: Orchard Parkway San Jose, CA

LOCATION See Exploration Plan

Northing: 37.3782205 Easting: -121.9335425

Surface Elev.: 27.06 (Ft.)

MODEL LAYER	DEPTH (Ft.)	DESCRIPTION	ELEVATION (Ft.)
1	5.0	**FAT CLAY (CH)**, trace sand, fine grained, dark brown, very stiff	22
2	10.0	**SANDY LEAN CLAY (CL)**, fine grained, brown to dark brown, medium stiff to stiff	17
2	16.0	**LEAN CLAY (CL)**, fine grained, brown with red brown mottling, stiff to very stiff	11
3	24.5	**POORLY GRADED SAND (SP)**, fine to medium grained, gray, dense to very dense	2.5
3	26.0	**WELL GRADED SAND (SW)**, fine to coarse grained, grayish brown, very loose	1
1		**FAT CLAY (CH)**, some sand, fine grained, gray, stiff to very stiff	

FIELD TEST RESULTS	LABORATORY HP (tsf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
7-16-17	>4.5	10	91	59-21-38	90
8-8-10 N=18	4.5 (HP)	24			
10-16-21	4.0 (HP)	20	100		
2-3-4 N=7	1.25 (HP)	22			
9-6-10	2.0 (HP)	29	93	41-23-18	97
7-9-9 N=18		15			
10-31-46		16	112		33
1-1-1 N=2	1.0 (HP)	30			
8-12-11	1.0 (HP)	32	89		

DEPTH (Ft.): 5, 10, 15, 20, 25, 30, 35

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
6" Hollow Stem Auger

Abandonment Method:
Boring backfilled with bentonite grout upon completion

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by HMH

Notes:

Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.
Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.

WATER LEVEL OBSERVATIONS

▽ *At completion of drilling*

Terracon
5075 Commercial Cir Ste E
Concord, CA

Boring Started: 10-26-2020	Boring Completed: 10-26-2020
Drill Rig: CME 95	Driller: Baja Exploration
Project No.: ND205079	

BORING LOG NO. B1

PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc
Columbus, OH

SITE: Orchard Parkway
San Jose, CA

LOCATION See Exploration Plan

Northing: 37.3782205 Easting: -121.9335425

Surface Elev.: 27.06 (Ft.)

MODEL LAYER	DEPTH (ft.)	MATERIAL DESCRIPTION	ELEVATION (Ft.)	DEPTH (In.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	LABORATORY HP (tsf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1		**FAT CLAY (CH)**, some sand, fine grained, gray, stiff to very stiff *(continued)*					7-4-8 N=12	1.75 (HP)			52-22-30	99
				40			6-11-12	2.0 (HP)	36			
	45.0		-18									
2	46.0	**SANDY LEAN CLAY (CL)**, fine grained, brown, very hard	-19	45			6-39-45 N=84	1.25 (HP)	30			
3		**WELL GRADED SAND (SW)**, fine to coarse grained, brown and black, medium dense										
	51.5		-24.5	50			3-6-9 No Recovery		22	98		

Boring Terminated at 51.5 Feet

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
6" Hollow Stem Auger

Abandonment Method:
Boring backfilled with bentonite grout upon completion

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by HMH

Notes:

WATER LEVEL OBSERVATIONS

At completion of drilling

Terracon
5075 Commercial Cir Ste E
Concord, CA

Boring Started: 10-26-2020	Boring Completed: 10-26-2020
Drill Rig: CME 95	Driller: Baja Exploration
Project No.: ND205079	

BORING LOG NO. B2

PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc Columbus, OH

SITE: Orchard Parkway San Jose, CA

LOCATION See Exploration Plan

Northing: 37.3785126 Easting: -121.9331984

Surface Elev.: 26.92 (Ft.)

Model Layer	Depth (Ft.)	Description	Elevation (Ft.)
3	0.0–5.5	**SILTY SAND (SM)**, fine grained, brown, medium dense	21.5
2	5.5–10.0	**SANDY LEAN CLAY (CL)**, fine to medium grained, brown with gray mottling, stiff to very stiff	17
2	10.0–15.0	**LEAN CLAY (CL)**, trace sand, fine grained, gray with black, medium stiff to stiff	12
3	15.0–20.5	**POORLY GRADED SAND (SP)**, fine to medium grained, grayish brown, very dense	6.5
3	20.5–30.0	**WELL GRADED SAND (SW)**, trace gravel, medium to coarse grained, dark gray, dense to very dense	-3
3	30.0–	**POORLY GRADED SAND WITH CLAY (SP)**, fine to medium grained, gray, medium dense	

Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Laboratory HP (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
1		Split spoon	6-11-5 N=16	3.5 (HP)	5			
2.5		Ring	7-9-12	3.5 (HP)	6	90		
5	▽							
5		Split spoon	2-5-10 N=15	2.5 (HP)	17			
7.5		Ring	7-10-18	2.0 (HP)	22	100	32-20-12	63
10		Split spoon	6-1-6 N=7	2.25 (HP)	32			
15		Ring	9-25-50		18	105		
20	▽							
20		Split spoon	14-25-46 N=71		17			
25		Ring	6-16-25		15	93		
30		Split spoon	4-7-8 N=15		23			

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
6" Hollow Stem Auger

Abandonment Method:
Boring backfilled with bentonite grout upon completion

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by HMH

Notes:

Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.
Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.

WATER LEVEL OBSERVATIONS

▽ *While sampling*

▽ *At completion of drilling*

Terracon
5075 Commercial Cir Ste E
Concord, CA

Boring Started: 10-26-2020	Boring Completed: 10-26-2020
Drill Rig: CME 95	Driller: Baja Exploration
Project No.: ND205079	

BORING LOG NO. B2

PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc
Columbus, OH

SITE: Orchard Parkway
San Jose, CA

LOCATION See Exploration Plan

Northing: 37.3785126 Easting: -121.9331984

Surface Elev.: 26.92 (Ft.)

ELEVATION (Ft.)

MODEL LAYER	GRAPHIC LOG	Depth (Ft.)	Description	Elevation (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	LABORATORY HP (tsf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
3		36.0		-9				5-10-15		16	105		
2		40.0	**LEAN CLAY (CL)**, fine grained, gray, stiff to very stiff	-13	40					30			
		46.0	**LEAN CLAY (CL)**, fine grained, gray to dark gray, very stiff	-19	45			18-50/-1"					
3		51.5	**WELL GRADED SAND WITH GRAVEL (SW)**, fine to coarse grained, brown, very dense	-24.5	50			12-21-33 N=54		12	108		

Boring Terminated at 51.5 Feet

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
6" Hollow Stem Auger

Abandonment Method:
Boring backfilled with bentonite grout upon completion

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by HMH

Notes:

WATER LEVEL OBSERVATIONS

While sampling

At completion of drilling

Terracon
5075 Commercial Cir Ste E
Concord, CA

Boring Started: 10-26-2020	Boring Completed: 10-26-2020
Drill Rig: CME 95	Driller: Baja Exploration
Project No.: ND205079	

BORING LOG NO. B3

PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc Columbus, OH

SITE: Orchard Parkway San Jose, CA

LOCATION See Exploration Plan

Northing: 37.3777266 Easting: -121.9324945

Surface Elev.: 27.98 (Ft.)

MODEL LAYER	DEPTH (Ft.)	ELEVATION (Ft.)	DESCRIPTION
3	0.0 – 3.0	25	**SILTY SAND (SM)**, fine grained, brown, loose to medium dense
2	3.0 – 5.0	23	**LEAN CLAY (CL)**, fine grained, grayish brown with red brown mottling, medium stiff to stiff
2	5.0 – 7.5	20.5	**LEAN CLAY (CL)**, trace sand, fine to medium grained, brown, stiff
2	7.5 – 10.0	18	**SANDY LEAN CLAY (CL)**, fine to medium grained, brown, stiff
2	10.0 – 15.0	13	**SANDY LEAN CLAY (CL)**, fine to medium grained, grayish brown with red brown mottling, stiff
2	15.0 – 20.0	8	**LEAN CLAY WITH SAND (CL)**, fine grained, grayish brown, soft to medium stiff
3	20.0 – 25.0	3	**POORLY GRADED SAND (SP)**, fine to medium grained, gray, medium dense
2	25.0 – 35.0	-7	**LEAN CLAY (CL)**, fine grained, dark gray, medium stiff

DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	LABORATORY HP (tsf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1 – 2.5		Split spoon	6-6-4 N=10		5			
2.5 – 4		Split spoon	2-2-5 N=7	4.0 (HP)	5			
5 – 6.5		Split spoon	2-3-7 N=10	4.0 (HP)	19			
7.5 – 9		Split spoon	4-5-8 N=13	3.5 (HP)	23			
10 – 11.5		Split spoon	2-4-6 N=10	1.5 (HP)	25			
15 – 16.5		Split spoon	1-2-2 N=4	<0.25	36		34-18-16	80
20 – 21.5	▽ (approx. 19.7)	Split spoon	5-8-14 N=22		26			
25 – 27.5		Tube					32-19-13	81
30 – 32.5		Tube						

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
6" Hollow Stem Auger

Abandonment Method:
Boring backfilled with bentonite grout upon completion

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by HMH

Notes:

Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.
Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.

WATER LEVEL OBSERVATIONS

▽ *At completion of drilling*

Terracon
5075 Commercial Cir Ste E
Concord, CA

Boring Started: 10-27-2020	Boring Completed: 10-27-2020
Drill Rig: CME 95	Driller: Baja Exploration
Project No.: ND205079	

BORING LOG NO. B3

PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc Columbus, OH

SITE: Orchard Parkway San Jose, CA

LOCATION See Exploration Plan

Northing: 37.3777266 Easting: -121.9324945

Surface Elev.: 27.98 (Ft.)

MODEL LAYER	GRAPHIC LOG	DEPTH / ELEVATION (Ft.)	DESCRIPTION	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	LABORATORY HP (tsf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
2			**SANDY LEAN CLAY (CL)**, fine to medium grained, gray, stiff				8-9-12		27	98		
		40.0 / -12	**LEAN CLAY (CL)**, fine grained, gray to grayish brown with brown motting, stiff	40			4-6-7 N=13	2.5 (HP)	30		39-23-16	100
				45			6-9-12	2.25 (HP)	28	93		
		51.0 / -23		50			20-18-20 N=38		21			
3		51.5 / -23.5	**WELL GRADED SAND (SW)**, fine grained, reddish brown, dense									
			Boring Terminated at 51.5 Feet									

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
6" Hollow Stem Auger

Abandonment Method:
Boring backfilled with bentonite grout upon completion

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by HMH

Notes:

WATER LEVEL OBSERVATIONS

At completion of drilling

Terracon
5075 Commercial Cir Ste E
Concord, CA

Boring Started: 10-27-2020	Boring Completed: 10-27-2020
Drill Rig: CME 95	Driller: Baja Exploration
Project No.: ND205079	

BORING LOG NO. B4

PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc Columbus, OH

SITE: Orchard Parkway San Jose, CA

LOCATION See Exploration Plan

Northing: 37.3780735 Easting: -121.9321738

Surface Elev.: 28.22 (Ft.)

MODEL LAYER	DEPTH	DESCRIPTION	ELEVATION (Ft.)
3	1.5	**POORLY GRADED SAND WITH CLAY (SP)**, fine to medium grained, brown and gray, medium dense	26.5
1	5.0	**FAT CLAY (CH)**, fine to medium grained, dark brown, very stiff	23
2	7.5	**LEAN CLAY WITH SAND (CL)**, fine to medium grained, dark brown, very stiff	20.5
1	15.0	**FAT CLAY (CH)**, fine grained, grayish brown with red brown mottling, medium stiff	13
2	20.5	**SANDY LEAN CLAY (CL)**, fine to medium grained, grayish brown with red brown mottling, stiff	7.5
2	26.5	**SANDY LEAN CLAY (CL)**, some silt, fine grained, gray, stiff	1.5
2		**LEAN CLAY (CL)**, fine grained, gray, medium stiff	

FIELD TEST RESULTS	LABORATORY HP (tsf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
6-15-18	>4.5	16	80		
6-9-11 N=20	>4.5	22		76-31-45	95
7-10-20	4.5 (HP)	24	100		
3-6-7 N=13	2.5 (HP)	31			
5-4-7	2.0 (HP)	31	92	55-23-32	85
1-2-10 N=12	0.25 (HP)	19			
6-8-10	1.25 (HP)	32	92		
1-2-6 N=8		23			81

DEPTH (Ft.): 5, 10, 15, 20, 25, 30, 35

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
6" Hollow Stem Auger

Abandonment Method:
Boring backfilled with bentonite grout upon completion

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by HMH

Notes:

Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.
Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.

WATER LEVEL OBSERVATIONS

At completion of drilling

Terracon
5075 Commercial Cir Ste E
Concord, CA

Boring Started: 10-27-2020	Boring Completed: 10-27-2020
Drill Rig: CME 95	Driller: Baja Exploration
Project No.: ND205079	

BORING LOG NO. B4

PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc Columbus, OH

SITE: Orchard Parkway San Jose, CA

LOCATION See Exploration Plan

Northing: 37.3780735 Easting: -121.9321738

Surface Elev.: 28.22 (Ft.)

MODEL LAYER	GRAPHIC LOG	DEPTH (Ft.)	DESCRIPTION	ELEVATION (Ft.)	DEPTH (In.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	LABORATORY HP (tsf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
2		35.5	**LEAN CLAY (CL)**, fine grained, orange and brown, stiff	-7.5				5-6-8 N=14	1.25 (HP)	23			
					40			6-9-10	1.0 (HP)	30	92		
		45.0	**LEAN CLAY (CL)**, fine grained, gray with brown, stiff to very stiff	-17	45			4-5-8 N=13	1.25 (HP)	31			
					50			5-19-22	3.25 (HP)	26	99		
		51.5	*Boring Terminated at 51.5 Feet*	-23.5									

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
6" Hollow Stem Auger

Abandonment Method:
Boring backfilled with bentonite grout upon completion

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by HMH

Notes:

WATER LEVEL OBSERVATIONS

At completion of drilling

Terracon
5075 Commercial Cir Ste E
Concord, CA

Boring Started: 10-27-2020	Boring Completed: 10-27-2020
Drill Rig: CME 95	Driller: Baja Exploration
Project No.: ND205079	

BORING LOG NO. B5

PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc
Columbus, OH

SITE: Orchard Parkway
San Jose, CA

LOCATION See Exploration Plan

Northing: 37.3788802 Easting: -121.9324507

Surface Elev.: 27.6 (Ft.)

MODEL LAYER	GRAPHIC LOG	DEPTH (Ft.)	DESCRIPTION	ELEVATION (Ft.)
1		0 – 5.5	**SANDY FAT CLAY WITH GRAVEL (CH)**, fine to coarse grained, dark brown, stiff to very stiff	22
2		5.5 – 7.5	**LEAN CLAY (CL)**, fine grained, grayish brown, stiff	20
2		7.5 – 15.5	**LEAN CLAY (CL)**, trace sand, fine to medium grained, light brown with red brown mottling, stiff to very stiff	12
3		15.5 – 20.0	**SILTY SAND (SM)**, fine to medium grained, gray, medium dense	7.5
3		20.0 – 25.5	**POORLY GRADED SAND WITH CLAY AND GRAVEL (SP)**, fine to coarse grained, gray, dense	2
3		25.5 – 31.5	**CLAYEY SAND (SC)**, with silt, fine to medium grained, gray, medium dense	-4
3		31.5 – 32.5	**POORLY GRADED SAND (SP)**, fine to medium grained, gray, loose	-5

Boring Terminated at 32.5 Feet

DEPTH (In.)	FIELD TEST RESULTS	LABORATORY HP (tsf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
	6-7-6 N=13	>4.5	13			
	12-13-15	>4.5	22			
5	3-4-7 N=11	4.5 (HP)	26			
	10-12-20	3.5 (HP)	25			
10	3-4-6 N=10	2.0 (HP)	28			
15	4-10-20		18	107		
20	8-14-17 N=31		19			
25	7-6-11	1.0 (HP)	29	93		
30						

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
6" Hollow Stem Auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by HMH

Notes:

Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.
Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.

WATER LEVEL OBSERVATIONS

- ▽ *While drilling*
- ▽ *At completion of drilling*

Terracon
5075 Commercial Cir Ste E
Concord, CA

Boring Started: 10-23-2020	Boring Completed: 10-23-2020
Drill Rig: CME 95	Driller: Baja Exploration
Project No.: ND205079	

BORING LOG NO. B6

PROJECT: Bay AZ1 Site (Northtown)

**CLIENT: Burns & McDonnell Engineering Company Inc
Columbus, OH**

**SITE: Orchard Parkway
San Jose, CA**

LOCATION See Exploration Plan

Northing: 37.3791793 Easting: -121.931615

Surface Elev.: 30.98 (Ft.)

MODEL LAYER	DEPTH (Ft.)	ELEVATION (Ft.)	DESCRIPTION
2	0.0–10.0	21	**LEAN CLAY WITH SAND (CL)**, some gravel, fine to medium grained, brown to dark brown, very stiff
2	10.0–15.0	16	**LEAN CLAY (CL)**, trace sand, fine grained, brown with white mottling, very stiff
2	15.0–20.0	11	**SANDY LEAN CLAY (CL)**, fine grained, grayish brown with red brown mottling, medium stiff to stiff
3	20.0–26.0	5	**CLAYEY SAND (SC)**, fine to medium grained, brown to gray, loose
4	26.0–31.5	-0.5	**SANDY SILT (ML)**, fine grained, gray, very stiff

Boring Terminated at 31.5 Feet

Water level observation (while drilling) at approximately 20 ft.

FIELD TEST RESULTS	LABORATORY HP (tsf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
19-10-10 N=20	>4.5	10		48-23-25	78
7-14-21	>4.5	8			
5-11-10 N=21	>4.5	14			
12-10-13	4.5 (HP)	19			
3-6-10 N=16	4.0 (HP)	21			
6-5-6	1.25 (HP)	28	93		
3-3-4 N=7		19			
10-8-8	2.5 (HP)	21	102		
2-3-13 N=16		19		21-18-3	52

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
6" Hollow Stem Auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by HMH

Notes:

Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.
Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.

WATER LEVEL OBSERVATIONS

While drilling

Terracon
5075 Commercial Cir Ste E
Concord, CA

Boring Started: 10-27-2020	Boring Completed: 10-27-2020
Drill Rig: CME 95	Driller: Baja Exploration
Project No.: ND205079	

BORING LOG NO. B7

PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc Columbus, OH

SITE: Orchard Parkway San Jose, CA

LOCATION See Exploration Plan

Northing: 37.3791155 Easting: -121.9321857

Surface Elev.: 27.46 (Ft.)

MODEL LAYER	DEPTH (Ft.)	DESCRIPTION	ELEVATION (Ft.)	DEPTH (ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	LABORATORY HP (tsf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1		**FAT CLAY (CH)**, fine grained, dark brown, stiff to very stiff					10-14-27	>4.5	9			
							6-8-6 N=14	4.5 (HP)	27		77-23-54	93
	4.0		23.5									
		FAT CLAY WITH SAND (CH), fine grained, dark brown, very stiff		5			12-18-24	4.5 (HP)	23		72-27-45	84
	7.0		20.5									
2		**LEAN CLAY (CL)**, trace sand, fine to medium grained, light brown with grayish white mottling, red brown blue motting, and red brown motting, stiff					4-3-6 N=9	2.5 (HP)	24			
				10			4-6-10	3.5 (HP)	24			
	15.0		12.5									
		LEAN CLAY (CL), fine grained, grayish brown, medium stiff		15			1-2-3 N=5	1.0 (HP)	30			
	20.0		7.5	20	▽							
		LEAN CLAY (CL), fine grained, gray, very stiff to hard					15-24-40		33	87		
	24.5		3									
		SANDY LEAN CLAY (CL), fine to medium grained, gray, soft to stiff		25			4-1-2 N=3	1.0 (HP)	22			
				30			13-12-13	0.5 (HP)	23	102		
	31.5		-4									

Boring Terminated at 31.5 Feet

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
6" Hollow Stem Auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by HMH

Notes:

Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.
Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.

WATER LEVEL OBSERVATIONS

▽ *While drilling*

Terracon
5075 Commercial Cir Ste E
Concord, CA

Boring Started: 10-23-2020	Boring Completed: 10-23-2020
Drill Rig: CME 95	Driller: Baja Exploration
Project No.: ND205079	

BORING LOG NO. B8

PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc Columbus, OH

SITE: Orchard Parkway San Jose, CA

LOCATION See Exploration Plan

Northing: 37.3784456 Easting: -121.9325447

Surface Elev.: 27.31 (Ft.)

MODEL LAYER	DEPTH (Ft.)	DESCRIPTION	ELEVATION (Ft.)
2	4.0	**LEAN CLAY WITH SAND (CL)**, fine grained, dark brown, stiff to very stiff	23.5
2	7.5	**LEAN CLAY (CL)**, fine grained, light brown, stiff	20
2	15.0	**LEAN CLAY WITH SAND (CL)**, fine grained, light brown with red brown mottling, stiff	12.5
2	20.0	**LEAN CLAY (CL)**, trace sand, fine grained, grayish brown, medium stiff	7.5
3	25.5	**WELL GRADED SAND (SW)**, fine to medium grained, gray, loose	2
2		**SANDY LEAN CLAY (CL)**, fine to medium grained, gray to dark gray, medium stiff to stiff	

FIELD TEST RESULTS	LABORATORY HP (tsf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
9-13-10 N=23	>4.5	12			
2-5-13	>4.5	28	77	49-23-26	80
1-7-7 N=14	3.75 (HP)	26			
7-11-15	1.5 (HP)	22	92	42-22-20	83
3-3-5 N=8	1.25 (HP)	24			
4-3-6	1.0 (HP)	26	91		
3-2-6 N=8					
3-4-4		13	108		
					58
5-5-9 N=14	2.5 (HP)	24		29-17-12	

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
6" Hollow Stem Auger

Abandonment Method:
Boring backfilled with bentonite grout upon completion

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by HMH

Notes:

Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.
Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.

WATER LEVEL OBSERVATIONS

▽ *While sampling*

Terracon
5075 Commercial Cir Ste E
Concord, CA

Boring Started: 10-26-2020	Boring Completed: 10-26-2020
Drill Rig: CME 95	Driller: Baja Exploration
Project No.: ND205079	

BORING LOG NO. B8

PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc Columbus, OH

SITE: Orchard Parkway San Jose, CA

LOCATION See Exploration Plan

Northing: 37.3784456 Easting: -121.9325447

Surface Elev.: 27.31 (Ft.)

MODEL LAYER	GRAPHIC LOG	DEPTH / DESCRIPTION	ELEVATION (Ft.)	DEPTH (In.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	LABORATORY HP (tsf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
2		**SANDY LEAN CLAY (CL)**, fine to medium grained, gray to dark gray, medium stiff to stiff *(continued)*					4-4-9	0.75 (HP)	24			62
		40.0	-12.5	40								
		LEAN CLAY WITH SAND (CL), fine grained, dark gray, stiff to very stiff					5-6-12 N=18	2.5 (HP)	32			
				45			7-14-19	3.75 (HP)	20		35-17-18	74
		51.5	-24	50			4-6-10 N=16	2.0 (HP)	34			
		Boring Terminated at 51.5 Feet										

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
6" Hollow Stem Auger

Abandonment Method:
Boring backfilled with bentonite grout upon completion

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by HMH

Notes:

WATER LEVEL OBSERVATIONS

While sampling

Terracon
5075 Commercial Cir Ste E
Concord, CA

Boring Started: 10-26-2020	Boring Completed: 10-26-2020
Drill Rig: CME 95	Driller: Baja Exploration
Project No.: ND205079	

BORING LOG NO. B9

PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc
Columbus, OH

SITE: Orchard Parkway
San Jose, CA

LOCATION See Exploration Plan

Northing: 37.3777845 Easting: -121.933206

Surface Elev.: 27.5 (Ft.)

MODEL LAYER	GRAPHIC LOG	DEPTH (Ft.)	DESCRIPTION	ELEVATION (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	LABORATORY HP (tsf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1			**FAT CLAY (CH)**, fine grained, dark brown, very stiff				9-19-22	>4.5	19			
							7-10-12 N=22	>4.5	26			
		5.0		22.5								
2			**LEAN CLAY (CL)**, fine grained, dark brown with light brown mottling, very stiff				7-19-25	>4.5	20			
		7.5		20								
			LEAN CLAY (CL), trace sand, coarse to medium grained, light brown, stiff				4-5-5 N=10		25			
		10.0		17.5	▽							
			LEAN CLAY (CL), fine grained, grayish brown with red brown mottling, medium stiff				3-4-6	0.75 (HP)	27	94		
					▽							
		15.0		12.5								
3			**CLAYEY SAND (SC)**, fine to medium grained, gray, loose				1-2-3 N=5		25			
		16.5		11								

Boring Terminated at 16.5 Feet

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
6" Hollow Stem Auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by HMH

Notes:

Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.

WATER LEVEL OBSERVATIONS

▽ *While drilling*

▽ *At completion of drilling*

Terracon
5075 Commercial Cir Ste E
Concord, CA

Boring Started: 10-23-2020	Boring Completed: 10-23-2020
Drill Rig: CME 95	Driller: Baja Exploration
Project No.: ND205079	

BORING LOG NO. B10

PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc Columbus, OH

SITE: Orchard Parkway San Jose, CA

LOCATION See Exploration Plan

Northing: 37.3789866 Easting: -121.9333547

Surface Elev.: 48.32 (Ft.)

MODEL LAYER	GRAPHIC LOG	DEPTH (ft) / Description / ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	LABORATORY HP (tsf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1		**FAT CLAY WITH GRAVEL (CH)**, fine to medium grained, brown, very stiff				15-15-12 N=27		5			
		3.5 — 45				5-11-18	3.5 (HP)	8	91		
3		**POORLY GRADED SAND (SP)**, fine to medium grained, light brown, medium dense									
		5.0 — 43.5	5								
1		**FAT CLAY (CH)**, some gravel, fine grained, brown with white mottling, very stiff				6-11-10 N=21	>4.5	13			
						8-12-17	4.5 (HP)	27			
		9.0 — 39.5									
1		**FAT CLAY (CH)**, some sand and gravel, fine to medium grained, dark brown with brown mottling, very stiff	10			6-7-10 N=17	4.5 (HP)	29			
			15								
		16.5 — 32				5-9-15	4.25 (HP)	22			
		Boring Terminated at 16.5 Feet									

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
6" Hollow Stem Auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by HMH

Notes:

Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.

WATER LEVEL OBSERVATIONS

Groundwater not encountered

Terracon
5075 Commercial Cir Ste E
Concord, CA

Boring Started: 10-27-2020	Boring Completed: 10-27-2020
Drill Rig: CME 95	Driller: Baja Exploration
Project No.: ND205079	

BORING LOG NO. B11

PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc
Columbus, OH

SITE: Orchard Parkway
San Jose, CA

LOCATION See Exploration Plan

Northing: 37.3779696 Easting: -121.9317919

Surface Elev.: 28.61 (Ft.)

MODEL LAYER	GRAPHIC LOG	DEPTH (ft.)	DESCRIPTION	ELEVATION (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	LABORATORY HP (tsf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1		5.0	**FAT CLAY (CH)**, fine grained, dark brown, very stiff	23.5			13-12-12 N=24	>4.5	10			
							10-11-13	>4.5	25	92		
2		10.0	**LEAN CLAY WITH SAND (CL)**, fine to medium grained, brown with red brown mottling, very stiff	18.5			5-7-10 N=17	4.25 (HP)	25			
							8-11-19	2.5 (HP)	22	102		
		15.0	**LEAN CLAY (CL)**, some gravel, fine to medium grained, brown with gray brown mottling, medium stiff	13.5	▽		2-3-3 N=6		18			
3		16.5	**WELL GRADED SAND (SW)**, some clay, fine to coarse grained, brown and black, medium dense	12			6-18-11		19	104		

Boring Terminated at 16.5 Feet

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
6" Hollow Stem Auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by HMH

Notes:

Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.

WATER LEVEL OBSERVATIONS

▽ *While drilling*

Terracon
5075 Commercial Cir Ste E
Concord, CA

Boring Started: 10-23-2020	Boring Completed: 10-23-2020
Drill Rig: CME 95	Driller: Baja Exploration
Project No.: ND205079	

BORING LOG NO. B12

PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc Columbus, OH

SITE: Orchard Parkway San Jose, CA

LOCATION See Exploration Plan

Northing: 37.3781626 Easting: -121.9340656

Surface Elev.: 26.92 (Ft.)

MODEL LAYER	DESCRIPTION (Depth Ft. / ELEVATION Ft.)	DEPTH (ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	LABORATORY HP (tsf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1	**FAT CLAY (CH)**, fine grained, dark brown, stiff to very stiff				6-7-8 N=15	>4.5	14			
					8-17-34		21	91		
	5.0 / 22	5								
2	**LEAN CLAY (CL)**, fine grained, light brown with white mottling, stiff				3-5-6 N=11	4.25 (HP)	21			
	7.5 / 19.5									
3	**CLAYEY SAND (SC)**, fine to coarse grained, brown, loose				4-5-7		21	100		
	10.0 / 17	10	▽							
2	**LEAN CLAY WITH SAND (CL)**, fine grained, gray with reddish brown mottling, stiff to very stiff				0-4-5 N=9	1.25 (HP)	29			
			▽							
	15.0 / 12	15								
4	**SANDY SILT (ML)**, fine to medium grained, gray, very stiff				5-12-20		23	102		
	16.5 / 10.5									
	Boring Terminated at 16.5 Feet									

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
6" Hollow Stem Auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by HMH

Notes:

Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.

WATER LEVEL OBSERVATIONS

▽ *While drilling*

▽ *At completion of drilling*

Terracon
5075 Commercial Cir Ste E
Concord, CA

Boring Started: 10-23-2020	Boring Completed: 10-23-2020
Drill Rig: CME 95	Driller: Baja Exploration
Project No.: ND205079	

BORING LOG NO. B13

PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc Columbus, OH

SITE: Orchard Parkway San Jose, CA

LOCATION See Exploration Plan

Northing: 37.3784349 Easting: -121.9345573

Surface Elev.: 29.47 (Ft.)

MODEL LAYER	DEPTH / ELEVATION (Ft.)	DESCRIPTION	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	FIELD TEST RESULTS	LABORATORY HP (tsf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
3	2.5 / 27	**CLAYEY SAND (SC)**, fine to medium grained, light brown, loose			8-3-4		8			
2	7.5 / 22	**LEAN CLAY WITH SAND (CL)**, some sand, fine to medium grained, dark brown with white mottling, very stiff			5-8-10 N=18	>4.5	15		38-19-19	73
			5		8-13-15	4.5 (HP)	22	97		
3	9.5 / 20	**CLAYEY SAND (SC)**, fine to medium grained, light brown with white mottling, medium dense			5-11-14 N=25		17			
	11.0 / 18.5	**POORLY GRADED SAND (SP)**, trace clay, fine to medium grained, brown with black, medium dense	10		5-10-18		8			
	20.0 / 9.5	**CLAYEY SAND (SC)**, trace silt, fine to medium grained, brown, medium dense	15		2-6-11 N=17		29			33
2	25.0 / 4.5	**LEAN CLAY WITH SAND (CL)**, fine to medium grained, light brown, stiff	20	▽	2-5-12		22	103		
	32.5 / -3	**LEAN CLAY (CL)**, fine grained, gray, soft to medium stiff	25		2-1-3 N=4	1.25 (HP)	29			95
			30							

Boring Terminated at 32.5 Feet

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
6" Hollow Stem Auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by HMH

Notes:

Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.
Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.

WATER LEVEL OBSERVATIONS

▽ *While drilling*

Terracon
5075 Commercial Cir Ste E
Concord, CA

Boring Started: 10-23-2020	Boring Completed: 10-23-2020
Drill Rig: CME 95	Driller: Baja Exploration
Project No.: ND205079	

BORING LOG NO. B14

PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc Columbus, OH

SITE: Orchard Parkway San Jose, CA

LOCATION See Exploration Plan

Northing: 37.3790698 Easting: -121.934504

Surface Elev.: 35.89 (Ft.)

MODEL LAYER	GRAPHIC LOG	DEPTH (Ft.)	DESCRIPTION	ELEVATION (Ft.)	DEPTH (In.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	LABORATORY HP (tsf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1		2.5	**FAT CLAY (CH)**, fine grained, brown to dark brown, very stiff	33.5				13-16-16	>4.5	19			
4		5.5	**SANDY SILT (ML)**, fine grained, light brown, stiff	30.5	5			4-7-8 N=15	>4.5	17			77
2		10.0	**LEAN CLAY (CL)**, some sand, fine to medium grained, dark brown, stiff to very stiff	26				9-13-12	4.5 (HP)	18	94	48-25-23	91
								5-8-10 N=18	>4.5	22			
3		11.0	**SILTY SAND (SM)**, fine grained, light brown, medium dense	25	10			6-9-16	>4.5	19	94		
2		15.0	**LEAN CLAY (CL)**, fine to medium grained, dark brown, very stiff	21									
		20.0	**SANDY LEAN CLAY (CL)**, fine to medium grained, light brown with greyish white mottling, very stiff	16	15			3-9-11 N=20	2.5 (HP)	17			
3		25.0	**POORLY GRADED SAND (SP)**, some gravel, fine to coarse grained, brown with black and reddish brown, dense	11	20			7-15-29		13	118		
		27.0	**WELL GRADED SAND (SW)**, fine to coarse grained, black with white and gray, loose	9	25	▽		1-2-5 N=7		27			
2		30.0	**SANDY LEAN CLAY (CL)**, fine grained, gray, medium stiff	6		▽							
4		31.5	**SANDY SILT (ML)**, fine grained, gray to black, stiff	4.5	30			6-5-5 N=10		21			

Boring Terminated at 31.5 Feet

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
6" Hollow Stem Auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by HMH

Notes:

Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.
Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.

WATER LEVEL OBSERVATIONS

▽ *While drilling*

▽ *At completion of drilling*

Terracon
5075 Commercial Cir Ste E
Concord, CA

Boring Started: 10-22-2020	Boring Completed: 10-22-2020
Drill Rig: CME 95	Driller: Baja Exploration
Project No.: ND205079	

BORING LOG NO. B15

PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc, Columbus, OH

SITE: Orchard Parkway, San Jose, CA

LOCATION See Exploration Plan

Northing: 37.3788994 Easting: -121.9343977

Surface Elev.: 40.89 (Ft.)

MODEL LAYER	DEPTH / ELEVATION (Ft.)	DESCRIPTION	DEPTH (In.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	LABORATORY HP (tsf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1		**FAT CLAY WITH SAND (CH)**, with sand and gravel, fine to coarse grained, brown and white, stiff to very stiff				4-7-2 N=9	>4.5	10			
						9-16-21	>4.5	16	83	62-21-41	82
	5.5 / 35.5		5								
2		**LEAN CLAY WITH SAND (CL)**, fine to medium grained, brown, very stiff				12-14-14 N=28	>4.5	13			
	10.0 / 31		10								
		LEAN CLAY (CL), trace sand, fine to medium grained, dark brown with grayish brown mottling, very stiff				5-8-9 N=17	4.0 (HP)	21			
		LEAN CLAY (CL), trace sand, fine to medium grained, dark brown, very stiff	15			7-12-20	4.5 (HP)	23	81		
	20.0 / 21		20								
		SANDY LEAN CLAY (CL), fine to medium grained, light brown with gray mottling, stiff to very stiff				6-7-9 N=16	3.0 (HP)	17			
	25.0 / 16		25								
3		**POORLY GRADED SAND (SP)**, trace gravel, fine to medium grained, light brown with gray, medium dense				3-10-11		23			
	30.0 / 11		30								
2		**LEAN CLAY (CL)**, fine grained, gray, medium stiff				1-3-5 N=8	0.5 (HP)	32			
	35.0 / 6		35								

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
6" Hollow Stem Auger

Abandonment Method:
Boring backfilled with bentonite grout upon completion

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by HMH

Notes:

Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.
Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.

WATER LEVEL OBSERVATIONS

Terracon
5075 Commercial Cir Ste E
Concord, CA

Boring Started: 10-22-2020	Boring Completed: 10-22-2020
Drill Rig: CME 95	Driller: Baja Exploration
Project No.: ND205079	

BORING LOG NO. B15

PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc Columbus, OH

SITE: Orchard Parkway San Jose, CA

LOCATION See Exploration Plan

Northing: 37.3788994 Easting: -121.9343977

Surface Elev.: 40.89 (Ft.)

MODEL LAYER	GRAPHIC LOG	Description / Depth / ELEVATION (Ft.)	DEPTH (ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	LABORATORY HP (tsf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
2		**LEAN CLAY (CL)**, fine grained, black with gray mottling, stiff				4-6-8	1.5 (HP)	30			
		40.0 — 1	40								
		LEAN CLAY (CL), trace sand, fine grained, black with brown mottling, medium stiff to stiff									
		45.0 — -4	45								
		LEAN CLAY WITH SAND (CL), fine grained, gray, stiff				7-9-14	2.0 (HP)	26	100	40-18-22	88
			50			2-4-7 N=11		19			
		51.5 — -10.5									

Boring Terminated at 51.5 Feet

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
6" Hollow Stem Auger

Abandonment Method:
Boring backfilled with bentonite grout upon completion

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by HMH

Notes:

WATER LEVEL OBSERVATIONS

Terracon
5075 Commercial Cir Ste E
Concord, CA

Boring Started: 10-22-2020	Boring Completed: 10-22-2020
Drill Rig: CME 95	Driller: Baja Exploration
Project No.: ND205079	

BORING LOG NO. P1

PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc Columbus, OH

SITE: Orchard Parkway San Jose, CA

LOCATION See Exploration Plan

Northing: 37.3783114 Easting: -121.9342412

Surface Elev.: 26.5 (Ft.)

MODEL LAYER	GRAPHIC LOG	DEPTH / ELEVATION (Ft.)	DESCRIPTION	DEPTH (In.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	LABORATORY HP (tsf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1			**FAT CLAY (CH)**, fine grained, brown to dark brown, very stiff									
							6-15-18	>4.5	26			
				5								
		7.0 / 19.5										
2			**SANDY LEAN CLAY (CL)**, fine to medium grained, light brown with white mottling, stiff				4-5-7 N=12	3.5 (HP)	18			
		10.0 / 16.5		10								
			SANDY LEAN CLAY (CL), trace gravel, fine to medium grained, light brown with red brown mottling, medium stiff				3-5-7	0.5 (HP)	21			
		11.5 / 15	***Boring Terminated at 11.5 Feet***									

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
6" Hollow Stem Auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by HMH

Notes:

Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.

WATER LEVEL OBSERVATIONS

Groundwater not encountered

5075 Commercial Cir Ste E
Concord, CA

Boring Started: 10-22-2020	Boring Completed: 10-22-2020
Drill Rig: CME 95	Driller: Baja Exploration
Project No.: ND205079	

BORING LOG NO. P2

PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc Columbus, OH

SITE: Orchard Parkway San Jose, CA

LOCATION See Exploration Plan

Northing: 37.3792005 Easting: -121.9332728

Surface Elev.: 41.85 (Ft.)

MODEL LAYER	GRAPHIC LOG	DESCRIPTION / DEPTH / ELEVATION (Ft.)	DEPTH (ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	LABORATORY HP (tsf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1		**FAT CLAY WITH SAND (CH)**, some gravel, fine to coarse grained, brown, stiff to very stiff									
						10-6-9 N=15	>4.5	8			
		5.0 / 37	5								
2		**LEAN CLAY (CL)**, fine to coarse grained, brown, very stiff				11-33-28	>4.5	13			
		11.5 / 30.5	10			4-7-11 N=18	3.75 (HP)	23			
		Boring Terminated at 11.5 Feet									

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
6" Hollow Stem Auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by HMH

Notes:

Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.

WATER LEVEL OBSERVATIONS

Groundwater not encountered

Terracon
5075 Commercial Cir Ste E
Concord, CA

Boring Started: 10-22-2020	Boring Completed: 10-22-2020
Drill Rig: CME 95	Driller: Baja Exploration
Project No.: ND205079	

BORING LOG NO. P3

PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc Columbus, OH

SITE: Orchard Parkway San Jose, CA

LOCATION See Exploration Plan

Northing: 37.3780553 Easting: -121.9329027

Surface Elev.: 27.28 (Ft.)

MODEL LAYER	GRAPHIC LOG	DEPTH (Ft.) / ELEVATION (Ft.)	DEPTH (In.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	LABORATORY HP (tsf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1		**FAT CLAY (CH)**, fine grained, brown, stiff									
		2.5 / 25									
2		**LEAN CLAY (CL)**, fine grained, gray with brown mottling, stiff				2-4-6 N=10		22			
			5								
		7.0 / 20.5									
		LEAN CLAY (CL), fine grained, brown with gray mottling, stiff to very stiff				10-11-11	3.25 (HP)	23			
			10			3-5-5 N=10	2.0 (HP)	31			
		11.5 / 16									

Boring Terminated at 11.5 Feet

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
6" Hollow Stem Auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by HMH

Notes:

Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.

WATER LEVEL OBSERVATIONS

Groundwater not encountered

Terracon
5075 Commercial Cir Ste E
Concord, CA

Boring Started: 10-26-2020	Boring Completed: 10-26-2020
Drill Rig: CME 95	Driller: Baja Exploration
Project No.: ND205079	

BORING LOG NO. P4

PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc
Columbus, OH

SITE: Orchard Parkway
San Jose, CA

LOCATION See Exploration Plan

Northing: 37.3786742 Easting: -121.9310569

Surface Elev.: 32.8 (Ft.)

MODEL LAYER	GRAPHIC LOG	DEPTH	DESCRIPTION	ELEVATION (Ft.)	DEPTH (ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	LABORATORY HP (tsf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
2			**SANDY LEAN CLAY (CL)**, fine to medium grained, grayish brown, stiff				Hand						
		2.0		31			SPT	8-13-12		21		32-15-17	53
			LEAN CLAY (CL), fine grained, dark gray, very stiff				Hand						
					5		SPT	8-11-15 N=26	>4.5	11			
		11.5		21.5	10		SPT	7-13-14	>4.5	22			

Boring Terminated at 11.5 Feet

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
6" Hollow Stem Auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by HMH

Notes:

Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.

WATER LEVEL OBSERVATIONS

Groundwater not encountered

Terracon
5075 Commercial Cir Ste E
Concord, CA

Boring Started: 10-27-2020	Boring Completed: 10-27-2020
Drill Rig: CME 95	Driller: Baja Exploration
Project No.: ND205079	

BORING LOG NO. P5

PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc Columbus, OH

SITE: Orchard Parkway San Jose, CA

LOCATION See Exploration Plan

Northing: 37.3772208 Easting: -121.932485

Surface Elev.: 28.94 (Ft.)

MODEL LAYER	GRAPHIC LOG	DEPTH / ELEVATION (Ft.)	DESCRIPTION	DEPTH (ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	LABORATORY HP (tsf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1			**FAT CLAY (CH)**, fine grained, dark brown, stiff to very stiff			Hand						
						SPT	6-8-9		18			
						Hand						
				5		SPT	7-10-10 N=20	4.5 (HP)	26			
		10.0 / 19										
2		11.5 / 17.5	**LEAN CLAY WITH SAND (CL)**, fine to medium grained, light brown with gray mottling, stiff	10		Hand			22			
			Boring Terminated at 11.5 Feet									

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
6" Hollow Stem Auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by HMH

Notes:

Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.

WATER LEVEL OBSERVATIONS

Groundwater not encountered

Terracon
5075 Commercial Cir Ste E
Concord, CA

Boring Started: 10-23-2020	Boring Completed: 10-23-2020
Drill Rig: CME 95	Driller: Baja Exploration
Project No.: ND205079	

BORING LOG NO. I1

PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc
Columbus, OH

SITE: Orchard Parkway
San Jose, CA

MODEL LAYER	GRAPHIC LOG	LOCATION See Exploration Plan Northing: 37.3786894 Easting: -121.9347755 Surface Elev.: 31.97 (Ft.) DEPTH / ELEVATION (Ft.)	DEPTH (In.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	LABORATORY HP (tsf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1		**FAT CLAY (CH)**, some sand, fine to medium grained, dark brown, stiff 5.0 27	5								
		Boring Terminated at 5 Feet									

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
6" Hollow Stem Auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by HMH

Notes:

Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.

WATER LEVEL OBSERVATIONS

Groundwater not encountered

Terracon
5075 Commercial Cir Ste E
Concord, CA

Boring Started: 10-22-2020	Boring Completed: 10-22-2020
Drill Rig: CME 95	Driller: Baja Exploration
Project No.: ND205079	

BORING LOG NO. I2

PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc
Columbus, OH

SITE: Orchard Parkway
San Jose, CA

LOCATION See Exploration Plan

Northing: 37.3789698 Easting: -121.9341981

Surface Elev.: 39.97 (Ft.)

MODEL LAYER	GRAPHIC LOG	DEPTH / ELEVATION (Ft.)	DEPTH (In.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	LABORATORY HP (tsf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1		**FAT CLAY (CH)**, fine grained, dark brown, stiff									
		5.0 / 35	5								
		Boring Terminated at 5 Feet									

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
6" Hollow Stem Auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by HMH

Notes:

Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.

WATER LEVEL OBSERVATIONS

Groundwater not encountered

5075 Commercial Cir Ste E
Concord, CA

Boring Started: 10-22-2020	Boring Completed: 10-22-2020
Drill Rig: CME 95	Driller: Baja Exploration
Project No.: ND205079	

BORING LOG NO. I3

PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc
Columbus, OH

SITE: Orchard Parkway
San Jose, CA

MODEL LAYER	GRAPHIC LOG	LOCATION See Exploration Plan Northing: 37.3793174 Easting: -121.9322947 Surface Elev.: 27.73 (Ft.) DEPTH / ELEVATION (Ft.)	DEPTH (In.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	LABORATORY HP (tsf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1		**FAT CLAY (CH)**, fine grained, dark brown, stiff 5.0 22.5	5								
		Boring Terminated at 5 Feet									

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
6" Hollow Stem Auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by HMH

Notes:

Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.

WATER LEVEL OBSERVATIONS

Groundwater not encountered

5075 Commercial Cir Ste E
Concord, CA

Boring Started: 10-23-2020	Boring Completed: 10-23-2020
Drill Rig: CME 95	Driller: Baja Exploration
Project No.: ND205079	

BORING LOG NO. I4

PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc Columbus, OH

SITE: Orchard Parkway San Jose, CA

MODEL LAYER	GRAPHIC LOG	LOCATION See Exploration Plan Northing: 37.3789737 Easting: -121.9315757 Surface Elev.: 32.04 (Ft.) ELEVATION (Ft.)	DEPTH (In.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	LABORATORY HP (tsf)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1		**FAT CLAY (CH)**, fine grained, brown, stiff 5.0 27	5								
		Boring Terminated at 5 Feet									

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
6" Hollow Stem Auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by HMH

Notes:

Northing & Easting and Elevations were provided by HMH Engineers in Boring Locations plan dated 10/30/2020.

WATER LEVEL OBSERVATIONS

Groundwater not encountered

Terracon
5075 Commercial Cir Ste E
Concord, CA

Boring Started: 10-23-2020	Boring Completed: 10-23-2020
Drill Rig: CME 95	Driller: Baja Exploration
Project No.: ND205079	

CPT LOG NO. CPT-01

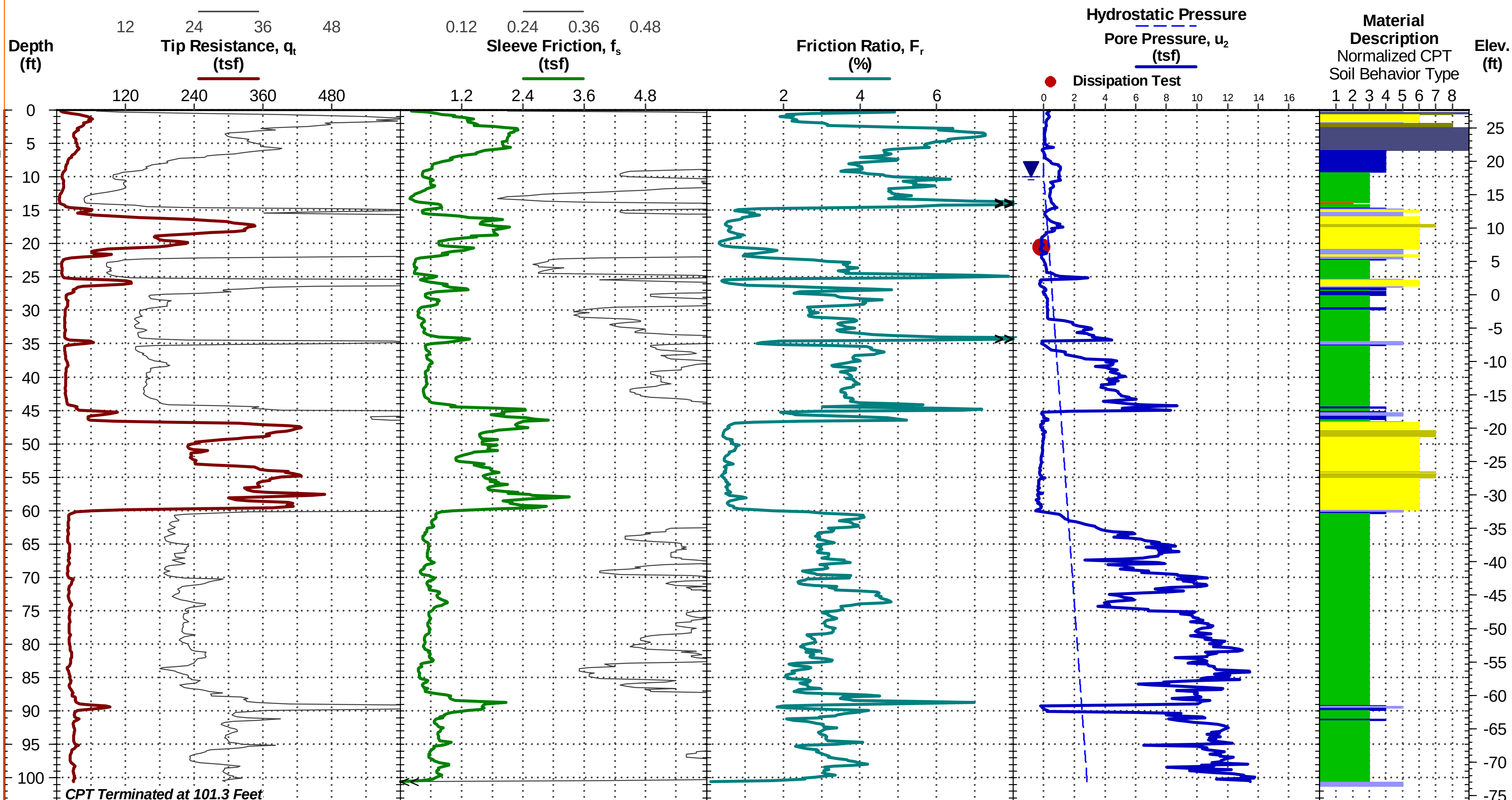
PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc
Columbus, OH

TEST LOCATION: See Exploration Plan

SITE: Orchard Parkway
San Jose, CA

Surface Elev.: 27.65 ft
Northing: 37.3777314
Easting: -121.9333134



See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).
Elevations were provided by HMH

Dead weight of rig used as reaction force.
CPT sensor calibration reports available upon request.

1 Sensitive, fine grained
2 Organic soils - clay
3 Clay - silty clay to clay
4 Silt mixtures - clayey silt to silty clay
5 Sand mixtures - silty sand to sandy silt
6 Sands - clean sand to silty sand
7 Gravelly sand to dense sand
8 Very stiff sand to clayey sand
9 Very stiff fine grained

WATER LEVEL OBSERVATION

10 ft measured water depth
(used in normalizations and correlations; See Supporting Information)

Probe no. DDG1530

Terracon
5075 Commercial Cir Ste E
Concord, CA

CPT Started: 10/23/2020	CPT Completed: 10/23/2020
Rig: CPT	Operator: Middle Earth
Project No.: ND205079	

CPT LOG NO. CPT-02

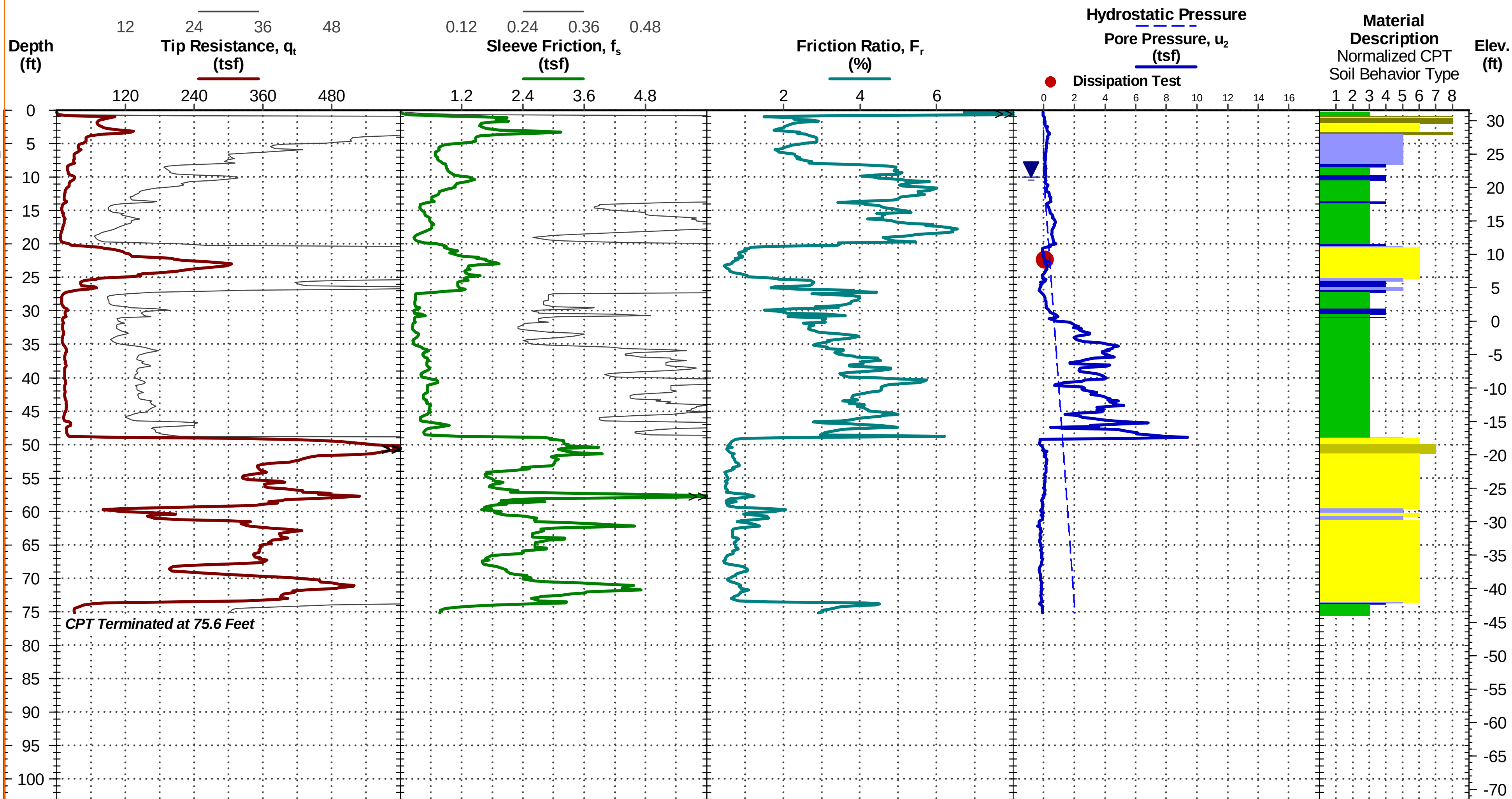
PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc
Columbus, OH

TEST LOCATION: See Exploration Plan

Surface Elev.: 31.53 ft
Northing: 37.3787553
Easting: -121.9333421

SITE: Orchard Parkway
San Jose, CA



See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).
Elevations were provided by HMH

Dead weight of rig used as reaction force.
CPT sensor calibration reports available upon request.

1 Sensitive, fine grained
2 Organic soils - clay
3 Clay - silty clay to clay
4 Silt mixtures - clayey silt to silty clay
5 Sand mixtures - silty sand to sandy silt
6 Sands - clean sand to silty sand
7 Gravelly sand to dense sand
8 Very stiff sand to clayey sand
9 Very stiff fine grained

WATER LEVEL OBSERVATION	Probe no. DDG1530	Terracon	CPT Started: 10/23/2020	CPT Completed: 10/23/2020
▼ 10 ft measured water depth (used in normalizations and correlations; See Supporting Information)		5075 Commercial Cir Ste E Concord, CA	Rig: CPT	Operator: Middle Earth
			Project No.: ND205079	

CPT LOG NO. CPT-03

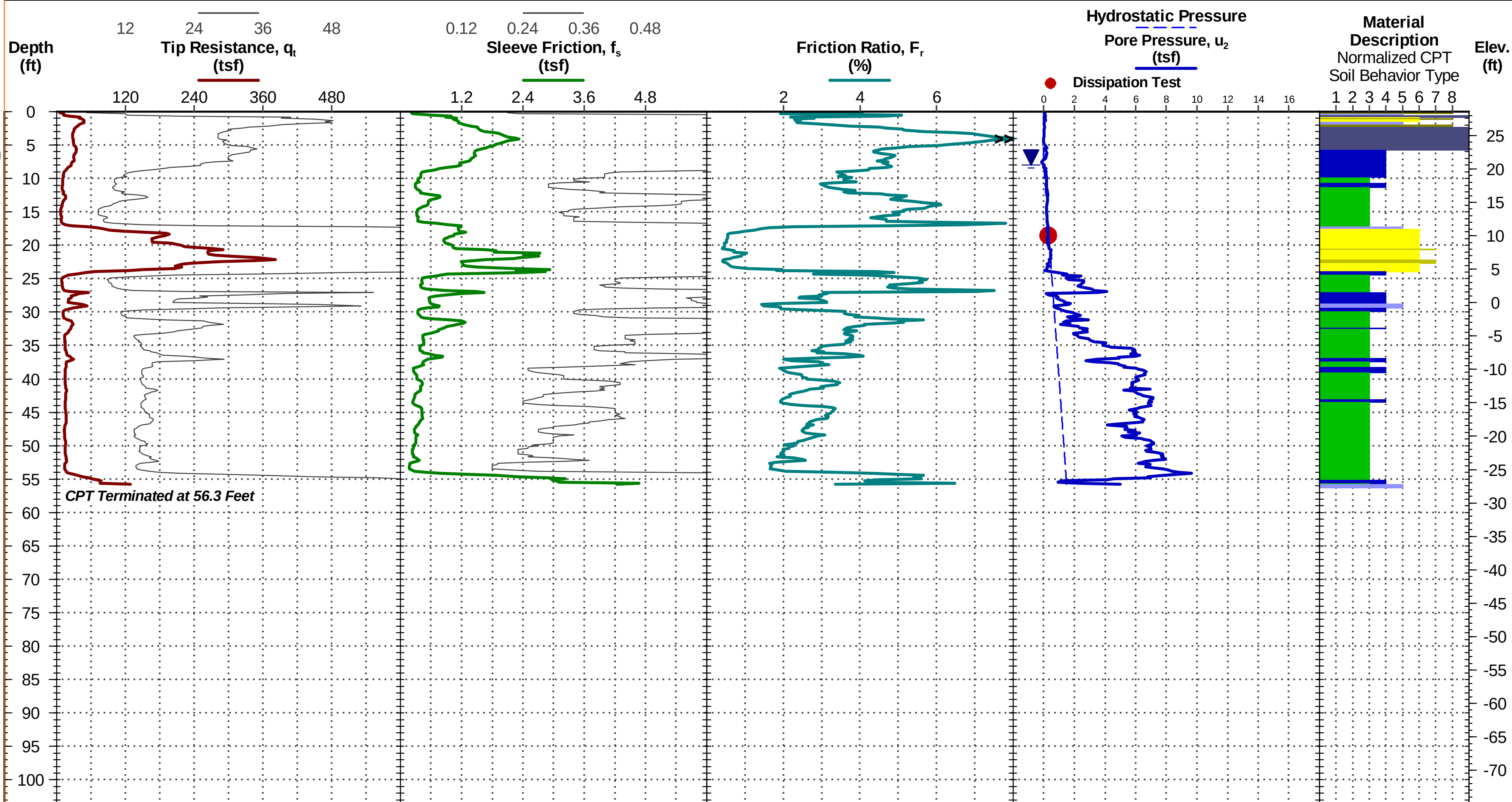
PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc
Columbus, OH

TEST LOCATION: See Exploration Plan

SITE: Orchard Parkway
San Jose, CA

Surface Elev.: 28.57 ft
Northing: 37.3793318
Easting: -121.9326379



See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).
Elevations were provided by HMH

Dead weight of rig used as reaction force.
CPT sensor calibration reports available upon request.

1 Sensitive, fine grained
2 Organic soils - clay
3 Clay - silty clay to clay
4 Silt mixtures - clayey silt to silty clay
5 Sand mixtures - silty sand to sandy silt
6 Sands - clean sand to silty sand
7 Gravelly sand to dense sand
8 Very stiff sand to clayey sand
9 Very stiff fine grained

WATER LEVEL OBSERVATION

8 ft measured water depth
(used in normalizations and correlations; See Supporting Information)

Probe no. DDG1530

Terracon
5075 Commercial Cir Ste E
Concord, CA

CPT Started: 10/22/2020	CPT Completed: 10/22/2020
Rig: CPT	Operator: Middle Earth
Project No.: ND205079	

CPT LOG NO. CPT-04

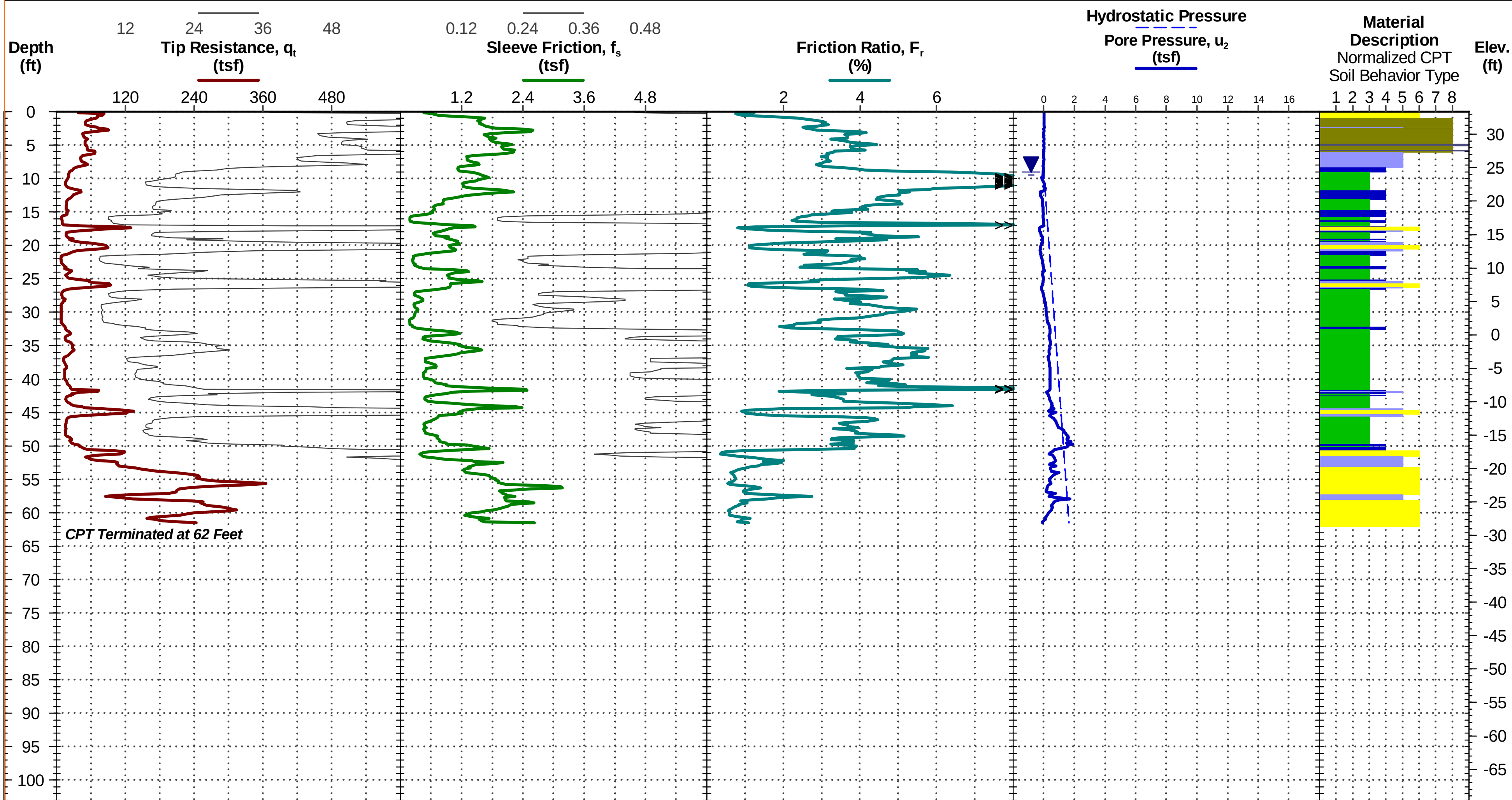
PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc
Columbus, OH

TEST LOCATION: See Exploration Plan

Surface Elev.: 33.38 ft
Northing: 37.3788936
Easting: -121.9312269

SITE: Orchard Parkway
San Jose, CA



See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).
Elevations were provided by HMH

Dead weight of rig used as reaction force.
CPT sensor calibration reports available upon request.

1 Sensitive, fine grained
2 Organic soils - clay
3 Clay - silty clay to clay
4 Silt mixtures - clayey silt to silty clay
5 Sand mixtures - silty sand to sandy silt
6 Sands - clean sand to silty sand
7 Gravelly sand to dense sand
8 Very stiff sand to clayey sand
9 Very stiff fine grained

WATER LEVEL OBSERVATION

9 ft measured water depth
(used in normalizations and correlations; See Supporting Information)

Probe no. DDG1530

Terracon
5075 Commercial Cir Ste E
Concord, CA

CPT Started: 10/22/2020	CPT Completed: 10/22/2020
Rig: CPT	Operator: Middle Earth
Project No.: ND205079	

CPT LOG NO. CPT-05

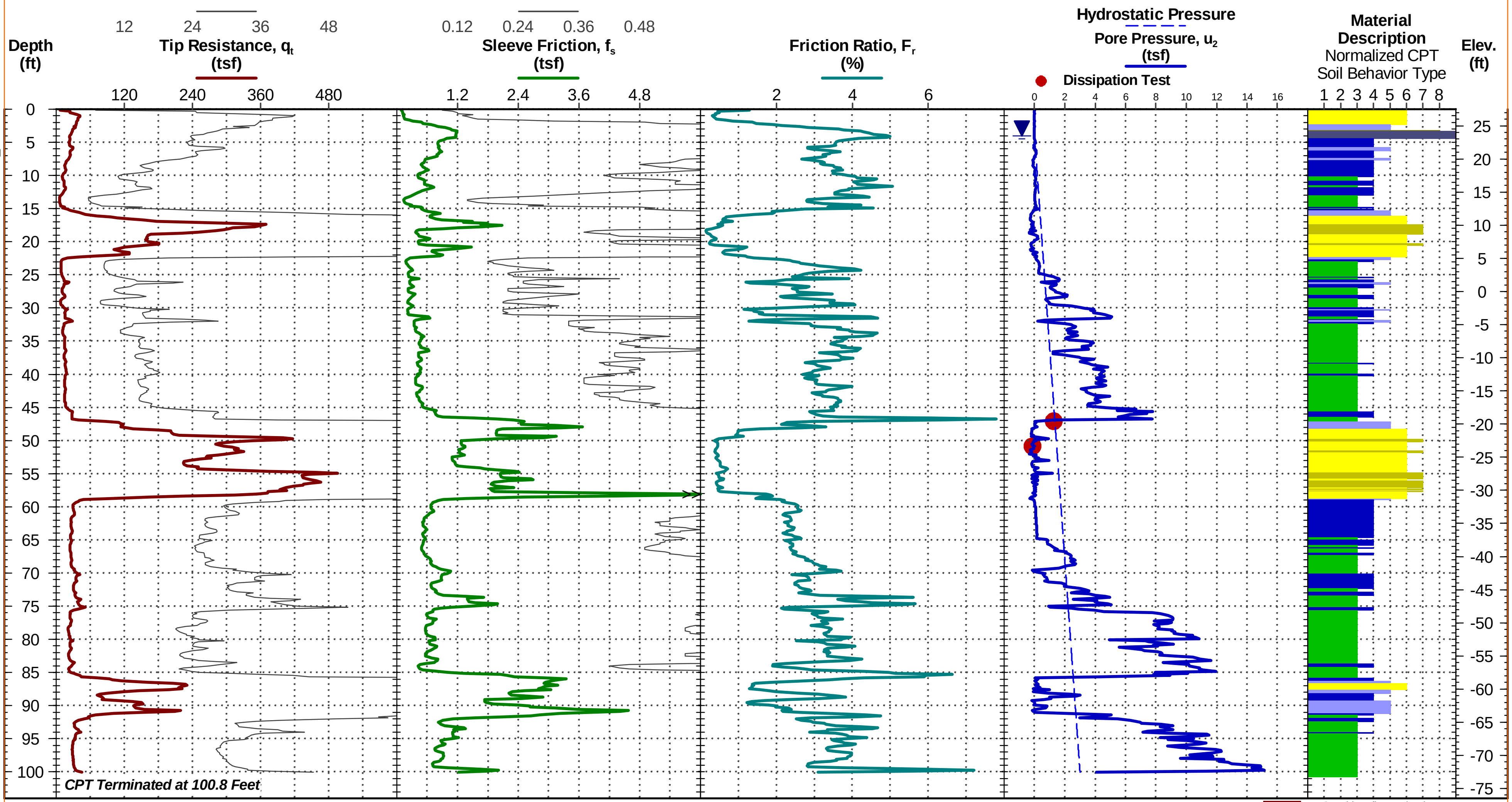
PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc
Columbus, OH

TEST LOCATION: See Exploration Plan

Surface Elev.: 27.51 ft
Northing: 37.3783481
Easting: -121.9326182

SITE: Orchard Parkway
San Jose, CA



See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).
Elevations were provided by HMH

Dead weight of rig used as reaction force.
CPT sensor calibration reports available upon request.

1 Sensitive, fine grained
2 Organic soils - clay
3 Clay - silty clay to clay
4 Silt mixtures - clayey silt to silty clay
5 Sand mixtures - silty sand to sandy silt
6 Sands - clean sand to silty sand
7 Gravelly sand to dense sand
8 Very stiff sand to clayey sand
9 Very stiff fine grained

WATER LEVEL OBSERVATION	Probe no. DDG1530		CPT Started: 10/22/2020	CPT Completed: 10/22/2020
4 ft measured water depth (used in normalizations and correlations; See Supporting Information)		5075 Commercial Cir Ste E Concord, CA	Rig: CPT	Operator: Middle Earth
			Project No.: ND205079	

CPT LOG NO. CPT-06

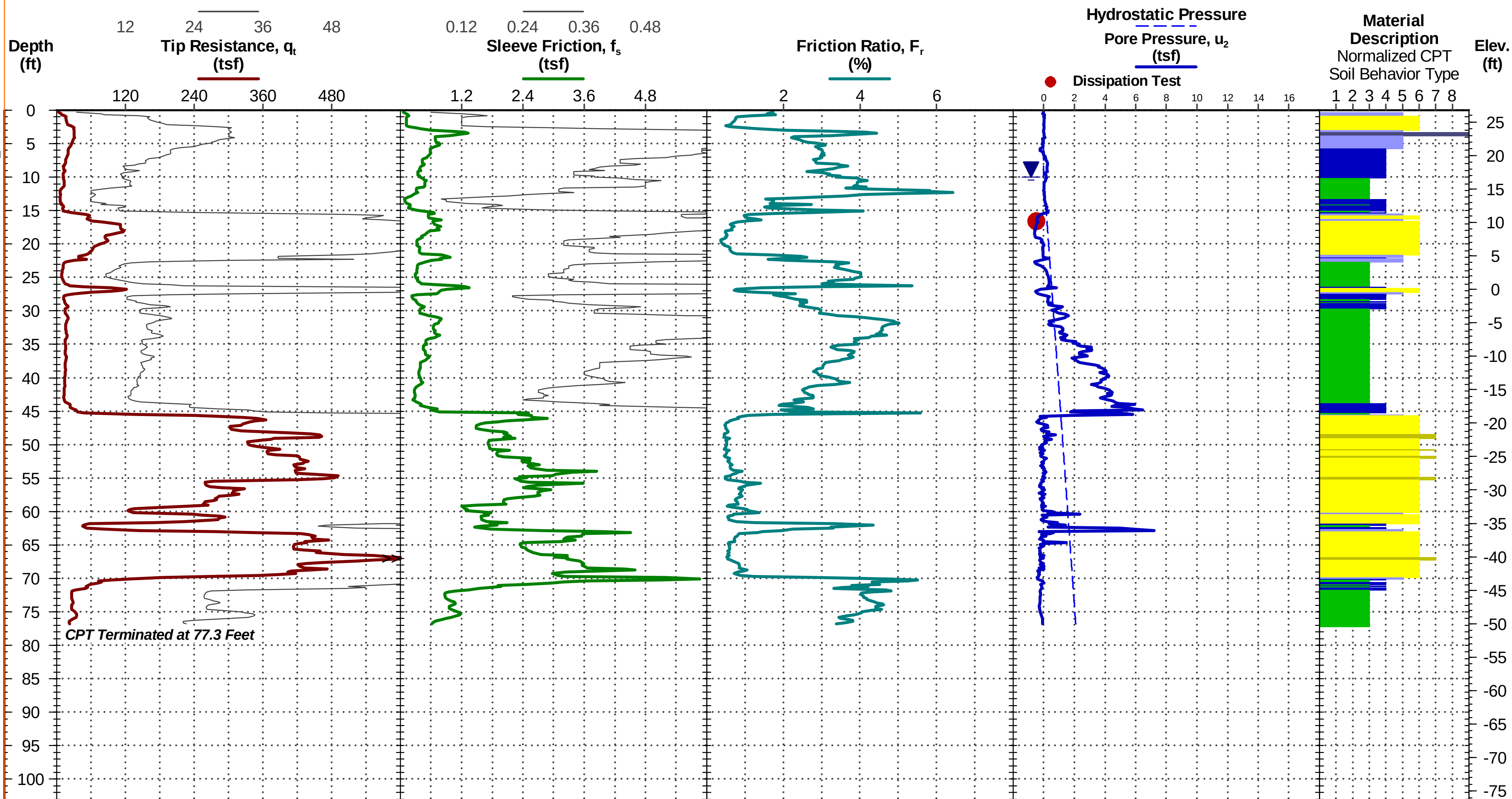
PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc
Columbus, OH

TEST LOCATION: See Exploration Plan

Surface Elev.: 26.77 ft
Northing: 37.378221
Easting: -121.9340598

SITE: Orchard Parkway
San Jose, CA



See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).
Elevations were provided by HMH

Dead weight of rig used as reaction force.
CPT sensor calibration reports available upon request.

1 Sensitive, fine grained
2 Organic soils - clay
3 Clay - silty clay to clay
4 Silt mixtures - clayey silt to silty clay
5 Sand mixtures - silty sand to sandy silt
6 Sands - clean sand to silty sand
7 Gravelly sand to dense sand
8 Very stiff sand to clayey sand
9 Very stiff fine grained

WATER LEVEL OBSERVATION

10 ft measured water depth
(used in normalizations and correlations; See Supporting Information)

Probe no. DDG1530

Terracon
5075 Commercial Cir Ste E
Concord, CA

CPT Started: 10/23/2020	CPT Completed: 10/23/2020
Rig: CPT	Operator: Middle Earth
Project No.: ND205079	

CPT LOG NO. CPT-07

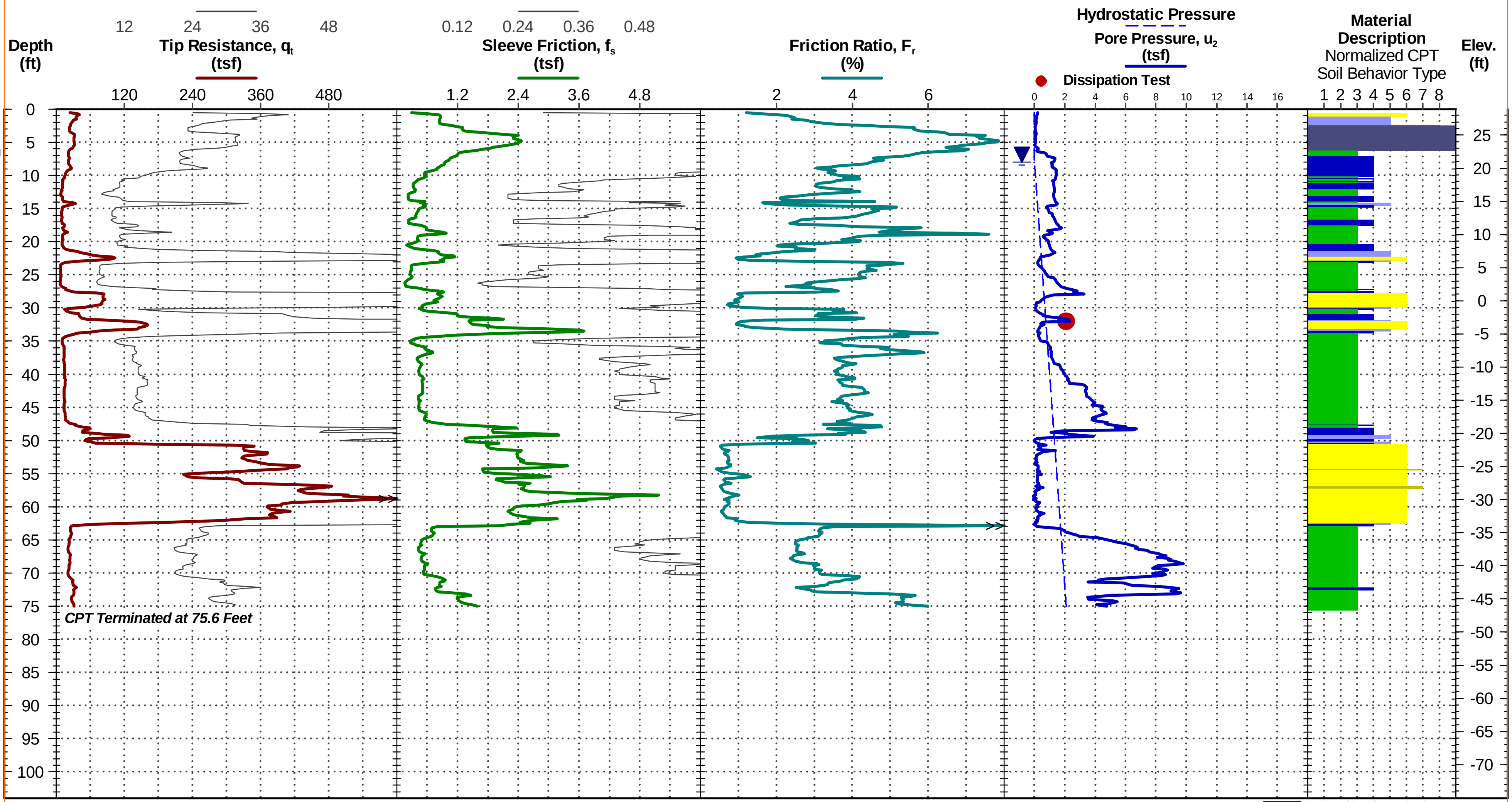
PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc
Columbus, OH

TEST LOCATION: See Exploration Plan

Surface Elev.: 28.91 ft
Northing: 37.3773138
Easting: -121.9325117

SITE: Orchard Parkway
San Jose, CA



See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).
Elevations were provided by HMH

Dead weight of rig used as reaction force.
CPT sensor calibration reports available upon request.

1 Sensitive, fine grained
2 Organic soils - clay
3 Clay - silty clay to clay
4 Silt mixtures - clayey silt to silty clay
5 Sand mixtures - silty sand to sandy silt
6 Sands - clean sand to silty sand
7 Gravelly sand to dense sand
8 Very stiff sand to clayey sand
9 Very stiff fine grained

WATER LEVEL OBSERVATION	Probe no. DDG1530	Terracon	CPT Started: 10/22/2020	CPT Completed: 10/22/2020
8 ft measured water depth (used in normalizations and correlations; See Supporting Information)		5075 Commercial Cir Ste E, Concord, CA	Rig: CPT	Operator: Middle Earth
			Project No.: ND205079	

CPT LOG NO. CPT-08

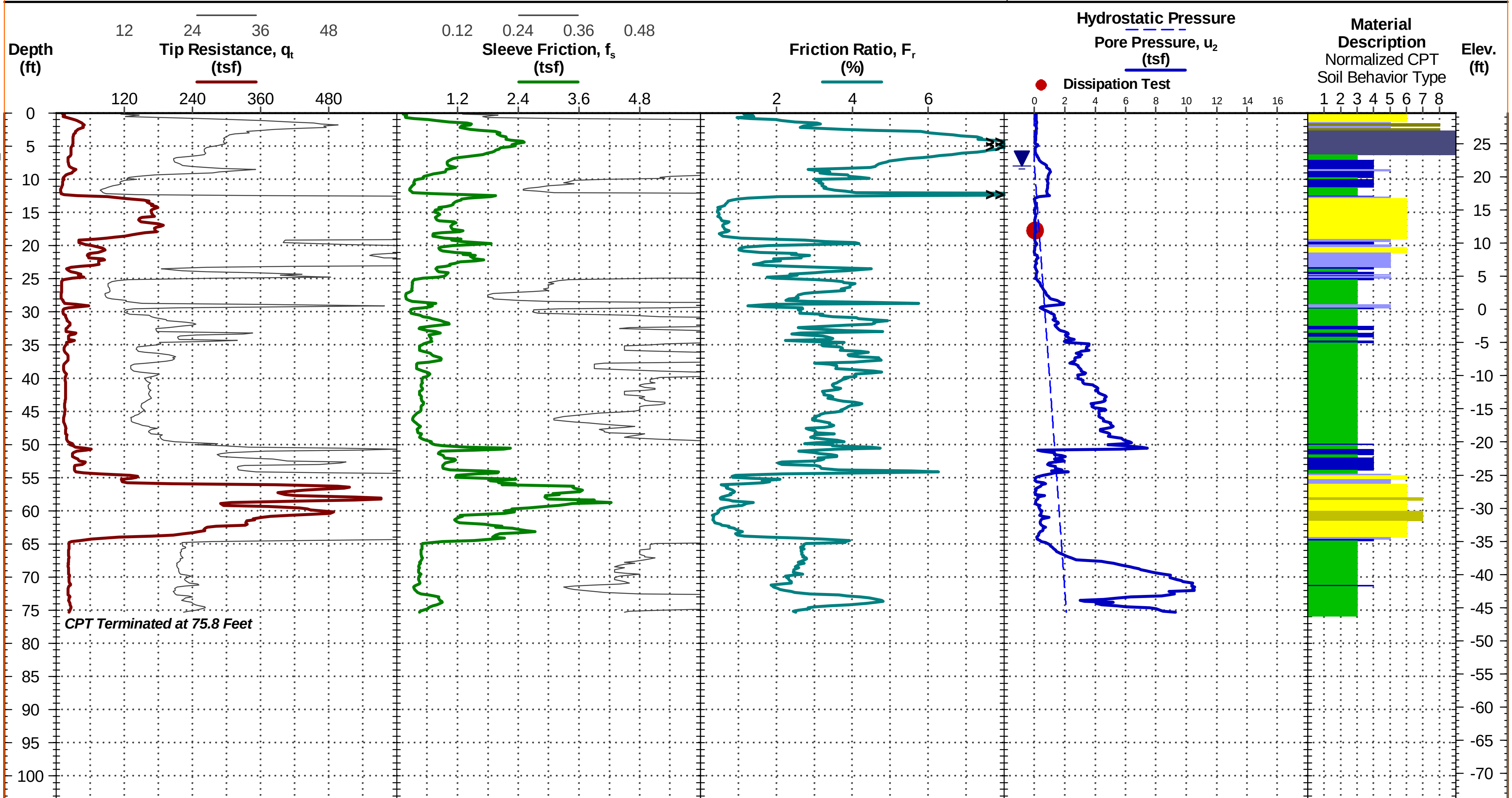
PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc
Columbus, OH

TEST LOCATION: See Exploration Plan

Surface Elev.: 29.61 ft
Northing: 37.3780836
Easting: -121.9316126

SITE: Orchard Parkway
San Jose, CA



See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).
Elevations were provided by HMH

Dead weight of rig used as reaction force.
CPT sensor calibration reports available upon request.

1 Sensitive, fine grained
2 Organic soils - clay
3 Clay - silty clay to clay
4 Silt mixtures - clayey silt to silty clay
5 Sand mixtures - silty sand to sandy silt
6 Sands - clean sand to silty sand
7 Gravelly sand to dense sand
8 Very stiff sand to clayey sand
9 Very stiff fine grained

WATER LEVEL OBSERVATION

8 ft measured water depth
(used in normalizations and correlations; See Supporting Information)

Probe no. DDG1530

Terracon
5075 Commercial Cir Ste E
Concord, CA

CPT Started: 10/22/2020	CPT Completed: 10/22/2020
Rig: CPT	Operator: Middle Earth
Project No.: ND205079	

CPT LOG NO. CPT-09

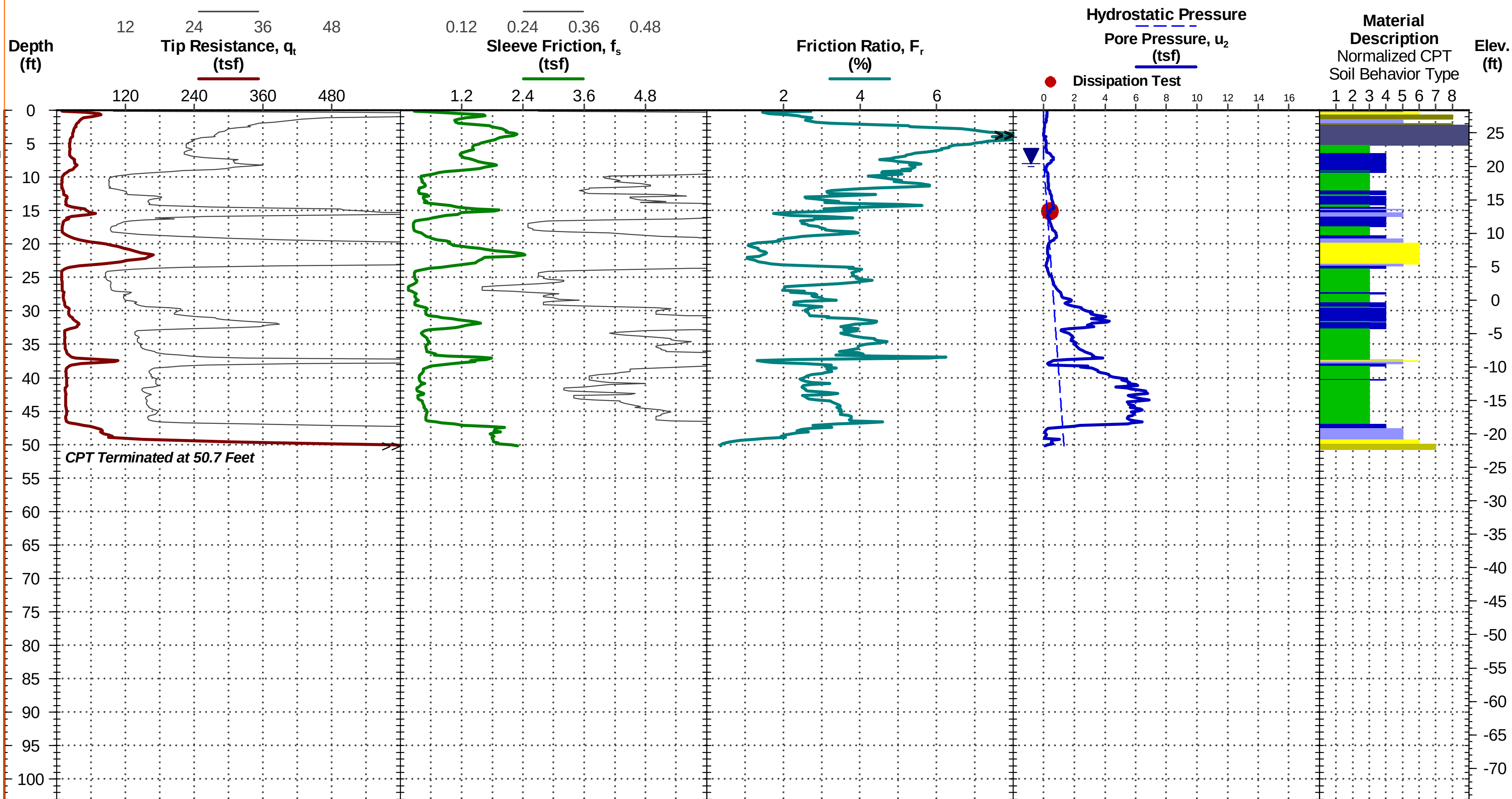
PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc
Columbus, OH

TEST LOCATION: See Exploration Plan

Surface Elev.: 28.39 ft
Northing: 37.3794703
Easting: -121.9313941

SITE: Orchard Parkway
San Jose, CA



See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).
Elevations were provided by HMH

Dead weight of rig used as reaction force.
CPT sensor calibration reports available upon request.

1 Sensitive, fine grained
2 Organic soils - clay
3 Clay - silty clay to clay
4 Silt mixtures - clayey silt to silty clay
5 Sand mixtures - silty sand to sandy silt
6 Sands - clean sand to silty sand
7 Gravelly sand to dense sand
8 Very stiff sand to clayey sand
9 Very stiff fine grained

WATER LEVEL OBSERVATION	Probe no. DDG1530	Terracon	CPT Started: 10/22/2020	CPT Completed: 10/22/2020
8 ft measured water depth (used in normalizations and correlations; See Supporting Information)		5075 Commercial Cir Ste E Concord, CA	Rig: CPT	Operator: Middle Earth
			Project No.: ND205079	

CPT LOG NO. CPT-10

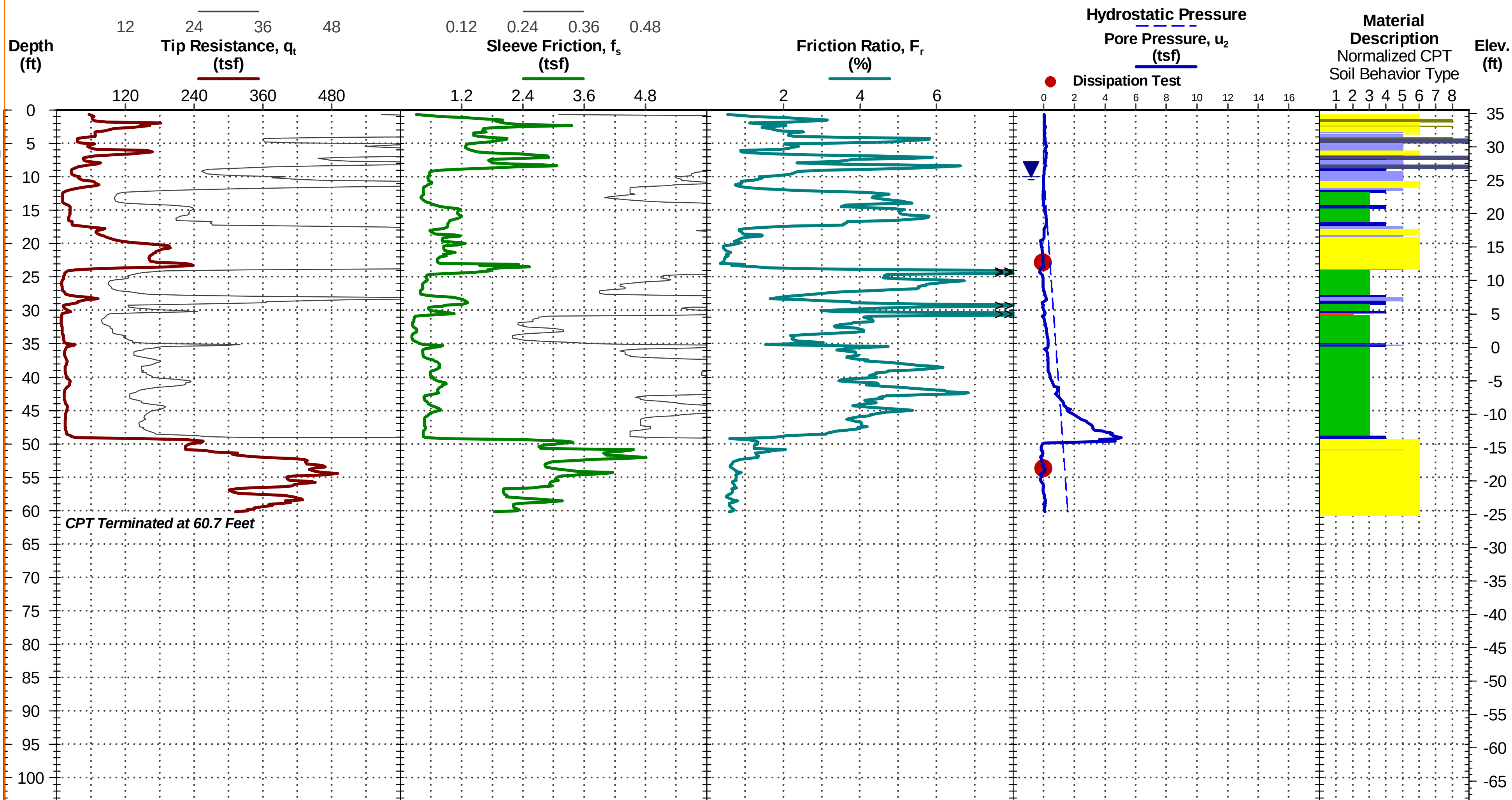
PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc
Columbus, OH

TEST LOCATION: See Exploration Plan

SITE: Orchard Parkway
San Jose, CA

Surface Elev.: 35.53 ft
Northing: 37.3786438
Easting: -121.9350387



See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).
Elevations were provided by HMH

Dead weight of rig used as reaction force.
CPT sensor calibration reports available upon request.

1 Sensitive, fine grained
2 Organic soils - clay
3 Clay - silty clay to clay
4 Silt mixtures - clayey silt to silty clay
5 Sand mixtures - silty sand to sandy silt
6 Sands - clean sand to silty sand
7 Gravelly sand to dense sand
8 Very stiff sand to clayey sand
9 Very stiff fine grained

WATER LEVEL OBSERVATION	Probe no. DDG1530	CPT Started: 10/23/2020	CPT Completed: 10/23/2020
10 ft measured water depth (used in normalizations and correlations; See Supporting Information)		Rig: CPT	Operator: Middle Earth
		Project No.: ND205079	

Terracon
5075 Commercial Cir Ste E
Concord, CA

CPT LOG NO. CPT-11

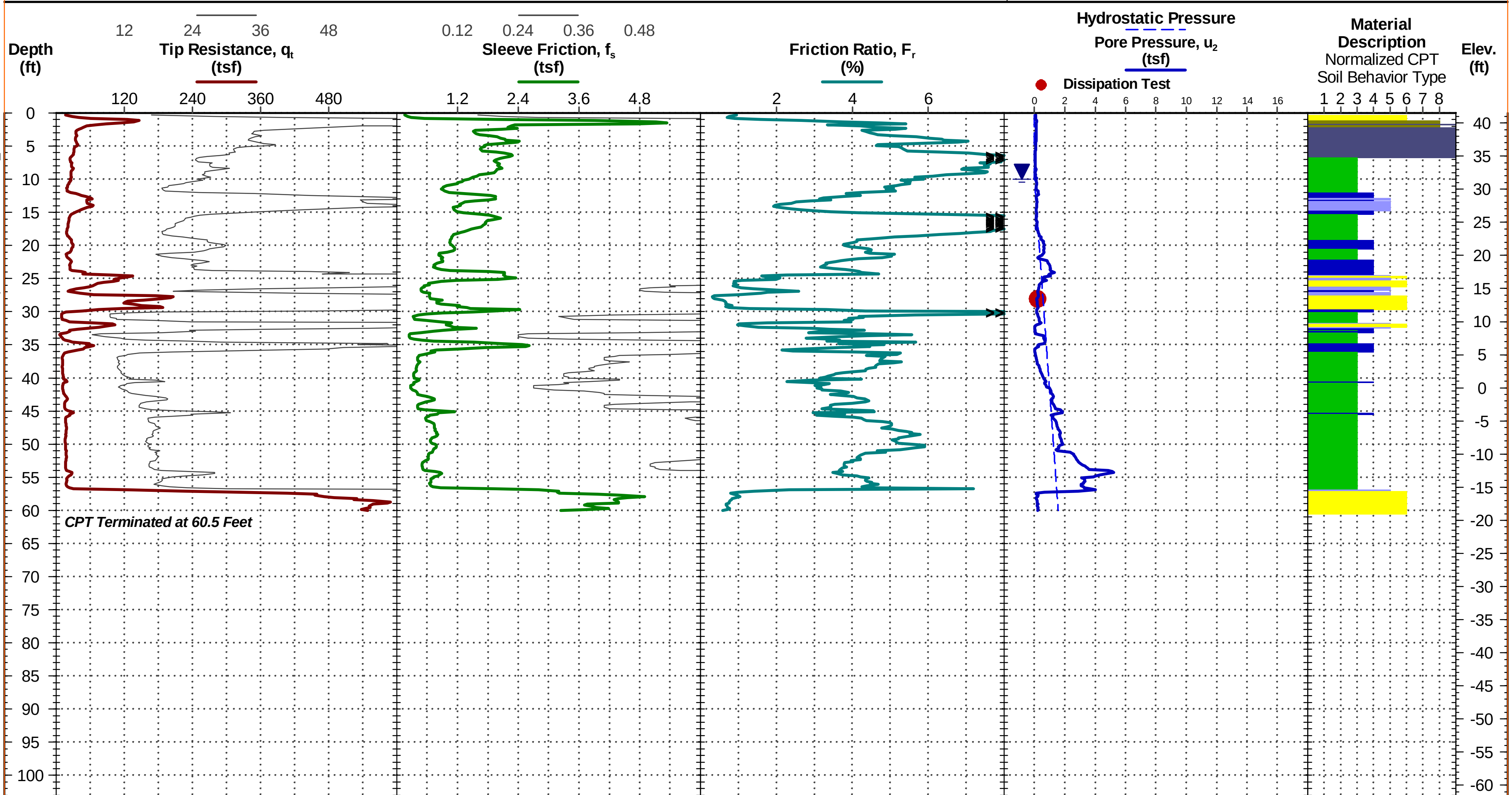
PROJECT: Bay AZ1 Site (Northtown)

CLIENT: Burns & McDonnell Engineering Company Inc
Columbus, OH

TEST LOCATION: See Exploration Plan

SITE: Orchard Parkway
San Jose, CA

Surface Elev.: 41.5 ft
Northing: 37.3789611
Easting: -121.9338514



See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).
Elevations were provided by HMH

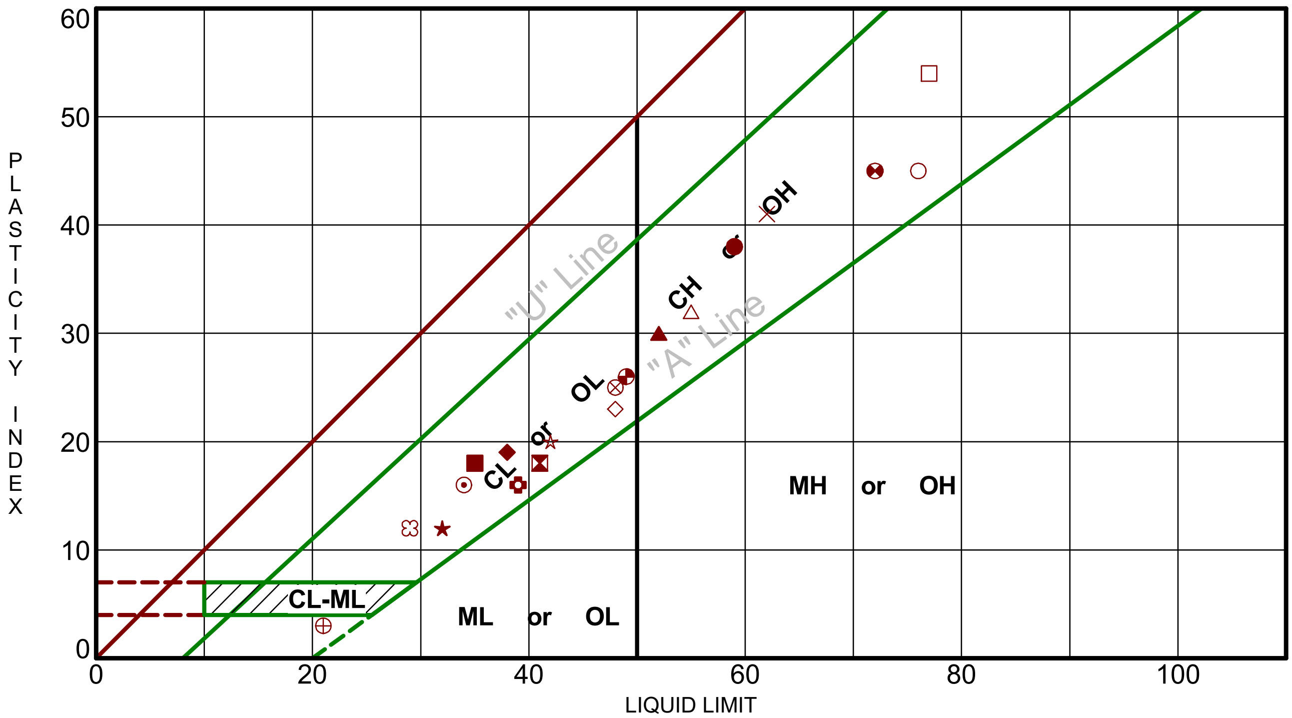
Dead weight of rig used as reaction force.
CPT sensor calibration reports available upon request.

1 Sensitive, fine grained
2 Organic soils - clay
3 Clay - silty clay to clay
4 Silt mixtures - clayey silt to silty clay
5 Sand mixtures - silty sand to sandy silt
6 Sands - clean sand to silty sand
7 Gravelly sand to dense sand
8 Very stiff sand to clayey sand
9 Very stiff fine grained

WATER LEVEL OBSERVATION	Probe no. DDG1530	Terracon 5075 Commercial Cir Ste E Concord, CA	CPT Started: 10/23/2020	CPT Completed: 10/23/2020
10 ft measured water depth (used in normalizations and correlations; See Supporting Information)			Rig: CPT	Operator: Middle Earth
			Project No.: ND205079	

ATTERBERG LIMITS RESULTS

ASTM D4318



	Boring ID	Depth	LL	PL	PI	Fines	USCS	Description
●	B1	1 - 2.5	59	21	38	89.6	CH	FAT CLAY
⧗	B1	10 - 11.5	41	23	18	96.5	CL	LEAN CLAY
▲	B1	35 - 36.5	52	22	30	98.8	CH	FAT CLAY
★	B2	7.5 - 9	32	20	12	62.8	CL	SANDY LEAN CLAY
⊙	B3	15 - 16.5	34	18	16	79.7	CL	LEAN CLAY with SAND
✿	B3	40 - 41.5	39	23	16	99.5	CL	LEAN CLAY
○	B4	2.5 - 4	76	31	45	95.1	CH	FAT CLAY
△	B4	10 - 11.5	55	23	32	85.4	CH	Fat Clay
⊗	B6	1 - 2.5	48	23	25	77.6	CL	LEAN CLAY with SAND
⊕	B6	30 - 31.5	21	18	3	52.4	ML	SANDY SILT
□	B7	2.5 - 4	77	23	54	93.2	CH	FAT CLAY
⊛	B7	5 - 6.5	72	27	45	84.1	CH	FAT CLAY with SAND
◕	B8	2.5 - 4	49	23	26	80.3	CL	LEAN CLAY with SAND
☆	B8	7.5 - 9	42	22	20	82.6	CL	LEAN CLAY with SAND
✾	B8	30 - 31.5	29	17	12			
■	B8	45 - 46.5	35	17	18	73.6	CL	LEAN CLAY with SAND
◆	B13	2.5 - 4	38	19	19	72.9	CL	LEAN CLAY with SAND
◇	B14	5 - 6.5	48	25	23	90.9	CL	LEAN CLAY
×	B15	2.5 - 4	62	21	41	81.8	CH	FAT CLAY with SAND

PROJECT: Bay AZ1 Site (Northtown)

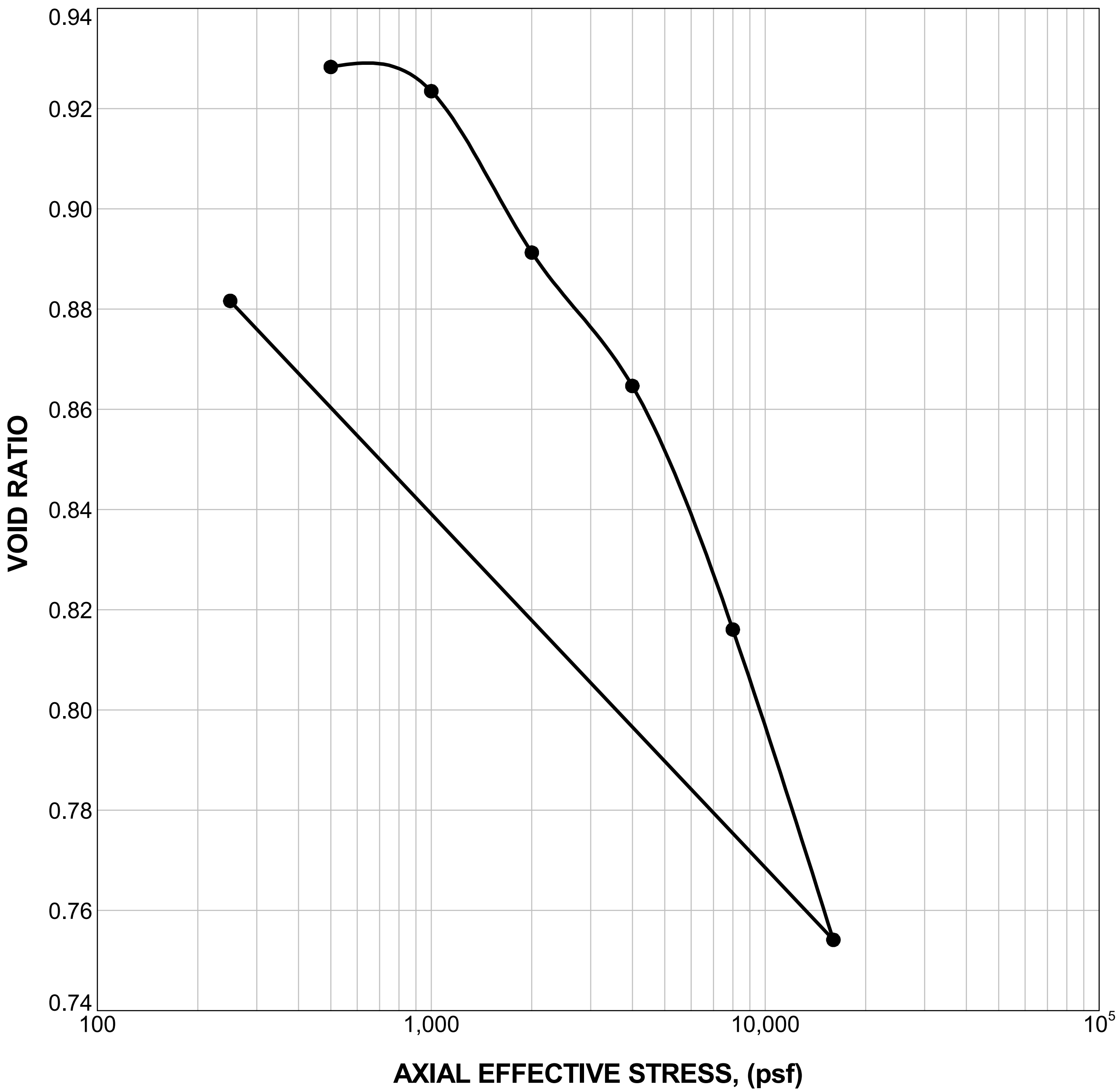
SITE: Orchard Parkway
San Jose, CA

Terracon
5075 Commercial Cir Ste E
Concord, CA

PROJECT NUMBER: ND205079

CLIENT: Burns & McDonnell Engineering Company Inc
Columbus, OH

CONSOLIDATION TEST (D2435)



Natural		Initial Dry Density (pcf)	LL	PI	Sp. Gr.	Overburden (psf)	P_c (psf)	C_c (vr / log stress)	C_r (vr / log stress)	Initial Void Ratio
Saturation	Moisture									
%	30.8 %	87.4	55	32	2.70					

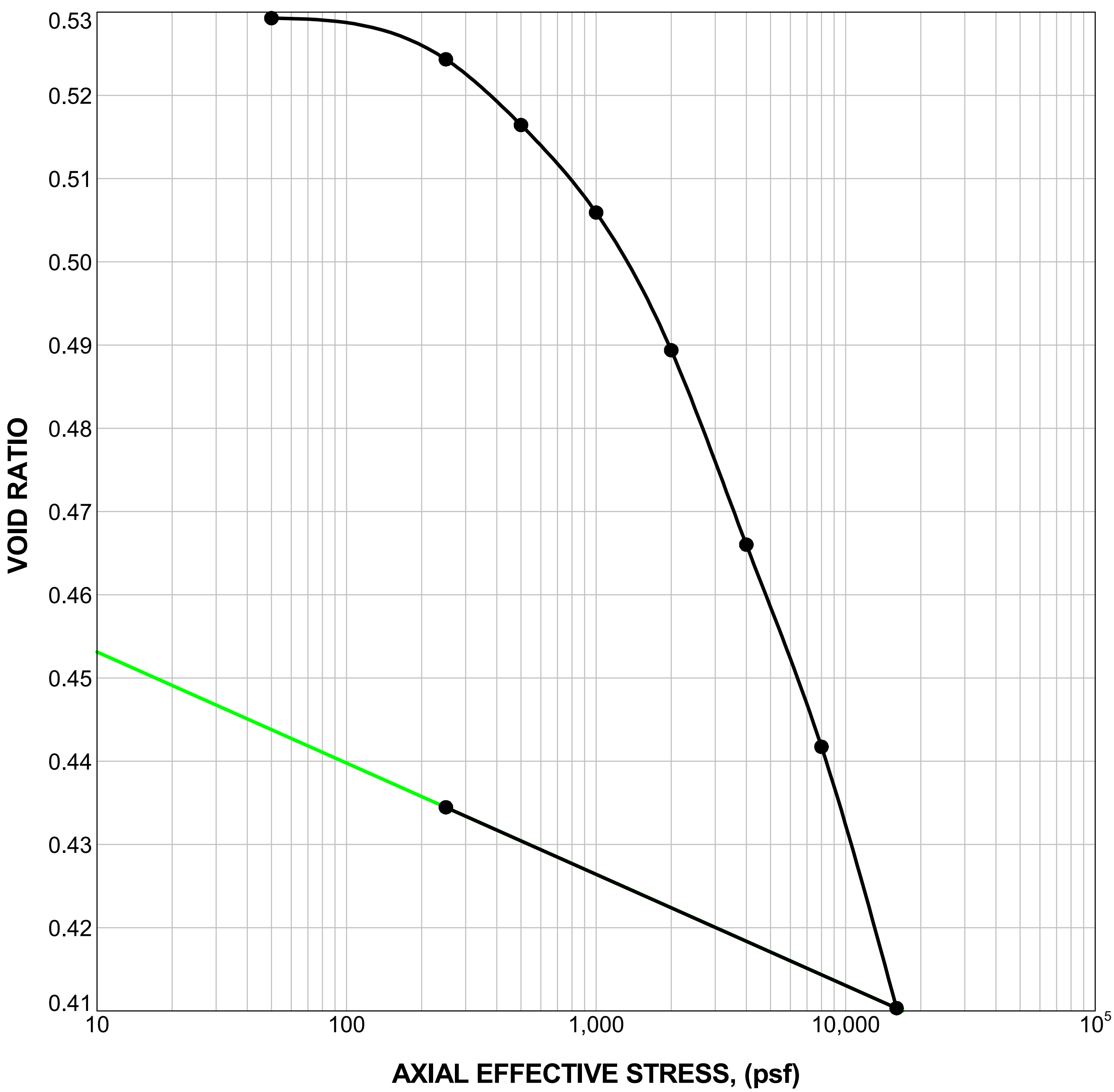
MATERIAL DESCRIPTION	USCS	AASHTO
Fat Clay	CH	A-7-6

NOTES:

Borehole: B4 Depth: 10 ft Specimen #: 5

PROJECT: Bay AZ1 Site (Northtown)	Terracon 5075 Commercial Cir Ste E Concord, CA	PROJECT NUMBER: ND205079
SITE: Orchard Parkway San Jose, CA		CLIENT: Burns & McDonnell Engineering Company Inc Columbus, OH

CONSOLIDATION TEST (D2435)



Natural		Initial Dry Density (pcf)	LL	PI	Sp. Gr.	Overburden (psf)	P_c (psf)	C_c (vr / log stress)	C_r (vr / log stress)	Initial Void Ratio
Saturation	Moisture									
97.7 %	19.8 %	109.0			2.70					0.547

MATERIAL DESCRIPTION	USCS	AASHTO
Sandy Lean Clay	CL	

NOTES:

Borehole: B8 Depth: 26.5 ft Specimen #: 10

PROJECT: Bay AZ1 Site (Northtown)

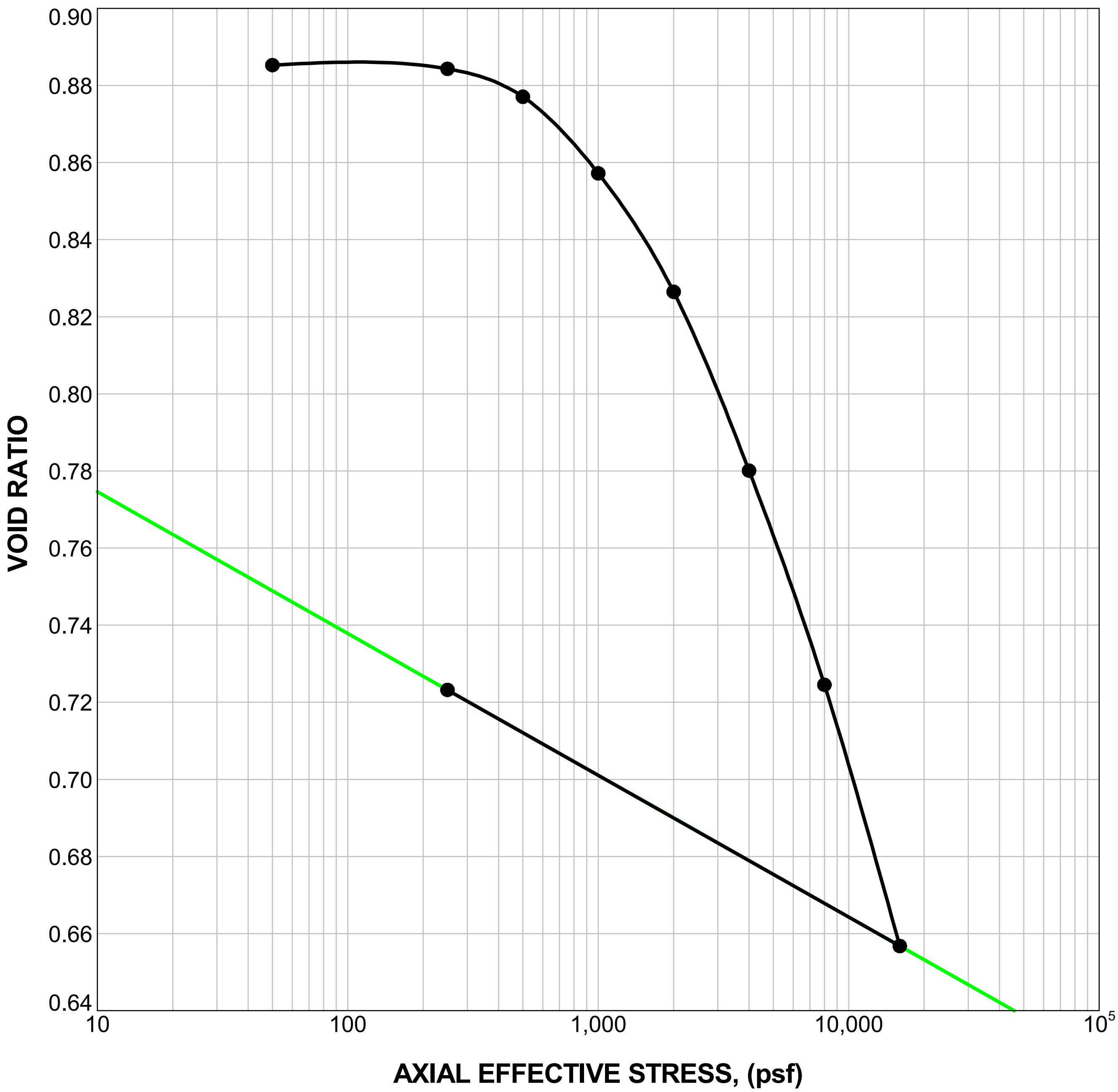
SITE: Orchard Parkway
San Jose, CA

Terracon
5075 Commercial Cir Ste E
Concord, CA

PROJECT NUMBER: ND205079

CLIENT: Burns & McDonnell Engineering Company Inc
Columbus, OH

CONSOLIDATION TEST (D2435)



Natural		Initial Dry Density (pcf)	LL	PI	Sp. Gr.	Overburden (psf)	P_c (psf)	C_c (vr / log stress)	C_r (vr / log stress)	Initial Void Ratio
Saturation	Moisture									
102.7 %	34.6 %	88.3			2.70					0.909

MATERIAL DESCRIPTION	USCS	AASHTO
Lean Clay	CL	

NOTES:

Borehole: B13 Depth: 26.5 ft Specimen #: 9

PROJECT: Bay AZ1 Site (Northtown)

SITE: Orchard Parkway
San Jose, CA

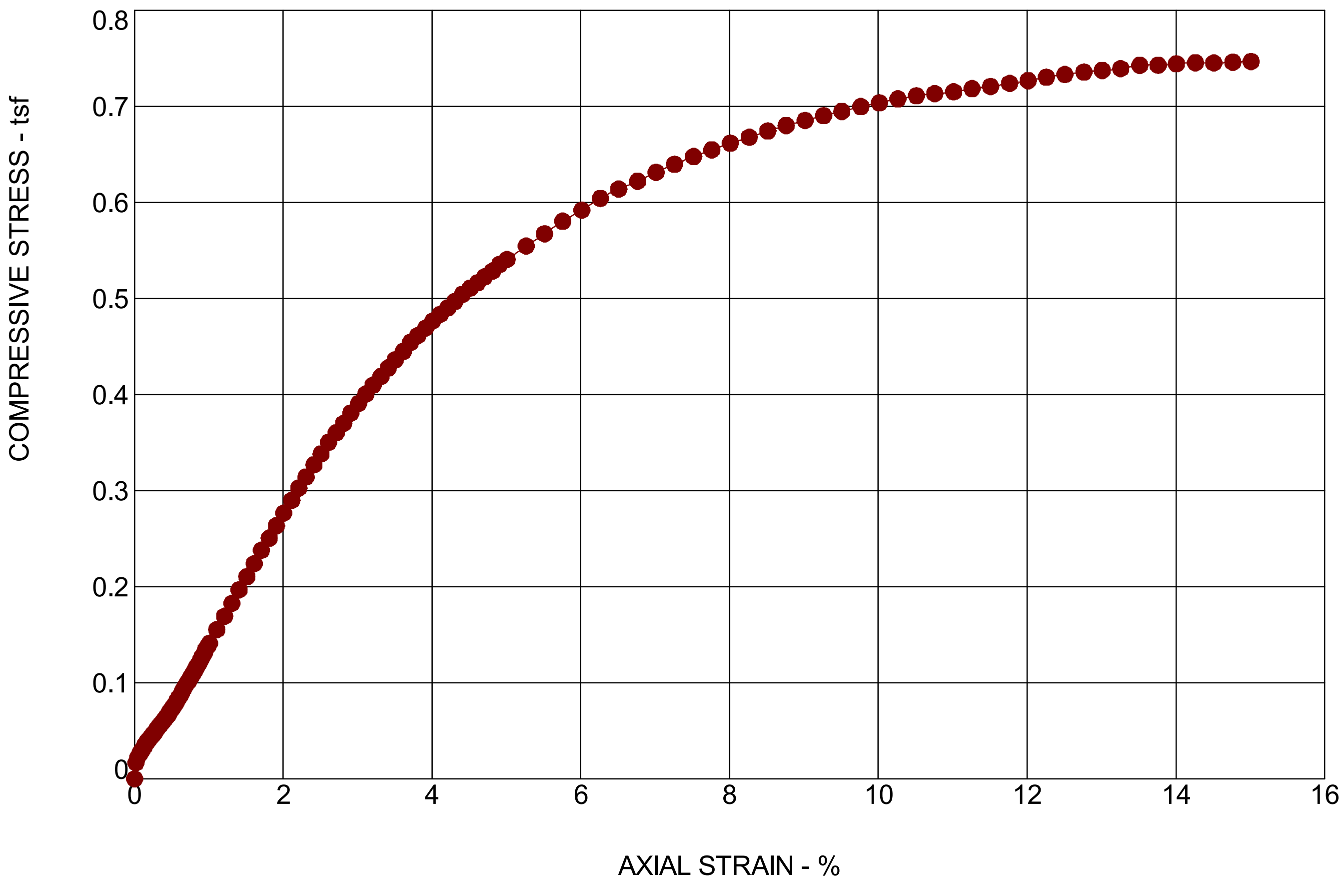
Terracon
5075 Commercial Cir Ste E
Concord, CA

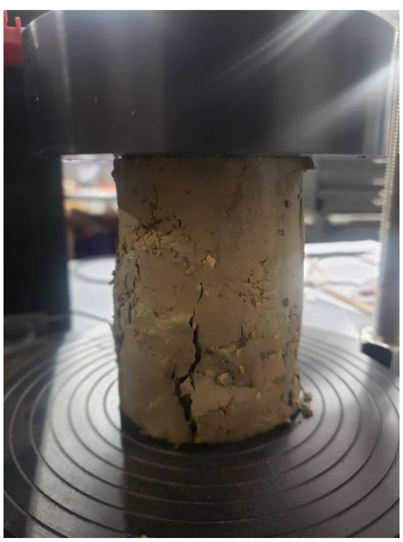
PROJECT NUMBER: ND205079

CLIENT: Burns & McDonnell Engineering Company Inc
Columbus, OH

UNCONFINED COMPRESSION TEST

ASTM D2166



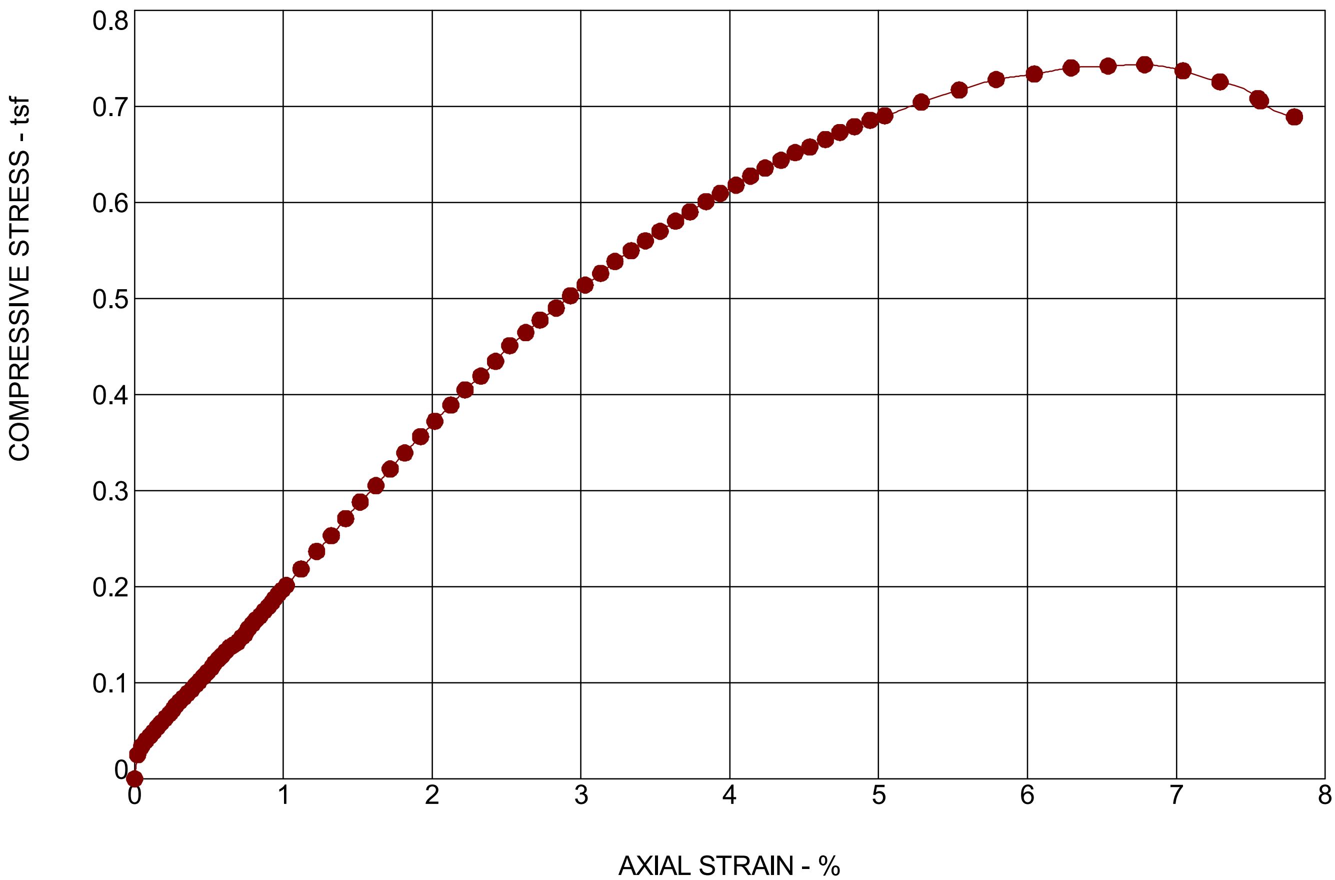
SPECIMEN FAILURE PHOTOGRAPH	SPECIMEN TEST DATA		
	Moisture Content:	%	29.3
	Dry Density:	pcf	98
	Diameter:	in.	2.37
	Height:	in.	3.90
	Height / Diameter Ratio:		1.65
	Calculated Saturation:	%	
	Calculated Void Ratio:		
	Assumed Specific Gravity:		
	Failure Strain:	%	15.00
	Unconfined Compressive Strength	(tsf)	0.75
	Undrained Shear Strength:	(tsf)	0.37
	Strain Rate:	in/min	
	Remarks:		

SAMPLE TYPE: CARS	SAMPLE LOCATION: B1 @ 10 - 11.5 feet			
DESCRIPTION: LEAN CLAY(CL)	LL 41	PL 23	PI 18	Percent < #200 Sieve 97

PROJECT: Bay AZ1 Site (Northtown)	Terracon 5075 Commercial Cir Ste E Concord, CA	PROJECT NUMBER: ND205079
SITE: Orchard Parkway San Jose, CA		CLIENT: Burns & McDonnell Engineering Company Inc Columbus, OH

UNCONFINED COMPRESSION TEST

ASTM D2166



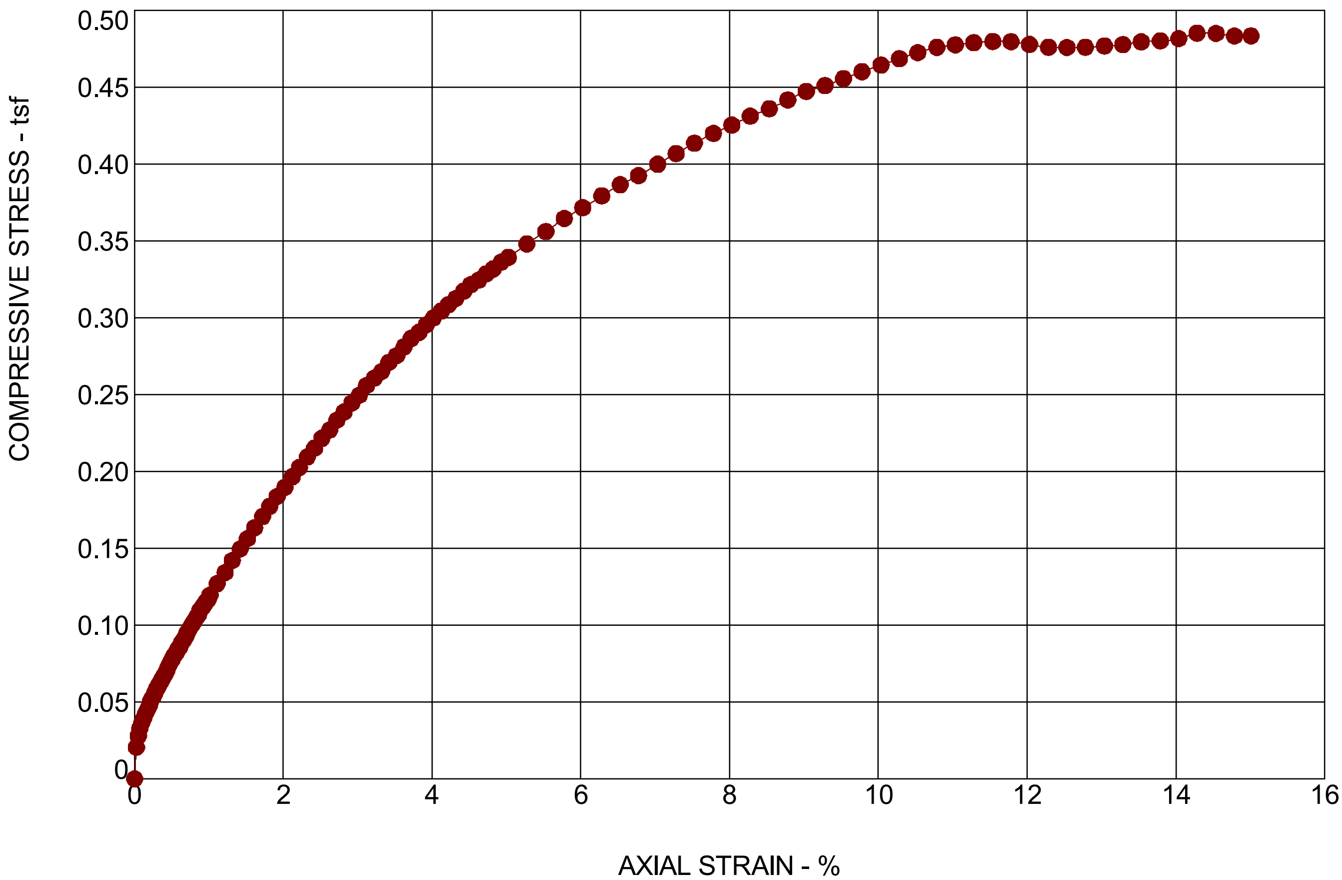
SPECIMEN FAILURE PHOTOGRAPH	SPECIMEN TEST DATA		
	Moisture Content:	%	21.7
	Dry Density:	pcf	105
	Diameter:	in.	2.41
	Height:	in.	4.07
	Height / Diameter Ratio:		1.69
	Calculated Saturation:	%	
	Calculated Void Ratio:		
	Assumed Specific Gravity:		
	Failure Strain:	%	6.79
	Unconfined Compressive Strength	(tsf)	0.74
	Undrained Shear Strength:	(tsf)	0.37
	Strain Rate:	in/min	
	Remarks:		

SAMPLE TYPE: CARS	SAMPLE LOCATION: B2 @ 7.5 - 9 feet			
DESCRIPTION: SANDY LEAN CLAY(CL)	LL	PL	PI	Percent < #200 Sieve
	32	20	12	63

PROJECT: Bay AZ1 Site (Northtown)	Terracon	PROJECT NUMBER: ND205079
SITE: Orchard Parkway San Jose, CA	5075 Commercial Cir Ste E Concord, CA	CLIENT: Burns & McDonnell Engineering Company Inc Columbus, OH

UNCONFINED COMPRESSION TEST

ASTM D2166



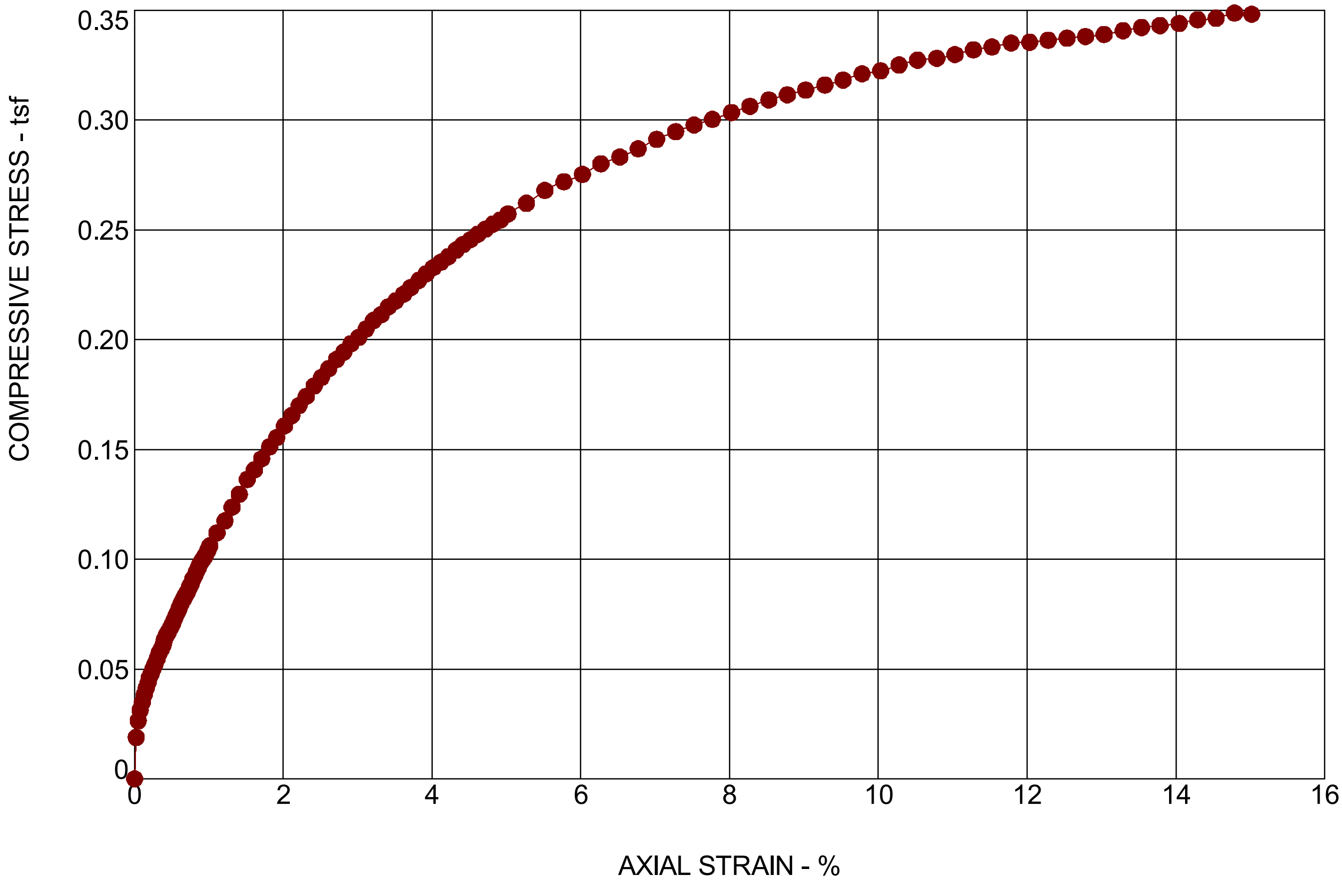
SPECIMEN FAILURE PHOTOGRAPH	SPECIMEN TEST DATA		
	Moisture Content:	%	16.3
	Dry Density:	pcf	117
	Diameter:	in.	2.30
	Height:	in.	3.83
	Height / Diameter Ratio:		1.66
	Calculated Saturation:	%	
	Calculated Void Ratio:		
	Assumed Specific Gravity:		
	Failure Strain:	%	14.28
	Unconfined Compressive Strength	(tsf)	0.49
	Undrained Shear Strength:	(tsf)	0.24
	Strain Rate:	in/min	
	Remarks:		

SAMPLE TYPE: CARS	SAMPLE LOCATION: B2 @ 35 - 36.5 feet			
DESCRIPTION: LEAN CLAY	LL	PL	PI	Percent < #200 Sieve

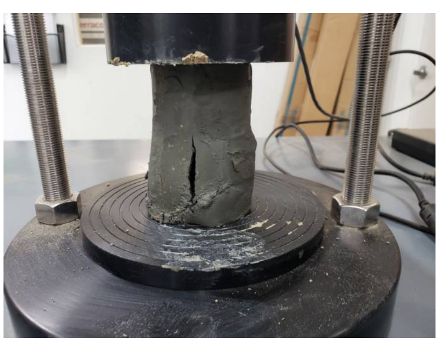
PROJECT: Bay AZ1 Site (Northtown)	Terracon 5075 Commercial Cir Ste E Concord, CA	PROJECT NUMBER: ND205079
SITE: Orchard Parkway San Jose, CA		CLIENT: Burns & McDonnell Engineering Company Inc Columbus, OH

UNCONFINED COMPRESSION TEST

ASTM D2166



SPECIMEN FAILURE PHOTOGRAPH



SPECIMEN TEST DATA		
Moisture Content:	%	29.8
Dry Density:	pcf	98
Diameter:	in.	2.35
Height:	in.	4.31
Height / Diameter Ratio:		1.83
Calculated Saturation:	%	
Calculated Void Ratio:		
Assumed Specific Gravity:		
Failure Strain:	%	14.79
Unconfined Compressive Strength	(tsf)	0.35
Undrained Shear Strength:	(tsf)	0.17
Strain Rate:	in/min	
Remarks:		

SAMPLE TYPE: CARS	SAMPLE LOCATION: B4 @ 40 - 41.5 feet			
DESCRIPTION: LEAN CLAY	LL	PL	PI	Percent < #200 Sieve

PROJECT: Bay AZ1 Site (Northtown)

SITE: Orchard Parkway
San Jose, CA

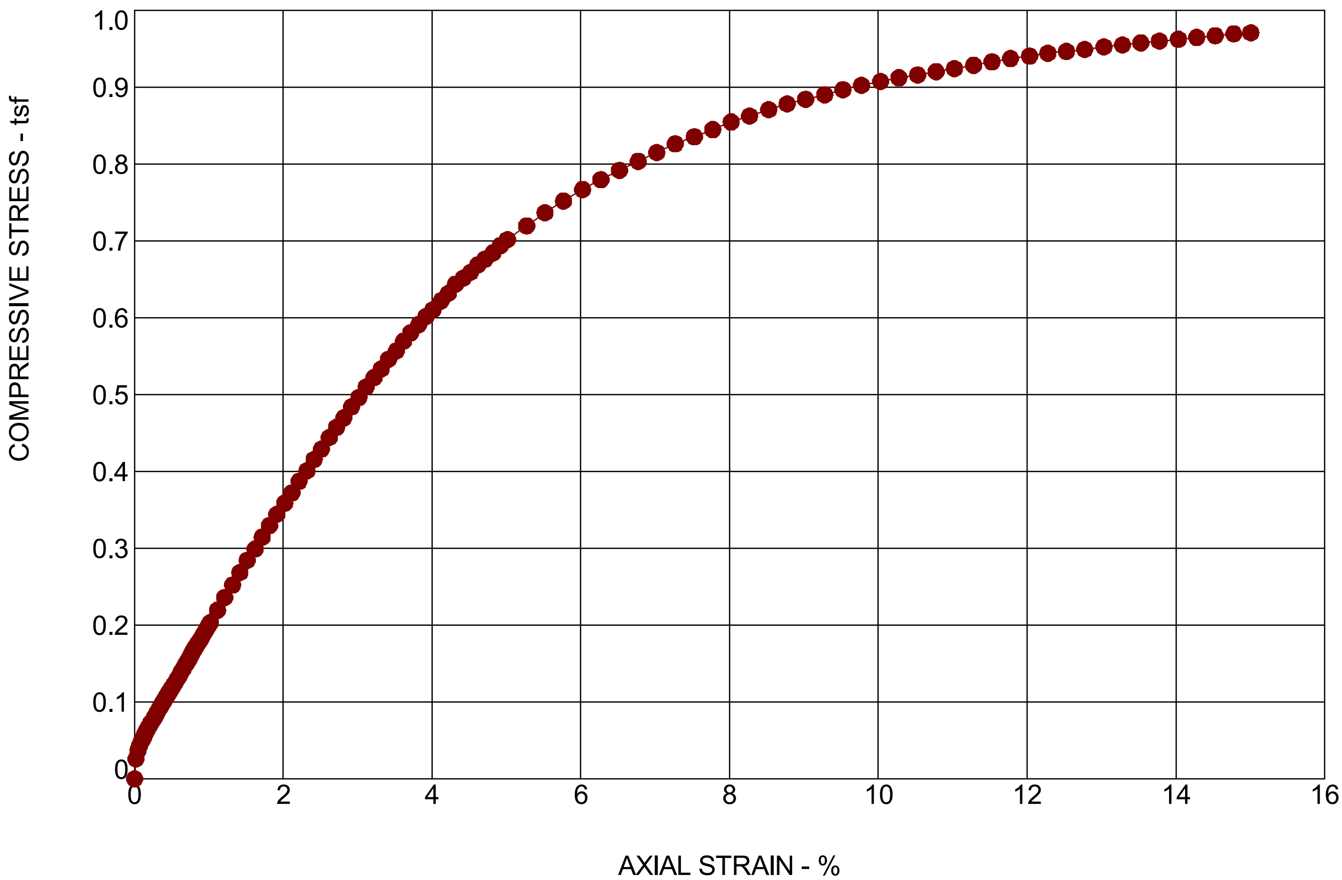
Terracon
5075 Commercial Cir Ste E
Concord, CA

PROJECT NUMBER: ND205079

CLIENT: Burns & McDonnell Engineering Company Inc
Columbus, OH

UNCONFINED COMPRESSION TEST

ASTM D2166



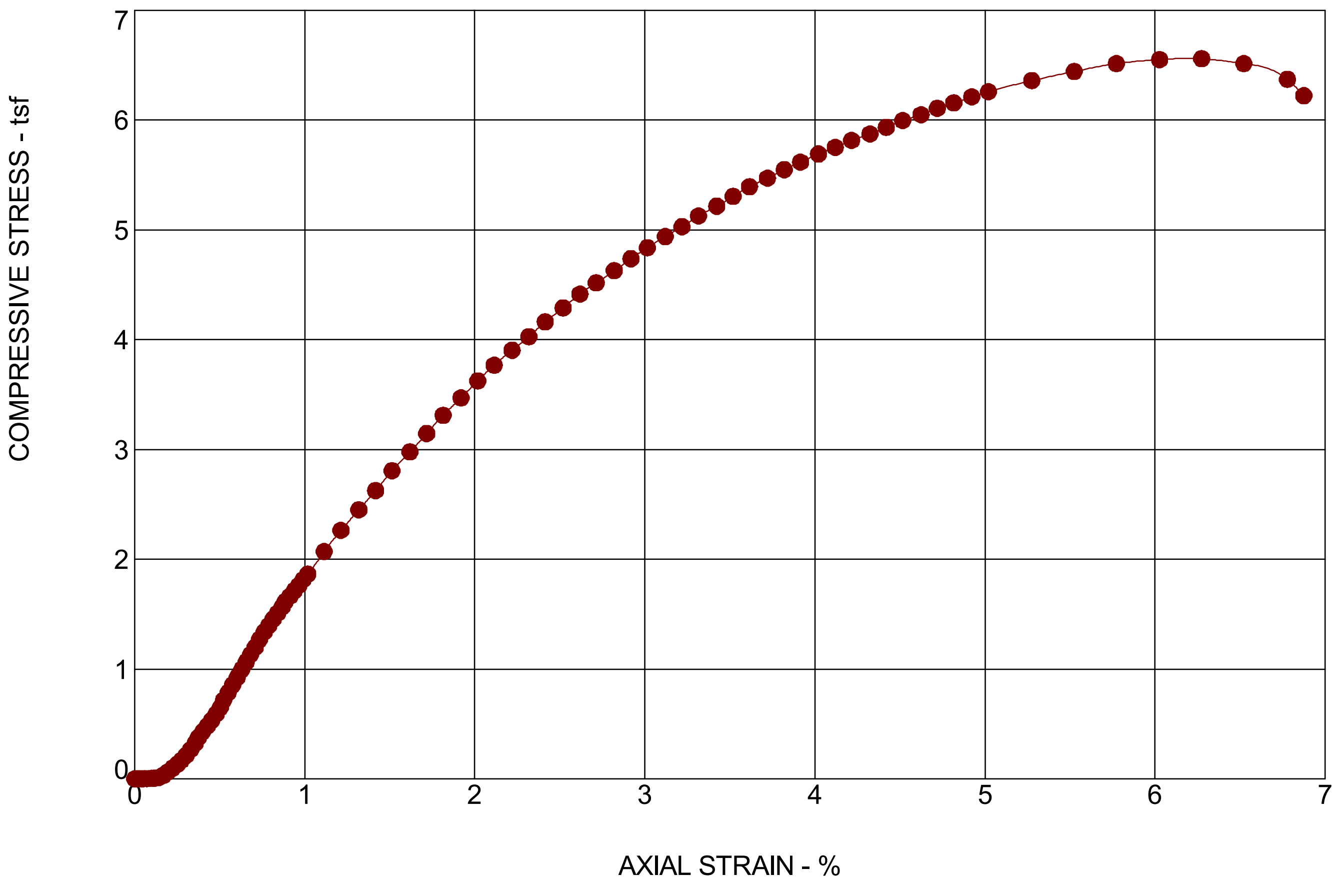
SPECIMEN FAILURE PHOTOGRAPH	SPECIMEN TEST DATA		
	Moisture Content:	%	32.9
	Dry Density:	pcf	91
	Diameter:	in.	2.40
	Height:	in.	4.14
	Height / Diameter Ratio:		1.72
	Calculated Saturation:	%	
	Calculated Void Ratio:		
	Assumed Specific Gravity:		
	Failure Strain:	%	15.00
	Unconfined Compressive Strength	(tsf)	0.97
	Undrained Shear Strength:	(tsf)	0.49
	Strain Rate:	in/min	
	Remarks:		

SAMPLE TYPE: CARS	SAMPLE LOCATION: B7 @ 20 - 21.5 feet			
DESCRIPTION: LEAN CLAY	LL	PL	PI	Percent < #200 Sieve

PROJECT: Bay AZ1 Site (Northtown)	Terracon 5075 Commercial Cir Ste E Concord, CA	PROJECT NUMBER: ND205079
SITE: Orchard Parkway San Jose, CA		CLIENT: Burns & McDonnell Engineering Company Inc Columbus, OH

UNCONFINED COMPRESSION TEST

ASTM D2166



SPECIMEN FAILURE PHOTOGRAPH	SPECIMEN TEST DATA		
	Moisture Content:	%	27.7
	Dry Density:	pcf	93
	Diameter:	in.	2.37
	Height:	in.	4.36
	Height / Diameter Ratio:		1.84
	Calculated Saturation:	%	
	Calculated Void Ratio:		
	Assumed Specific Gravity:		
	Failure Strain:	%	6.27
	Unconfined Compressive Strength	(tsf)	6.56
	Undrained Shear Strength:	(tsf)	3.28
	Strain Rate:	in/min	
	Remarks:		

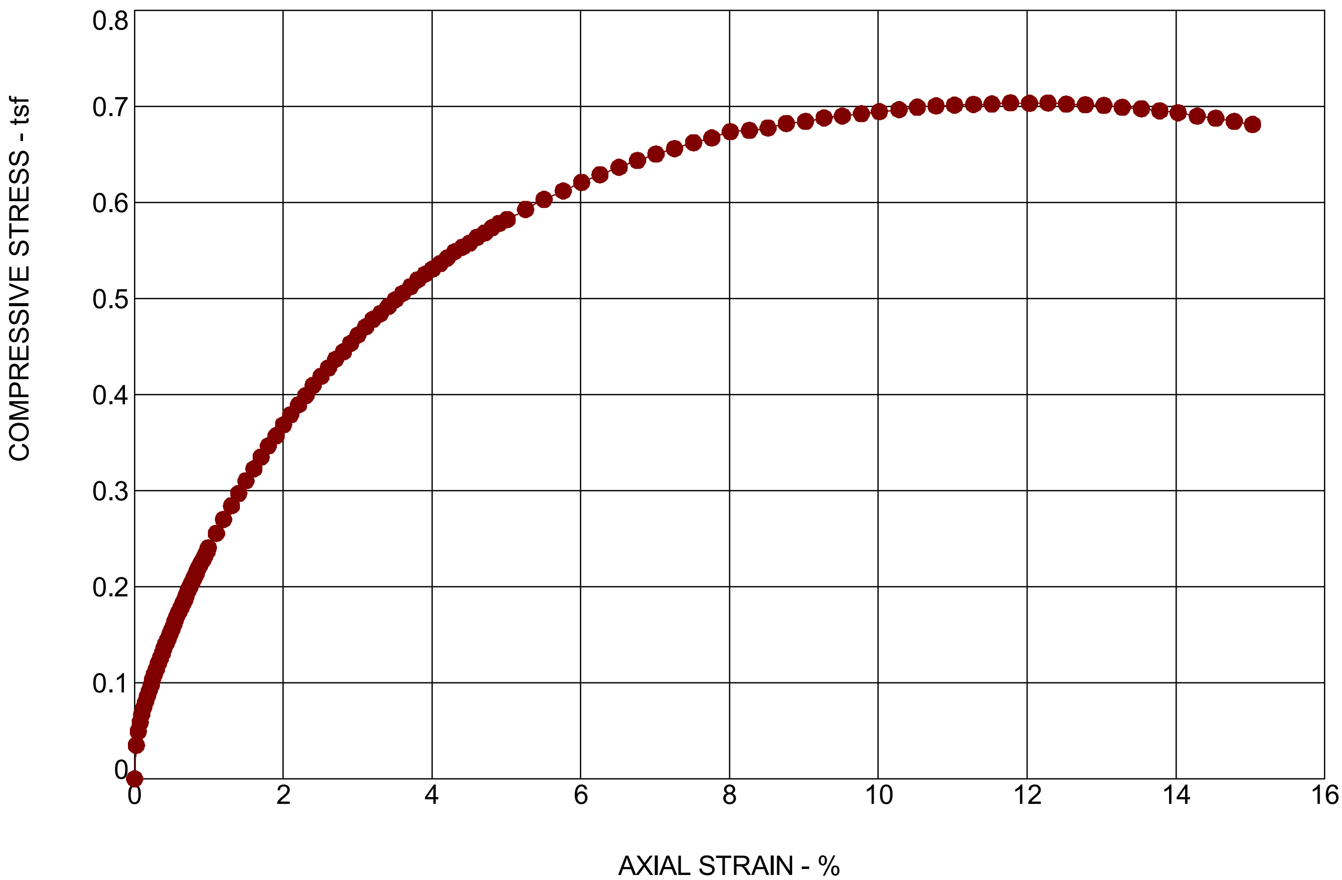
SAMPLE TYPE: CARS	SAMPLE LOCATION: B8 @ 2.5 - 4 feet			
DESCRIPTION: LEAN CLAY with SAND(CL)	LL	PL	PI	Percent < #200 Sieve
	49	23	26	80

PROJECT: Bay AZ1 Site (Northtown)	Terracon 5075 Commercial Cir Ste E Concord, CA	PROJECT NUMBER: ND205079
SITE: Orchard Parkway San Jose, CA		CLIENT: Burns & McDonnell Engineering Company Inc Columbus, OH

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. UNCONFINED WITH PHOTOS ND205079 BAY AZ1 SITE (NOR.GPJ TERRACON_DATATEMPLATE.GDT 11/30/20

UNCONFINED COMPRESSION TEST

ASTM D2166



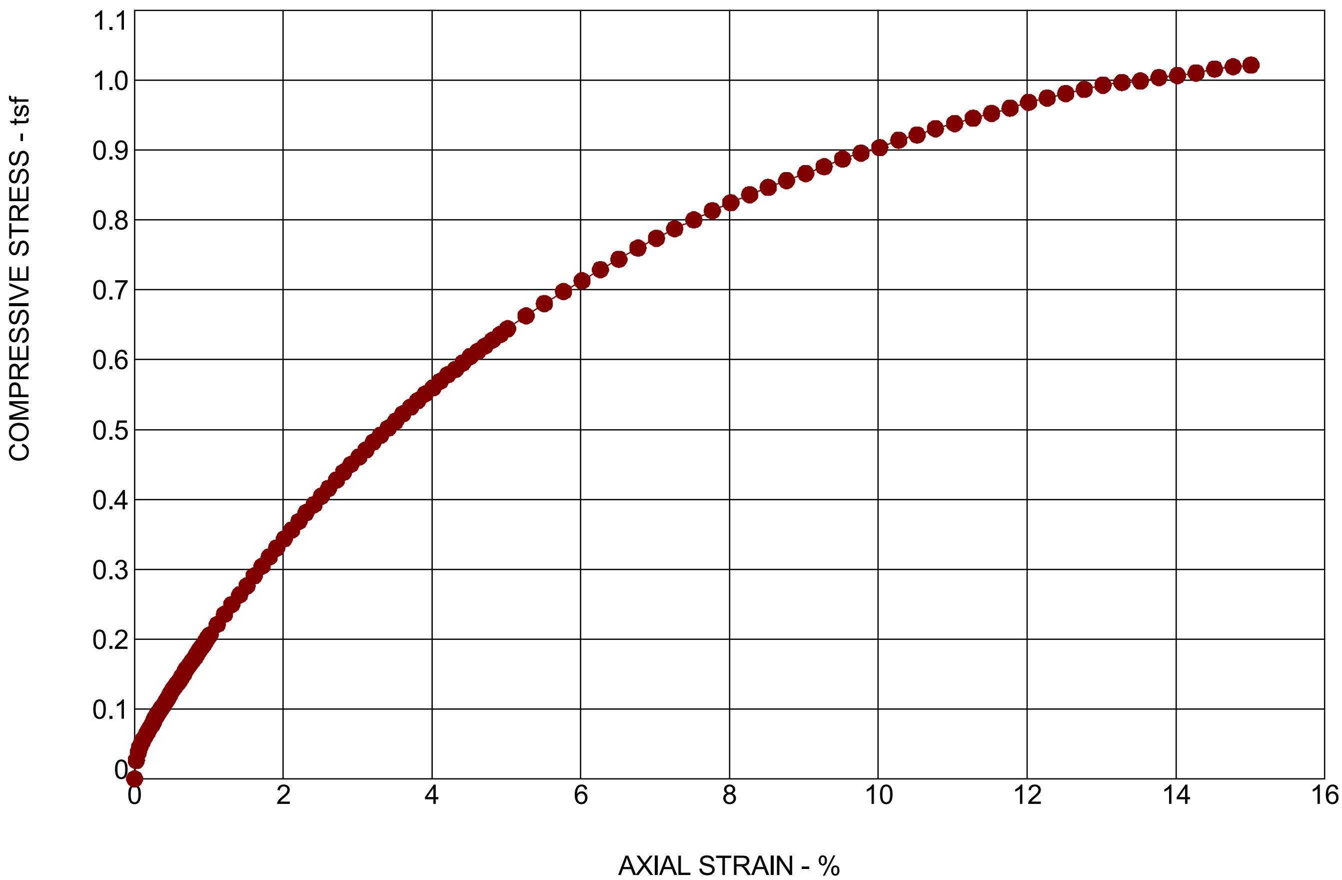
SPECIMEN FAILURE PHOTOGRAPH	SPECIMEN TEST DATA		
	Moisture Content:	%	26.4
	Dry Density:	pcf	99
	Diameter:	in.	2.36
	Height:	in.	4.18
	Height / Diameter Ratio:		1.77
	Calculated Saturation:	%	
	Calculated Void Ratio:		
	Assumed Specific Gravity:		
	Failure Strain:	%	11.78
	Unconfined Compressive Strength	(tsf)	0.70
	Undrained Shear Strength:	(tsf)	0.35
	Strain Rate:	in/min	
	Remarks:		

SAMPLE TYPE: CARS	SAMPLE LOCATION: B8 @ 15 - 16.5 feet			
DESCRIPTION: LEAN CLAY	LL	PL	PI	Percent < #200 Sieve

PROJECT: Bay AZ1 Site (Northtown)	Terracon 5075 Commercial Cir Ste E Concord, CA	PROJECT NUMBER: ND205079
SITE: Orchard Parkway San Jose, CA		CLIENT: Burns & McDonnell Engineering Company Inc Columbus, OH

UNCONFINED COMPRESSION TEST

ASTM D2166



SPECIMEN FAILURE PHOTOGRAPH	SPECIMEN TEST DATA		
	Moisture Content:	%	27.3
	Dry Density:	pcf	104
	Diameter:	in.	2.33
	Height:	in.	4.38
	Height / Diameter Ratio:		1.88
	Calculated Saturation:	%	
	Calculated Void Ratio:		
	Assumed Specific Gravity:		
	Failure Strain:	%	15.00
	Unconfined Compressive Strength	(tsf)	1.02
	Undrained Shear Strength:	(tsf)	0.51
	Strain Rate:	in/min	
	Remarks:		

SAMPLE TYPE: CARS	SAMPLE LOCATION: B9 @ 10 - 11.5 feet			
DESCRIPTION: LEAN CLAY	LL	PL	PI	Percent < #200 Sieve

PROJECT: Bay AZ1 Site (Northtown)

SITE: Orchard Parkway
San Jose, CA

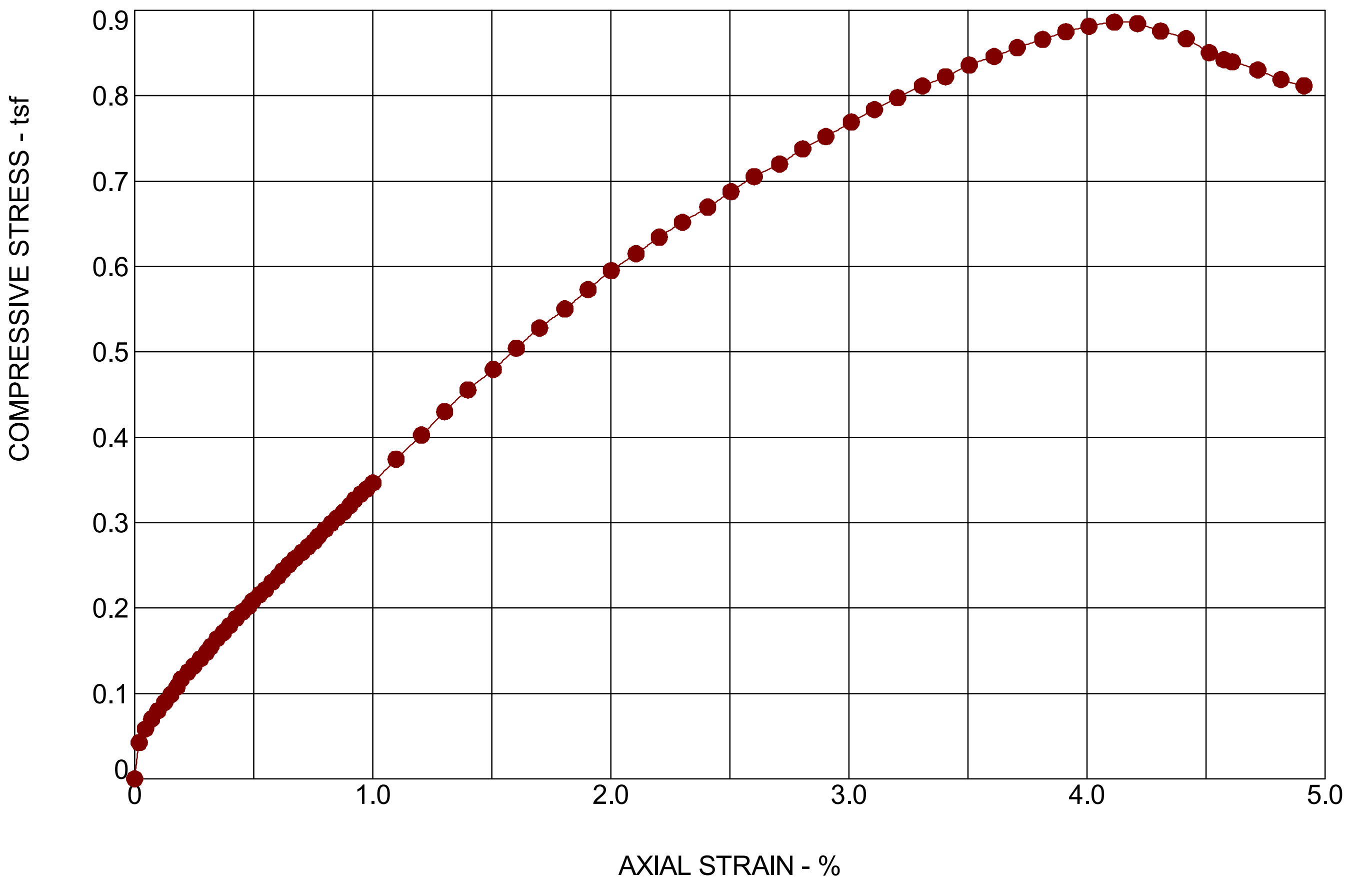
Terracon
5075 Commercial Cir Ste E
Concord, CA

PROJECT NUMBER: ND205079

CLIENT: Burns & McDonnell Engineering Company Inc
Columbus, OH

UNCONFINED COMPRESSION TEST

ASTM D2166



SPECIMEN FAILURE PHOTOGRAPH	SPECIMEN TEST DATA		
	Moisture Content:	%	19.4
	Dry Density:	pcf	112
	Diameter:	in.	2.36
	Height:	in.	4.22
	Height / Diameter Ratio:		1.79
	Calculated Saturation:	%	
	Calculated Void Ratio:		
	Assumed Specific Gravity:		
	Failure Strain:	%	4.11
	Unconfined Compressive Strength	(tsf)	0.89
	Undrained Shear Strength:	(tsf)	0.44
	Strain Rate:	in/min	
	Remarks:		

SAMPLE TYPE: CARS	SAMPLE LOCATION: B11 @ 7.5 - 9 feet			
DESCRIPTION: LEAN CLAY WITH SAND	LL	PL	PI	Percent < #200 Sieve

PROJECT: Bay AZ1 Site (Northtown)

SITE: Orchard Parkway
San Jose, CA

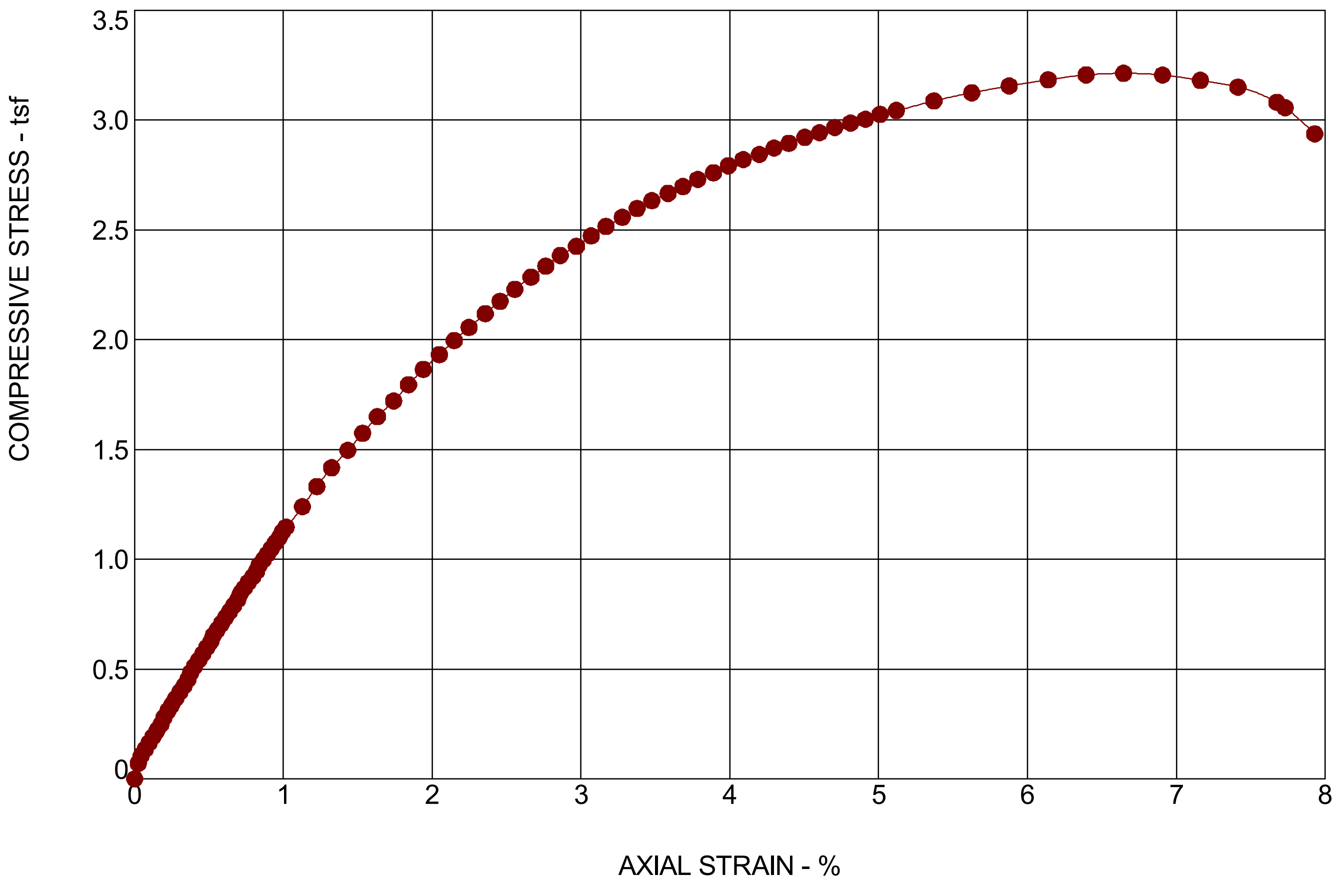
Terracon
5075 Commercial Cir Ste E
Concord, CA

PROJECT NUMBER: ND205079

CLIENT: Burns & McDonnell Engineering Company Inc
Columbus, OH

UNCONFINED COMPRESSION TEST

ASTM D2166



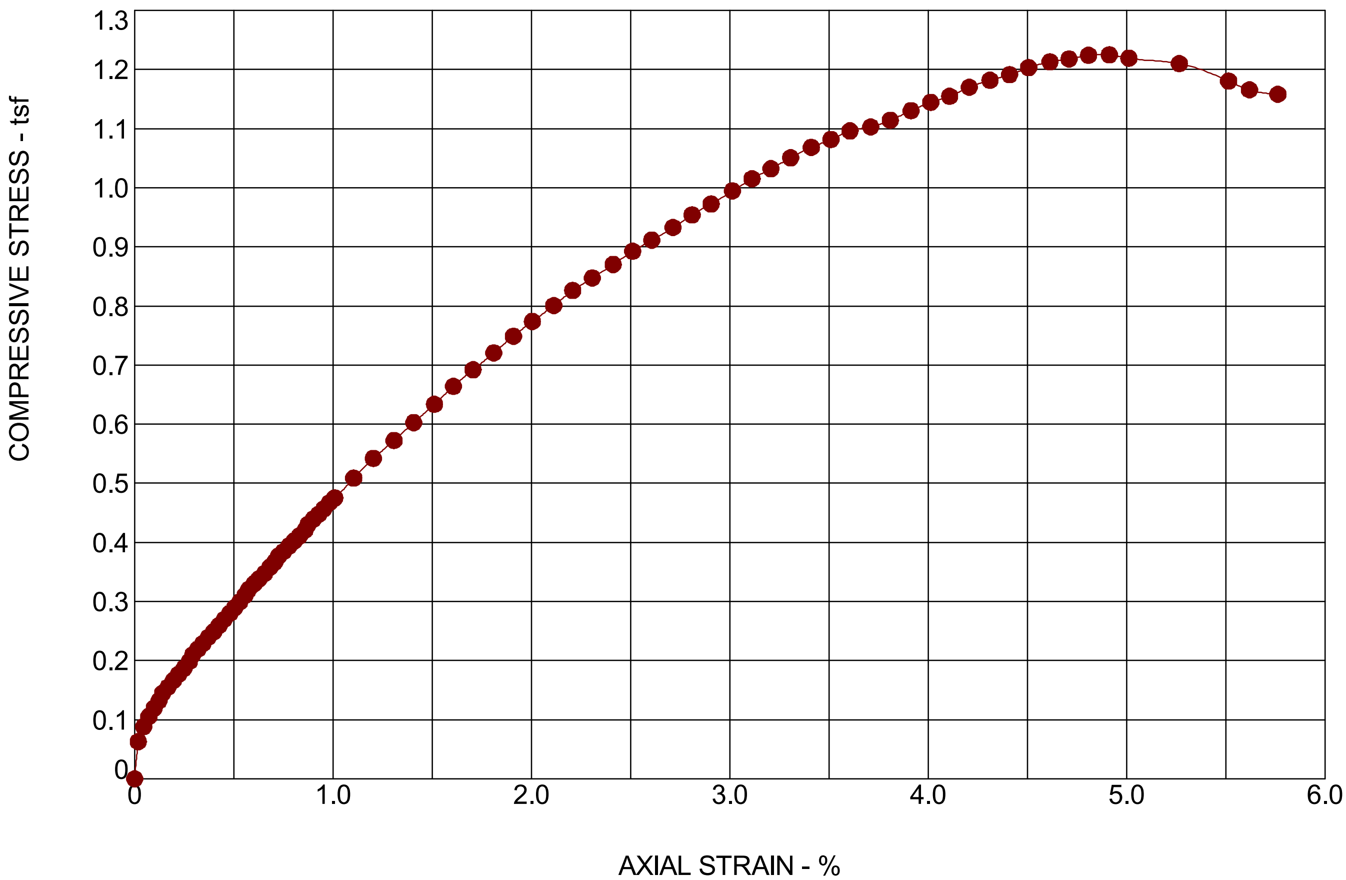
SPECIMEN FAILURE PHOTOGRAPH	SPECIMEN TEST DATA		
	Moisture Content:	%	23.1
	Dry Density:	pcf	99
	Diameter:	in.	2.38
	Height:	in.	4.13
	Height / Diameter Ratio:		1.73
	Calculated Saturation:	%	
	Calculated Void Ratio:		
	Assumed Specific Gravity:		
	Failure Strain:	%	6.65
	Unconfined Compressive Strength	(tsf)	3.21
	Undrained Shear Strength:	(tsf)	1.61
	Strain Rate:	in/min	
	Remarks:		

SAMPLE TYPE: CARS	SAMPLE LOCATION: B13 @ 5 - 6.5 feet			
DESCRIPTION: LEAN CLAY WITH SAND	LL	PL	PI	Percent < #200 Sieve

PROJECT: Bay AZ1 Site (Northtown)	Terracon 5075 Commercial Cir Ste E Concord, CA	PROJECT NUMBER: ND205079
SITE: Orchard Parkway San Jose, CA		CLIENT: Burns & McDonnell Engineering Company Inc Columbus, OH

UNCONFINED COMPRESSION TEST

ASTM D2166

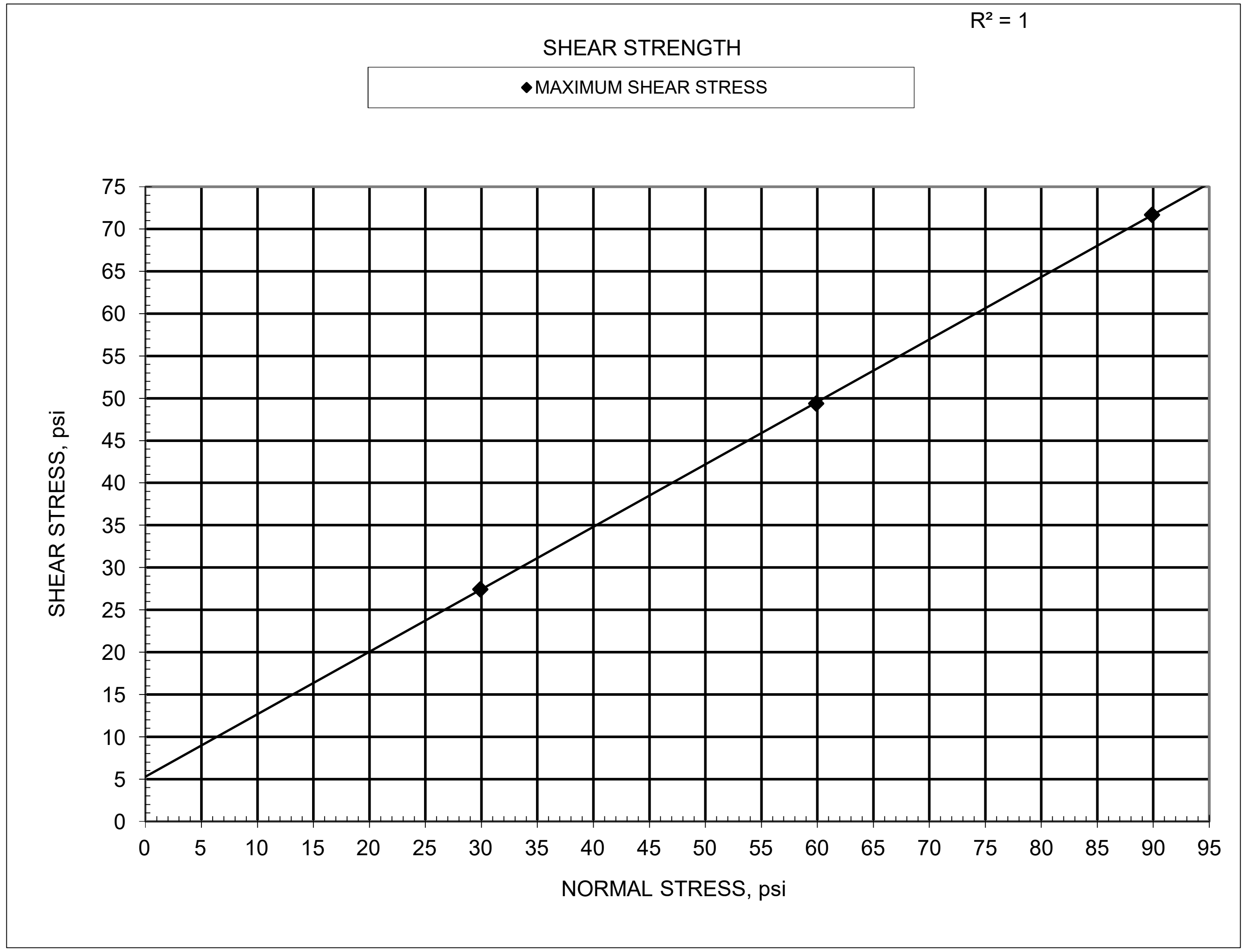


SPECIMEN FAILURE PHOTOGRAPH	SPECIMEN TEST DATA		
	Moisture Content:	%	23.3
	Dry Density:	pcf	95
	Diameter:	in.	2.37
	Height:	in.	3.34
	Height / Diameter Ratio:		1.41
	Calculated Saturation:	%	
	Calculated Void Ratio:		
	Assumed Specific Gravity:		
	Failure Strain:	%	4.91
	Unconfined Compressive Strength	(tsf)	1.22
	Undrained Shear Strength:	(tsf)	0.61
	Strain Rate:	in/min	
	Remarks:		

SAMPLE TYPE: CARS	SAMPLE LOCATION: B15 @ 15 - 16.5 feet			
DESCRIPTION: LEAN CLAY	LL	PL	PI	Percent < #200 Sieve

PROJECT: Bay AZ1 Site (Northtown)	Terracon 5075 Commercial Cir Ste E Concord, CA	PROJECT NUMBER: ND205079
SITE: Orchard Parkway San Jose, CA		CLIENT: Burns & McDonnell Engineering Company Inc Columbus, OH

DIRECT SHEAR TEST OF SOILS UNDER CONSOLIDATED DRAINED CONDITIONS
ASTM D3080



		FRICTION ANGLE		COHESION		NORMAL STRESS, psi	NORMAL STRESS, psi	NORMAL STRESS, psi
AT MAXIMUM SHEAR STRESS		36.4	deg	5.3	psi	29.91	59.90	89.89
INITIAL AREA, mm2	3166.9	INITIAL MOISTURE, %				16.6	18.1	19.5
INITIAL LENGTH, mm	25.15	INITIAL DRY DENSITY, pcf				111.7	109.1	109.0
SPECIFIC GRAVITY	2.70	INITIAL SATURATION, %				88	90	97
SG TESTED		INITIAL VOID RATIO				0.51	0.54	0.55
SG ASSUMED	X	FINAL MOISTURE, %				20.9	19.3	27.1
LIQUID LIMIT		FINAL SATURATION, %				126	102	122
PLASTIC LIMIT		FINAL VOID RATIO				0.45	0.51	0.60
PLASTICITY INDEX		MAXIMUM SHEAR STRESS, psi				27.41	49.37	71.68
SAMPLE TYPE	RECOMPACTED	RATE OF LOADING, in/min				0.0050	0.0050	0.0050
DESCRIPTION								

PROJECT NAME: Bay AZ1 Site (Northtown) BORING NO. B1

LOCATION: San Jose, CA SAMPLE NO. 7

JOB NO.: ND205079 DEPTH, feet 20 TO 21.5

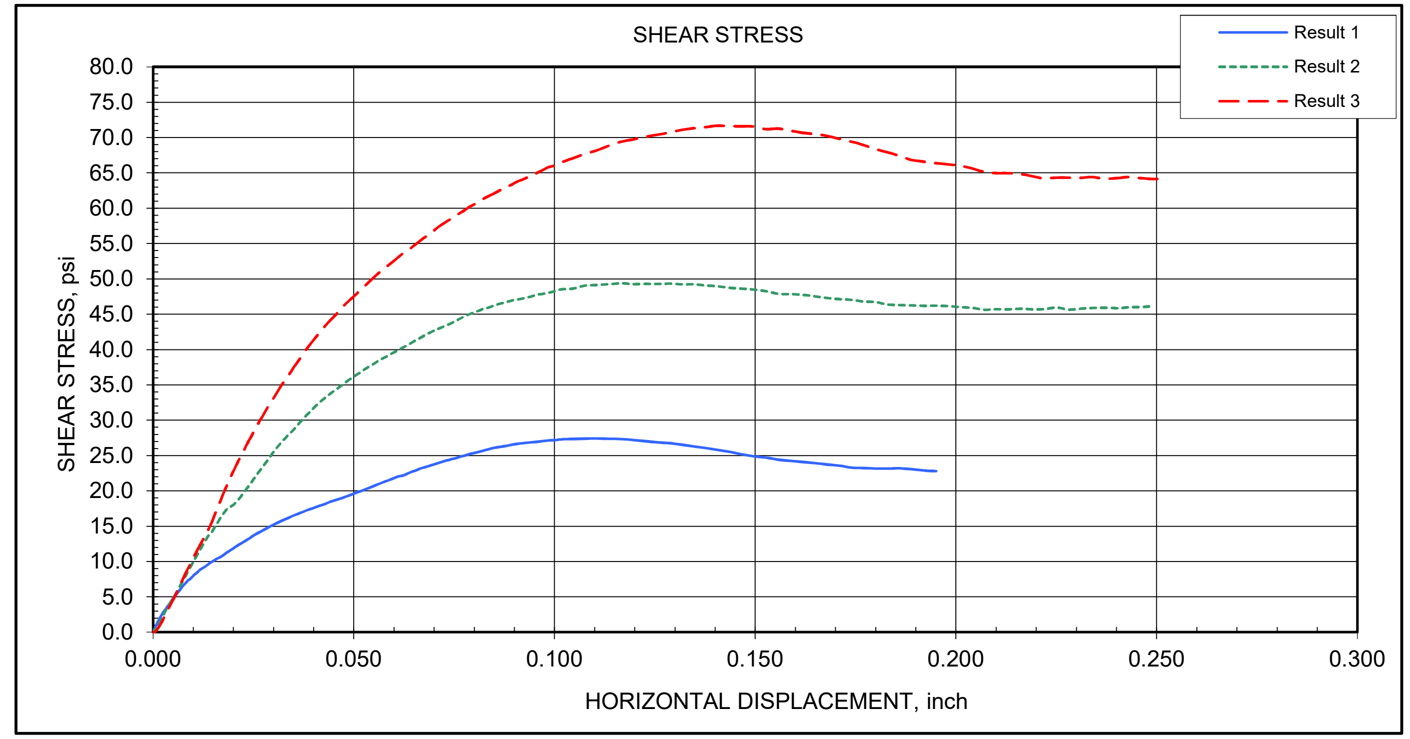
DATE: 11/10/2020

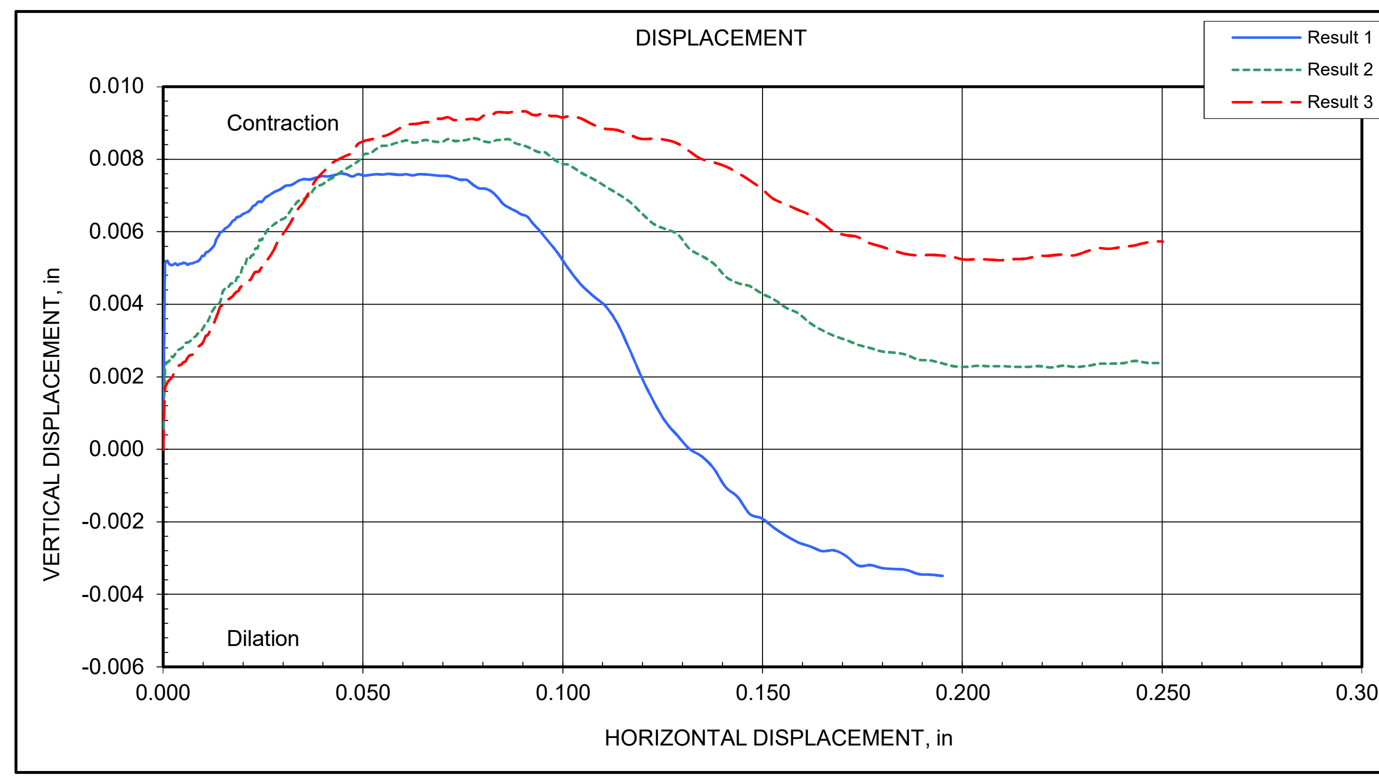
Terracon

Bay AZ1 Site (Northtown)

ND205079
11/10/2020

BORING NO. B1
SAMPLE NO. 7
DEPTH, feet 20 TO 21.5

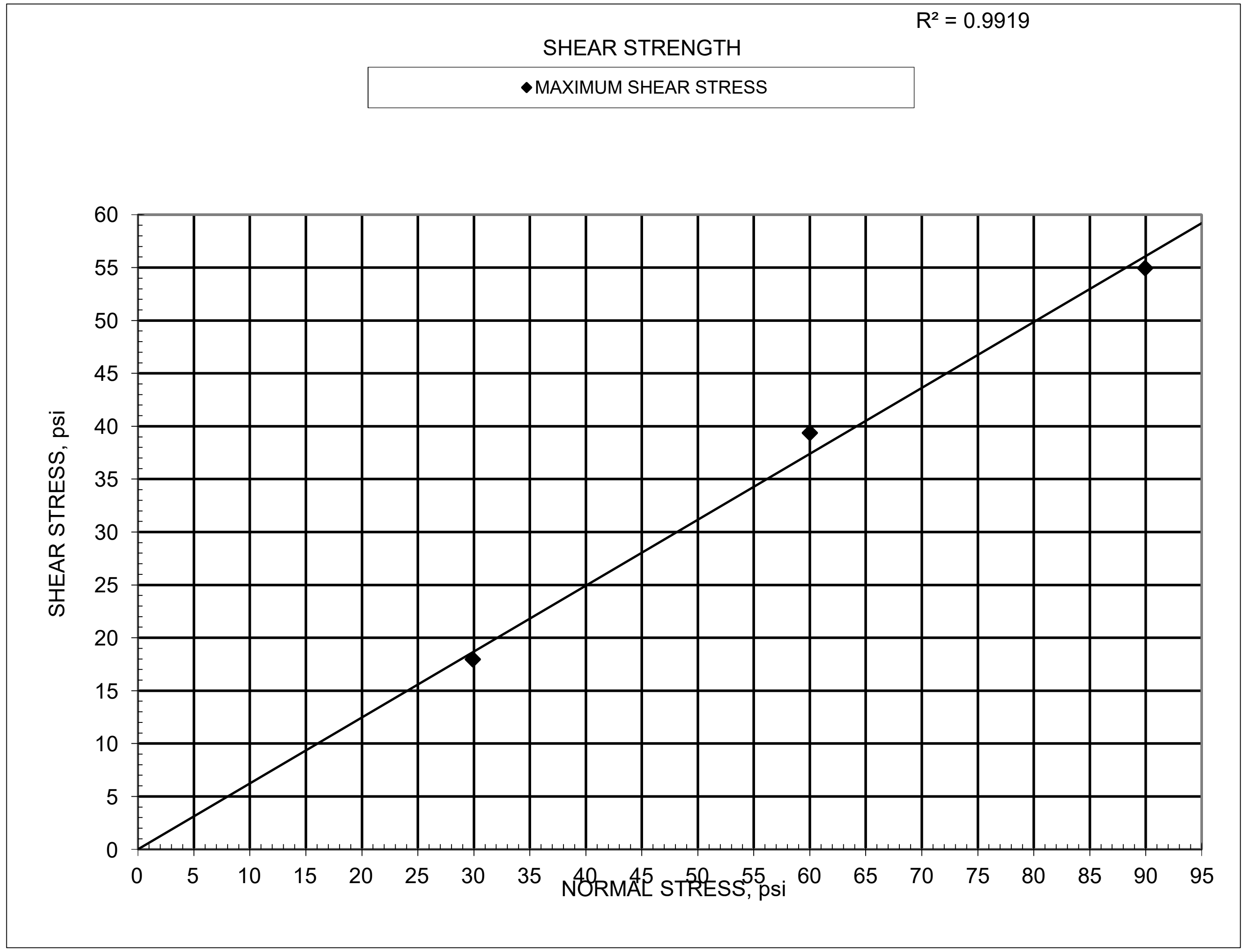




N:\Projects\2020\ND205079\Working Files\Laboratory-Field Data-Boring Logs\[Direct Shear Data_B1-7-1.xls]Report

Terracon

DIRECT SHEAR TEST OF SOILS UNDER CONSOLIDATED DRAINED CONDITIONS
ASTM D3080



		FRICTION ANGLE		COHESION		NORMAL STRESS, psi	NORMAL STRESS, psi	NORMAL STRESS, psi
AT MAXIMUM SHEAR STRESS		31.6	deg	0.5	psi	29.91	60.00	89.94
INITIAL AREA, mm2	3166.9	INITIAL MOISTURE, %				21.2	19.5	19.6
INITIAL LENGTH, mm	24.89	INITIAL DRY DENSITY, pcf				84.1	83.7	83.7
SPECIFIC GRAVITY	2.70	INITIAL SATURATION, %				57	52	52
SG TESTED		INITIAL VOID RATIO				1.00	1.01	1.01
SG ASSUMED	X	FINAL MOISTURE, %				22.1	20.4	20.9
LIQUID LIMIT		FINAL SATURATION, %				242	107	103
PLASTIC LIMIT		FINAL VOID RATIO				0.25	0.51	0.55
PLASTICITY INDEX		MAXIMUM SHEAR STRESS, psi				17.97	39.36	54.94
SAMPLE TYPE	RECOMPACTED	RATE OF LOADING, in/min				0.0030	0.0030	0.0030
DESCRIPTION								

PROJECT NAME: Bay AZ1 Site (Northtown)
BORING NO. B2-2-1

LOCATION: San Jose, CA
SAMPLE NO. 2

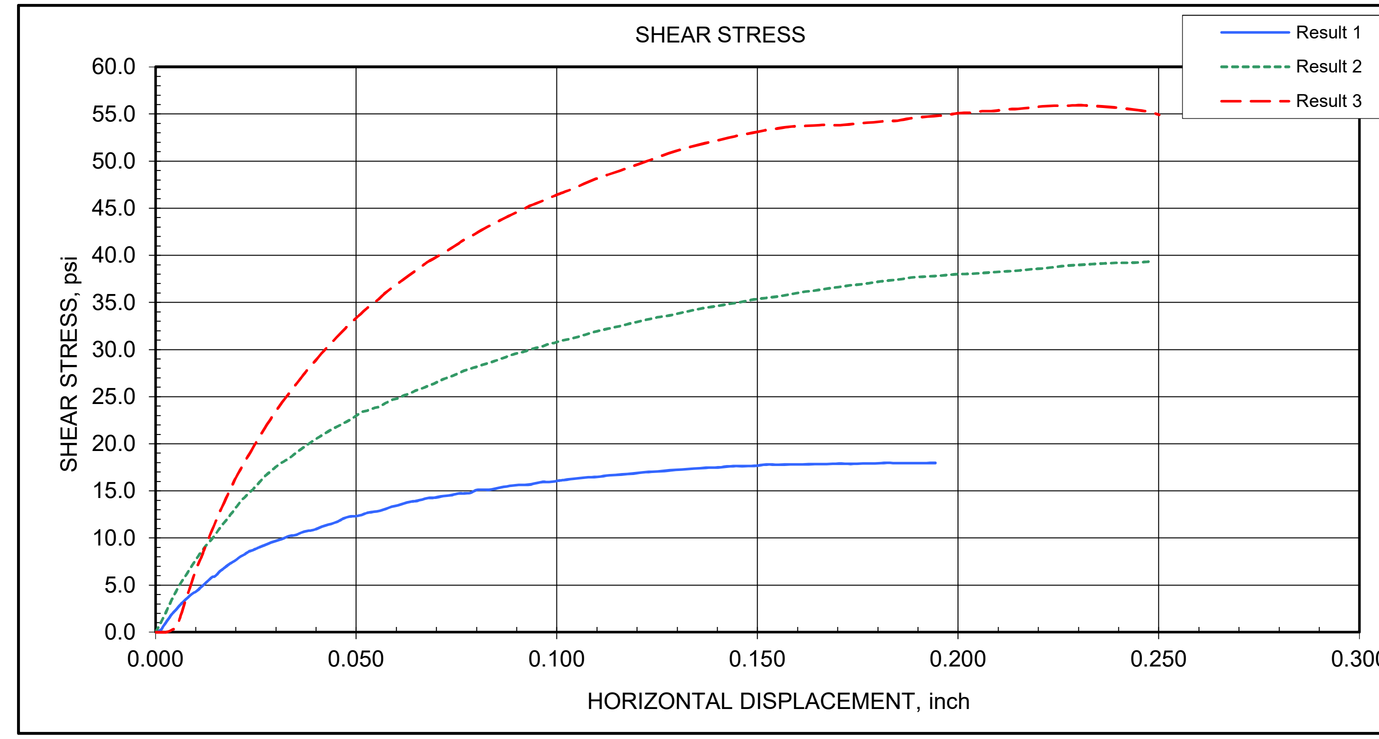
JOB NO.: ND205079
DEPTH, feet 2.5 TO 4

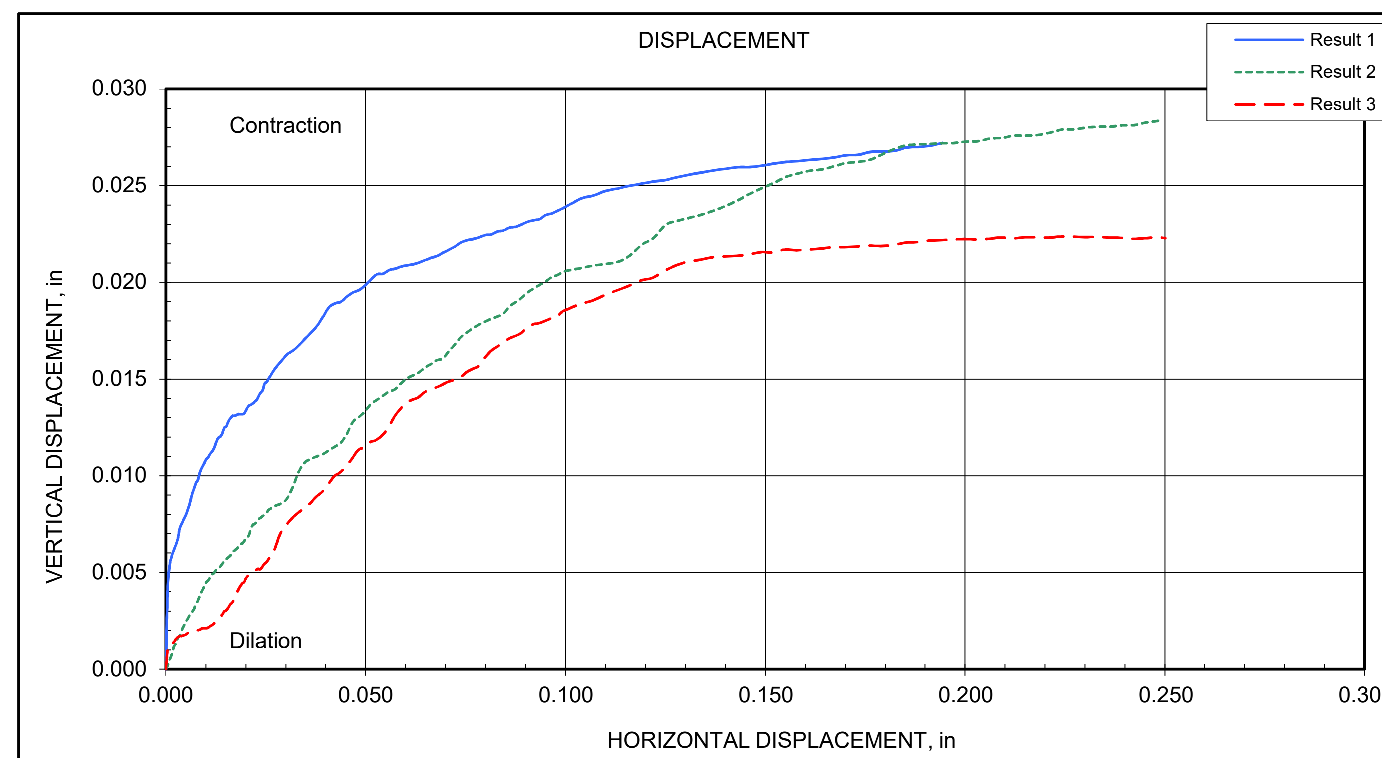
DATE: 11/12/2020

Bay AZ1 Site (Northtown)

ND205079
11/12/2020

BORING NO. B2-2-1
SAMPLE NO. 2
DEPTH, feet 2.5 TO 4





750 Pilot Road, Suite F
Las Vegas, Nevada 89119
(702) 597-9393

Terracon GeoReport

Client

Burns & McDonnell Engineering Company Inc
Colombus, OH

Project

Bay AZ1 Site (Northtown)

Sample Submitted By: Terracon (ND) **Date Received:** 11/10/2020 **Lab No.: 20-1214**

Results of Corrosion Analysis

Sample Number	1	7	3	2
Sample Location	B2	B3	B5	B6
Sample Depth (ft.)	1.0-2.5	20.0-21.5	5.0-6.5	2.5-4.0
pH Analysis, ASTM G 51	7.81	8.04	7.55	7.24
Water Soluble Sulfate (SO4), ASTM C 1580 (mg/kg)	133	182	122	103
Sulfides, AWWA 4500-S D, (mg/kg)	Nil	Nil	Nil	Nil
Chlorides, ASTM D 512, (mg/kg)	38	60	113	55
Red-Ox, ASTM G 200, (mV)	+686	+685	+686	+684
Total Salts, AWWA 2540, (mg/kg)	1131	1579	1971	1540
Resistivity (Saturated), ASTM G 57, (ohm-cm)	2959	1455	664	1067

Analyzed By: Trisha Campo

Trisha Campo
Chemist

750 Pilot Road, Suite F
Las Vegas, Nevada 89119
(702) 597-9393

Client

Burns & McDonnell Engineering Company Inc
Colombus, OH

Project

Bay AZ1 Site (Northtown)

Sample Submitted By: Terracon (ND) **Date Received:** 11/10/2020 **Lab No.: 20-1214**

Results of Corrosion Analysis

Sample Number	3	1
Sample Location	B13	B14
Sample Depth (ft.)	5.0-6.5	1.0-2.5
pH Analysis, ASTM G 51	7.44	7.76
Water Soluble Sulfate (SO4), ASTM C 1580 (mg/kg)	212	115
Sulfides, AWWA 4500-S D, (mg/kg)	Nil	Nil
Chlorides, ASTM D 512, (mg/kg)	45	53
Red-Ox, ASTM G 200, (mV)	+685	+687
Total Salts, AWWA 2540, (mg/kg)	1333	1355
Resistivity (Saturated), ASTM G 57, (ohm-cm)	1213	1164

Analyzed By: Trisha Campo

Trisha Campo
Chemist

21239 FM529 Rd., Bldg. F
Cypress, TX 77433
Tel: 281-985-9344
Fax: 832-427-1752
info@geothermusa.com
http://www.geothermusa.com

November 17, 2020

Terracon Consultants, Inc.
5075 Commercial Circle, Suite E
Concord, CA 94520
Attn: Hoda Alinasab

Re: Thermal Analysis of Native Soil Samples
Bay AZ1- San Jose, CA (Project No. ND205079)

The following is the report of thermal dryout characterization tests conducted on the eight (8) native soil samples from the referenced project sent to our laboratory.

Thermal Resistivity Tests: The samples were tested their 'optimum' moisture content and 90% of maximum dry density ***provided by Terracon***. The tests were conducted in accordance with the IEEE standard 442-2017. The results are tabulated below and the thermal dryout curves are presented in **Figures 1 to 8.**

Sample ID, Description, Thermal Resistivity, Moisture Content and Density

Sample ID	Depth (ft)	Description (Terracon)	Thermal Resistivity (°C-cm/W)		Moisture Content (%)	Dry Density (lb/ft^3)
			Wet	Dry		
B5	1'-2.5'	Sandy fat clay w/ gravel	85	206	17	94
B8	1'-2.5'	Fat clay	94	196	19	93
B9	1'-2.5'	Fat clay	83	189	18	99
B10	1'-2.5'	Fat clay w/ gravel	81	142	9	115
B11	1'-2.5'	Fat clay	88	186	15	101
P1	0'-2'	Fat clay	91	174	14	105
P5	0'-1'	Fat clay	96	189	14	102
I4	0'-5'	Fat clay	99	185	13	103

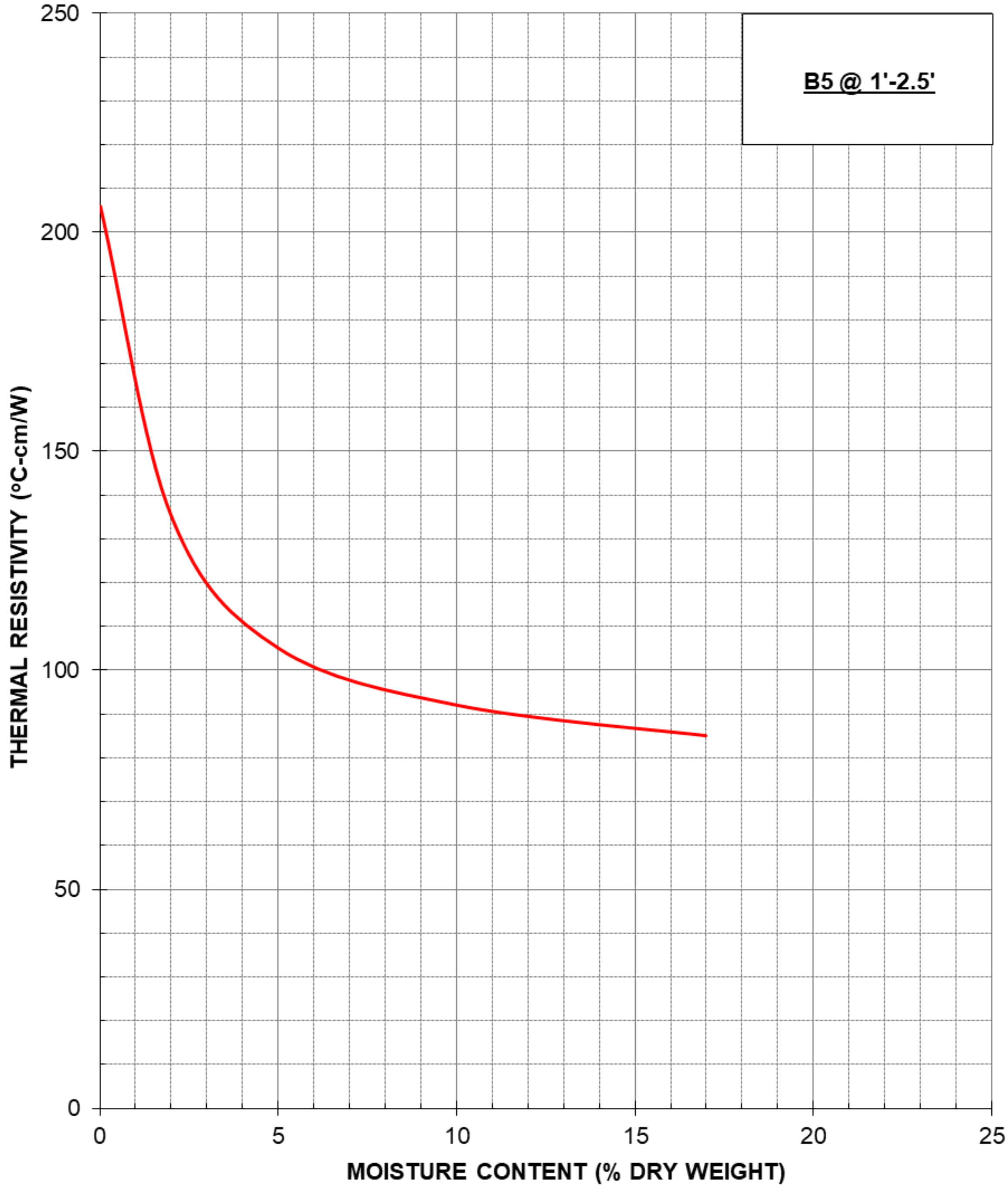
Comments: The thermal characteristic depicted in the dryout curves apply for the soils at their respective test dry density.

Please contact us if you have any questions or if we can be of further assistance.

Geotherm USA

Deepak Parmar

THERMAL DRYOUT CURVE

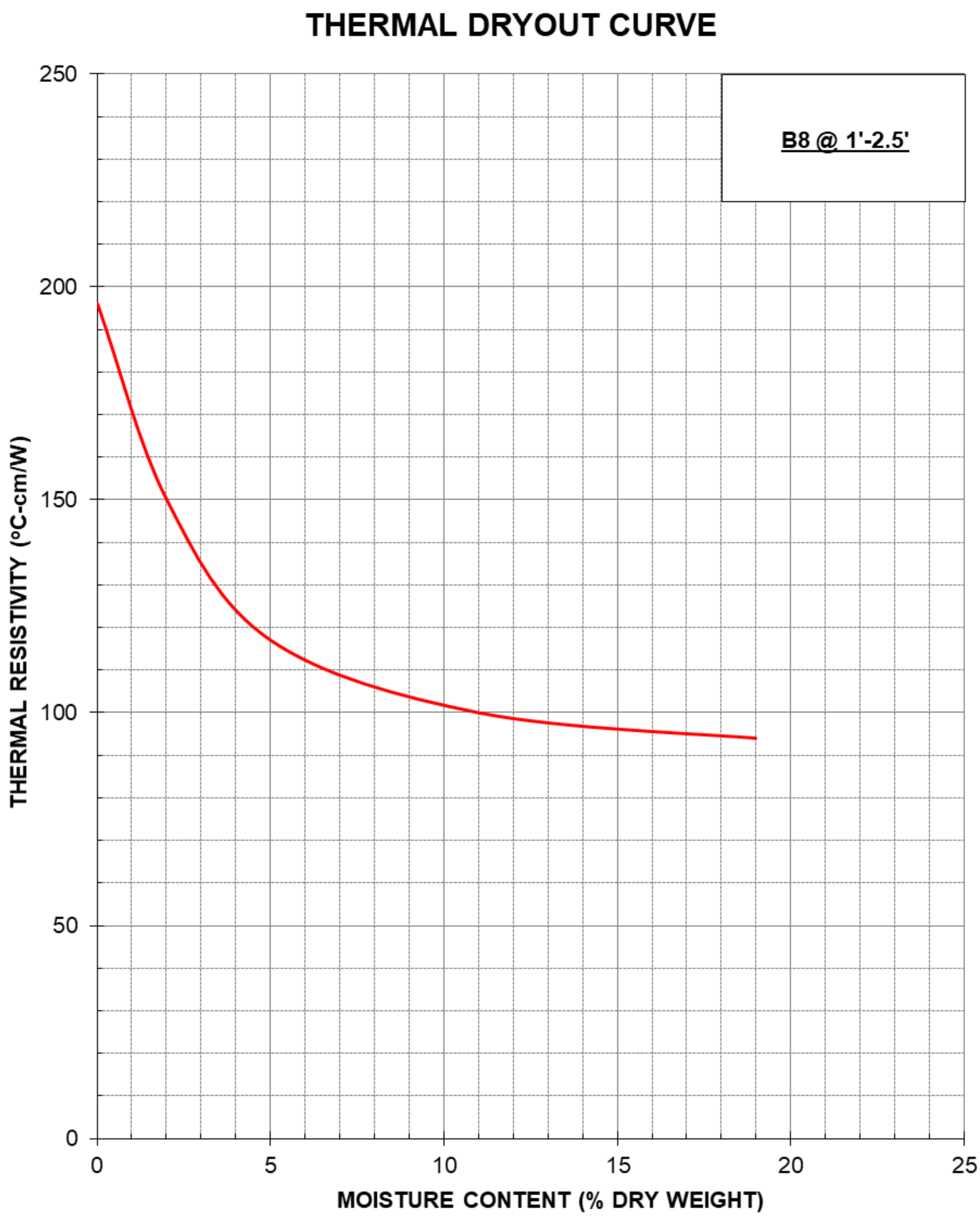


Terracon Consultants, Inc. (Project No. ND205079)

Thermal Analysis of Native Soils

Bay AZ1- San Jose, CA

November 2020 **Figure 1**

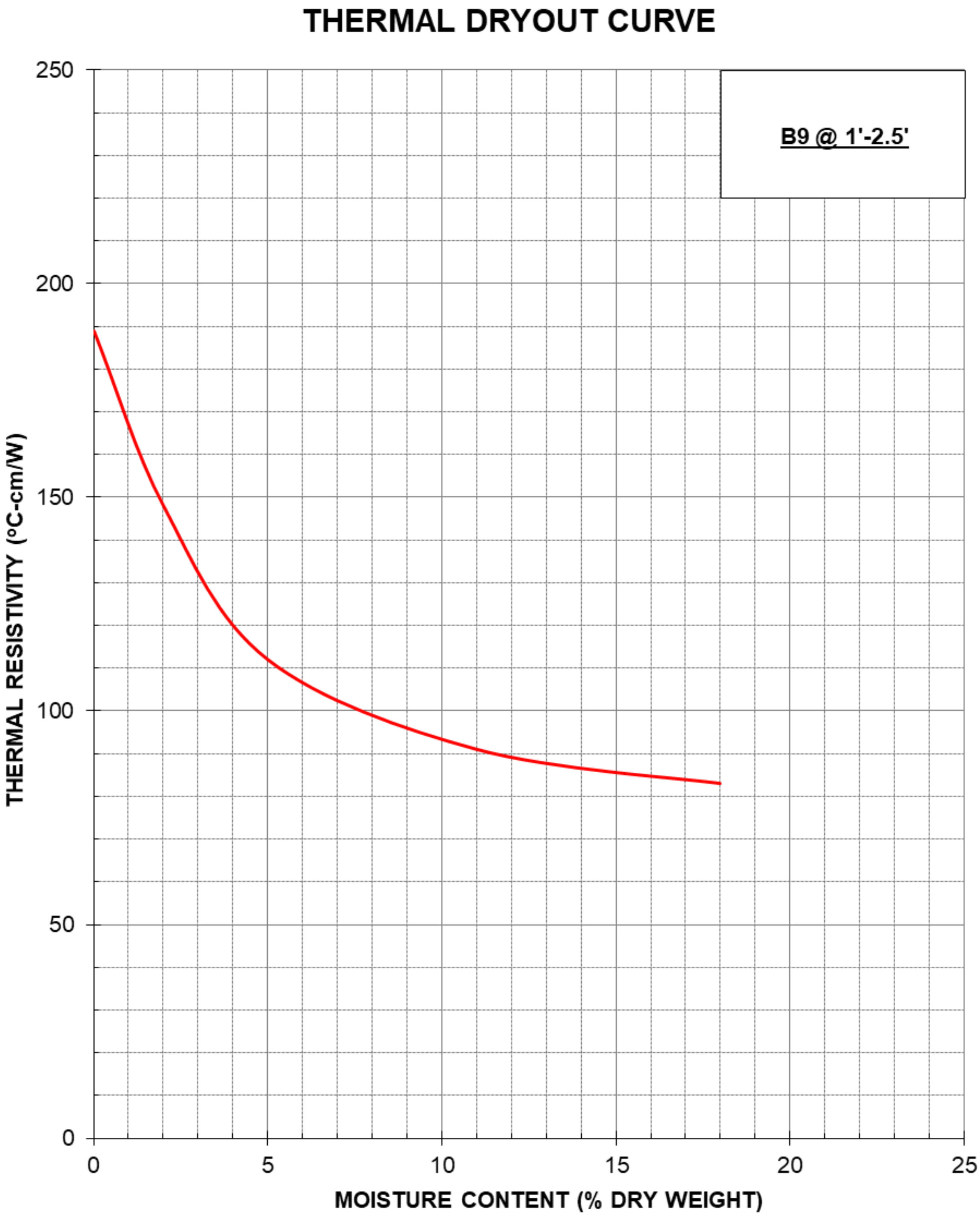


Terracon Consultants, Inc. (Project No. ND205079)

Thermal Analysis of Native Soils

Bay AZ1- San Jose, CA

November 2020 **Figure 2**



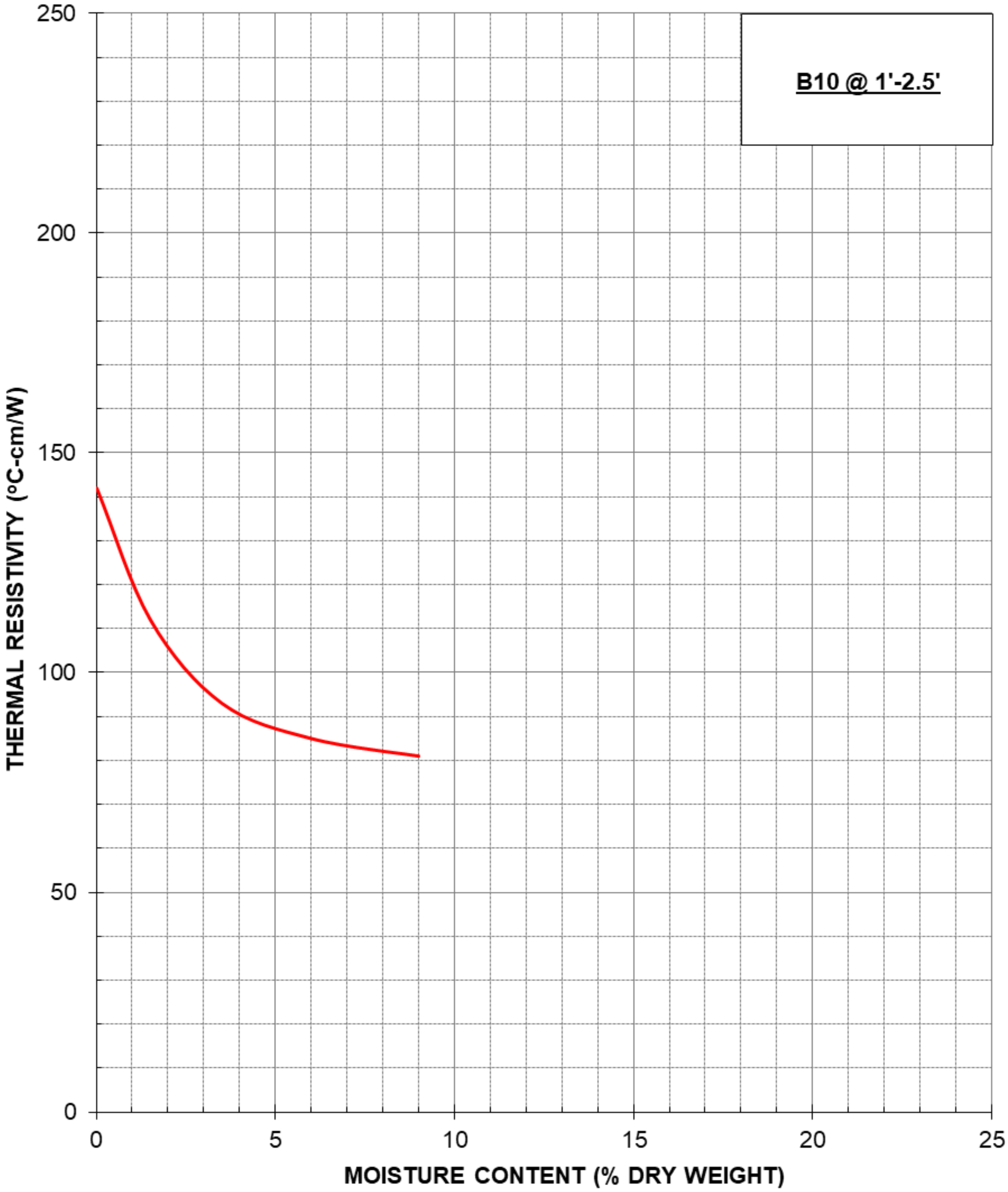
Terracon Consultants, Inc. (Project No. ND205079)

Thermal Analysis of Native Soils

Bay AZ1- San Jose, CA

November 2020 **Figure 3**

THERMAL DRYOUT CURVE

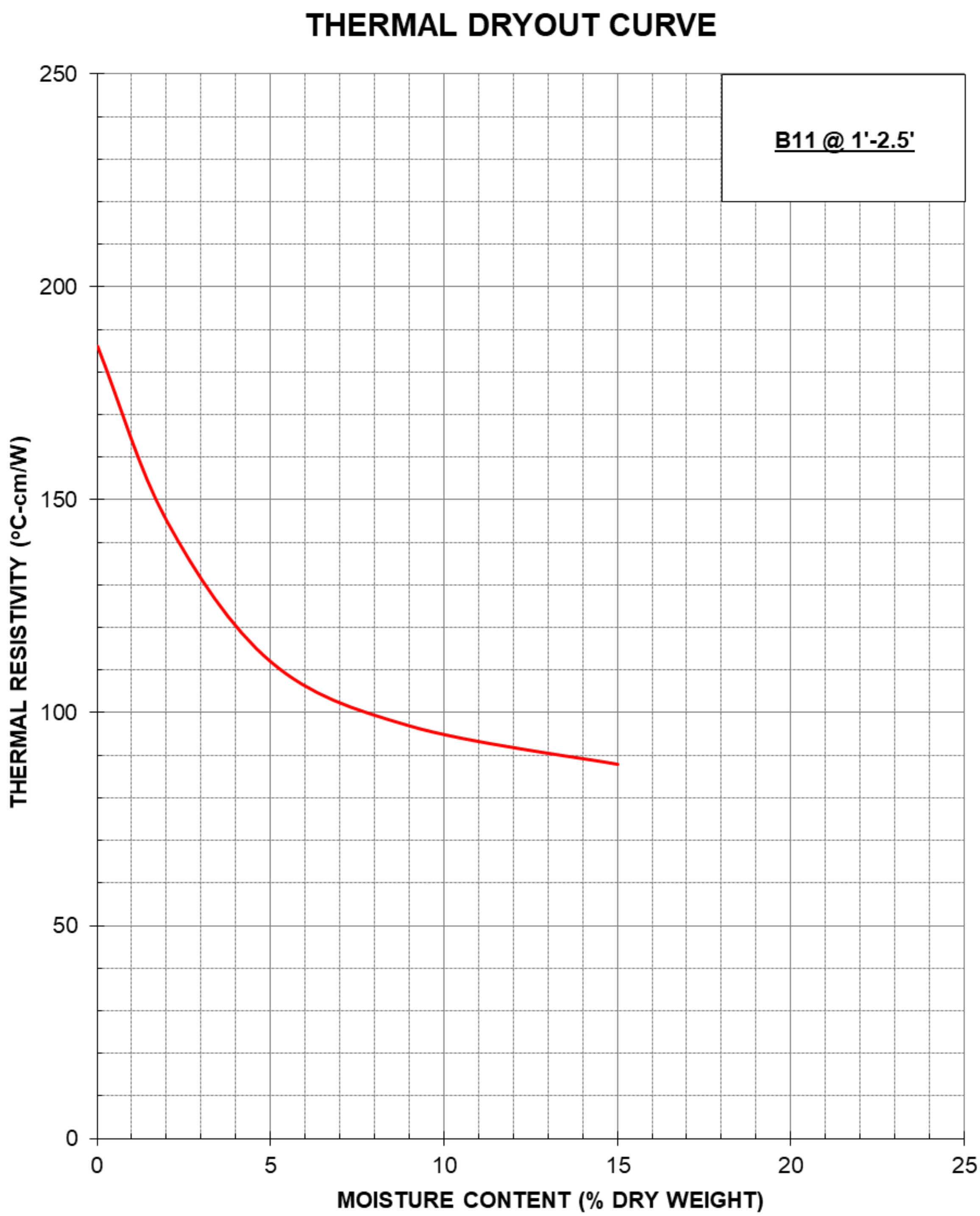


Terracon Consultants, Inc. (Project No. ND205079)

Thermal Analysis of Native Soils

Bay AZ1- San Jose, CA

November 2020 **Figure 4**

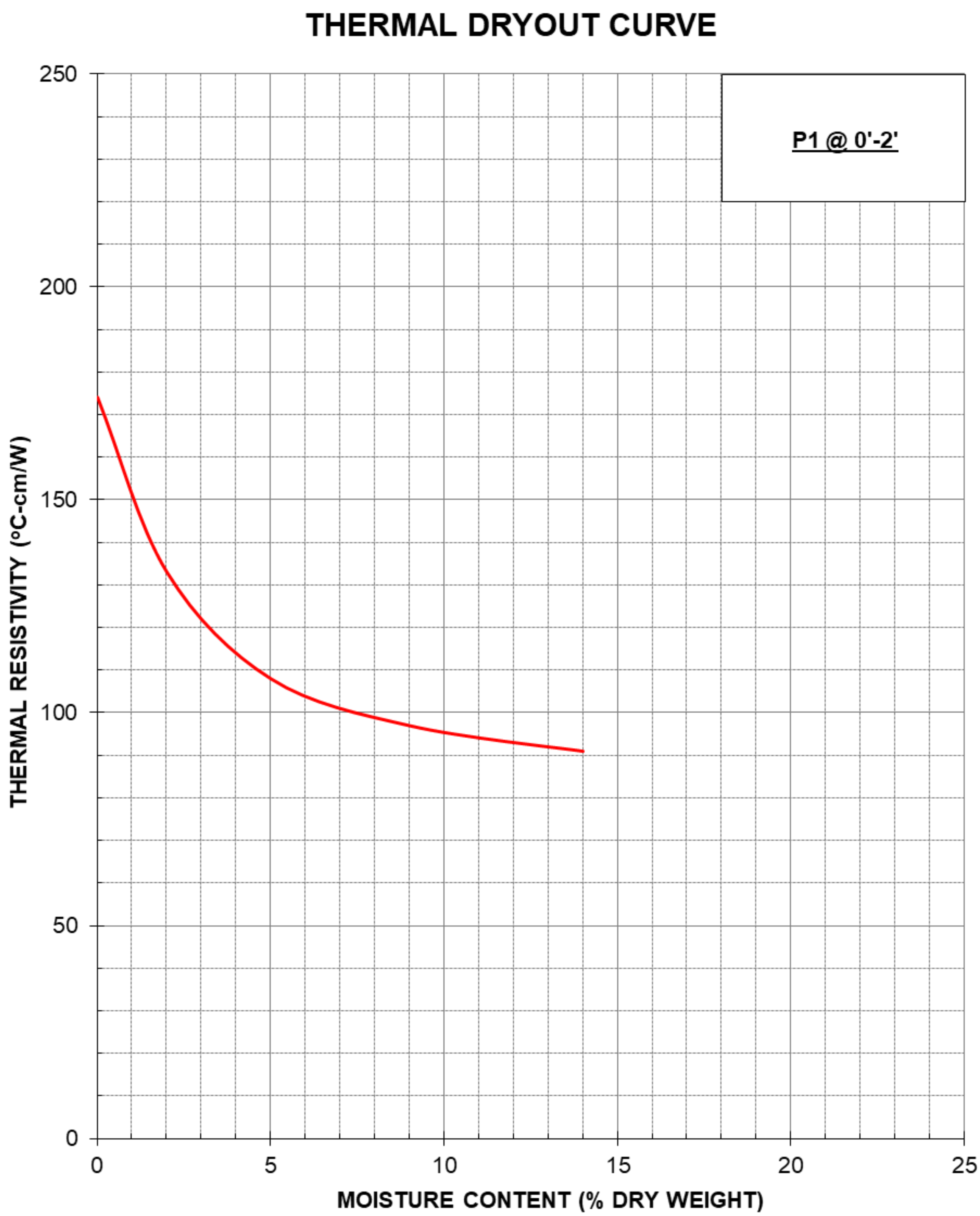


Terracon Consultants, Inc. (Project No. ND205079)

Thermal Analysis of Native Soils

Bay AZ1- San Jose, CA

November 2020 **Figure 5**

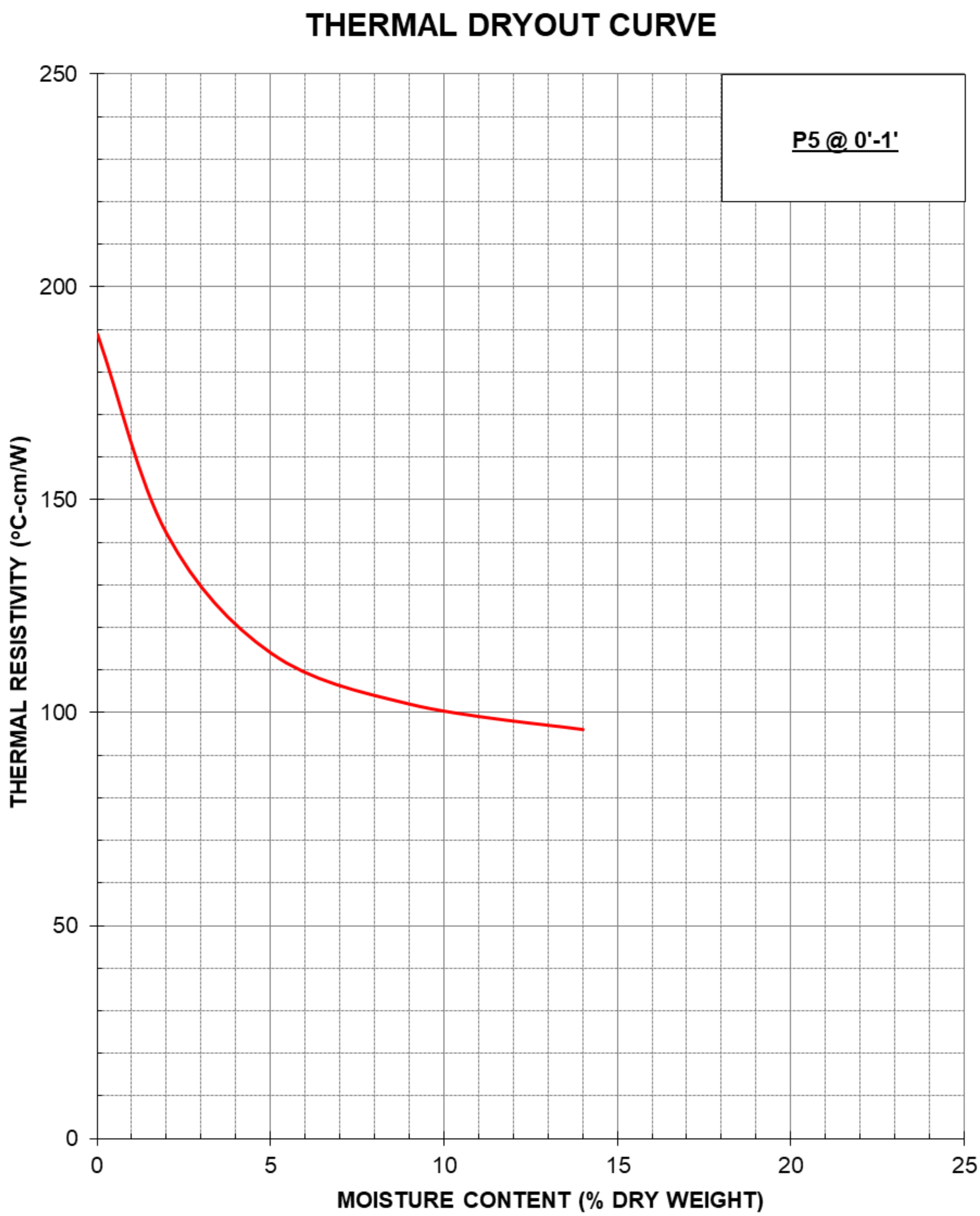


Terracon Consultants, Inc. (Project No. ND205079)

Thermal Analysis of Native Soils

Bay AZ1- San Jose, CA

November 2020 **Figure 6**



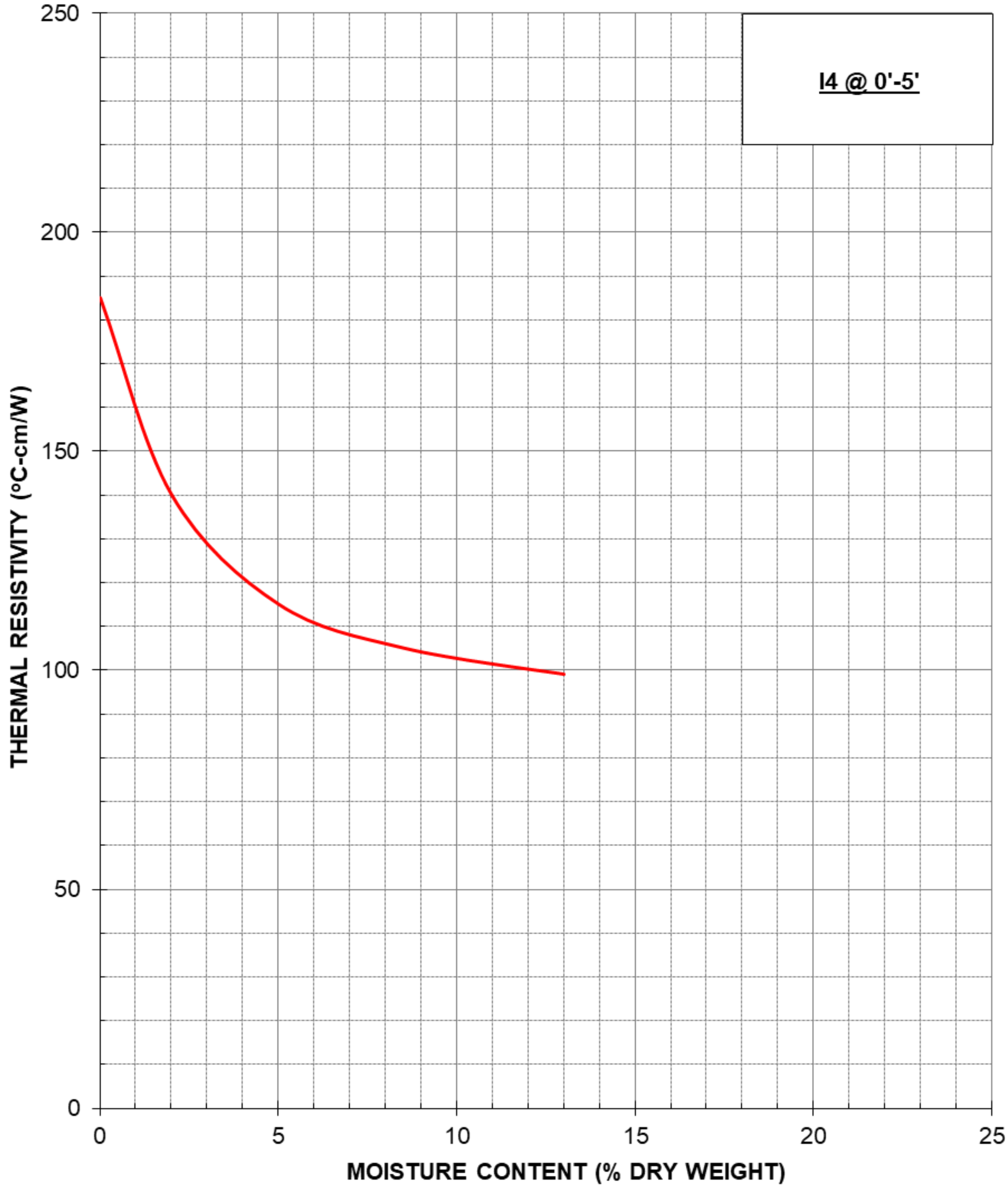
Terracon Consultants, Inc. (Project No. ND205079)

Thermal Analysis of Native Soils

Bay AZ1- San Jose, CA

November 2020 | **Figure 7**

THERMAL DRYOUT CURVE



Terracon Consultants, Inc. (Project No. ND205079)

Thermal Analysis of Native Soils

Bay AZ1- San Jose, CA

November 2020 **Figure 8**

Geophysical Report

Geophysical Survey
Bay AZ1
Orchard Parkway
San Jose, California

November 12, 2020
NORCAL JOB NO. NS205134

Prepared for:

BURNS McDONNELL

530 West Spring Street, Suite 200
Columbus, Ohio 43215

Prepared by:

NORCAL GEOPHYSICAL CONSULTANTS INC.
A Terracon COMPANY

NORCAL Geophysical Consultants, Inc. 321A Blodgett Street Cotati, California 94931
P (707) 796-7170 F (707) 796-7175 norcalgeophysical.com

November 12, 2020

BURNS & McDONNELL

530 West Spring Street, Suite 200
Columbus, Ohio 43215

Subject: Geophysical Survey
Bay AZ1
Orchard Parkway
San Jose, California

NORCAL Project No. NS205134

Attention: Ms. Brandi Sauter:

Dear Ms. Sauter:

This report presents the findings of a geophysical survey performed by NORCAL Geophysical Consultants, Inc. for Burns & McDonnell Engineering Company, Inc at your Bay AZ1 site on Orchard Parkway in San Jose, California. The work was authorized under a Terracon Inter-Office Service agreement for Terracon Concord Project No. ND205079. NORCAL Professional Geophysicist David T. Hagin (CA PGp No. 1033) and Senior Geophysical Technician Travis W. Black performed the survey on November 3, 2020.

The scope of NORCAL's services for this project consisted of using geophysical methods to characterize the subsurface. The accuracy of our findings is subject to specific site conditions and limitations inherent to the techniques used. We performed our services in a manner consistent with the standard of care ordinarily exercised by members of the profession currently employing similar methods. No warranty, with respect to the performance of services or products delivered under this agreement, expressed or implied, is made by NORCAL.

We appreciate having the opportunity to provide our services for this project. If you have any questions or require additional geophysical services, please do not hesitate to call on us.

Sincerely,

NORCAL Geophysical Consultants, Inc.

David T. Hagin
California Professional Geophysicist
PGp 1033

PROFESSIONAL GEOPHYSICIST
DAVID T HAGIN
11-12-2020
No. 1033
STATE OF CALIFORNIA

Donald J. Kirker
California Professional Geophysicist
PGp No. 997

PROFESSIONAL GEOPHYSICIST
DONALD KIRKER
No. 997
STATE OF CALIFORNIA

Geophysical Report

Geophysical Survey
Bay AZ1 - Orchard Parkway
San Jose, California

November 12, 2020

1.0 INTRODUCTION

This report presents the results of a geophysical survey performed to aid in planning and design for proposed improvements at the Bay AZ1 site in San Jose, California. The survey consists of two parts:

- Seismic Multichannel Analysis of Surface Waves (MASW) survey, to measure the shear-wave velocities of the subsurface as a function of depth. The survey method is a sounding, producing one-dimensional (1-D) data that is presented in tabular form. Descriptions of the MASW methodology, our data acquisition and analysis procedures, and the instrumentation we used for the MASW method are provided in **Appendix A – MASW Survey**.

- Vertical Electrical Sounding (VES) survey, using the Wenner 4-Pin method, to measure the Electrical Resistivity (ER) of the shallow sub-surface. The survey results are presented in tabular form. Descriptions of the VES methodology, our data acquisition and analysis procedures, and the instrumentation we used are provided in **Appendix B – VES Survey**.

Figure 1 shows an aerial photograph of the site, indicating the locations of the one (1) MASW sounding (yellow line), designated as MASW-1, and the four (4) VES, designated as FR1 through FR4 (white crosses). Each VES consists of data measured by two perpendicular electrode arrays with a common center point. The purpose of measuring along perpendicular arrays is to account for possible anisotropy in the electrical properties of the sub-surface.

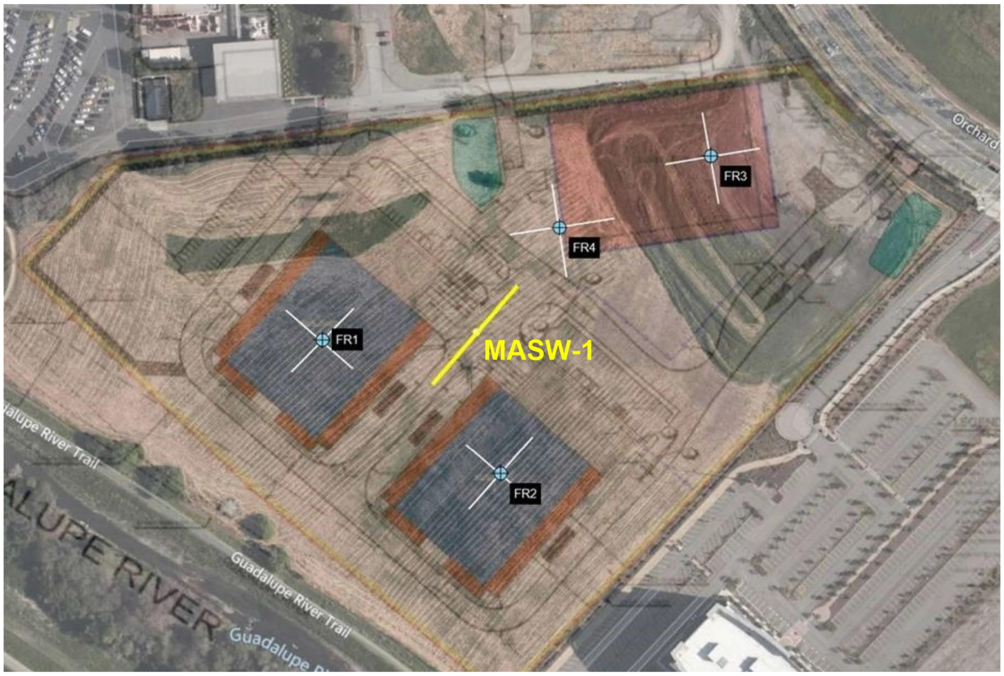


Figure 1: *Site map showing MASW-1 and VES-FR1 through FR4.*

2.0 SITE CONDITIONS

The following description of site conditions is derived from our site visit and our review of publicly available geologic and topographic maps.

Item	Description
Site information	The Bay AZ1 site is located to the northwest of the intersection of Orchard Parkway and Component Drive. The approximate coordinates of the MASW sounding are: MASW-1 (37.378442º N, 121.932817º W). The VES coordinates are approximately: VES-FR1 (37.378411º N, 121.933637º W), VES-FR2 (37.377845º N, 121.932691º W), VES-FR3 (37.379190º N, 121.931576º W), VES-FR4 (37.378887º N, 121.932379º W),
Existing improvements	The site is currently undeveloped and has no existing improvements, aside from fencing around the perimeter.
Current ground cover	The site consists of grass-covered terrain. At the time of the survey, the field had been recently mowed.
Existing topography	Based on Google Earth data, the survey area topography is flat, with a ground surface elevation ranging from about 28-ft to 48-ft (NAVD88).
Site geology	Available geologic maps (USGS, 2003; CGS 2010) indicate that the site is underlain by Quaternary age alluvial deposits.

3.0 SCOPE OF WORK

Our scope of work consisted of obtaining MASW data along a single geophone array, as shown in yellow on Figure 1, and VES data along soundings comprised of two orthogonal electrode arrays, as displayed in white on Figure 1. Terracon Concord personnel determined the positions of the soundings prior to our arrival on site. The survey sites were located by navigation to pre-determined coordinates using a Trimble Geo 7X GPS. Our scope of work also consists of processing the geophysical data and presenting our findings in a written report.

4.0 MASW RESULTS

The MASW results are listed in Table A for the vertical sounding, which is designated as MASW-1. The left column of the table contains the calculated depth range of each seismic layer (feet below ground surface) and the right column comprises the associated shear (S-) wave values in feet per second (ft/sec).

Table A : *MASW-1 Seismic Velocity vs Depth*

DEPTH RANGE (FT)	S-WAVE VELOCITY (FT/SEC)
0 - 3	470
3 - 6	530
6 - 10	470
10 - 16	1,220
16 - 22	1,070
22 - 30	410
30 - 41	610
41 - 53	1,260
53 - 70	1,480
70 - 100	1,820

The measured Vs values range from a low of 410 ft/sec to a maximum of 1,820 ft/sec. Generally, Vs tends to increase with increasing depth; however, a significant velocity inversion (a decrease in Vs with depth) is apparent between 22-ft and 41-ft.

The standard method of reporting MASW data is to consider the location of the 1D velocity vs. depth model as the center point of the MASW spread. However, this does not mean that the measured velocity values represent materials solely beneath that location. In fact, the subsurface conditions underlying the entire length of the array, and for several tens of feet to either side, contribute to the measured velocity values.

5.0 VES RESULTS

The results of the VES survey are listed in Tables B through E. The left column of the table contains the a-spacing and the right two columns comprise the associated electrical resistivity values in ohm-meters.

Table B : *VES-FR1 Electrical Resistivity vs. A-Spacing*

A-SPACING (FT)	ELECTRICAL RESISTIVITY (OHM-METERS)	
	Northwest-Southeast	Southwest-Northeast
2.5	31.70	52.44
5	45.19	53.48
10	45.12	45.45
20	57.64	44.70
50	70.98	69.43

Table C : *VES-FR2 Electrical Resistivity vs. A-Spacing*

A-SPACING (FT)	ELECTRICAL RESISTIVITY (OHM-METERS)	
	Northwest-Southeast	Southwest-Northeast
2.5	532.7	377.5
5	180.8	227.5
10	48.76	74.09
20	45.08	46.18
50	60.64	61.23

Table D : *VES-FR3 Electrical Resistivity vs. A-Spacing*

A-SPACING (FT)	ELECTRICAL RESISTIVITY (OHM-METERS)	
	North-South	East-West
2.5	266.5	228.8
5	165.3	133.4
10	63.83	61.09
20	43.46	43.86
50	58.49	58.22

Table E : *VES-FR4 Electrical Resistivity vs. A-Spacing*

A-SPACING (FT)	ELECTRICAL RESISTIVITY (OHM-METERS)	
	North-South	East-West
2.5	50.51	98.77
5	52.03	40.95
10	32.01	32.95
20	38.64	41.67
50	60.93	60.68

The overall range of values measured varies from 31.7 to over 500 ohm-meters. Generally, we note agreement between the orthogonal readings at each location, especially at greater depth. While near-surface readings vary from one location to another, the data show substantial consistency between the deeper readings, with most values at a-spacings of 10-ft or greater falling between approximately 40 and 70 ohm-meters.

APPENDIX A:

1-D Multi-Channel Analysis of Surface Waves (MASW)

APPENDIX A:

1-D Multi-Channel Analysis of Surface Waves (MASW)

1.0 METHODOLOGY

When seismic energy is generated at or near the ground surface, both body and surface waves are produced. Body waves expand omni-directionally throughout the subsurface. They consist of both compressional (P) and shear (S) waves. Surface waves (e.g., Rayleigh, Love, etc.) radiate along the ground surface at velocities that are proportional to shear wave velocity (Vs). Rayleigh waves are characterized by retrograde elliptical particle motion, and travel at approximately 0.9 times the velocity of S-waves.

If a vertical impact source is used, approximately two-thirds of the seismic energy that is produced is in the form of ground roll. As a result, surface waves are typically the most prominent signal on multi-channel seismic records. In addition, surface waves have dispersion properties that body waves lack. That is, different wavelengths have different penetration depths and, therefore, propagate at different velocities. By analyzing the dispersion of surface waves, it is possible to obtain an S-wave versus depth velocity profile. Since S-wave velocity is directly proportional to shear modulus, this provides a direct indication in the variation of stiffness (or rigidity) of subsurface materials with depth.

Surface waves can be recorded and analyzed using a method referred to as Multichannel Analysis of Surface Waves (MASW). This method is used to collect surface wave data using a fixed array of geophones and shot points. This is referred to as a sounding and results in a 1D model depicting variation in S-wave velocity versus depth beneath the center of the array. However, the subsurface conditions underlying the entire length of the array, and for several tens of feet to either side, contribute to the measured velocity values. The method requires an energy source that is capable of producing ground roll and geophones that are capable of detecting low frequencies (<10 Hz) signals.

2.0 DATA ACQUISITION

We acquired one MASW sounding, denoted MASW-1, as indicated by the yellow line on Figure 1 of the report. The location and orientation of the MASW-1 were determined by Terracon Concord. The sounding was configured using a seismic array consisting of four shot points and 24-geophones distributed at 6-ft intervals in a collinear array, as shown in Figure 1 of this appendix, below. The center of the array is considered the 1-D sounding location.

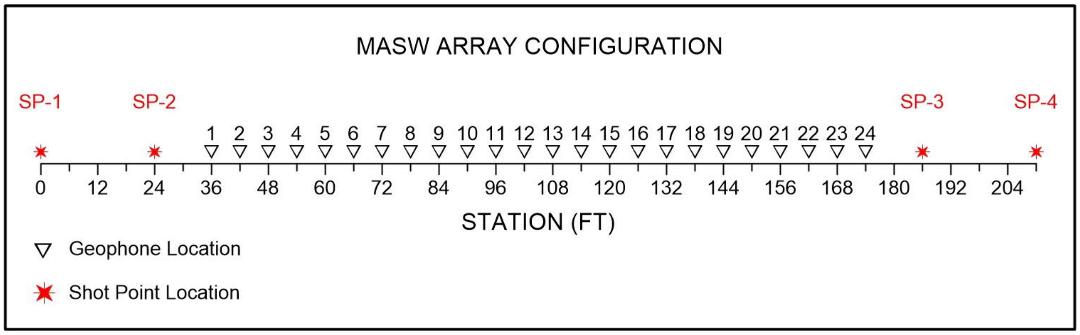


Figure 1: *MASW Array Configuration*

Seismic energy was produced at each shot point using a 16-pound sledgehammer striking a polyurethane plate on the ground surface. The resulting seismic waveforms were detected by an array of 24 Oyo ***Geospace*** geophones with a natural frequency of 8-Hz and recorded using a Geometrics ***Geode*** 24-channel distributed array engineering seismograph. The seismic waveforms were digitized, processed and amplified by the Geode and transmitted via a ruggedized Ethernet cable to a field computer. The recorded data were archived for subsequent processing and displayed on the computer screen in the form of seismograms for quality assurance purposes.

3.0 DATA ANALYSIS

The seismic wave-traces (shot gathers) recorded at each shot point were analyzed using the computer program ***SURFSEIS*** developed by the Kansas Geological Survey (Version 5.0, 2016). This interactive program converts the data acquired from all four shot points in a given sounding into a dispersion curve representing phase velocity versus frequency. This curve is then inverted to produce a 1D model indicating S-wave velocity versus depth. The steps involved in this procedure are as follows:

1) The shot gathers are converted to KGS format.
2) Stations are assigned to the geophone and shot point locations.
3) The resulting records are viewed to determine their overall quality. If necessary, portions of the records are muted to remove interference from refractions, reflections and higher mode events.
4) For each formatted (and/or muted) record, the program produces what is referred to as an "overtone plot". This is a colored cross-section indicating phase velocity versus frequency and amplitude. The vertical axis represents phase velocity (increasing upward); the horizontal axis represents frequency (increasing to the right); and signal amplitude is indicated by various colors, with the hottest colors (orange to red to dark brown) representing the greatest signal to noise ratio. Typically, the strongest signals align in a

curved pattern with a symmetry similar to a “hockey stick” where the blade is pointing upward at the lower end of the frequency spectrum (higher velocity at greater depth) and the handle projects to the right in the direction of increasing frequencies indicating lower velocities.

5) The overtone plots compiled from the four shot points are reviewed to determine their overall quality and the best among them (possibly all) are merged to form a single overtone. This enhances the overall signal to noise ratio of the survey and incorporates data from both ends of the spread (if feasible).
6) The resulting overtone plot is used as a guide in deriving a dispersion curve representing phase velocity versus frequency. This is done by fitting the curve along the center of the hockey stick where the signal to noise ratio is highest.
7) The resulting dispersion curve is inverted through an iterative process to compute a 1D model representing S-wave velocity versus depth.

The calculated seismic S-wave velocities in each depth range for MASW-1 are presented in Table A, in the main body of the report.

APPENDIX B:

Electrical Resistivity Sounding

APPENDIX B:

Electrical Resistivity Sounding

1.0 METHODOLOGY

1.1 ELECTRICAL RESISTIVITY: DEFINITION AND APPLICATIONS

Electrical resistivity (ER) is the resistance of a volume of earth material to the flow of electrical current. The ER of sedimentary earth materials is directly affected by factors such as grain size, porosity, mineralogy, moisture content and groundwater salinity. However, it has been our experience through numerous ER surveys conducted throughout the Bay Area that, in unconsolidated materials, grain size seems to have the largest effect on ER of all these parameters. Specifically, fine grained materials such as clays and silts typically have relatively low ER whereas coarse grained materials such as sands and gravels have relatively high ER.

The ER of rock is affected primarily by mineralogy and the degree of weathering and fracturing. Rock formations that are deeply buried and not exposed to chemical weathering are generally impermeable, contain little water, and have a relatively high electrical resistivity. Conversely, highly weathered and fractured rock that contains moisture typically has lower resistivity values. Alternatively, some rocks contain conductive minerals that can result in the rock having relatively low ER.

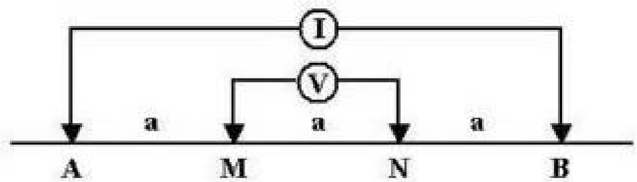
Given the relationships described above, geophysical methods that measure subsurface ER can be used to determine the depth, thickness and lateral extent of groundwater aquifers, the depth to groundwater, the depth to rock, the depth, thickness and lateral extent of clay layers and the depth, thickness and lateral extent of sand/gravel deposits. ER measurements can also be used to evaluate soil corrosion potential and to provide parameters for the design of electrical grounding systems.

1.2 ELECTRICAL RESISTIVITY SOUNDING

Measuring the variation in ER versus depth beneath a fixed point is referred to as a vertical electric sounding (VES). This involves transmitting electrical current *(I)* into the ground between two electrodes, and measuring the resulting electrical potential or voltage drop (*V*) between two other electrodes. There are many different electrode configurations that can be used. The most common are the Wenner and Schlumberger arrays. With both techniques, the four electrodes are arranged in a collinear array. Current is transmitted between the outer two electrodes (referred to as A and B) and the resulting voltage is measured across the inner two electrodes (referred to as

M and N). Readings are typically taken with many different electrode separations, ranging from less than one foot to several hundreds of feet. The larger the separation, the deeper the current is forced to flow to complete a circuit. The actual current flow occurs within a generally hemi-spherical volume of earth between the current electrodes. The readings obtained with each electrode separation are used to compute a value referred to as apparent resistivity *(ρ_a).* The term “apparent” is used because the value represents the resistivity of a volume of earth with varying resistivity values rather than a discrete layer with consistent resistivity. The location of the sounding is defined as the center of the electrode array.

For ER surveys involving the design of grounding systems, including this survey, the Four Pin Wenner Array is typically used. With this array the electrode separation, “a”, is uniform between all four electrodes but varies from one reading to the next. The array configuration diagram and the equation that is used to compute apparent resistivity are illustrated in Figure 1.



$$\rho_A = 2\pi\ a \frac{V}{I}$$

Figure 1: *Physical layout of the Wenner Array and associated Apparent Resistivity (ρ_a) Equation.*

2.0 INSTRUMENTATION

We collected VES data using a ***SuperSting R1*** Resistivity Meter, manufactured by Advanced Geosciences Incorporated (AGI). The SuperSting is a self-contained unit that transmits current at outputs ranging from 1 to 2,000 milliamps (mA). The instrument measures the potential drop (voltage) caused by the current influx and converts the data to values of apparent resistivity. The data are stored in internal memory and can be downloaded to a computer for subsequent archiving.

3.0 DATA ACQUISITION

The VES survey consisted of four soundings, each comprised of two orthogonal arrays. The VES are designated as FR1 through FR4, as shown on Figure 1 of the report. The ***SuperSting R1*** was connected to the four electrodes in the array using 14-gauge insulated single conductor wires. Once programmed with the a-spacing for a given measurement, the instrument transmitted electrical current through the outer electrodes (A and B) and measured the voltage drop across the inner pair (M and N). Each measurement was made twice, and the results compared to make sure that there was no more than 2% deviation between the measurements. The averaged readings were then used to automatically calculate the apparent resistivity values (ρ_a) according to the equation shown in Figure 1. This procedure was repeated for every prescribed a-spacing starting with small values (a=2.5-ft) and expanding with each subsequent measurement to the largest spacing (a=50-ft). The survey results are presented in Tables B through E, in the main body of the report.

SUPPORTING INFORMATION

Contents:

Note: All attachments are one page unless noted above.

GENERAL NOTES

DESCRIPTION OF SYMBOLS AND ABBREVIATIONS

Bay AZ1 Site (Northtown) ■ San Jose, CA

Terracon Project No. ND205079

SAMPLING	WATER LEVEL	FIELD TESTS
Auger Cuttings; Modified California Ring Sampler; Grab Sample; Shelby Tube; Standard Penetration Test	**Water Initially Encountered**; **Water Level After a Specified Period of Time**; **Water Level After a Specified Period of Time**; **Cave In Encountered**	**N** — **Standard Penetration Test Resistance (Blows/Ft.)**
	Water levels indicated on the soil boring logs are the levels measured in the borehole at the times indicated. Groundwater level variations will occur over time. In low permeability soils, accurate determination of groundwater levels is not possible with short term water level observations.	**(HP)** — **Hand Penetrometer**
		(T) — **Torvane**
		(DCP) — **Dynamic Cone Penetrometer**
		UC — **Unconfined Compressive Strength**
		(PID) — **Photo-Ionization Detector**
		(OVA) — **Organic Vapor Analyzer**

DESCRIPTIVE SOIL CLASSIFICATION

Soil classification as noted on the soil boring logs is based Unified Soil Classification System. Where sufficient laboratory data exist to classify the soils consistent with ASTM D2487 "Classification of Soils for Engineering Purposes" this procedure is used. ASTM D2488 "Description and Identification of Soils (Visual-Manual Procedure)" is also used to classify the soils, particularly where insufficient laboratory data exist to classify the soils in accordance with ASTM D2487. In addition to USCS classification, coarse grained soils are classified on the basis of their in-place relative density, and fine-grained soils are classified on the basis of their consistency. See "Strength Terms" table below for details. The ASTM standards noted above are for reference to methodology in general. In some cases, variations to methods are applied as a result of local practice or professional judgment.

LOCATION AND ELEVATION NOTES

Exploration point locations as shown on the Exploration Plan and as noted on the soil boring logs in the form of Latitude and Longitude are approximate. See Exploration and Testing Procedures in the report for the methods used to locate the exploration points for this project. Surface elevation data annotated with +/- indicates that no actual topographical survey was conducted to confirm the surface elevation. Instead, the surface elevation was approximately determined from topographic maps of the area.

STRENGTH TERMS

RELATIVE DENSITY OF COARSE-GRAINED SOILS (More than 50% retained on No. 200 sieve.) Density determined by Standard Penetration Resistance			**CONSISTENCY OF FINE-GRAINED SOILS** (50% or more passing the No. 200 sieve.) Consistency determined by laboratory shear strength testing, field visual-manual procedures or standard penetration resistance			
Descriptive Term (Density)	**Standard Penetration or N-Value Blows/Ft.**	**Ring Sampler Blows/Ft.**	**Descriptive Term (Consistency)**	**Unconfined Compressive Strength Qu, (tsf)**	**Standard Penetration or N-Value Blows/Ft.**	**Ring Sampler Blows/Ft.**
Very Loose	0 - 3	0 - 6	Very Soft	less than 0.25	0 - 1	< 3
Loose	4 - 9	7 - 18	Soft	0.25 to 0.50	2 - 4	3 - 4
Medium Dense	10 - 29	19 - 58	Medium Stiff	0.50 to 1.00	4 - 8	5 - 9
Dense	30 - 50	59 - 98	Stiff	1.00 to 2.00	8 - 15	10 - 18
Very Dense	> 50	> 99	Very Stiff	2.00 to 4.00	15 - 30	19 - 42
			Hard	> 4.00	> 30	> 42

RELEVANCE OF SOIL BORING LOG

The soil boring logs contained within this document are intended for application to the project as described in this document. Use of these soil boring logs for any other purpose may not be appropriate.

CPT GENERAL NOTES

DESCRIPTION OF SYMBOLS AND ABBREVIATIONS

Bay AZ1 Site (Northtown) ■ San Jose, CA

Terracon Project No. ND205079

DESCRIPTION OF MEASUREMENTS AND CALIBRATIONS

To be reported per ASTM D5778:

Uncorrected Tip Resistance, q_c
Measured force acting on the cone divided by the cone's projected area

Corrected Tip Resistance, q_t
Cone resistance corrected for porewater and net area ratio effects
$q_t = q_c + u_2(1 - a)$

Where a is the net area ratio, a lab calibration of the cone typically between 0.70 and 0.85

Pore Pressure, u
Pore pressure measured during penetration
u_1 - sensor on the face of the cone
u_2 - sensor on the shoulder (more common)

Sleeve Friction, f_s
Frictional force acting on the sleeve divided by its surface area

Normalized Friction Ratio, F_r
The ratio as a percentage of f_s to q_t, accounting for overburden pressure

To be reported per ASTM D7400, if collected:

Shear Wave Velocity, V_s
Measured in a Seismic CPT and provides direct measure of soil stiffness

DESCRIPTION OF GEOTECHNICAL CORRELATIC

Normalized Tip Resistance, Q_{tn}
$Q_{tn} = ((q_t - \sigma_{V0})/P_a)(P_a/\sigma'_{V0})^n$
$n = 0.381(I_c) + 0.05(\sigma'_{V0}/P_a) - 0.15$

Over Consolidation Ratio, OCR
OCR (1) $= 0.25(Q_{tn})^{1.25}$
OCR (2) $= 0.33(Q_{tn})$

Undrained Shear Strength, S_u
$S_u = Q_{tn} \times \sigma'_{V0}/N_{kt}$
N_{kt} is a soil-specific factor (shown on S_u plot)

Sensitivity, S_t
$S_t = (q_t - \sigma_{V0}/N_{kt}) \times (1/f_s)$

Effective Friction Angle, ϕ'
ϕ' (1) $= \tan^{-1}(0.373[\log(q_t/\sigma'_{V0}) + 0.29])$
ϕ' (2) $= 17.6 + 11[\log(Q_{tn})]$

Unit Weight, γ
$\gamma = (0.27[\log(F_r)]+0.36[\log(q_t/atm)]+1.236) \times \gamma_{water}$
σ_{V0} is taken as the incremental sum of the unit weights

Small Strain Shear Modulus, G_0
G_0 (1) $= \rho V_s^2$
G_0 (2) $= 0.015 \times 10^{(0.55Ic + 1.68)}(q_t - \sigma_{V0})$

Soil Behavior Type Index, I_c
$I_c = [(3.47 - \log(Q_{tn})^2 + (\log(F_r) + 1.22)^2]^{0.5}$

SPT N_{60}
$N_{60} = (q_t/atm) / 10^{(1.1268 - 0.2817Ic)}$

Elastic Modulus, E_s (assumes $q/q_{ultimate} \sim 0.3$, i.e. FS = 3)
E_s (1) $= 2.6\psi G_0$ where $\psi = 0.56 - 0.33\log Q_{tn,clean\ sand}$
E_s (2) $= G_0$
E_s (3) $= 0.015 \times 10^{(0.55Ic + 1.68)}(q_t - \sigma_{V0})$
E_s (4) $= 2.5q_t$

Constrained Modulus, M
$M = \alpha_M(q_t - \sigma_{V0})$
For $I_c > 2.2$ (fine-grained soils)
$\alpha_M = Q_{tn}$ with maximum of 14
For $I_c < 2.2$ (coarse-grained soils)
$\alpha_M = 0.0188 \times 10^{(0.55Ic + 1.68)}$

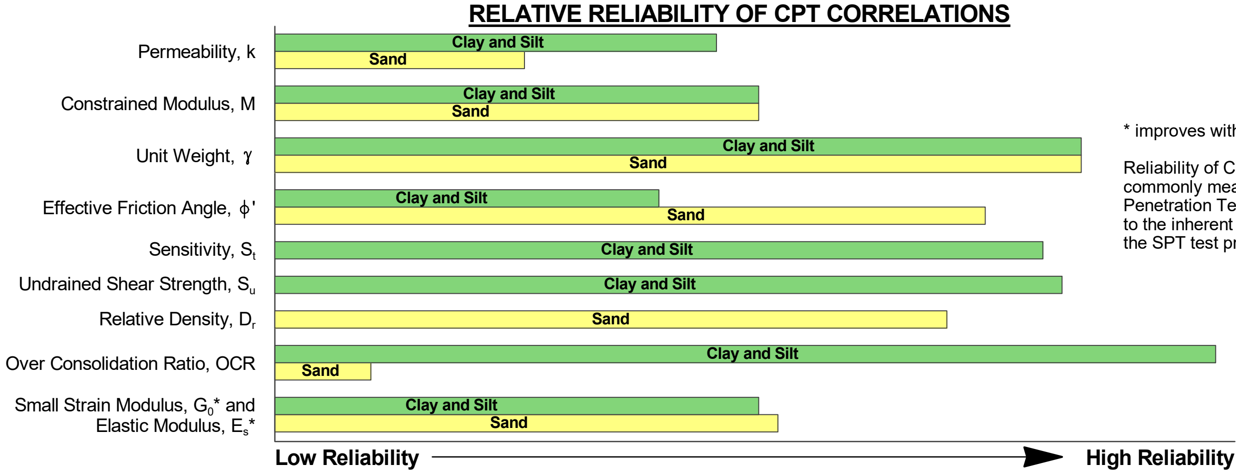
Hydraulic Conductivity, k
For $1.0 < I_c < 3.27$ $k = 10^{(0.952 - 3.04Ic)}$
For $3.27 < I_c < 4.0$ $k = 10^{(-4.52 - 1.37Ic)}$

Relative Density, D_r
$D_r = (Q_{tn} / 350)^{0.5} \times 100$

REPORTED PARAMETERS

CPT logs as provided, at a minimum, report the data as required by ASTM D5778 and ASTM D7400 (if applicable). This minimum data include q_t, f_s, and u. Other correlated parameters may also be provided. These other correlated parameters are interpretations of the measured data based upon published and reliable references, but they do not necessarily represent the actual values that would be derived from direct testing to determine the various parameters. To this end, more than one correlation to a given parameter may be provided. The following chart illustrates estimates of reliability associated with correlated parameters based upon the literature referenced below.

RELATIVE RELIABILITY OF CPT CORRELATIONS



* improves with seismic V_s measurements

Reliability of CPT-predicted N_{60} values as commonly measured by the Standard Penetration Test (SPT) is not provided due to the inherent inaccuracy associated with the SPT test procedure.

WATER LEVEL

The groundwater level at the CPT location is used to normalize the measurements for vertical overburden pressures and as a result influences the normalized soil behavior type classification and correlated soil parameters. The water level may either be "measured" or "estimated:"

Measured - Depth to water directly measured in the field

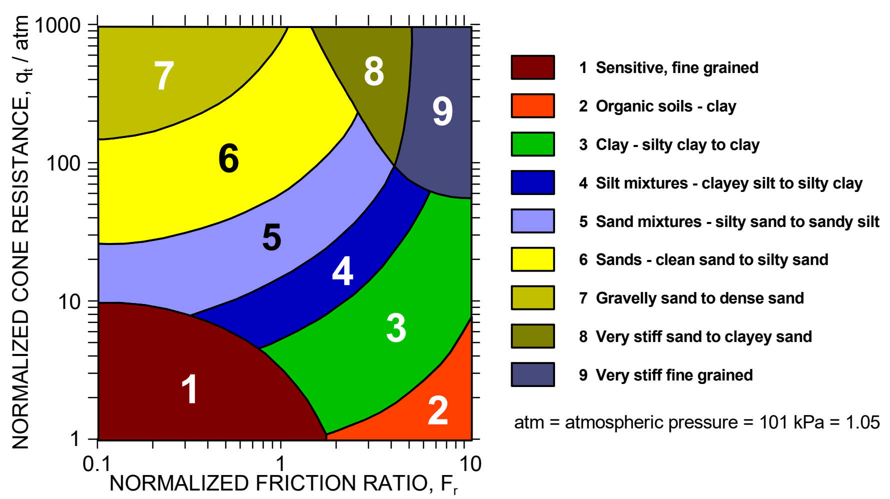
Estimated - Depth to water interpolated by the practitioner using pore pressure measurements in coarse grained soils and known site conditions

While groundwater levels displayed as "measured" more accurately represent site conditions at the time of testing than those "estimated," in either case the groundwater should be further defined prior to construction as groundwater level variations will occur over time.

CONE PENETRATION SOIL BEHAVIOR TYPE

The estimated stratigraphic profiles included in the CPT logs are based on relationships between corrected tip resistance (q_t), friction resistance (f_s), and porewater pressure (u_2). The normalized friction ratio (F_r) is used to classify the soil behavior type.

Typically, silts and clays have high F_r values and generate large excess penetration porewater pressures; sands have lower F_r's and do not generate excess penetration porewater pressures. The adjacent graph (Robertson *et al.*) presents the soil behavior type correlation used for the logs. This normalized SBT chart, generally considered the most reliable, does not use pore pressure to determine SBT due to its lack of repeatability in onshore CPTs.



atm = atmospheric pressure = 101 kPa = 1.05 tsf

REFERENCES

Kulhawy, F.H., Mayne, P.W., (1997). "Manual on Estimating Soil Properties for Foundation Design," Electric Power Research Institute, Palo Alto, CA.
Mayne, P.W., (2013). "Geotechnical Site Exploration in the Year 2013," Georgia Institue of Technology, Atlanta, GA.
Robertson, P.K., Cabal, K.L. (2012). "Guide to Cone Penetration Testing for Geotechnical Engineering," Signal Hill, CA.
Schmertmann, J.H., (1970). "Static Cone to Compute Static Settlement over Sand," *Journal of the Soil Mechanics and Foundations Division*, 96(SM3), 1011-1043.

UNIFIED SOIL CLASSIFICATION SYSTEM

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests [A]					Soil Classification	
					Group Symbol	Group Name [B]
Coarse-Grained Soils: More than 50% retained on No. 200 sieve	**Gravels:** More than 50% of coarse fraction retained on No. 4 sieve	**Clean Gravels:** Less than 5% fines [C]	$Cu \geq 4$ and $1 \leq Cc \leq 3$ [E]		GW	Well-graded gravel [F]
			$Cu < 4$ and/or [$Cc<1$ or $Cc>3.0$] [E]		GP	Poorly graded gravel [F]
		Gravels with Fines: More than 12% fines [C]	Fines classify as ML or MH		GM	Silty gravel [F, G, H]
			Fines classify as CL or CH		GC	Clayey gravel [F, G, H]
	Sands: 50% or more of coarse fraction passes No. 4 sieve	**Clean Sands:** Less than 5% fines [D]	$Cu \geq 6$ and $1 \leq Cc \leq 3$ [E]		SW	Well-graded sand [I]
			$Cu < 6$ and/or [$Cc<1$ or $Cc>3.0$] [E]		SP	Poorly graded sand [I]
		Sands with Fines: More than 12% fines [D]	Fines classify as ML or MH		SM	Silty sand [G, H, I]
			Fines classify as CL or CH		SC	Clayey sand [G, H, I]
Fine-Grained Soils: 50% or more passes the No. 200 sieve	**Silts and Clays:** Liquid limit less than 50	**Inorganic:**	PI > 7 and plots on or above “A” line [J]		CL	Lean clay [K, L, M]
			PI < 4 or plots below “A” line [J]		ML	Silt [K, L, M]
		Organic:	Liquid limit - oven dried	< 0.75	OL	Organic clay [K, L, M, N]
			Liquid limit - not dried			Organic silt [K, L, M, O]
	Silts and Clays: Liquid limit 50 or more	**Inorganic:**	PI plots on or above “A” line		CH	Fat clay [K, L, M]
			PI plots below “A” line		MH	Elastic Silt [K, L, M]
		Organic:	Liquid limit - oven dried	< 0.75	OH	Organic clay [K, L, M, P]
			Liquid limit - not dried			Organic silt [K, L, M, Q]
Highly organic soils:	Primarily organic matter, dark in color, and organic odor				PT	Peat

[A] Based on the material passing the 3-inch (75-mm) sieve.

[B] If field sample contained cobbles or boulders, or both, add “with cobbles or boulders, or both” to group name.

[C] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.

[D] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay.

[E] $Cu = D_{60}/D_{10}$ $\quad Cc = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$

[F] If soil contains ≥ 15% sand, add “with sand” to group name.

[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.

[H] If fines are organic, add “with organic fines” to group name.

[I] If soil contains ≥ 15% gravel, add “with gravel” to group name.

[J] If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.

[K] If soil contains 15 to 29% plus No. 200, add “with sand” or “with gravel,” whichever is predominant.

[L] If soil contains ≥ 30% plus No. 200 predominantly sand, add “sandy” to group name.

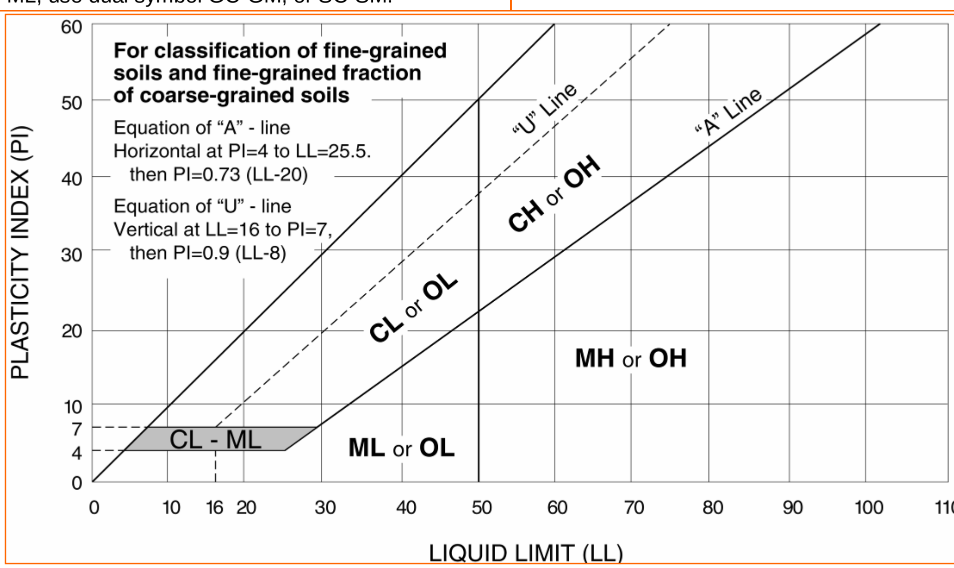
[M] If soil contains ≥ 30% plus No. 200, predominantly gravel, add “gravelly” to group name.

[N] PI ≥ 4 and plots on or above “A” line.

[O] PI < 4 or plots below “A” line.

[P] PI plots on or above “A” line.

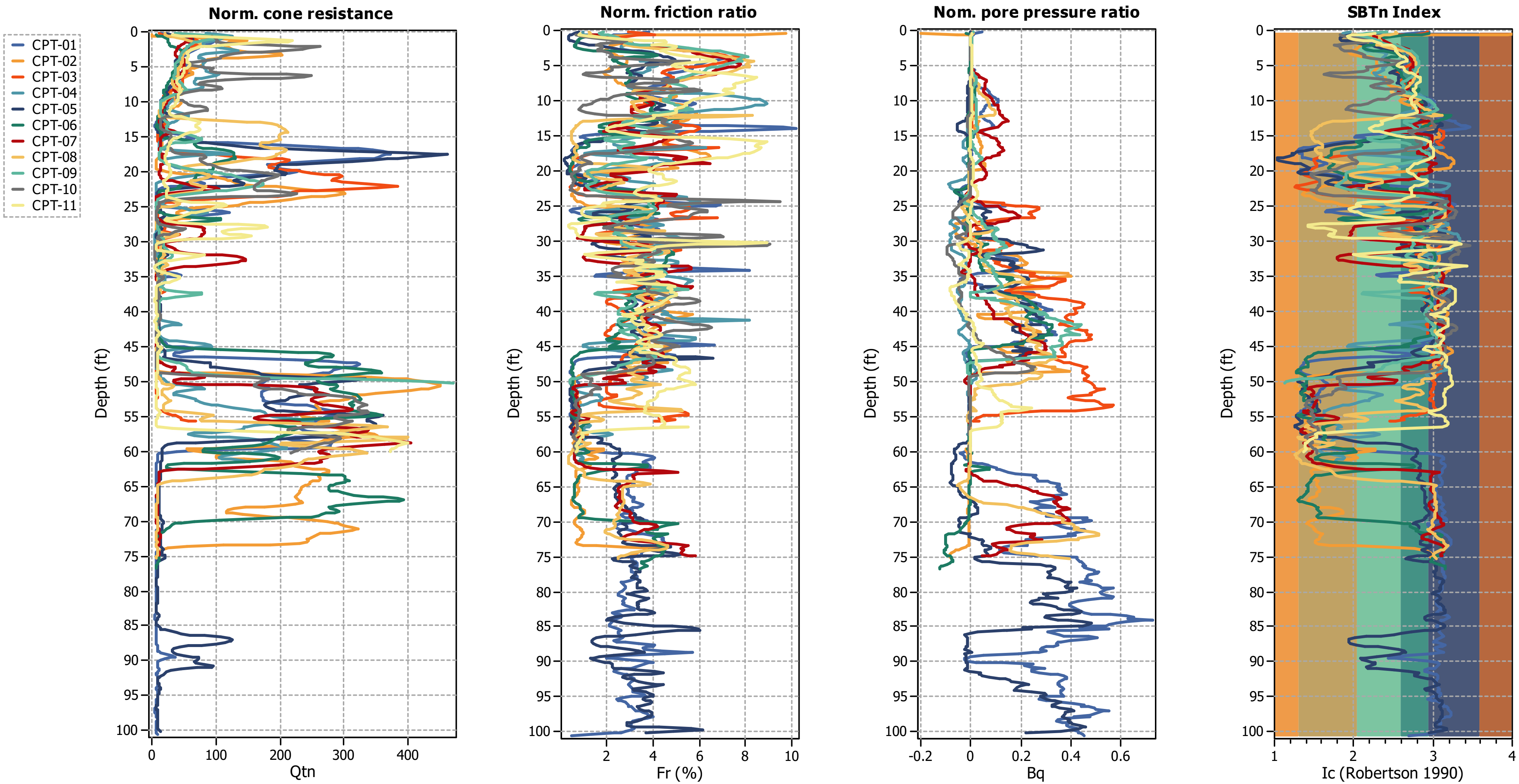
[Q] PI plots below “A” line.



Terracon Consultants, Inc.
5075 Commercial Cir.,Suite E
Concord, CA 94520
www.terracon.com

Project: BAY AZ1

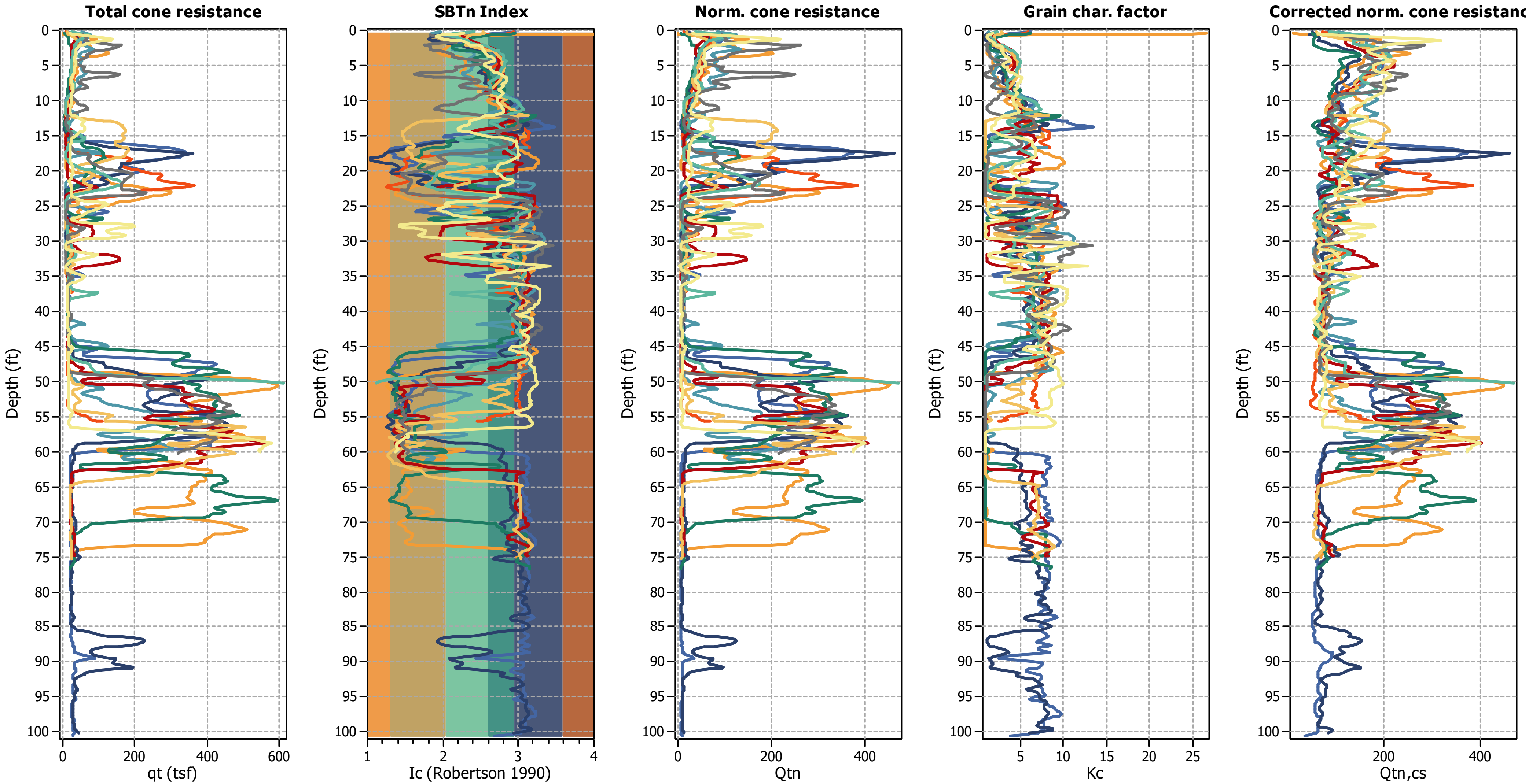
Overlay Normalized Plots



Terracon Consultants, Inc.
5075 Commercial Cir.,Suite E
Concord, CA 94520
www.terracon.com

Project: BAY AZ1

Overlay Intermediate Results

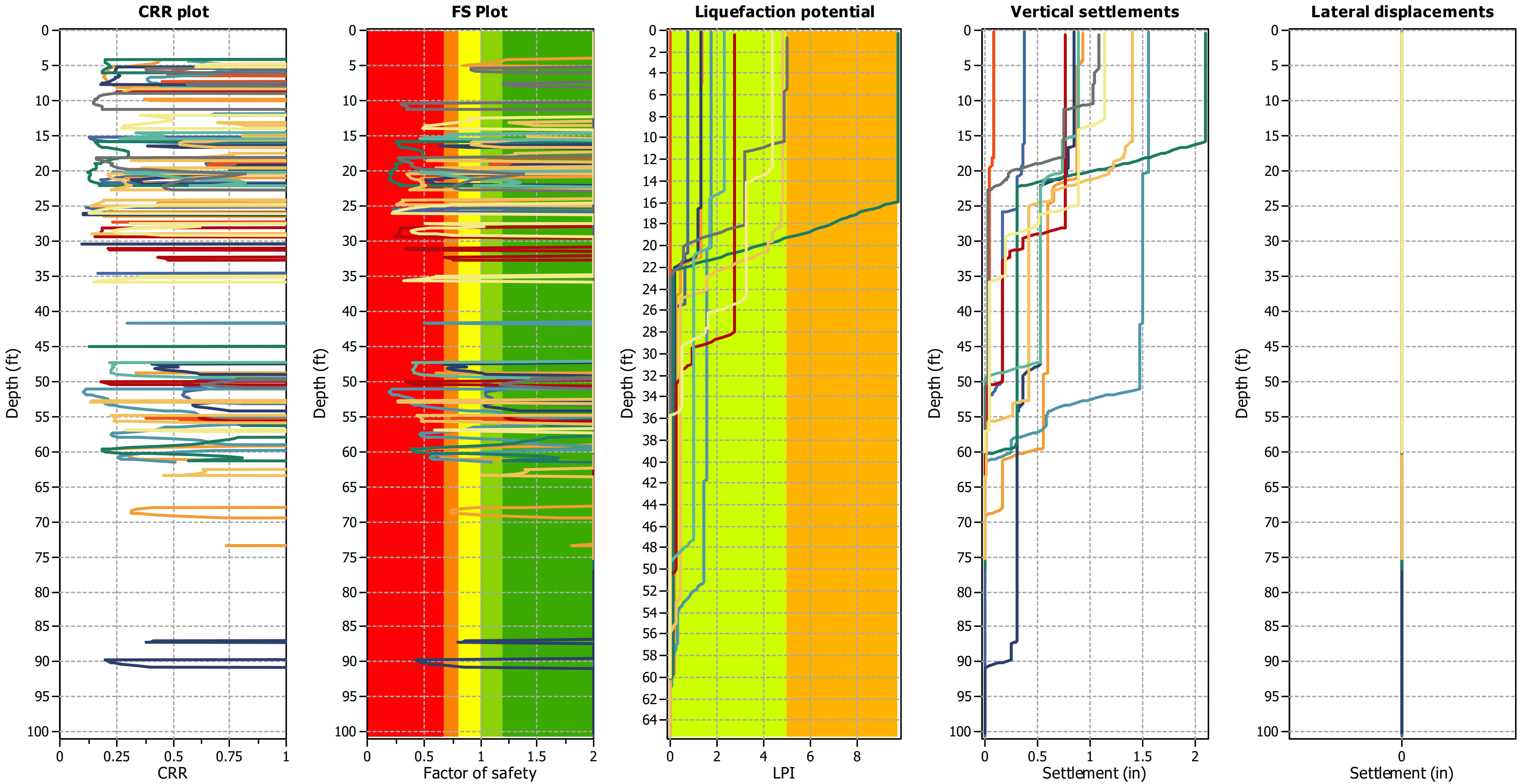


Project file: C:\Users\ntsmith\OneDrive - Terracon Consultants Inc\Desktop\Bay AZ1\CPT\ND205079 BAY AZ1 - liquefaction.clq

Terracon Consultants, Inc.
5075 Commercial Cir.,Suite E
Concord, CA 94520
www.terracon.com

Project: BAY AZ1

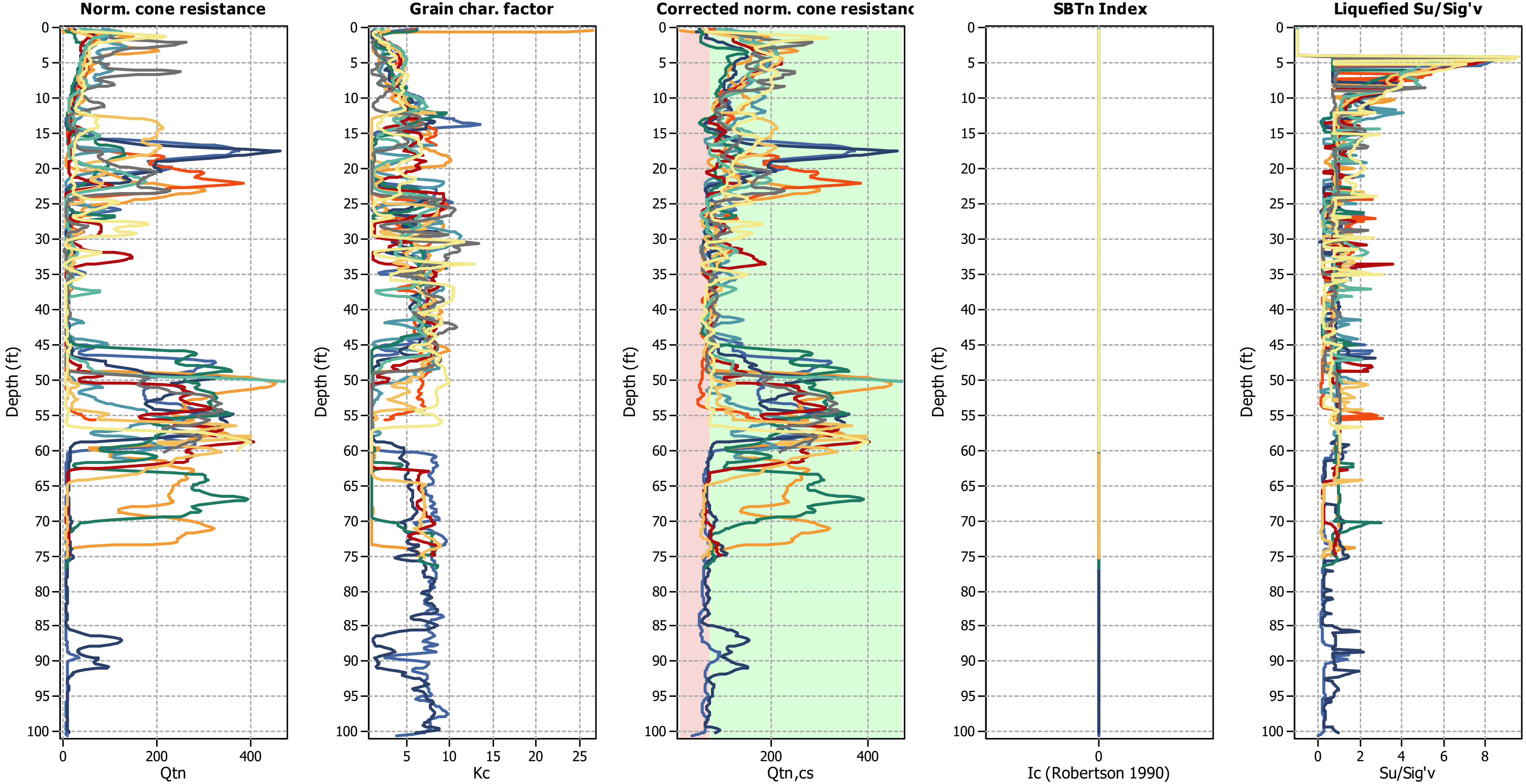
Overlay Cyclic Liquefaction Plots



Terracon Consultants, Inc.
5075 Commercial Cir.,Suite E
Concord, CA 94520
www.terracon.com

Project: BAY AZ1

Overlay Strength Loss Plots



December 15, 2020

Burns & McDonnell Engineering Company, Inc.
530 W. Spring Street, Suite 200
Columbus, Ohio 43215

Attn: Ms. Brandi Sauter
P: (614) 499-1009
E: blsauter@burnsmcd.com

Re: Report of Site-Specific Ground Motion Study
Bay AZ1 Site (Northtown)
Orchard Parkway
San Jose, California
Terracon Project No. ND205079

Dear Ms. Sauter:

Terracon Consultants, Inc. (Terracon) has completed the site-specific ground motion study for the above-referenced project. Our December 15, 2020 Geotechnical Engineering Report describes the site and project and provides recommendations for earthwork, foundations, and other components of the project. The ground motion study services were performed in general accordance with Terracon Proposal No. PND205079 dated September 28, 2020 and Supplement to Agreement for Services dated October 15, 2020.

REGIONAL FAULTS AND SEISMIC SETTING

The site is located within the Santa Clara Valley that is within the southern portion of the Coast Ranges geomorphic province, characterized by northwest-trending ranges and valleys formed in Cenozoic and Mesozoic age strata. The Coast Ranges are bounded by the Pacific Ocean and Great Valley. The channel of the Guadalupe River is adjacent to the site and flows northwest to San Francisco Bay. Geologic maps indicate the subsurface conditions at the site consist of alluvial gravel, sand, and clay. Site explorations encountered clays and fine-grained sands.

The site is not located in a State-designated Alquist-Priolo earthquake fault zone; therefore, the potential for surface rupture due to faulting at the site is low.

Faults in the site region include:

Fault Name	Maximum Moment Magnitude	Approx. Distance and Direction
Hayward	7.2	8 kms northeast
Calaveras	7.3	13 kms east-northeast

Fault Name	Maximum Moment Magnitude	Approx. Distance and Direction
Monte Vista – Shannon	6.5	13½ kms southwest
San Andreas (Peninsula Section)	8.0	19½ kms southwest

According to our deterministic evaluation, seismic shaking hazard at the site is dominated by the Hayward Fault at short periods and the San Andreas Fault at long periods.

FIELD WAVE VELOCITY TESTING AND SITE CLASS

Terracon performed two seismic cone penetration tests (sCPT) at the site and one seismic Multichannel Analysis of Surface Waves (MASW) survey. The shear wave velocity profiles for both the sCPT and the MASW survey are included in a Geophysical Survey Report (Project No. ND205134, Dated November 12, 2020) appended to our December 15, 2020 Geotechnical Engineering Report. The time-averaged shear wave velocity for the upper 100 feet (30 meters) of the soil profile ($V_{S,30}$) computed from the two sCPT soundings was 843 and 831 feet/second. The $V_{S,30}$ value from the MASW survey was 942 feet/second. The $V_{S,30}$ values from the sCPT soundings and the MASW survey indicated a NEHRP Site Class D for seismic design per Table 20.3-1 of ASCE 7-16, but the Site Class is F because of the soil liquefaction hazard.

APPLICABLE CODES AND STANDARDS

In general, the basis of design is the 2019 California Building Code which is a state-amended version of the 2018 International Building Code (2018 IBC). The 2018 IBC states that structures shall be designed and constructed to resist the effects of earthquake motions in accordance with ASCE 7-16. Per ASCE 7-16, the design earthquake ground motions are two-thirds of the risk-targeted Maximum Considered Earthquake (MCE_R) which is defined as a 1 percent chance of structure collapse in 50 years.

SITE-SPECIFIC GROUND MOTION STUDY

The site-specific ground motion study in this report included both a ground motion hazard analysis and a site response analysis. The procedures outlined in ASCE 7-16 Chapters 11, 20 and 21 were utilized in part for preparation of site-specific spectra for the proposed project.

Ground Motion Hazard Analysis

The ground motion hazard analysis considered regional seismic sources including known faults and background seismicity consistent with the 2014 USGS seismic model. We performed both a deterministic seismic hazard analysis (DSHA) and a probabilistic seismic hazard analysis (PSHA) for two values of $V_{S,30}$ (= 1,200 and 1,820 feet/second) given the variability between the sCPT and MASW shear wave velocity results at different locations across the site.

The deterministic analysis was based on evaluation of scenario events on the Hayward and San Andreas faults and employed the Next Generation Attenuation West 2 (NGA-West 2) ground motion models (GMMs). The equally-weighted spectral values from the GMMs of Abrahamson and others (ASK 2014), Boore and others (BSSA 2014), Campbell and Borzognia (CB 2014) and Chiou and Youngs (CY 2014) were used for computation of the deterministic maximum considered earthquake (MCE) spectrum. The MCE spectrum represents 84th-percentile, 5-percent-damped spectral response acceleration in the direction of maximum horizontal response (maximum rotated) for each oscillator period. Maximum rotated values were obtained from geomean values using the scaling factors of Shahi and Baker (2014). The deterministic MCE spectrum was based on the values from the Hayward scenario spectrum at periods from 0 (PGA) to 3 seconds and from the San Andreas Fault at periods of 4 and 5 seconds.

The probabilistic MCE spectrum was developed using spectral values obtained with the USGS Hazard Tool (https://earthquake.usgs.gov/hazards/interactive/) web-based software application. The USGS software (version 4.2.0) uses the same set of GMMs as used in the deterministic evaluation. The values so obtained were scaled from geomean to maximum rotated values using the factors of Shahi and Baker (2014). Gridded seismic sources were included in the probabilistic model. The probabilistic MCE spectrum was converted to a risk-targeted spectrum (MCE_R) using Method 1 of ASCE 7.16 Section 21.2.1.1 in which C_{RS} = 0.950 and C_{R1} = 0.929.

The probabilistic MCE_R exceeded the deterministic MCE for all periods for both values of $V_{S,30}$; consequently, we selected the deterministic MCE as the site-specific response spectrum per ASCE 7-16 Section 21.2.3. Figure 1 compares the probabilistic and deterministic maximum rotated component (MRC) spectra for both values of $V_{S,30}$.

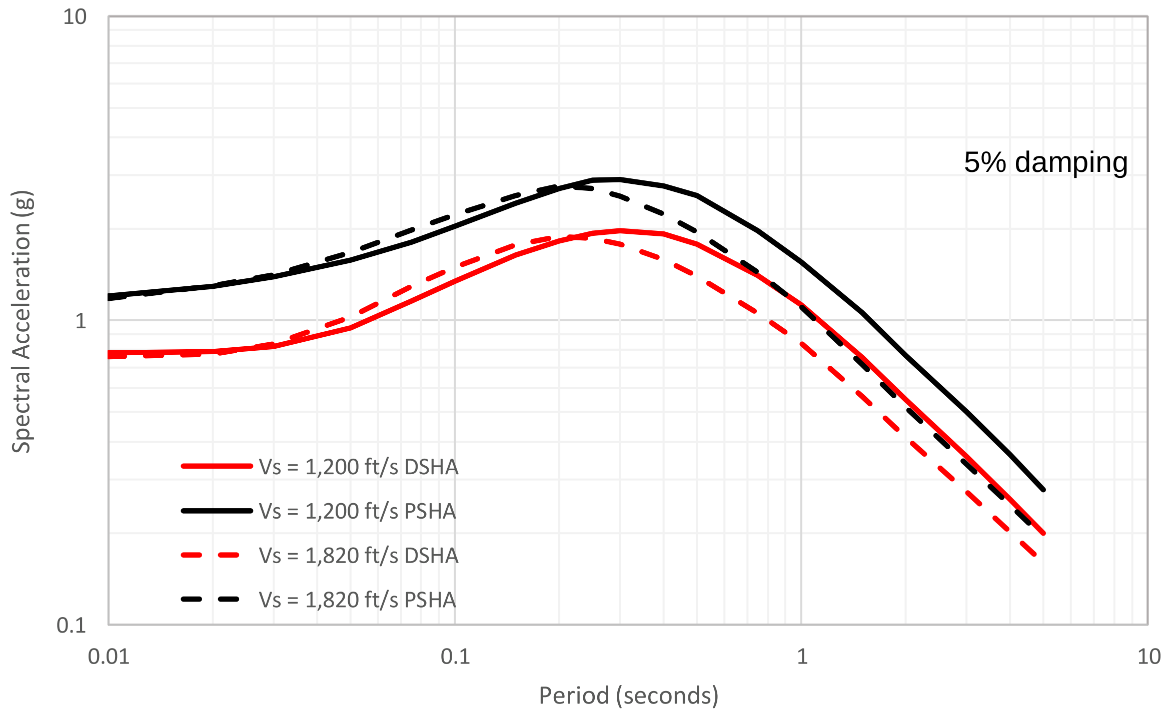


Figure 1. Comparison of Deterministic and Probabilistic MRC Spectra for Both Values of $V_{S,30}$

Selection and Scaling of Earthquake Records

The MCE_R spectral acceleration values at the base of our soil model (i.e., $V_{S,30}$ = 1,200 and 1,820 feet/second) were used as target spectra for selecting and scaling time histories of acceleration. The time histories served as input motions to our site response analysis. Deaggregation of the probabilistic seismic hazard analysis for a period of 1 second allowed us to define our search criteria for the time histories. The recorded motions were obtained from the Pacific Earthquake Engineering Research (PEER) Center. We selected recordings from a database of strike-slip events with a moment magnitude range of 6.5 to 8.5 and a source-to-site distance range of 5 to 20 km. Characteristics of the recordings and their scale factors are listed in Table 1.

Table 1. Characteristics of Selected Time Histories

Earthquake	Moment Magnitude	Station	Distance (km)	Component	Scale Factor
1979 Imperial Valley, California	6.5	El Centro Array #4	7.0	140	2.0|1.7[1]
2010 El Mayor-Cucapah, Mexico	7.2	Michoacan De Ocampo	15.9	000	1.8|1.5[1]
1987 Superstition Hills, California	6.5	Westmoreland Fire Station	13.0	090	4.3|3.6[1]
2010 Darfield, New Zealand	7.0	DSLC	8.5	N27W	3.2|2.6[1]

Earthquake	Moment Magnitude	Station	Distance (km)	Component	Scale Factor
1999 Kocaeli, Turkey[2]	7.5	Duzce	15.4	180	2.6
1999 Kocaeli, Turkey[3]	7.5	Gebze	10.9	000	3.6

[1]Scale factor for each value of $V_{S,30}$ (i.e., 1,200|1,820 feet/second), see report text.
[2]Recording for $V_{S,30}$ = 1,200 feet/second target spectrum, see report text.
[3]Recording for $V_{S,30}$ = 1,820 feet/second target spectrum, see report text.

The selected recordings provided the minimum mean squared error after scaling relative to each respective target spectrum within the period range of 0.16 and 2.1 seconds. We based this range on the potential range of fundamental periods of vibration for the building (i.e., 0.78 to 1.4 seconds) provided to us by Burns & McDonnell on November 6, 2020. Figures 2a and 2b compare the respective MCE_R target spectra to computed spectra from the scaled time histories and the average of the scaled time histories.

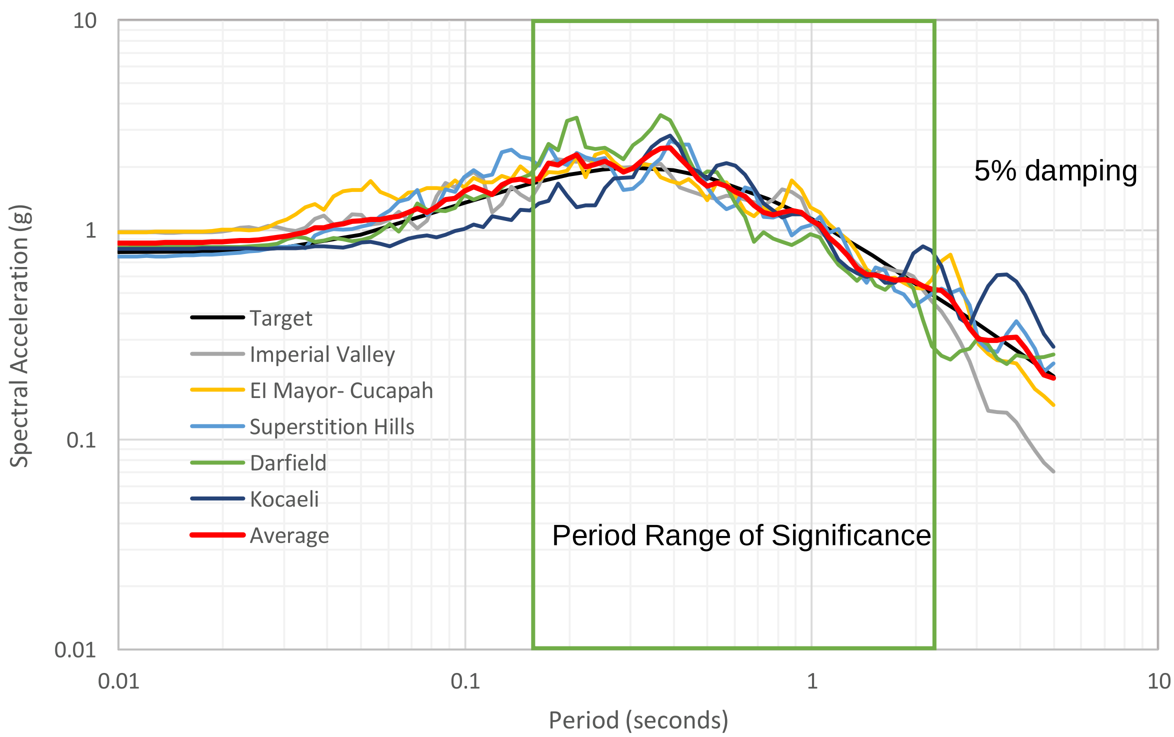


Figure 2a. Comparison of Target Spectrum (V_s = 1,200 ft/s) to Computed Response Spectra

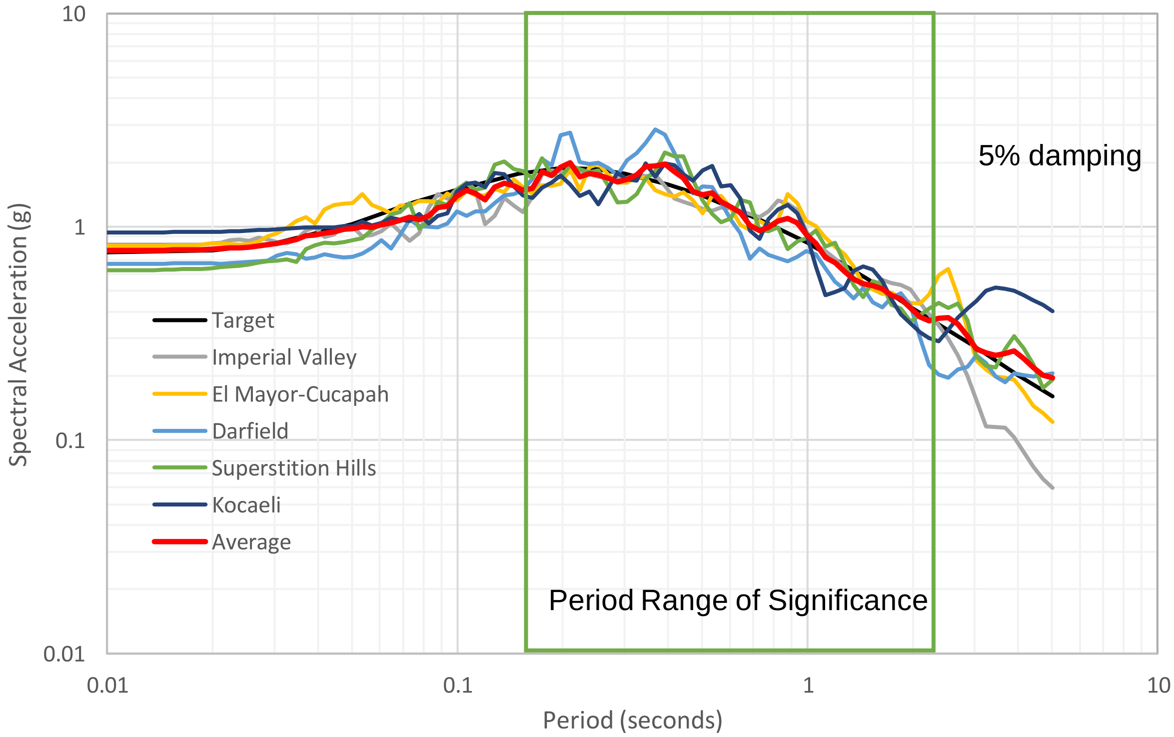


Figure 2b. Comparison of Target Spectrum (V_s = 1,820 ft/s) to Computed Response Spectra

Site Response Analysis

We evaluated the one-dimensional, nonlinear, effective stress and total stress response of the site soils as described in ASCE 7-16 Section 21.1. Effective stress analysis allows for pore water pressure generation and dissipation from cyclic loading. The site response analyses were performed using the computer program DEEPSOIL v.7.0.26.0 (Hashash and others, 2017). We developed two soil models for the site from the field and laboratory testing to capture uncertainty in soil properties. One model assigned V_S = 1,200 feet/second at a depth of 110 feet and the other model assigned V_S = 1,820 feet/second at a depth of 70 feet. The softer profile included a dense sand layer between the depths of 45 and 60 feet and the stiffer profile included a dense sand layer between the depths of 10 and 20 feet.

The time domain modeling employed the General Quadratic/Hyperbolic Model (GC/H) soil model with non-Masing rules for hysteretic unloading and reloading. Shear modulus reduction curves are required to simulate the change of soil stiffness with shear strain, while damping curves represent the amount of energy dissipated by the soils with shear strain. The DEEPSOIL program includes a variety of pre-programmed relationships for shear modulus and damping. We selected the pressure-dependent Darendeli (2001) curves with the high strain values of shear modulus corrected for soil strength (Phillips and Hashash, 2009). The pore water pressure generation model for sands was Matasovic and Vucetic (1993) and for clays was Matasovic and Vucetic (1995) with curve fitting parameters suggested by Carlton (2014) for the site geological and geotechnical conditions. The material below the soil model was assumed to be an elastic half-space. An equivalent linear, frequency domain analysis was performed in parallel with the nonlinear analyses as a check of model outputs.

We followed ASCE 7-16 Chapter 21 procedures to develop the site-specific MCE_R response spectrum at the ground surface. That is, ratios of 5 percent damped response spectra of surface ground motions to input base ground motions were calculated at select periods and the ratio at each period was multiplied by the MCE_R response spectrum of the base motion. Figure 3 shows the site amplification factors for the effective stress analyses and both values of $V_{S,30}$.

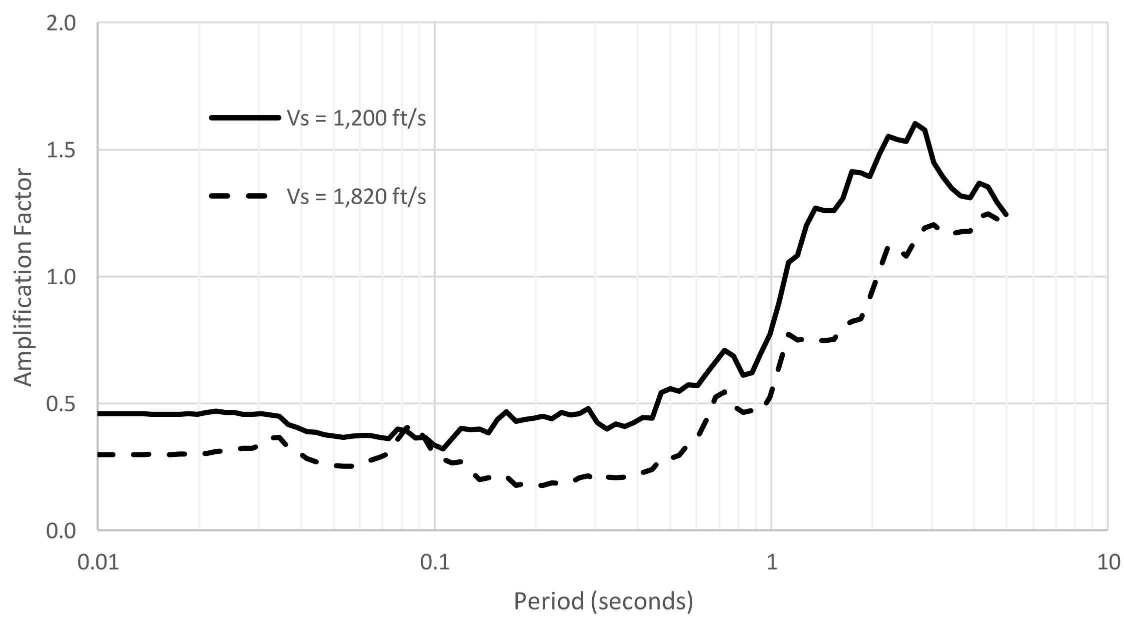


Figure 3. Comparison of Site Amplification Factors for Both Values of $V_{S,30}$

The plot of site amplification factors demonstrates that the input motions were deamplified at shorter periods because of nonlinear soil behavior. That is, relatively large shear strains develop in the medium stiff clay and medium dense sand layers. The natural period of the site shifts to longer periods because of the reduction in stiffness and development of excess pore water pressure in the sands.

Design Response Spectrum and Design Acceleration Parameters

ASCE 7-16 Section 21.3 states that the design spectral response acceleration at any period shall be determined by reducing the site-specific MCE_R spectral response accelerations by one-third. However, the design spectral response acceleration at any period shall not be taken as less than 80 percent of the spectral acceleration determined in accordance with the general procedure of ASCE 7-16 Section 11.4 where F_a = 1.0 and F_v = 2.5 for Site Class D. Figure 4 compares the general procedure design response spectra and the spectra from the site-specific study. We recommend that the site-specific design response spectrum envelope the site-specific results given the variability between the sCPT and MASW shear wave velocity results at different locations across the site. Therefore, the site-specific design response spectrum (DRS) is equivalent to 80 percent of the design response spectrum determined in accordance with the general procedure for periods equal to and less than 1 second and is equivalent to 2/3 of the site-specific MCE_R spectrum for V_S = 1,200 feet/second for periods greater than 1 second. The attached Table 2 lists the acceleration response spectral values (5 percent damping) and factors used to generate the site-specific DRS.

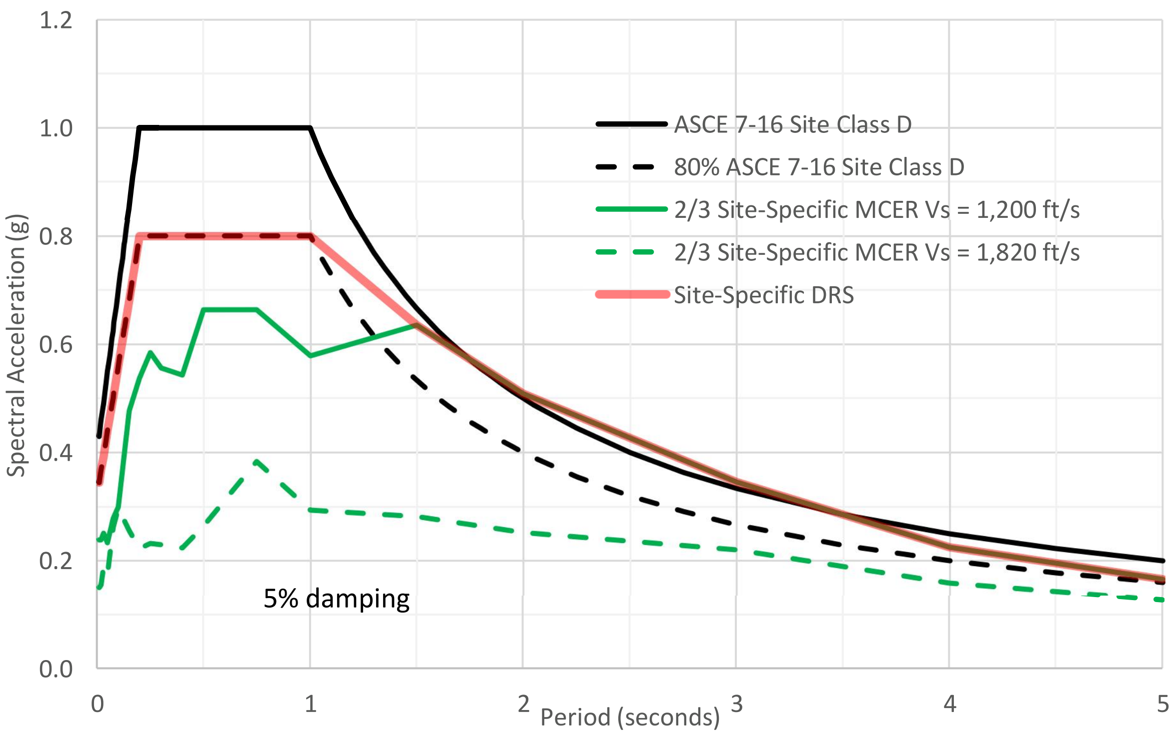


Figure 4. Comparison of Design Response Spectra

Using ASCE 7-16 Section 21.4, the site-specific seismic design parameters are defined as follows:

- S_{DS} = 0.80g, based on the spectral acceleration at a period of 0.2 seconds
- S_{D1} = 1.036g, based on 3 times the spectral acceleration at a period of 3.0 seconds
- S_{MS} = 1.20g, based on 1.5 times S_{DS}
- S_{M1} = 1.554g, based on 1.5 times S_{D1}

Site-Specific Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Acceleration

The site-specific MCE_G peak ground acceleration, PGA_M, is taken as the lesser of the probabilistic and deterministic values; however, it shall not be taken as less than 80 percent of the PGA_M determined from the general procedure for Site Class D. The site-specific MCE_G peak ground acceleration is 0.512g and is equivalent to 80 percent of the PGA_M value for Site Class D.

USGS NATIONAL SEISMIC HAZARD MAP VALUES

For structures other than the buildings with a provided fundamental period between 0.78 and 1.4 seconds, the USGS National Seismic Hazard Map values for S_S and S_1 are 1.5g and 0.6g, respectively. The applicable ATC Hazards by Location data for the site is attached.

GENERAL COMMENTS

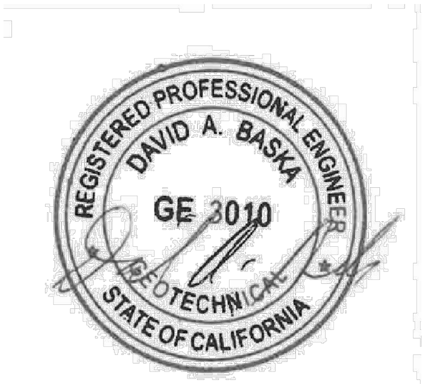
The analysis and recommendations presented in this report are based upon the data obtained from explorations performed by Terracon and from other information discussed in this report. This report does not reflect variations that may occur between explorations, across the site, or due to the modifying effects of construction or weather. The nature and extent of such variations may not become evident until during or after construction. If variations appear, we should be immediately notified so that further evaluation and supplemental recommendations can be provided.

This report has been prepared for the exclusive use of our client for specific application to the project discussed and has been prepared in accordance with generally accepted geotechnical engineering practices. No warranties, either expressed or implied, are intended or made. In the event that changes in the nature, design, or location of the project as outlined in this report are planned, the conclusions and recommendations contained in this report shall not be considered valid unless Terracon reviews the changes and either verifies or modifies the conclusions of this report in writing.

CLOSURE

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report, or if we may be of further service, please contact us.

Sincerely,
Terracon Consultants, Inc.



David A. Baska, Ph.D., P.E., G.E.
Principal Seismologist/Earthquake Engineer

Noah T. Smith, P.E., G.E.
Principal

Attachments: Table 2. Acceleration Response Spectral Values (5% damping) and Factors
ATC Hazards by Location accessed November 20, 2020.

REFERENCES

Abrahamson, N.A., Walter J. Silva, and Ronnie Kamai (2014), Summary of the ASK14 Ground Motion Relation for Active Crustal Regions. Earthquake Spectra: August 2014, Vol. 30, No. 3, pp. 1025-1055.

American Society of Civil Engineers (ASCE), 2016, Minimum Design Loads for Buildings and Other Structures, ASCE Standard 7-16. American Society of Civil Engineers/Structural Engineering Institute. Reston, VA.

American Society of Civil Engineers (ASCE), 2018, ASCE Standard 7-16 - Minimum Design Loads for Buildings and Other Structures – Supplement 1 Effective December 12, 2018. American Society of Civil Engineers/Structural Engineering Institute. Reston, VA.

Boore, D.M., Jonathan P. Stewart, Emel Seyhan, and Gail M. Atkinson (2014), NGA-West2 Equations for Predicting PGA, PGV, and 5% Damped PSA for Shallow Crustal Earthquakes. Earthquake Spectra: August 2014, Vol. 30, No. 3, pp. 1057-1085.

Campbell, K.W. and Yousef Bozorgnia (2014), NGA-West2 Ground Motion Model for the Average Horizontal Components of PGA, PGV, and 5% Damped Linear Acceleration Response Spectra. Earthquake Spectra: August 2014, Vol. 30, No. 3, pp. 1087-1115.

Carlton, B. (2014), An Improved Description of the Seismic Response of Sites with High Plasticity Soils, Organic Clays, and Deep Soft Soil Deposits. PhD Thesis. University of California, Berkeley.

Chiou, B. S. J. and Robert R. Youngs (2014), Update of the Chiou and Youngs NGA Model for the Average Horizontal Component of Peak Ground Motion and Response Spectra. Earthquake Spectra: August 2014, Vol. 30, No. 3, pp. 1117-1153.

Darendeli M.B. (2001), Development of a New Family of Normalized Modulus Reduction and Material Damping Curves. Department of Civil, Architectural and Environmental Engineering, The University of Texas, Austin, Texas.

Hartzell, S., Harmsen, S., Williams, R.A., Carver, D., Frankel, A., Choy, G., Liu, P.C., Jachnes, R.C., Brocher, T.M., and Wentworth, C.M. (2006), Modeling of a 3D Velocity Structure for the Santa Clara Valley, California, for Seismic-Wave Simulations. Bulletin of the Seismological Society of America, vol. 96, no. 5.

Hashash, Y.M.A., Musgrove, M.I., Harmon, J.A., Ilhan, O., Groholski, D.R., Phillips, C.A., and Park, D. (2017), DEEPSOIL 7.0, User Manual.

IBC (2018), 2018 International Building Code. International Code Council. Country Club Hills, IL.

Matasovic, N., and Vucetic, M. (1993), Cyclic Characterization of Liquefiable Sands. ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 119, No. 11, pp 1805-1822.

Matasovic, N., and Vucetic, M. (1995), Generalized Cyclic Degradation-Pore Pressure Generation Model for Clays. ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 121, No. 1, pp 33-42.

Petersen, M.D., Moschetti, M.P., Powers, P.M., Mueller, C.S., Haller, K.M., Frankel, A.D., Zeng, Yuehua, Rezaeian, Sanaz, Harmsen, S.C., Boyd, O.S., Field, Ned, Chen, Rui, Rukstales, K.S., Luco, Nico, Wheeler, R.L., Williams, R.A., and Olsen, A.H. (2014), Documentation for the 2014 update of the United States national seismic hazard maps: U.S. Geological Survey Open-File Report 2014–1091, 243 p., https://dx.doi.org/10.3133/ofr20141091.

Phillips, C. and Hashash, Y. (2009), Damping Formulation for Non-Linear 1D Site Response Analyses. Soil Dynamics and Earthquake Engineering, Vol. 29, pp 1143-1158.

Shahi, S.K. and Baker, J.W. (2014), NGA-West2 Models for Ground Motion Directionality. Earthquake Spectra, 30(3), pp. 1285-1300.

Working Group on California Earthquake Probabilities (2003), USGS Open-File Report 03-214.

Table 2. Acceleration Response Spectral Values (5% damping) and Factors

Period (sec)	DSHA 84^{th}-Percentile Geomean MCE (g) for Vs = 1,200 ft/sec	MRC Factor	Input Base Motion MCE_R (g)	Surface to Input Base Motion Site Amplification Factor	Surface Site-Specific MCE_R (g)	2/3 Site-Specific MCE_R (g) for Vs = 1,200 ft/sec	80% ASCE 7-16 Site Class D DRS (g)	Site-Specific DRS (g)
0.01	0.657	1.19	0.782	0.458	0.358	0.239	0.344	0.344
0.02	0.663	1.19	0.789	0.456	0.360	0.240	0.368	0.368
0.03	0.691	1.19	0.822	0.458	0.377	0.251	0.392	0.392
0.05	0.796	1.19	0.947	0.370	0.351	0.234	0.440	0.440
0.075	0.973	1.19	1.158	0.360	0.417	0.278	0.500	0.500
0.1	1.129	1.19	1.344	0.335	0.450	0.300	0.560	0.560
0.15	1.367	1.20	1.641	0.436	0.715	0.477	0.680	0.680
0.2	1.509	1.21	1.826	0.441	0.805	0.537	0.800	0.800
0.25	1.587	1.22	1.936	0.453	0.876	0.584	0.800	0.800
0.3	1.617	1.22	1.972	0.423	0.833	0.556	0.800	0.800
0.4	1.564	1.23	1.924	0.424	0.815	0.544	0.800	0.800
0.5	1.450	1.23	1.784	0.558	0.996	0.664	0.800	0.800
0.75	1.133	1.24	1.405	0.709	0.996	0.664	0.800	0.800
1	0.909	1.24	1.127	0.770	0.868	0.579	0.800	0.800
1.5	0.611	1.24	0.757	1.258	0.953	0.635	0.533	0.635
2	0.443	1.24	0.549	1.391	0.764	0.509	0.400	0.509
3	0.286	1.25	0.358	1.446	0.518	0.345	0.267	0.345
4	0.205	1.26	0.258	1.309	0.338	0.225	0.200	0.225
5	0.159	1.26	0.200	1.243	0.249	0.166	0.160	0.166

Search Information

Coordinates: 37.3781, -121.9332

Elevation: 28 ft

Timestamp: 2020-11-20T05:35:08.123Z

Hazard Type: Seismic

Reference Document: ASCE7-16

Risk Category: II

Site Class: D

Basic Parameters

Name	Value	Description
S_S	1.5	MCE_R ground motion (period=0.2s)
S_1	0.6	MCE_R ground motion (period=1.0s)
S_{MS}	1.5	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	1	Numeric seismic design value at 0.2s SA
S_{D1}	* null	Numeric seismic design value at 1.0s SA

* See Section 11.4.8

Additional Information

Name	Value	Description
SDC	* null	Seismic design category
F_a	1	Site amplification factor at 0.2s
F_v	* null	Site amplification factor at 1.0s
CR_S	0.95	Coefficient of risk (0.2s)
CR_1	0.929	Coefficient of risk (1.0s)
PGA	0.582	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.64	Site modified peak ground acceleration
T_L	12	Long-period transition period (s)
SsRT	2.202	Probabilistic risk-targeted ground motion (0.2s)
SsUH	2.317	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)

SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.816	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.878	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.6	Factored deterministic acceleration value (1.0s)
PGAd	0.582	Factored deterministic acceleration value (PGA)

* See Section 11.4.8

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.

While the information presented on this website is believed to be correct, ATC and its sponsors and contributors assume no responsibility or liability for its accuracy. The material presented in the report should not be used or relied upon for any specific application without competent examination and verification of its accuracy, suitability and applicability by engineers or other licensed professionals. ATC does not intend that the use of this information replace the sound judgment of such competent professionals, having experience and knowledge in the field of practice, nor to substitute for the standard of care required of such professionals in interpreting and applying the results of the report provided by this website. Users of the information from this website assume all liability arising from such use. Use of the output of this website does not imply approval by the governing building code bodies responsible for building code approval and interpretation for the building site described by latitude/longitude location in the report.

This page was left blank intentionally.

APPENDIX 7.2

GEOTECHNICAL REPORT ADDENDUM

January 7, 2021

Burns & McDonnell Engineering Company, Inc.
530 W. Spring Street, Suite 200
Columbus, Ohio 43215

Attn: Ms. Brandi Sauter
P: (614) 499 1009
E: blsauter@burnsmcd.com

Subject: Geotechnical Engineering Report Addendum – Pavement
Bay AZ1 Site (Northtown)
Orchard Parkway
San Jose, California
Terracon Project No. ND205079

Dear Ms. Sauter:

Terracon Consultants, Inc. (Terracon) prepared a geotechnical engineering report (Project No. ND205079, Dated December 15, 2020 for the subject project. Our geotechnical engineering report provided recommendations concerning earthwork and the design and construction of foundations and floor slabs. At the time our report was published, pavement recommendations were still being prepared for the project. This addendum provides pavement recommendations for use for the subject project.

PAVEMENTS

General Pavement Comments

Concrete and asphalt pavement alternatives have been considered for this project. Pavement thickness design is dependent upon:

- the anticipated traffic conditions during the life of the pavement,
- subgrade and paving material characteristics, and
- climatic conditions of the region.

A critical aspect of pavement performance is site preparation. Pavement designs noted in this section must be applied to the site which has been prepared as recommended in the **Earthwork** section of our December 15, 2020 geotechnical engineering report.

Support characteristics of subgrade for pavement design do not account for shrink/swell movements of an expansive clay subgrade, such as the soils encountered on this project. Thus, the pavement may be adequate from a structural standpoint, yet still experience cracking and deformation due to shrink/swell related movement of the subgrade. Mitigation measures to reduce the potential for differential subgrade expansion are discussed in the **Earthwork** section of our December 15, 2020 report.

The pavement sections were designed using the American Association of State and Highway Transportation Officials (AASHTO) Guide for Design of Pavement Structures (1993). Development of layer thicknesses were determined using the layered elastic design methodology as outlined in the AASHTO 93 Design Guide, Part II, Section 3.1.5 Layered Design Analysis.

Design Traffic Analysis

Terracon assumed traffic frequencies for the light-duty and heavy-duty based on an HS-20 traffic loading as requested in the Site Due Diligence and Master Planning Scope of Services document for the project. These traffic frequencies were converted into flexible AASHTO pavement 18-kip equivalent single axle loads (ESALs) for use in asphalt concrete (AC) pavement thickness design, and into rigid AASHTO pavement 18-kip ESALs for portland cement concrete (PCC) design, as noted in the following table. Terracon should be contacted if there are any changes in the reported traffic patterns or frequency to review the enclosed values. A more detailed calculation of the design ESALs is attached to this addendum.

Design Equivalent Single-Axle Loads (ESALs) for Light and Heavy Duty Pavements				
Vehicle	Light Duty		Heavy Duty	
	Flexible	Rigid	Flexible	Rigid
Autos/Light Trucks	1,098	1,441	1,098	1,441
Light Delivery and Trash Collection	--	--	145,939	198,897
Tractor-Trailer Trucks	--	--	59,905	107,018
Total	**1,098**	**1,441**	**206,942**	**307,356**

1. Assumes 20-year design life

Pavement Subgrade Parameters

Subgrade support was estimated from laboratory-prepared remolded California Bearing Ratio (CBR) tests of collected composite bulk samples. CBR values of 7 and 8 were obtained from these tests. A design CBR value of 7 was used as the basis for pavement design. This value corresponds to a subgrade Resilient Modulus (M_r) of 8,877 psi (pounds per square inch) for use for flexible pavement design. An Effective Modulus of Subgrade Reaction (k) of 146 pci (pounds per cubic inch) at the top of the aggregate base layer was used in designing the rigid pavement sections.

Note that if actual subgrade conditions differ from the soil conditions and characteristics described here, we should be contacted to assess the construction conditions and review the pavement design recommendations.

Pavement Design Parameters

Analyses for the pavement design of the project have been based on the procedures of the AASHTO Guide for Design of Pavement Structures (1993). The following design parameters were utilized for pavement engineering analyses and the determination of design alternatives for the project:

Pavement Design Parameters		
Reliability	Level of Reliability	85%
	Flexible Overall Standard Deviation	0.45
	Rigid Overall Standard Deviation	0.35
Serviceability	Flexible Initial PSI	4.2
	Flexible Terminal PSI	2.0
	Rigid Terminal PSI	2.5
	Rigid Initial PSI	4.5
Subgrade Conditions	Subgrade Design CBR	7
	Correlated Resilient Modulus, M_r	8,877 psi
	Effective Modulus of Subgrade Reaction at the Top of the Aggregate Base Course, k	146 pci
Layer Properties	Asphalt Concrete (AC) Layer Coefficient	0.44
	Aggregate Base (ABC) Layer Coefficient	0.14
	Aggregate Base (ABC) Drainage Coefficient	1.0
	Aggregate Base (ABC) Resilient Modulus [1]	36,000 psi
	Load Transfer Coefficient J [2]	2.8 (doweled) 3.9 (non-doweled)
	Compressive Strength of Concrete f'_c	4,000 psi

1. Reinforced AB Resilient Modulus values are limited to 5x the subgrade Resilient Modulus for the purposes of layered design analysis for flexible pavements.
2. Load transfer coefficient of 2.8 for dowel reinforced concrete joints.

The design period is considered the interval over which, with proper maintenance, the pavement will not require major repairs. We recommend a continuing regular maintenance program be implemented to maintain satisfactory serviceability over the design life. Please refer to **Pavement Maintenance** for additional information.

Asphalt Concrete Pavement Recommendations

The following table provides our recommended flexible pavement sections for this project:

Flexible Pavement Thickness Design Recommendations for Bay AZ1 Site					
LAYER	Materials [2] (CalTrans Grading)	Thickness (in) [1]			
		Light Duty		Heavy Duty	
		Alt A	Alt B	Alt C	Alt D
Surface Course[2]	CalTrans Section 39, Type A HMA, ½ inch, PG 64-16	3	3	4	4
Aggregate [2]	CalTrans Section 26 Aggregate Base Class 2	6	6	8	6
Chemically Stabilized Subgrade	Prepared in accordance with the **Earthwork** section of our December 15, 2020 report.	--	12	--	12
Total Pavement Section (in.)		**9**	**21**	**12**	**22**

1. The individual and total material thickness values presented herein represent minimum thickness values, not averages.
2. Materials should meet the Standard Specifications State of California State Transportation Agency Department of Transportation, Sections 39 for Asphalt and Section 26 for Aggregate Base Course.

Terracon considered the climate conditions and traffic to determine the appropriate asphalt binder for this project. This was accomplished using the LTPPBind Version 3.1 Beta software, dated September 15, 2015 provided by the Federal Highway Administration (FHWA). This software utilizes historical temperature data from the 5 weather stations nearest the project and considers traffic speed and traffic loading to establish a recommended Performance Graded (PG) binder grade of asphalt concrete. Terracon then compared the software output to the binders that were indicated to be locally available, based on the CalTrans website, to determine the recommended binder selection for the project. The number of binders selected was limited for this recommendation to reduce the number of mix designs needed to construct the pavements.

Portland Cement Concrete Pavement Recommendations

The following table outlines our recommendations for Jointed Plain Concrete Pavements (JCPC) for the project.

Rigid Pavement Thickness Design Recommendations for Bay AZ1 Site			
LAYER	**MATERIAL**	**Thickness (in)** [1]	
		LIGHT DUTY	**HEAVY DUTY**[3]
Surface Course	Portland Cement Concrete [2, 3]	5	6½
Aggregate	CalTrans Section 26 Aggregate Base Class 2 [4]	4	4
Subgrade	Prepared in accordance with the **Earthwork** section of our December 15, 2020 report.		
Total Pavement Section [1]		**9**	**10**

1. The individual and total material thickness values presented herein represent minimum thickness values, not averages.
2. The concrete should meet the Standard Specifications State of California State Transportation Agency Department of Transportation Section 40. It should be air entrained in accordance with a minimum compressive strength of 4,000 psi after 28 days of laboratory curing per ASTM C-31.
3. Heavy-duty concrete pavements should include 1-inch diameter by 13-inch long dowel bars spaced at 12 inches center to center in all longitudinal and transverse contraction joints.
4. Materials should meet the Standard Specifications State of California State Transportation Agency Department of Transportation, Section 26 for Aggregate Base Course.

The recommendations presented above require dowel reinforcement in longitudinal and transverse contraction joints as shown in ACI 330.2R-17. In locations where concrete slabs are used in isolated areas such as dumpster pads, joint reinforcement is not required. In these locations however, an additional two inches should be added to the thicknesses presented in the table above to alleviate cracking of unsupported edges.

If Portland cement concrete is selected for use in general pavement areas, proper design and detailing of longitudinal and transverse control joints, tie bars and joint dowels will be required. In this situation, we should be contacted to provide more detailed recommendations and to review final jointing plans and details for the project. The following general recommendations are presented for doweled PCC pavements:

Jointed Plain Concrete Pavement Jointing Requirements for Heavy Duty Pavements	
Item	**Description**
Contraction Joints	■ Joints should be reinforced with dowels in accordance with ACI 330.2R-17[1]. ■ Alternate joint reinforcement devices such as plate dowels will be considered if the device manufacturers recommendations showing they are equivalent to the dowel bar size and spacing presented in the concrete pavement section table above is submitted and approved by the engineer. ■ Joint cuts should be 1/5 of the depth of the concrete and should be cut as soon as the slab can support the weight of a man and the saw and can be cut without dislodging coarse aggregate particles from the surface. ■ Joints should have a maximum spacing no greater than 15 feet for heavy-duty sections or 12½ feet for light duty sections, as described in ACI 330.2R
Expansion (Isolation) Joints	■ Expansion joints are recommended to isolate fixed objects abutting or within the paved area, such as around light poles and drainage inlet structures. ■ Joints should be full depth and filled with pre-molded materials per ACI 330.2R-17. ■ Pavement edges at expansion joints located in areas that encounter wheel loads should be thickened by two inches wherever practical; the transition in thickness should occur over a minimum distance of five feet.
Construction Joints	■ Joints dowels should be provided at the same size and spacing as required for Contraction Joints as noted above. ■ For a butt end construction joint, an adequate number of ½ inch diameter (#4 bar) deformed steel tie bars, 30 inches in length and spaced no greater than 36 inches apart, are also recommended to tie the exterior curb and gutter to the outer concrete pavement edge to keep the outside slab from separating from the curb and gutter.

[1] *Guide for the Design and Construction of Concrete Site Paving for Industrial and Trucking Facilities, American Concrete Institute, ACI 330.2R-17.*

Pavement Drainage

Pavements should be sloped to provide rapid drainage of surface water. Water allowed to pond on or adjacent to the pavements could saturate the subgrade and contribute to premature pavement deterioration. In addition, the pavement subgrade should be graded to provide positive drainage within the granular base section. Appropriate sub-drainage or connection to a suitable daylight outlet should be provided to remove water from the granular subbase.

Based on the possibility of shallow and/or perched groundwater, we recommend installing a pavement subdrain system to control groundwater, improve stability, and improve long-term pavement performance.

Cold Weather Paving

Asphalt Concrete Pavement

Construction and quality of pavements, especially concrete pavements, can be negatively impacted when colder temperatures exist at the time of material placement.

The primary concern for asphalt concrete is the ability to adequately densify the pavement layer before it cools below the minimum allowable temperature for compaction. We recommend that pavement construction be performed in accordance with the *Standard Specifications State of California State Transportation Agency Department of Transportation, Section 39.*

When circumstances dictate, asphalt pavement construction during cold weather conditions may be necessary. However, placing asphalt in cold weather will increase the risk that adequate compaction is not achieved, resulting in a higher probability of premature cracking and increase rutting, stripping and raveling. If these increased risks are acceptable, we recommend that the following practices be observed:

- Compaction time should be calculated to determine the compaction equipment needed to complete compaction within the limited time. Tools are available for estimating the compaction. One example is the PaveCool application published by the Minnesota Department of Transportation: http://dot.state.mn.us/app/pavecool/index.html. Paving is not recommended if the calculated compaction time is less than required to adequately compact the pavement layer.
- Compaction of the asphalt should be avoided when the mixture temperature is less than 185º F.
- Warm mix asphalt additives can be used as a compaction aide.
- Additional and higher capacity rollers may be required, staged immediately behind the paver to provide immediate compaction.
- Layer lift thickness less than 2 inches should not be attempted.
- Increase the mix temperature and tightly control the mix temperature to reduce variability.
- Asphalt concrete loads should be tightly tarped to maintain uniform temperatures throughout the load. Tarps should tightly cover the load and seal over the sides of the truck bed.
- Minimize the time/length of haul to the jobsite.
- Haul trucks should be staged to unload immediately upon arrive at the job site.
- If the asphalt concrete is to be placed on an aggregate base, the aggregate base materials must not be excessively wet or below minimum allowable temperatures.
- Do not place asphalt concrete on frozen subgrade.

- Hand-worked areas should be avoided during cold weather conditions. If hand-work is necessary, these areas should be considered temporary and subject to replacement when favorable weather conditions permit.

Pavement Maintenance

The pavement sections represent minimum recommended thicknesses and, as such, periodic maintenance should be anticipated. Therefore, preventive maintenance should be planned and provided for through an on-going pavement management program. Maintenance activities are intended to slow the rate of pavement deterioration and to preserve the pavement investment. Maintenance consists of both localized maintenance (e.g., crack and joint sealing and patching) and global maintenance (e.g., surface sealing). Preventive maintenance is usually the priority when implementing a pavement maintenance program. Additional engineering observation is recommended to determine the type and extent of a cost-effective program. Even with periodic maintenance, some movements and related cracking may still occur, and repairs may be required.

Pavement performance is affected by its surroundings. In addition to providing preventive maintenance, the civil engineer should consider the following recommendations in the design and layout of pavements:

- Slope final grade adjacent to paved areas down from the edges at a minimum 2%.
- Slope subgrade and pavement surfaces with a minimum grade of 2% to promote proper surface drainage.
- Install below pavement drainage systems surrounding areas anticipated for frequent wetting.
- Install joint sealant and seal cracks immediately.
- Seal all landscaped areas in or adjacent to pavements to reduce moisture migration to subgrade soils.
- Place compacted, low permeability backfill against the exterior side of curb and gutter.
- Place curb, gutter and/or sidewalk directly on clay subgrade soils rather than on unbound granular base course materials.

All other conclusions, recommendations, and limitations presented in our December 15, 2020 geotechnical engineering report are still valid and should be followed.

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this addendum, or if we may be of further service, please contact us.

Sincerely,
Terracon Consultants, Inc.

Noah T. Smith, P.E., G.E.	Kirk D. Jackson, P.E.
Principal	Project Engineer

Attached: CBR test results
Design ESAL calculations

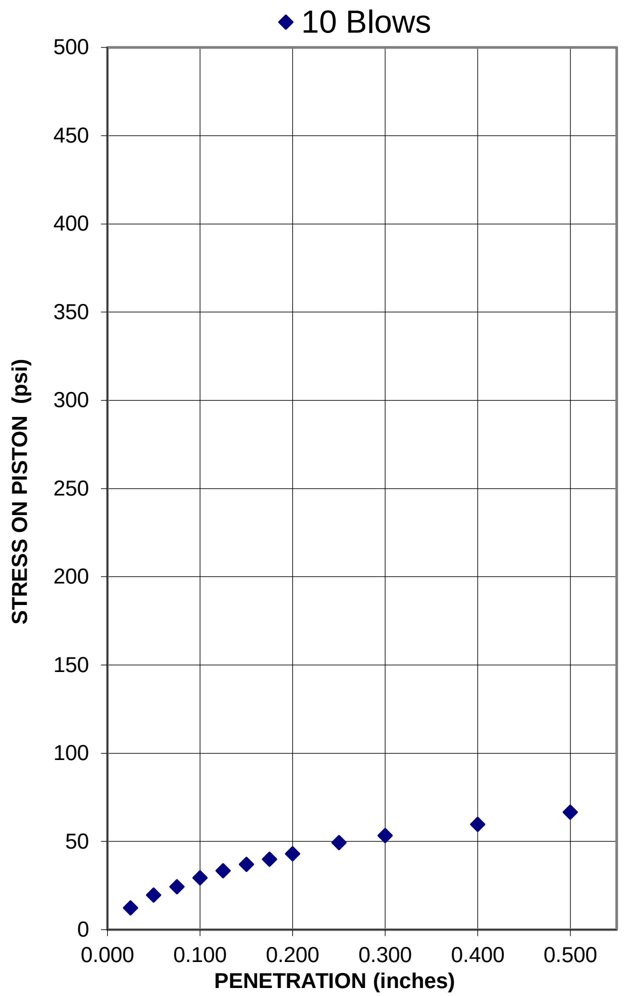
CBR(CALIFORNIA BEARING RATIO) OF LABORATORY-COMPACTED SOILS
ASTM D1883 (SOAKED)

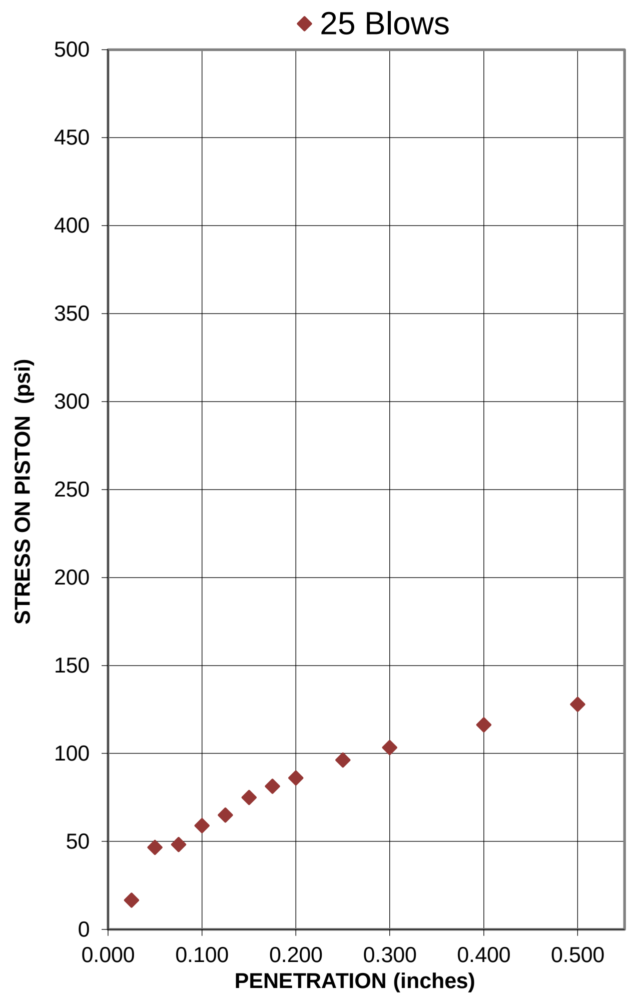
Terracon

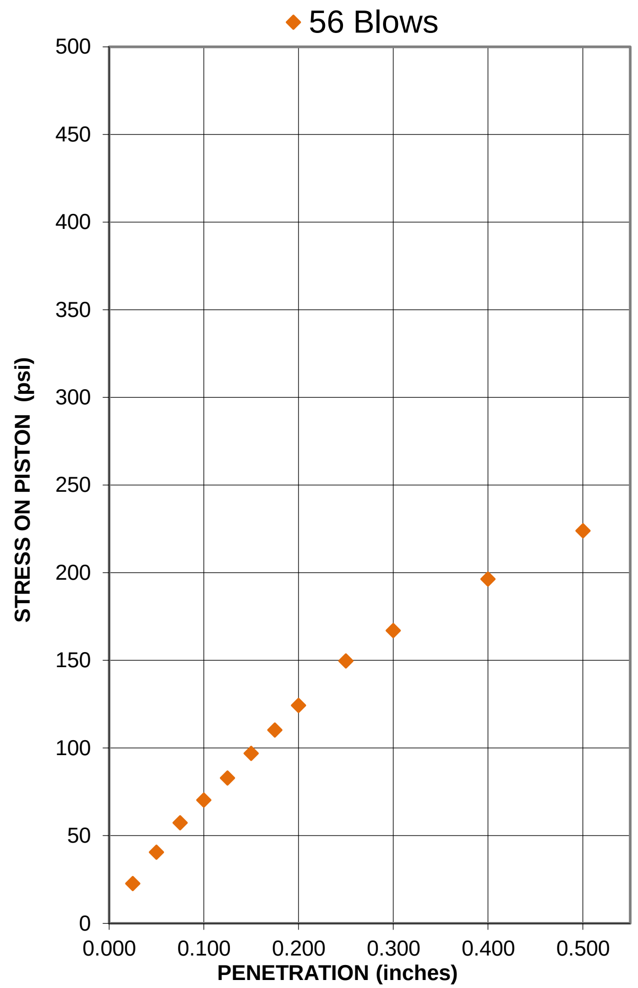
PROJECT: Bay AZ1 Site (Northtown)
LOCATION: P1/P2
MATERIAL:
SAMPLE SOURCE:

PROJECT NO: ND205079
WORK ORDER NO:
LAB NO:
DATE SAMPLED:

	Blows per Lift	Compaction %	Compaction Method	Surcharge Weight (Lb.)	Sample Condition	Percent Swell	Before Soak Dry Density	Before Soak Moisture %	After Soak Dry Density	After Soak Moisture %
Point 1	10	89.6	AASHTO T180	10	Soak	2.47	112.8	8.5	111.5	19.1
Point 2	25	94.2	AASHTO T180	10	Soak	2.44	118.6	8.7	115.3	19.1
Point 3	56	100.8	AASHTO T180	10	Soak	2.23	126.9	8.6	121.6	16.4







REVIEWED BY

CBR(CALIFORNIA BEARING RATIO) OF LABORATORY-COMPACTED SOILS
ASTM D1883 (SOAKED)

Terracon

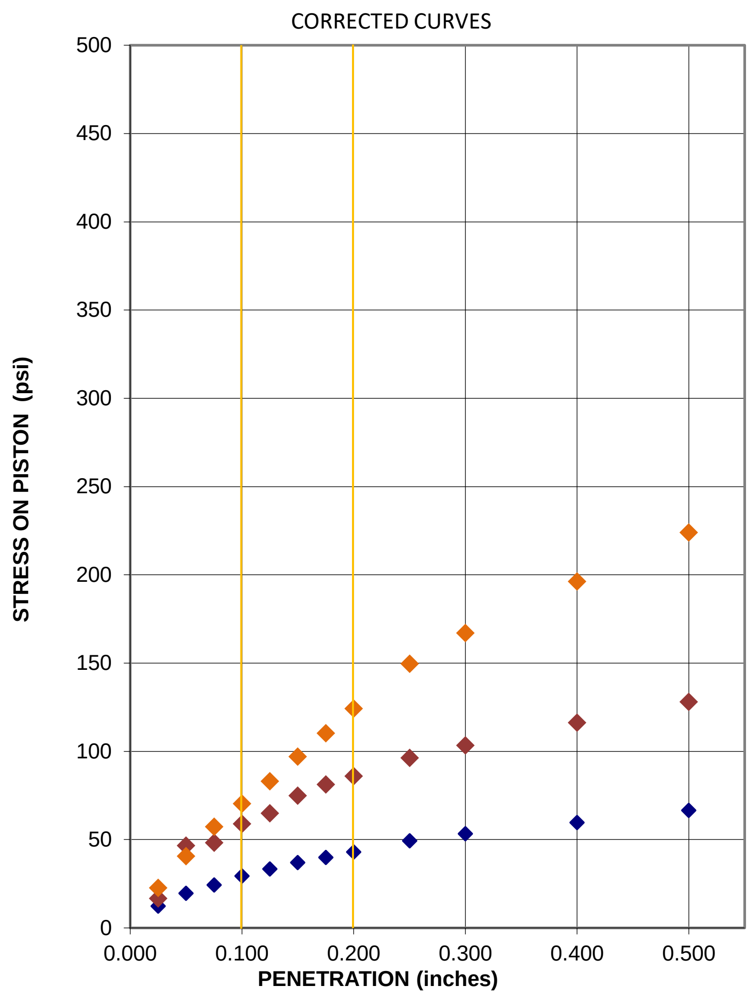
PROJECT:	**Bay AZ1 Site (Northtown)**	**JOB NO:**	**ND205079**
LOCATION:	**P1/P2**	**WORK ORDER NO:**	
MATERIAL:		**LAB NO:**	
SAMPLE SOURCE:		**DATE SAMPLED:**	

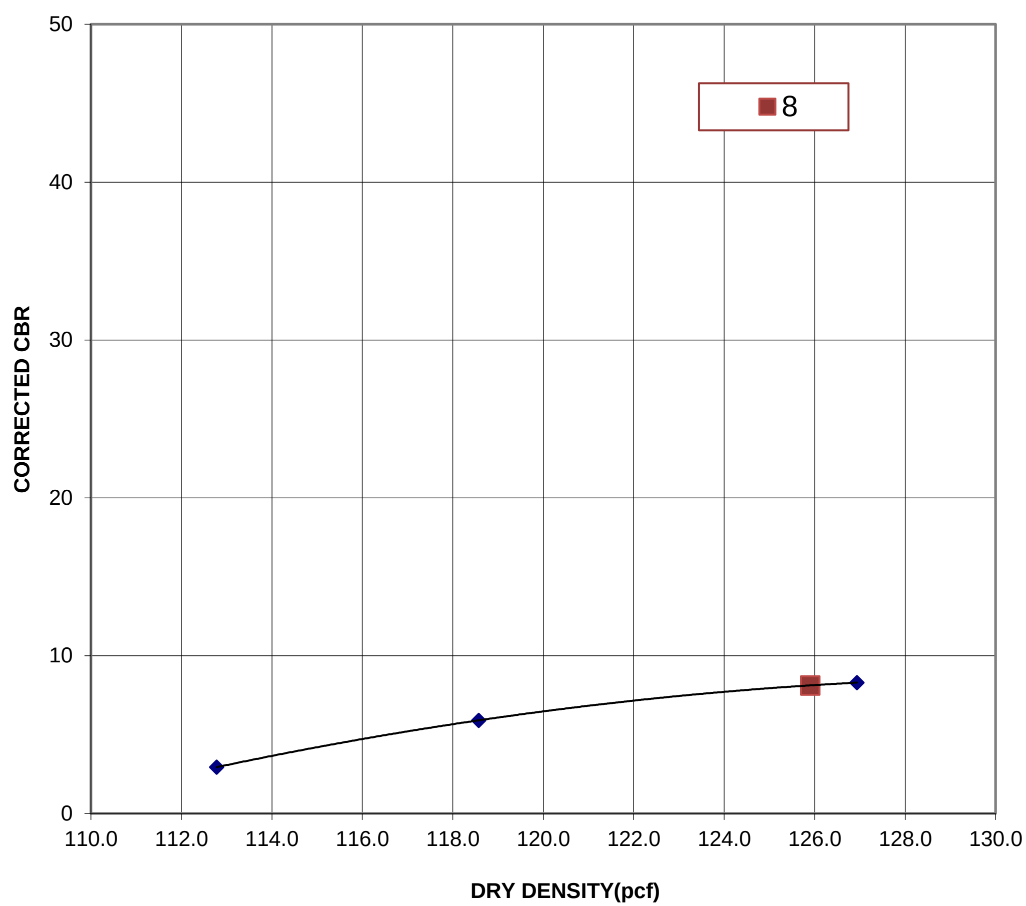
	Correction Factor 0.1"	Correction Factor 0.2"	Corrected CBR 0.1"	Corrected CBR 0.2"	CBR
Point 1	0.00	0.00	3	3	3
Point 2	0.00	0.00	6	6	6
Point 3	0.00	0.00	7	8	8

AASHTO T180				
Moisture	Dry Density (pcf)			
%	100%	95%	90%	
6.6	125.9	119.6	113.3	

100% COMPACTION CBR (.2")

8





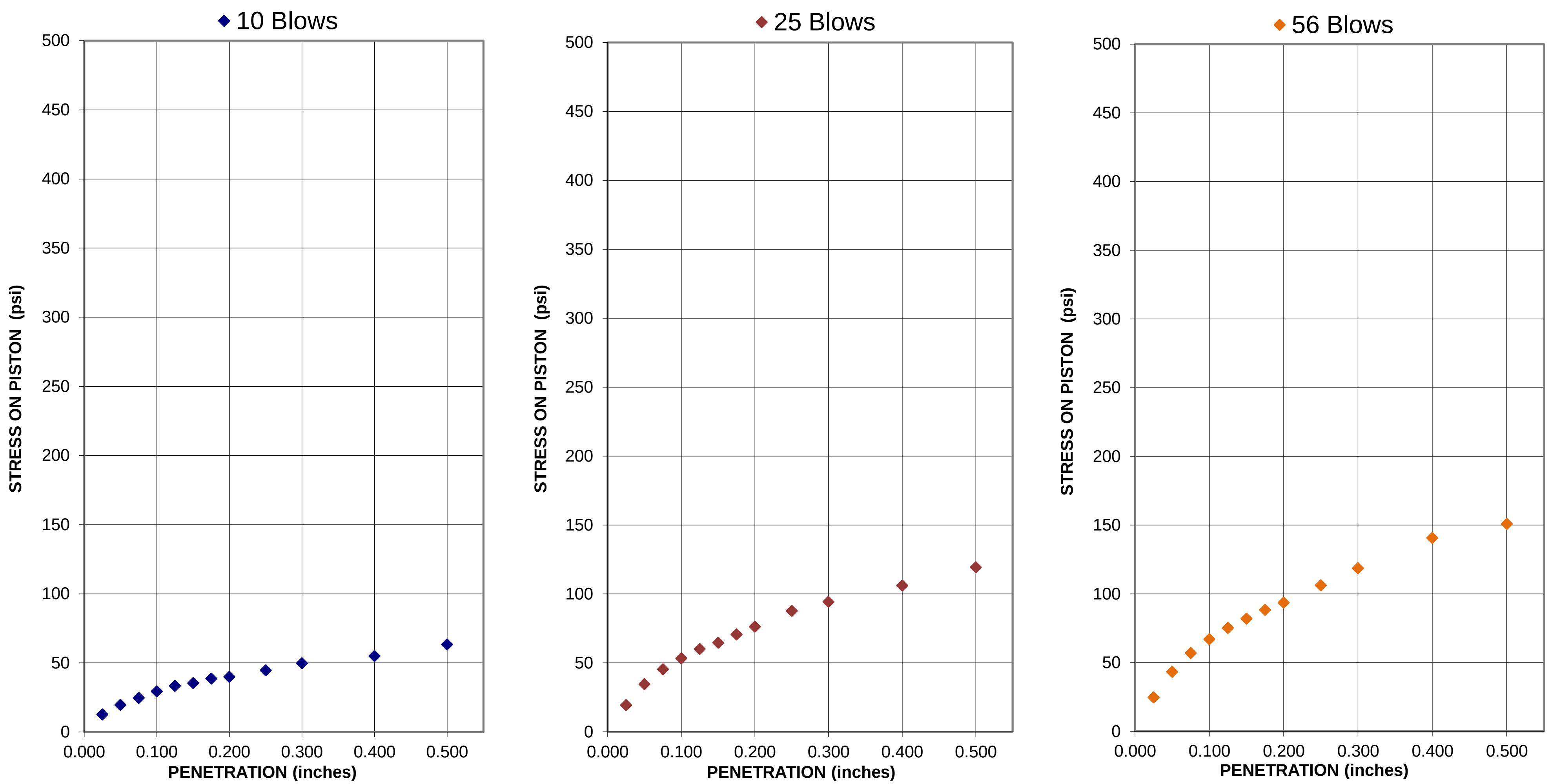
REVIEWED BY

CBR(CALIFORNIA BEARING RATIO) OF LABORATORY-COMPACTED SOILS
ASTM D1883 (SOAKED)

PROJECT: Bay AZ1 Site (Northtown)
LOCATION: P4
MATERIAL:
SAMPLE SOURCE:

PROJECT NO: ND205079
WORK ORDER NO:
LAB NO:
DATE SAMPLED:

	Blows per Lift	Compaction %	Compaction Method	Surcharge Weight (Lb.)	Sample Condition	Percent Swell	Before Soak		After Soak	
							Dry Density	Moisture %	Dry Density	Moisture %
Point 1	10	88.3	AASHTO T180	10	Soak	2.25	115.2	8.4	112.1	19.5
Point 2	25	92.2	AASHTO T180	10	Soak	1.51	120.4	12.3	118.2	19.3
Point 3	65	100.6	AASHTO T180	10	Soak	1.79	131.2	10.3	123.4	16.6



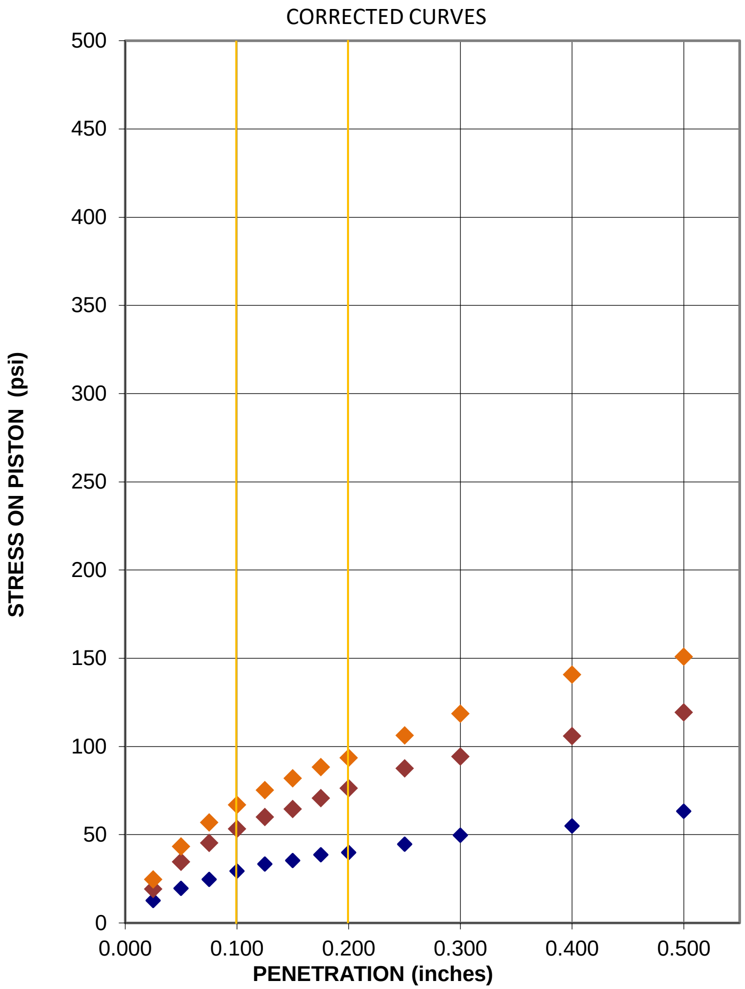
REVIEWED BY

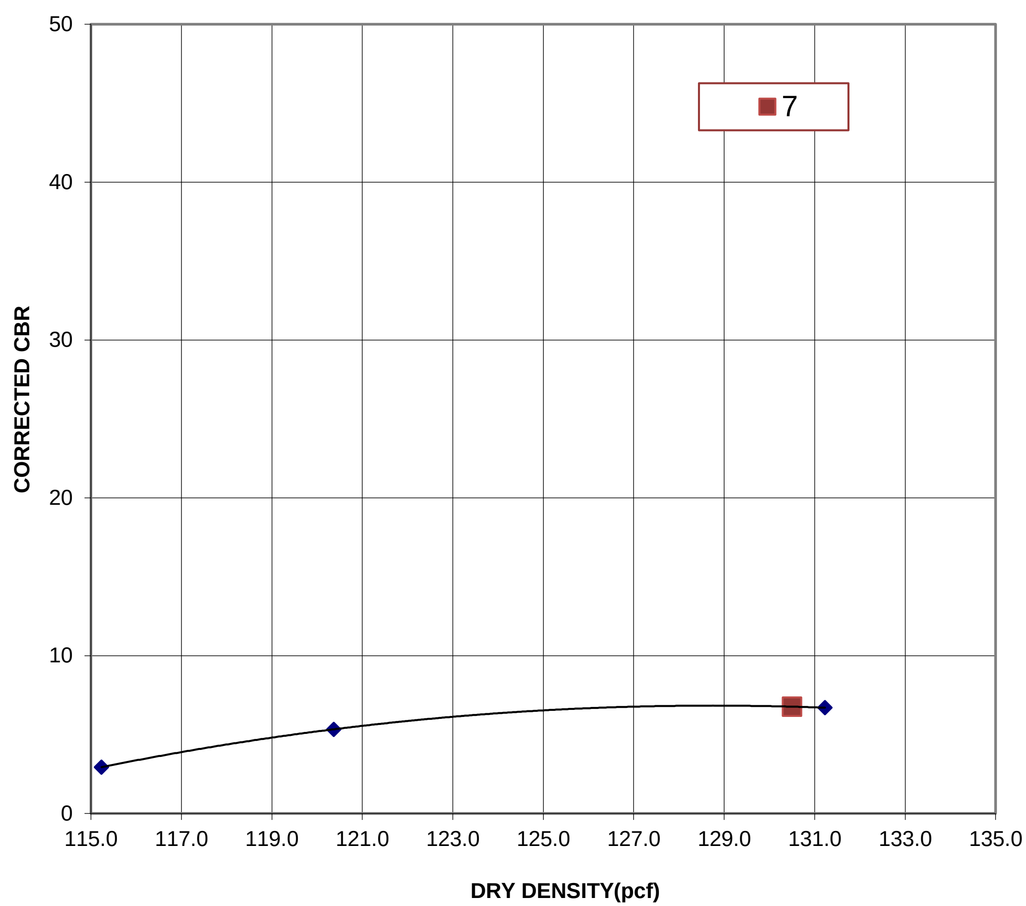
CBR(CALIFORNIA BEARING RATIO) OF LABORATORY-COMPACTED SOILS
ASTM D1883 (SOAKED)

PROJECT:	Bay AZ1 Site (Northtown)	JOB NO:	ND205079
LOCATION:	P4	WORK ORDER NO:	
MATERIAL:		LAB NO:	
SAMPLE SOURCE:		DATE SAMPLED:	

	Correction Factor		Corrected CBR		
	0.1"	0.2"	0.1"	0.2"	CBR
Point 1	0.00	0.00	3	3	3
Point 2	0.00	0.00	5	5	5
Point 3	0.00	0.00	7	6	7

AASHTO T180				
Moisture	Dry Density (pcf)			
%	100%	95%	90%	100% COMPACTION CBR (.2")
8.3	130.5	124.0	117.5	7





REVIEWED BY

AASHTO 1993 ESAL Calculator for Flexible Pavements

Vehicle Description	Traffic Volume: Quantity in the Design Lane	Traffic Volume: Days per Week	Traffic Volume: Weeks per Year	Analysis Period (years)	Axle Load and Type: Axle 1 (kips)		Axle 2 (kips)		Axle 3 (kips)		Gross Weight (pounds)	Equivalency Factors: Axle 1	Axle 2	Axle 3	ESAL's
Passenger car	**500**	**7**	**52**	**20**	2	S	2	S			4,000	0.0002	0.0002	0	1,098
Pick-up truck or van					2	S	4	S			6,000	0.0002	0.0017	0	0
Recreational vehicle					4	S	4	S			8,000	0.0017	0.0017	0	0
School bus					6	S	14	S			20,000	0.0087	0.3294	0	0
TARC bus					8	S	14	S			22,000	0.029	0.3294	0	0
Greyhound MC-12 bus					13.4	S	18.4	S	6	S	37,800	0.2715	1.102	0.0087	0
Package delivery truck					4	S	14	S			18,000	0.0017	0.3294	0	0
Beverage delivery truck					6	S	12	S	12	S	30,000	0.0087	0.1672	0.1672	0
Garbage/dumpster truck	**50**	**1**	**52**	**20**	20	S	35	T			55,000	1.592	1.2145	0	145,939
Concrete truck (full)					20	S	48	R			68,000	1.592	0.9848	0	0
Dump truck (full)					20	S	48	R			68,000	1.592	0.9848	0	0
Semi-tractor (no trailer)					8	S	2	T			10,000	0.029	3E-05	0	0
Semi-tractor trailer (empty)					8	S	8	T	6	T	22,000	0.029	0.0024	0.0008	0
Semi-tractor trailer	**25**	**1**	**52**	**20**	12	S	34	T	34	T	80,000	0.1672	1.0684	1.0684	59,905
M1097 HMMWV					3.5	S	6.5	S			10,000	0.0011	0.0121	0	0
Diesel Forklifts					26.432	S	134.41	S			160,843	5.3909	7686.3	0	0
Fire Truck (aerial platform)					24.3	S	28.35	S	28.35	S	81,000	3.74	7.298	7.298	0
Vehicle type H10					4	S	16	S			20,000	0.0017	0.5941	0	0
Vehicle type H15					6	S	24	S			30,000	0.0087	3.5427	0	0
Vehicle type H20					8	S	32	S			40,000	0.029	12.278	0	0
Vehicle type 3					16	S	34	T			50,000	0.5941	1.0684	0	0
Vehicle type HS15					6	S	24	S	24	S	54,000	0.0087	3.5427	3.5427	0
Vehicle type HS20					8	S	32	S	32	S	72,000	0.029	12.278	12.278	0
Vehicle type 3S2					10	S	31	T	31	T	72,000	0.0755	0.7102	0.7102	0

Terminal Serviceability, r_t	**2.0**
Assumed Structural Number, SN	**6.3**
Traffic Growth Rate, %/yr	**0**

Summary:		
	Total AASHTO ESAL's	**206,942**
	Traffic Category	**Heavy Duty**

Project: Bay AZ1 Site (Northtown) Location:

Job No.: ND205079 Date: 1/8/2021

Terracon

AASHTO 1993 ESAL Calculator for Concrete Pavements

Vehicle Description	Traffic Volume: Quantity in the Design Lane	Traffic Volume: Days per Week	Traffic Volume: Weeks per Year	Analysis Period (years)	Axle Load and Type: Axle 1 (kips)		Axle 2 (kips)		Axle 3 (kips)		Gross Weight (pounds)	Equivalency Factors: Axle 1	Axle 2	Axle 3	ESAL's
Passenger car	**500**	**7**	**52**	**20**	2	S	2	S			4,000	0.0002	0.0002	0	1,441
Pick-up truck or van					2	S	4	S			6,000	0.0002	0.0021	0	0
Recreational vehicle					4	S	4	S			8,000	0.0021	0.0021	0	0
School bus					6	S	14	S			20,000	0.0099	0.3356	0	0
TARC bus					8	S	14	S			22,000	0.0317	0.3356	0	0
Greyhound MC-12 bus					13.4	S	18.4	S	6	S	37,800	0.2779	1.101	0.0099	0
Package delivery truck					4	S	14	S			18,000	0.0021	0.3356	0	0
Beverage delivery truck					6	S	12	S	12	S	30,000	0.0099	0.1732	0.1732	0
Garbage/dumpster truck	**50**	**1**	**52**	**20**	20	S	35	T			55,000	1.5876	2.2373	0	198,897
Concrete truck (full)					20	S	48	R			68,000	1.5876	2.6059	0	0
Dump truck (full)					20	S	48	R			68,000	1.5876	2.6059	0	0
Semi-tractor (no trailer)					8	S	2	T			10,000	0.0317	8E-05	0	0
Semi-tractor trailer (empty)					8	S	8	T	6	T	22,000	0.0317	0.0053	0.0019	0
Semi-tractor trailer	**25**	**1**	**52**	**20**	12	S	34	T	34	T	80,000	0.1732	1.9714	1.9714	107,018
M1097 HMMWV					3.5	S	6.5	S			10,000	0.0013	0.0136	0	0
Diesel Forklifts					26.432	S	134.41	S			160,843	5.4504	7310.9	0	0
Fire Truck (aerial platform)					24.3	S	28.35	S	28.35	S	81,000	3.7522	7.4439	7.4439	0
Vehicle type H10					4	S	16	S			20,000	0.0021	0.5982	0	0
Vehicle type H15					6	S	24	S			30,000	0.0099	3.5512	0	0
Vehicle type H20					8	S	32	S			40,000	0.0317	12.774	0	0
Vehicle type 3					16	S	34	T			50,000	0.5982	1.9714	0	0
Vehicle type HS15					6	S	24	S	24	S	54,000	0.0099	3.5512	3.5512	0
Vehicle type HS20					8	S	32	S	32	S	72,000	0.0317	12.774	12.774	0
Vehicle type 3S2					10	S	31	T	31	T	72,000	0.0801	1.3191	1.3191	0

Terminal Serviceability, r_t	**2.0**
Assumed Pavement Thickness, in.	**15**
Traffic Growth Rate, %/yr	**0**

Summary:		
	Total AASHTO Rigid ESAL's	**307,356**
	Traffic Category	**Heavy Duty**

Project: Bay AZ1 Site (Northtown) Location:

Job No.: ND205079 Date: 1/8/2021

Terracon

AASHTO 1993 ESAL Calculator for Flexible Pavements

Vehicle Description	Traffic Volume			Analysis	Axle Load and Type						Gross	Equivalency Factors			ESAL's
	Quantity in the Design Lane	Days per Week	Weeks per Year	Period (years)	Axle 1 (kips)		Axle 2 (kips)		Axle 3 (kips)		Weight (pounds)	Axle 1	Axle 2	Axle 3	
Passenger car	**500**	**7**	**52**	**20**	2	S	2	S			4,000	0.0002	0.0002	0	1,098
Pick-up truck or van					2	S	4	S			6,000	0.0002	0.0017	0	0
Recreational vehicle					4	S	4	S			8,000	0.0017	0.0017	0	0
School bus					6	S	14	S			20,000	0.0087	0.3294	0	0
TARC bus					8	S	14	S			22,000	0.029	0.3294	0	0
Greyhound MC-12 bus					13.4	S	18.4	S	6	S	37,800	0.2715	1.102	0.0087	0
Package delivery truck					4	S	14	S			18,000	0.0017	0.3294	0	0
Beverage delivery truck					6	S	12	S	12	S	30,000	0.0087	0.1672	0.1672	0
Garbage/dumpster truck					20	S	35	T			55,000	1.592	1.2145	0	0
Concrete truck (full)					20	S	48	R			68,000	1.592	0.9848	0	0
Dump truck (full)					20	S	48	R			68,000	1.592	0.9848	0	0
Semi-tractor (no trailer)					8	S	2	T			10,000	0.029	3E-05	0	0
Semi-tractor trailer (empty)					8	S	8	T	6	T	22,000	0.029	0.0024	0.0008	0
Semi-tractor trailer					12	S	34	T	34	T	80,000	0.1672	1.0684	1.0684	0
M1097 HMMWV					3.5	S	6.5	S			10,000	0.0011	0.0121	0	0
Diesel Forklifts					26.432	S	134.41	S			160,843	5.3909	7686.3	0	0
Fire Truck (aerial platform)					24.3	S	28.35	S	28.35	S	81,000	3.74	7.298	7.298	0
Vehicle type H10					4	S	16	S			20,000	0.0017	0.5941	0	0
Vehicle type H15					6	S	24	S			30,000	0.0087	3.5427	0	0
Vehicle type H20					8	S	32	S			40,000	0.029	12.278	0	0
Vehicle type 3					16	S	34	T			50,000	0.5941	1.0684	0	0
Vehicle type HS15					6	S	24	S	24	S	54,000	0.0087	3.5427	3.5427	0
Vehicle type HS20					8	S	32	S	32	S	72,000	0.029	12.278	12.278	0
Vehicle type 3S2					10	S	31	T	31	T	72,000	0.0755	0.7102	0.7102	0

Terminal Serviceability, r_t	**2.0**
Assumed Structural Number, SN	**6.3**
Traffic Growth Rate, %/yr	**0**

Summary:		
	Total AASHTO ESAL's	**1,098**
	Traffic Category	**Light Duty**

Project: Bay AZ1 Site (Northtown) Location:

Job No.: ND205079 Date: 1/8/2021

Terracon

AASHTO 1993 ESAL Calculator for Concrete Pavements

Vehicle Description	Traffic Volume: Quantity in the Design Lane	Traffic Volume: Days per Week	Traffic Volume: Weeks per Year	Analysis Period (years)	Axle Load and Type: Axle 1 (kips)		Axle 2 (kips)		Axle 3 (kips)		Gross Weight (pounds)	Equivalency Factors: Axle 1	Axle 2	Axle 3	ESAL's
Passenger car	**500**	**7**	**52**	**20**	2	S	2	S			4,000	0.0002	0.0002	0	1,441
Pick-up truck or van					2	S	4	S			6,000	0.0002	0.0021	0	0
Recreational vehicle					4	S	4	S			8,000	0.0021	0.0021	0	0
School bus					6	S	14	S			20,000	0.0099	0.3356	0	0
TARC bus					8	S	14	S			22,000	0.0317	0.3356	0	0
Greyhound MC-12 bus					13.4	S	18.4	S	6	S	37,800	0.2779	1.101	0.0099	0
Package delivery truck					4	S	14	S			18,000	0.0021	0.3356	0	0
Beverage delivery truck					6	S	12	S	12	S	30,000	0.0099	0.1732	0.1732	0
Garbage/dumpster truck					20	S	35	T			55,000	1.5876	2.2373	0	0
Concrete truck (full)					20	S	48	R			68,000	1.5876	2.6059	0	0
Dump truck (full)					20	S	48	R			68,000	1.5876	2.6059	0	0
Semi-tractor (no trailer)					8	S	2	T			10,000	0.0317	8E-05	0	0
Semi-tractor trailer (empty)					8	S	8	T	6	T	22,000	0.0317	0.0053	0.0019	0
Semi-tractor trailer					12	S	34	T	34	T	80,000	0.1732	1.9714	1.9714	0
M1097 HMMWV					3.5	S	6.5	S			10,000	0.0013	0.0136	0	0
Diesel Forklifts					26.432	S	134.41	S			160,843	5.4504	7310.9	0	0
Fire Truck (aerial platform)					24.3	S	28.35	S	28.35	S	81,000	3.7522	7.4439	7.4439	0
Vehicle type H10					4	S	16	S			20,000	0.0021	0.5982	0	0
Vehicle type H15					6	S	24	S			30,000	0.0099	3.5512	0	0
Vehicle type H20					8	S	32	S			40,000	0.0317	12.774	0	0
Vehicle type 3					16	S	34	T			50,000	0.5982	1.9714	0	0
Vehicle type HS15					6	S	24	S	24	S	54,000	0.0099	3.5512	3.5512	0
Vehicle type HS20					8	S	32	S	32	S	72,000	0.0317	12.774	12.774	0
Vehicle type 3S2					10	S	31	T	31	T	72,000	0.0801	1.3191	1.3191	0

Terminal Serviceability, r_t	**2.0**
Assumed Pavement Thickness, in.	**15**
Traffic Growth Rate, %/yr	**0**

Summary:		
	Total AASHTO Rigid ESAL's	**1,441**
	Traffic Category	**Light Duty**

Project: Bay AZ1 Site (Northtown) Location:

Job No.: ND205079 Date: 1/8/2021

BURNS & McDONNELL

burnsmcd.com | Offices Worldwide

DEPARTMENT OF PLANNING, BUILDING AND CODE ENFORCEMENT

Purpose of the Compliance Checklist

In 2020, the City adopted a Greenhouse Gas Reduction Strategy (GHGRS) that outlines the actions the City will undertake to achieve its proportional share of State greenhouse gas (GHG) emission reductions for the interim target year 2030. The purpose of the Greenhouse Gas Reduction Strategy Compliance Checklist (Checklist) is to:

- Implement GHG reduction strategies from the 2030 GHGRS to new development projects.
- Provide a streamlined review process for proposed new development projects that are subject to discretionary review and trigger environmental review pursuant to the California Environmental Quality Act (CEQA).

The 2030 GHGRS presents the City's comprehensive path to reduce GHG emissions to achieve the 2030 reduction target, based on SB 32, BAAQMD, and OPR. Additionally, the 2030 GHGRS leverages other important City plans and policies; including the General Plan, Climate Smart San José, and the City Municipal Code in identifying reductions strategies that achieve the City's target. CEQA Guidelines Section 15183.5 allows for public agencies to analyze and mitigate GHG emissions as part of a larger plan for the reduction of greenhouse gases. Accordingly, the City of San José's 2030 GHGRS represents San José's qualified climate action plan in compliance with CEQA.

As described in the 2030 GHGRS, these GHG reductions will occur through a combination of City initiatives in various plans and policies and will provide reductions from both existing and new developments. This Compliance Checklist specifically applies to proposed discretionary projects that require environmental review pursuant to CEQA. Therefore, the Checklist is a critical implementation tool in the City's overall strategy to reduce GHG emissions. Implementation of applicable reduction actions in new development projects will help the City achieve incremental reductions toward its target. Per the 2030 GHGRS, the City will monitor strategy implementation and make updates, as necessary, to maintain an appropriate trajectory to the 2030 GHG target.

Pursuant to CEQA Guidelines Sections 15064(h)(3), 15130(d), and 15183(b), a project's incremental contribution to a cumulative GHG emissions effect may be determined not to be cumulatively considerable if it complies with the requirements of the GHGRS.

Instructions for Compliance Checklist

Applicants shall complete the following sections to demonstrate conformance with the City of San José 2030 Greenhouse Gas Reduction Strategy for the proposed project. All projects must complete Section A. General Plan Policy Conformance and Section B. Greenhouse Gas Reduction Strategies. Projects that propose alternative GHG mitigation measures must also complete Section C. Alternative Project Measures and Additional GHG Reductions.

A. General Plan Policy Compliance

Projects need to demonstrate consistency with the Envision San José 2040 General Plan's relevant policies for Land Use & Design, Transportation, Green Building, and Water Conservation, enumerated in Table A. All applicants shall complete the following steps.

1. Complete Table A, Item #1 to demonstrate the project's consistency with the General Plan Land Use and Circulation Diagram.
2. Complete Table A, Items #2 through #4 to demonstrate the project's consistency with General Plan policies[1] related to green building; pedestrian, bicycle & transit site design; and water conservation and urban forestry, as applicable. For each policy listed, mark the relevant yes/no check boxes to indicate project consistency, and provide a qualitative description of how the policy is implemented in the proposed project or why the policy is not applicable to the proposed project. Qualitative descriptions can be included in Table A or provided as separate attachments. This explanation will provide the basis for analysis in the CEQA document.

B. Greenhouse Gas Reduction Strategies

Table B identifies the GHGRS strategies and recommended consistency options. Projects need to demonstrate consistency with the GHGRS reduction strategies listed in Table B or document why the strategies are not applicable or are infeasible. The corresponding GHGRS strategies are indicated in the table to provide additional context, with the full text of the strategies preceding Table B.

Residential projects must complete Table B, Part 1 and 2; Non-residential projects must complete Table B, Part 2 only. All applicants shall complete the following steps for Table B.

1. Review the project consistency options described in the column titled 'GHGRS Strategy and Consistency Options'.
2. Use the check boxes in the column titled "Project Conformance" to indicate if the strategy is 'Proposed', 'Not Applicable', 'Not Feasible', or if there is an 'Alternative Measure Proposed'.

[1] The lists in items # 2-4 do not represent all General Plan policies but allow projects to demonstrate consistency and achievement of policies that are related to quantified reduction estimates in the 2030 GHGRS.

3. Provide a qualitative analysis of the proposed project's compliance with the GHGRS strategies in the column titled "Description of Project Measure". This will be the basis for CEQA analysis to demonstrate compliance with the 2030 GHGRS and by extension, with SB 32. The qualitative analysis should provide:

   a. A description of which consistency options are included as part of the proposed project, or

   b. A description of why the strategy is not applicable to the proposed project, or

   c. A description of why the consistency options are infeasible. If applicants select 'Not Feasible' or 'Alternative Measure Proposed', they must complete Table C to document what alternative project measures will be implemented to achieve a similar level of greenhouse gas reduction and how those reduction estimates were calculated.

C. Alternative Project Measures and Additional GHG Reductions

Projects that propose alternative GHG mitigation measures to those identified in Table B or propose to include additional GHG mitigation measures beyond those described in Tables A and B, shall provide a summary explanation of the proposed measures and demonstrate efficiency or greenhouse gas reductions achievable though the proposed measures. Documentation for these alternative or additional project measures shall be documented in Table C. Any applicants who select 'Not Feasible' or 'Alternative Measure Proposed' in Table B must complete the following steps for Table C.

1. In the column titled "Description of Proposed Measure" provide a qualitative description of what measure will be implemented, why it is proposed, and how it will reduce GHG emissions.
2. In the column titled "Description of GHG Reduction Estimate" demonstrate how the alternative project measure would achieve the same or greater level of greenhouse gas reductions as the GHGRS strategy it replaces. Documentation or calculation files can be attached separately.
3. In the column titled "Proposed Measure Implementation" identify how the measure will be implemented: incorporated as part of the project design or as an additional measure that is not part of the project (e.g., purchase of carbon offsets).

Compliance Checklist

Evaluation of Project Conformance with the 2030 Greenhouse Gas Reduction Strategy

Table A: General Plan Consistency

Development Type: ☐ Commercial ☐ Residential ☐ Office ■ Other: Specify

Industrial data center project (SJC04 Data Center)

1) Consistency with the Land Use/Transportation Diagram (Land Use and Density)	**Yes**	**No**
Is the proposed Project consistent with the Land Use/Transportation Diagram?	☒	☐
If not, and the proposed project includes a General Plan Amendment, does the proposed amendment decrease GHG emissions (in absolute terms or per capita, per employee, per service population) below the level assumed in the GHGRS based on the existing planned land use? (The project could have a higher density, mix of uses, or other features that would reduce GHG emissions compared to the planned land use).[2]	☐	☐
If not, would the proposed project and the General Plan Amendment increase GHG emissions (in absolute terms or per capita, per employee, per service population)? Project is not consistent with GHGRS and further modeling will be required to determine if additional mitigation measures are necessary.	☐	☐

Response documentation: *[Either here or as an attachment]*

The project site is designated Combined Industrial/Commercial and Industrial Park. The Combined Industrial/Commercial land use designation allows for commercial, office, or industrial developments or a compatible mix of these uses. The Industrial Park land use designation allows for a wide variety of industrial users such as research and development, manufacturing, assembly, testing and offices. The proposed project consists of a data center and associated facilities, such as a substation. These uses would be compatible with both land use designations.

[2] For example, a General Plan Amendment to change use from single-family residential to multi-family residential or a General Plan Amendment to change the use from regional-serving commercial to mixed-use urban in a transit-served area might reduce travel demand, and therefore GHG emissions from mobile sources.

2) *Implementation of Green Building Measures*	Yes	No
MS-2.2: *Encourage maximized use of on-site generation of renewable energy for all new and existing buildings.*	☐	☐
Not applicable	☒	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] The project owner will either participate in the San Jose Clean Energy (SJCE) at the Total Green Level (i.e., 100% carbon-free electricity) for electricity accounts associated with the project or enter into an electricity contract with SJCE or participate in a clean energy program that accomplishes the same goals of 100 percent carbon-free electricity as the SJCE Total Green Level. As a result, onsite renewable energy generation is not needed to offset the project's emissions.		
MS-2.3: *Encourage consideration of solar orientation, including building placement, landscaping, design and construction techniques for new construction to minimize energy consumption.*	☒	☐
Not applicable	☐	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] Unlike typical structures which may utilize windows to take advantage of sun exposure to reduce energy consumption, one of the primary concerns of data center structures is interior cooling. As a result, the data center buildings are designed with minimal windows and sun exposure to the data hall areas to reduce energy consumption associated with cooling.		
MS-2.7: *Encourage the installation of solar panels or other clean energy power generation sources over parking areas.*	☐	☒
Not applicable	☐	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] Due to site constraints and City parking requirements, it is not feasible to include solar panels on the roof of the proposed parking garage as it would reduce the number of parking spaces below the required level. The project owner will either participate in the SJCE at the Total Green Level (i.e., 100 percent carbon-free electricity) for electricity accounts associated with the project, or enter into an electricity contract with SJCE or participate in a clean energy program that accomplishes the same goals of 100% carbon-free electricity as the SJCE Total Green Level. As a result, onsite renewable energy generation is not needed to offset the project's emissions.		
MS-2.11: *Require new development to incorporate green building practices, including those required by the Green Building Ordinance. Specifically, target reduced energy use through construction techniques (e.g., design of building envelopes and systems to maximize energy performance), through architectural design (e.g., design to maximize cross ventilation and interior daylight) and through site design techniques (e.g., orienting buildings on sites to maximize the effectiveness of passive solar design).*	☒	☐
Not applicable	☐	☐

Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] The project will pursue LEED v4 BD+C Gold certification for Data Centers. The project would also be built in accordance with Title 24 and CalGreen required standards. In addition, the project would be constructed to be an all-electric building pursuant with the City of San José's natural gas infrastructure prohibition and reach code ordinances (Ordinance No. 30502). This reach code prohibits natural gas infrastructure.		
***MS-16.2**: Promote neighborhood-based distributed clean/renewable energy generation to improve local energy security and to reduce the amount of energy wasted in transmitting electricity over long distances.*	☐	☐
Not applicable	☒	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] The project owner will either participate in the SJCE at the Total Green Level (i.e., 100 percent carbon-free electricity) for electricity accounts associated with the project or enter into an electricity contract with SJCE or participate in a clean energy program that accomplishes the same goals of 100 percent carbon-free electricity as the SJCE Total Green Level. As a result, additional renewable energy generation is not needed to offset the project's emissions.		

3) Pedestrian, Bicycle & Transit Site Design Measures	**Yes**	**No**
***CD-2.1**: Promote the Circulation Goals and Policies in the Envision San José 2040 General Plan. Create streets that promote pedestrian and bicycle transportation by following applicable goals and policies in the Circulation section of the Envision San José 2040 General Plan.*		
a) Design the street network for its safe shared use by pedestrians, bicyclists, and vehicles. Include elements that increase driver awareness.	☒	☐
b) Create a comfortable and safe pedestrian environment by implementing wider sidewalks, shade structures, attractive street furniture, street trees, reduced traffic speeds, pedestrian-oriented lighting, mid-block pedestrian crossings, pedestrian-activated crossing lights, bulb-outs and curb extensions at intersections, and on-street parking that buffers pedestrians from vehicles.	☒	☐
c) Consider support for reduced parking requirements, alternative parking arrangements, and Transportation Demand Management strategies to reduce area dedicated to parking and increase area dedicated to employment, housing, parks, public art, or other amenities. Encourage de-coupled parking to ensure that the value and cost of parking are considered in real estate and business transactions.	☒	☐
Not applicable	☐	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] The proposed project would construct a multi-use trail extension (Class I bike path) along the southern boundary of the project site to create a link between the Guadalupe River Trail and Orchard Parkway at the intersection with Component Drive. New bicycle facilities would close existing network gaps for bicycles. On-site circulation would provide sufficient access to all portions of the site and would be easily accessible from Orchard Parkway and Component Drive. The project would also be required to implement Transportation Demand Management measures to reduce significant vehicle miles traveled impacts.		

Transportation Demand Management measures would include pedestrian network improvements, bike access improvements, commute trip reductions marketing and education, car sharing program, and 100 percent subsidized transit passes. Based on this discussion, the project would be consistent with *Circulation Goals and Policies in the Envision San José 2040 General Plan.*

***CD-2.5**: Integrate Green Building Goals and Policies of the Envision San José 2040 General Plan into site design to create healthful environments. Consider factors such as shaded parking areas, pedestrian connections, minimization of impervious surfaces, incorporation of stormwater treatment measures, appropriate building orientations, etc.*	☒	☐
Not applicable	☐	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] The project will pursue LEED v4 BD+C Gold certification for Data Centers. The project would also be built in accordance with Title 24 and CalGreen required standards. In addition, the project would be constructed to be an all-electric building pursuant with the City of San José's natural gas infrastructure prohibition and reach code ordinances (Ordinance No. 30502). The project would also include two bioretention basins on-site as stormwater treatment measures.		

	Yes	No
***CD-2.11**: Within the Downtown and Urban Village Overlay areas, consistent with the minimum density requirements of the pertaining Land Use/Transportation Diagram designation, avoid the construction of surface parking lots except as an interim use, so that long-term development of the site will result in a cohesive urban form. In these areas, whenever possible, use structured parking, rather than surface parking, to fulfill parking requirements. Encourage the incorporation of alternative uses, such as parks, above parking structures.*	☐	☐
Not applicable	☒	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] The project is not within a Downtown or Urban Village Overlay area. This measure is not applicable.		
***CD-3.2**: Prioritize pedestrian and bicycle connections to transit, community facilities (including schools), commercial areas, and other areas serving daily needs. Ensure that the design of new facilities can accommodate significant anticipated future increases in bicycle and pedestrian activity.*	☒	☐
Not applicable	☐	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] The project would provide 10 bicycle parking spaces (five spaces per building) pursuant with the City's bicycle parking requirement. Also, as mentioned above, the project would construct a Class I bike path, which would provide additional multi-modal infrastructure in San José		
***CD-3.4**: Encourage pedestrian cross-access connections between adjacent properties and require pedestrian and bicycle connections to streets and other public spaces, with particular attention and priority given to providing convenient access to transit facilities. Provide pedestrian and vehicular connections with cross-access easements within and between new and existing developments to encourage walking and minimize interruptions by parking areas and curb cuts.*	☒	☐
Not applicable	☐	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] The project site would include adequate pedestrian connectivity on-site would include an internal network of sidewalks and crosswalks connecting the buildings, substation, storage tank area, and parking lots. The sidewalks and crosswalks would connect to the existing pedestrian facilities along Orchard Parkway and Component Drive, which would allow pedestrians to travel to adjacent properties. The project is also constructing a multi-use trail to connect to the Guadalupe River Trail.		

	Yes	No
LU-3.5: *Balance the need for parking to support a thriving Downtown with the need to minimize the impacts of parking upon a vibrant pedestrian and transit oriented urban environment. Provide for the needs of bicyclists and pedestrians, including adequate bicycle parking areas and design measures to promote bicyclist and pedestrian safety.*	☐	☐
Not applicable	☒	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] The project is not within the Downtown area. This measure is not applicable.		

	Yes	No
TR-2.8: *Require new development to provide on-site facilities such as bicycle storage and showers, provide connections to existing and planned facilities, dedicate land to expand existing facilities or provide new facilities such as sidewalks and/or bicycle lanes/paths, or share in the cost of improvements.*	☒	☐
Not applicable	☐	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] The project would provide a total of 10 bicycle parking spaces and a new multi-use trail extension. The trail would be paved and would include pavement markings and signage to indicate that bikes are allowed.		
TR-7.1: *Require large employers to develop TDM programs to reduce the vehicle trips and vehicle miles generated by their employees through the use of shuttles, provision for car-sharing, bicycle sharing, carpool, parking strategies, transit incentives and other measures.*	☒	☐
Not applicable	☐	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] The project would implement a Transportation Demand Management program consisting of reduced parking onsite, encouraging employees to utilize alternative forms of transportation, thus reducing VMT.		
TR-8.5: *Promote participation in car share programs to minimize the need for parking spaces in new and existing development.*	☒	☐
Not applicable	☐	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] The project would implement a Transportation Demand Management program consisting of reduced parking onsite, encouraging employees to utilize alternative forms of transportation such as car share programs, thus reducing VMT.		

4) Water Conservation and Urban Forestry Measures	**Yes**	**No**
***MS-3.1**: Require water-efficient landscaping, which conforms to the State's Model Water Efficient Landscape Ordinance, for all new commercial, institutional, industrial and developer-installed residential development unless for recreation needs or other area functions.*	☒	☐
Not applicable	☐	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] The project would utilized recycled water for landscaping. The project's landscaping would conform to the *State's Model Water Efficient Landscape Ordinance*.		

	Yes	**No**
***MS-3.2**: Promote the use of green building technology or techniques that can help reduce the depletion of the City's potable water supply, as building codes permit. For example, promote the use of captured rainwater, graywater, or recycled water as the preferred source for non-potable water needs such as irrigation and building cooling, consistent with Building Codes or other regulations.*	☒	☐
Not applicable	☐	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] The project would utilize recycled water for landscape irrigation and building cooling. Potable water would only be used for uses such as toilets, sinks, and water fountains.		

***MS-19.4**: Require the use of recycled water wherever feasible and cost-effective to serve existing and new development.*	☒	☐
Not applicable	☐	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] The project would utilize recycled water for landscape irrigation and building cooling.		
***MS-21.3**: Ensure that San José's Community Forest is comprised of species that have low water requirements and are well adapted to its Mediterranean climate. Select and plant diverse species to prevent monocultures that are vulnerable to pest invasions. Furthermore, consider the appropriate placement of tree species and their lifespan to ensure the perpetuation of the Community Forest.*	☒	☐
Not applicable	☐	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] The landscaping proposed by the project would be reviewed by the City prior to receiving building permits, which would ensure that the plant species selected would be appropriate and comply with the City's Community Forest guidelines.		

	Yes	No
MS-26.1*: As a condition of new development, require the planting and maintenance of both street trees and trees on private property to achieve a level of tree coverage in compliance with and that implements City laws, policies or guidelines.*	☒	☐
Not applicable	☐	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] The project will comply with the City's laws, policies, and/or guidelines regarding the planting and maintenance of street trees or trees.		

	Yes	No
ER-8.7*: Encourage stormwater reuse for beneficial uses in existing infrastructure and future development through the installation of rain barrels, cisterns, or other water storage and reuse facilities.*	☐	☒
Not applicable	☐	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] The project is not proposing any rain barrels, cisterns, or other water storage facilities. The designers do not believe rainwater harvesting or the use of water storage facilities is feasible in Santa Clara County. Rainfall comes in a 3- or 4-month period at a time when irrigation is at its minimum. Storage of water for use during the dry weather has the potential for vector problems. The project is using recycled water for landscape irrigation and building cooling, minimizing the need for potable water.		

GHGRS Strategies

GHGRS #1: The City will implement the San José Clean Energy program to provide residents and businesses access to cleaner energy at competitive rates.

GHGRS #2: The City will implement its building reach code ordinance (adopted September 2019) and its prohibition of natural gas infrastructure ordinance (adopted October 2019) to guide the city's new construction toward zero net carbon (ZNC) buildings.

GHGRS #3: The City will expand development of rooftop solar energy through the provision of technical assistance and supportive financial incentives to make progress toward the Climate Smart San José goal of becoming a one-gigawatt solar city.

GHGRS #4: The City will support a transition to building decarbonization through increased efficiency improvements in the existing building stock and reduced use of natural gas appliances and equipment.

GHGRS #5: As an expansion to Climate Smart San José, the City will update its Zero Waste Strategic Plan and reassess zero waste strategies. Throughout the development of the update, the City will continue to divert 90 percent of waste away from landfills through source reduction, recycling, food recovery and composting, and other strategies.

GHGRS #6: The City will continue to be a partner in the Caltrain Modernization Project to enhance local transit opportunities while simultaneously improving the city's air quality.

GHGRS #7: The City will expand its water conservation efforts to achieve and sustain long-term per capita reductions that ensure a reliable water supply with a changing climate, through regional partnerships, sustainable landscape designs, green infrastructure, and water-efficient technology and systems.

Table B: 2030 Greenhouse Gas Reduction Strategy Compliance

GHGRS Strategy and Consistency Options	Description of Project Measure	Project Conformance
PART 1: RESIDENTIAL PROJECTS ONLY		
Zero Net Carbon Residential Construction 1. Achieve/exceed the City's Reach Code, and 2. Exclude natural gas infrastructure in new construction, or 3. Install on-site renewable energy systems or participate in a community solar program to offset 100% of the project's estimated energy demand, or 4. Participate in San José Clean Energy at the Total Green level (i.e., 100% carbon-free electricity) for electricity accounts associated with the project until which time SJCE achieves 100% carbon-free electricity for all accounts. **Supports Strategies**: GHGRS #1, GHGRS #2, GHGRS #3	*Describe which, if any, project consistency options from the leftmost column you are implementing.* *OR,* *Describe why this strategy is not applicable to your project.* *OR,* *Describe why such measures are infeasible.* The project is an industrial development that would include data centers, emergency generators, a substation, and other associated facilities. It is not a residential land use; therefore, this measure is not applicable.	☐ Proposed ☒ Not Applicable ☐ Not Feasible* ☐ Alternative Measure Proposed ** The 2030 GHGRS assumed this strategy would be feasible for 50% of residential units constructed between 2020 and 2030.*
PART 2: RESIDENTIAL AND NON-RESIDENTIAL PROJECTS		
Renewable Energy Development 1. Install solar panels, solar hot water, or other clean energy power generation sources on development sites, or 2. Participate in community solar programs to support development of renewable energy in the community, or	*Describe which, if any, project consistency options from the leftmost column you are implementing.* *OR,* *Describe why this strategy is not applicable to your project.* *OR,* *Describe why such measures are infeasible.*	☐ See Part 1 (Residential projects only) ☒ Proposed ☐ Not Applicable ☐ Not Feasible ☐ Alternative Measure Proposed

GHGRS Strategy and Consistency Options	Description of Project Measure	Project Conformance
3. Participate in San José Clean Energy at the Total Green level (i.e., 100% carbon-free electricity) for electricity accounts associated with the project. **Supports Strategies**: GHGRS #1, GHGRS #3	Compliance with this policy is demonstrated by employing one or more of the following options. The project proposes an Alternative Measure (see response to Table C below) that would allow it to either comply with Number 3 (i.e., participate at the Total Green Level) or participate in a clean energy program that accomplishes the same goals of 100% carbon-free electricity as the SJCE Total Green Level. 1. The project is not proposing onsite renewable energy generation. The project owner will participate in the SJCE at the Total Green Level (i.e., 100% carbon-free electricity) for electricity accounts associated with the project, or enter into an electricity contract with SJCE or participate in a clean energy program that accomplishes the same goals of 100% carbon-free electricity as the SJCE Total Green Level. As a result, onsite renewable energy generation is not needed to offset the project's emissions. 2. The project is not proposing to participate in community solar programs. 3. The project owner will either participate in the SJCE at the Total Green Level (i.e., 100% carbon-free electricity) for electricity accounts associated with the project, or enter into an electricity contract with SJCE or participate in a clean energy program that accomplishes the same goals of 100% carbon-free electricity as the SJCE Total Green Level.	
Building Retrofits – Natural Gas[3] This strategy only applies to projects that include a retrofit of an existing building. If the proposed project does not include a retrofit, select "Not Applicable" in the Project Conformance column. 1. Replace an existing natural gas appliance with an electric alternative (e.g., space heater, water heater, clothes dryer), or 2. Replace an existing natural gas appliance with a high-efficiency model	*Describe which, if any, project consistency options from the leftmost column you are implementing.* *OR,* *Describe why this strategy is not applicable to your project.* *OR,* *Describe why such measures are infeasible.* This is a new industrial development, and it would not include building retrofits. The measure is not applicable.	☐ Proposed ☒ Not Applicable ☐ Not Feasible ☐ Alternative Measure Proposed

[3] GHGRS Strategy #4 applies to existing building retrofits and not to new construction; Strategy #2 applies to new construction to reduce natural gas related GHG emissions

GHGRS Strategy and Consistency Options	Description of Project Measure	Project Conformance
Supports Strategies: GHGRS #4		
Zero Waste Goal 1. Provide space for organic waste (e.g., food scraps, yard waste) collection containers, and/or 2. Exceed the City's construction & demolition waste diversion requirement. **Supports Strategies:** GHGRS #5	*Describe which, if any, project consistency options from the leftmost column you are implementing.* *OR,* *Describe why this strategy is not applicable to your project.* *OR,* *Describe why such measures are infeasible.* The project would exceed the City's construction and demolition waste diversion requirement.	☒ Proposed ☐ Not Applicable ☐ Not Feasible ☐ Alternative Measure Proposed

GHGRS Strategy and Consistency Options	Description of Project Measure	Project Conformance
Caltrain Modernization 1. For projects located within ½ mile of a Caltrain station, establish a program through which to provide project tenants and/or residents with free or reduced Caltrain passes or 2. Develop a program that provides project tenants and/or residents with options to reduce their vehicle miles traveled (e.g., a TDM program), which could include transit passes, bike lockers and showers, or other strategies to reduce project related VMT. **Supports Strategies:** GHGRS #6	*Describe which, if any, project consistency options from the leftmost column you are implementing.* *OR,* *Describe why this strategy is not applicable to your project.* *OR,* *Describe why such measures are infeasible.* The project would implement a Transportation Demand Management program consisting of reduced parking onsite, encouraging employees to utilize alternative forms of transportation, thus reducing VMT.	☒ Proposed ☐ Not Applicable ☐ Not Feasible ☐ Alternative Measure Proposed
Water Conservation 1. Install high-efficiency appliances/fixtures to reduce water use, and/or include water-sensitive landscape design, and/or 2. Provide access to reclaimed water for outdoor water use on the project site. **Supports Strategies:** GHGRS #7	*Describe which, if any, project consistency options from the leftmost column you are implementing.* *OR,* *Describe why this strategy is not applicable to your project.* *OR,* *Describe why such measures are infeasible.* The project would install high-efficiency appliances and fixtures pursuant with Title 24 and CalGreen requirements and include a water-sensitive landscape design. The project would utilize recycled water for landscape irrigation and building cooling.	☒ Proposed ☐ Not Applicable ☐ Not Feasible ☐ Alternative Measure Proposed

Table C: Applicant Proposed Greenhouse Gas Reduction Measures

Description of Proposed Measure	Description of GHG Reduction Estimate	Proposed Measure Implementation
Description of Proposed Measure: The project owner shall participate in the SJCE at the Total Green Level (i.e., 100% carbon-free electricity) for electricity accounts associated with the project, or enter into an electricity contract with SJCE or participate in a clean energy program that accomplishes the same goals of 100% carbon-free electricity as the SJCE Total Green Level. **Supports Strategies/Sectors:** GHGRS #1	*Demonstrate the effectiveness of the proposed measure to reduce the project's GHG emissions.* By either participating in SJCE's Total Green Level or participating in a clean energy program that accomplishes the same goals of 100% carbon-free electricity as the SJCE Total Green Level, all GHG emissions associated with the project's electricity consumption would be offset.	☒ Part of Design ☐ Additional Measure
[Describe the proposed project measure and why it is proposed] **Supports Strategies/Sectors:** GHGRS #	*[Demonstrate the effectiveness of the proposed measure to reduce the project's GHG emissions.* *Include a description of how your measure will reduce emissions and provide supporting quantification documentation/assumptions.]*	☐ Part of Design ☐ Additional Measure
[Describe the proposed project measure and why it is proposed] **Supports Strategies/Sectors:** GHGRS #	*[Demonstrate the effectiveness of the proposed measure to reduce the project's GHG emissions.* *Include a description of how your measure will reduce emissions and provide supporting quantification documentation/assumptions.]*	☐ Part of Design ☐ Additional Measure
[Describe the proposed project measure and why it is proposed] **Supports Strategies/Sectors:** GHGRS #	*[Demonstrate the effectiveness of the proposed measure to reduce the project's GHG emissions.* *Include a description of how your measure will reduce emissions and provide supporting quantification documentation/assumptions.]*	☐ Part of Design ☐ Additional Measure