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GEOTECHNICAL REPORT

TOM BATES REGIONAL SPORTS COMPLEX PROJECT

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1.0 INTRODUCTION

1.1 GENERAL

Cal Engineering & Geology, Inc. (CE&G), a division of Haley & Aldrich, has provided geotechnical services to TranSystems for the Tom Bates Regional Sports Complex (the Project) located in Berkeley, California. The work has been completed to provide geotechnical design recommendations for design of a fieldhouse building, an U10 soccer field, six pickleball courts, a parking lot, and associated improvements.

1.2 PROJECT DESCRIPTION

The Tom Bates Regional Sports Complex is located at 400 Gilman Street, Berkeley, California as shown in Figure 1, Site Location. The site currently has two full-size soccer fields and a parking lot on the northern side, and a baseball field on the southwestern side. The proposed improvements will be located on the northwestern portion of the site, just west of the existing soccer fields. We understand the new parking lot consists of improving an existing asphalt-paved area on the western-most portion of the site, adjacent to San Francisco Bay.

1.3 PURPOSE AND SCOPE OF SERVICES

The purpose of CE&G's geotechnical investigation was to assess the surface and subsurface conditions in the immediate vicinity of the planned improvements and provide geotechnical recommendations for design. The scope of work completed for the geotechnical investigation and report included: consultation and coordination with TranSystems staff; reconnaissance to observe current site conditions and to mark for USA (Underground Service Alert); a subsurface exploration program consisting of 5 borings using a truck-mounted drill rig; laboratory testing of selected soil samples to determine key engineering properties; engineering analysis; development of geotechnical design recommendations; and preparation of this report.

2.0 SITE DESCRIPTION

The Tom Bates Sports Complex is located on the eastern shore of the San Francisco Bay in Berkeley, California, and is bounded by Golden Gate Fields Operations and Switch Board on the north, McLaughlin State Seashore on the south, San Francisco Bay on the west and Interstate 580 on the east. The eastern side of the site is The site currently has a paved parking lot on the northwestern corner, soccer fields on the northeastern corner, a dirt parking lot and open field on the south and a baseball field on the west. The site topography is generally flat with elevations ranging from 7 to 15 ft (NAVD 88). The northwestern portion of the site is lower than the remainder of the site with elevations between 7 and 8 feet (NAVD88). Key features are shown on Figure 2, Site Plan.

The proposed fieldhouse building will be located in the north-central portion of the proposed improvements. This area is currently paved and is located on the eastern edge of the existing parking lot and adjacent to the fill slope for the soccer fields to the east.

The proposed pickleball court sizes are each approximately 32 to 34 feet wide by 64 feet long, and the proposed soccer field is approximately 115 feet by 135 feet. It is anticipated that the pickleball courts and soccer field will be located on the southern end of the proposed improvements. This area is currently moderately vegetated with weeds and shrubs.

The proposed parking lot is shown to be approximately 360 ft by 64 ft and is anticipated to be located along the shoreline in the west area of the proposed improvements. This area is currently paved.

3.0 GEOLOGY

3.1 REGIONAL SETTING

The project site lies within the Coast Ranges geomorphic province of California. This province is characterized by northwest-southeast trending mountain ranges and intervening valleys, such as that occupied by San Francisco Bay and the Santa Clara Valley. The right-lateral strike-slip San Andreas fault system controls the northwest-southeast structural grain of the Coast Ranges and the Bay Area. The San Andreas fault system includes the Hayward-Rodgers Creek, Calaveras, Concord-Green Valley, and Greenville-Marsh Creek faults, among others, which have resulted in the uplift of the northwest-trending Diablo and Santa Cruz Mountain Ranges. San Francisco Bay is bounded on the east by the Diablo Range and the west by the Santa Cruz Mountains. As these ranges are uplifted, they shed erosional debris toward San Francisco Bay, resulting in relatively thick accumulations of alluvial sediment across San Francisco Bay Area. Various active river systems continue to flow from the surrounding mountain ranges and are responsible for the active incision of older alluvium and deposition of younger alluvium along the San Francisco Bay margin.

3.2 GEOLOGIC SETTING

The geologic setting is shown on the Regional Geology Map (Figure 3).

The general vicinity of the project site has been mapped several times, with geologic mapping having different emphases: Knudsen and others (2000); Graymer and others (2006); and Witter and others (2006). Knudsen and others (2000) mapped Quaternary geologic materials in detail for much of the San Francisco Bay Area. Much of Knudsen and others' mapping was incorporated or refined by Witter and others (2006). For this project, the Quaternary geologic mapping of Knudsen and others (2000), refined by Witter and others (2006) is the most detailed and pertinent.

The project site is mapped as being underlain by artificial fill over estuarine mud (bay mud), which is described as "material deposited by humans over sediments along the margins of San Francisco Bay" (Witter and others, 2006). The fill that overlays the bay mud may be engineered and/or non-engineered material (Witter and others, 2006).

3.3 SEISMICITY

The project site is located within the greater San Francisco Bay Area which is recognized as one of the more seismically active regions of California. The seismic activity in this region results from the complex movements along the transform boundary between the Pacific

Plate and the North American Plate. Along this transform boundary, the Pacific Plate is slowly moving to the northwest relative to the more stable North American Plate at approximately 40 mm/yr in the Bay Area (Page, 1992). The differential movements between the two crustal plates caused the formation of a series of active fault systems within the transform boundary. The transform boundary between the two plates extends across a broad zone of the North American Plate within which right-lateral strike-slip faulting predominates. In this broad transform boundary, the San Andreas fault accommodates less than half of the average total relative plate motion. Much of the remainder of the motion in this portion of the Bay Area is distributed across faults such as the Hayward, Calaveras, Concord, San Gregorio, and Rogers Creek fault zones.

Due to the site's location in the seismically active San Francisco Bay Area, it will likely experience strong ground shaking from a large (Moment Magnitude [Mw] 6.7) or greater earthquake along one or more of the nearby active faults during the design lifetime of the project (WGCEP, 2003). It should be noted that the third Uniform California Earthquake Rupture Forecast (UCERF3) time-independent model supports a magnitude-dependent methodology that accounts for historic open intervals on faults without a date of last event constraint. The exact factors influencing differences between UCERF2 and UCERF3 vary throughout the region and depend on the evaluation of specific seismogenic sources. For example, with the 30 yr M≥6.7 probabilities, the most significant changes from UCERF2 are a threefold increase on the Calaveras fault and a threefold decrease on the San Jacinto fault. The model also suggests that the average time between 6.7 Mw or larger events has increased from every 4.8 years to every 6.3 years. The UCERF3 model indicates that M≥6.7 probabilities may not be representative of other hazard or loss measures and the applicability of UCERF3 should be evaluated on a case-by-case basis if required during site-specific ground motion analyses or at the behest of the regulatory agencies (WGCEP, 2014).

Some contributors to seismic risk for the project include the Hayward, San Andreas, Concord and Calaveras faults. A large magnitude earthquake on any of these fault systems has the potential to cause significant ground shaking in the vicinity of the planned improvement. The intensity of ground shaking that is likely to occur in the area is generally dependent upon the magnitude of the earthquake and the distance to the epicenter.

Relevant seismic sources in the San Francisco Bay area and their distances from the site are summarized in Table 3-1.

Table-3-1. Distances to Selected Major Active Fault Surface Traces

Fault Name	Approximate Distance and Direction from Site to Mapped Surface Fault Traces
Hayward	3.9 km northeast
Calaveras	23 km east-southeast
Concord	26 km northeast
Serra	26 km southwest
San Andreas	27 km southwest
San Gregorio	30 km southwest
Rogers Creek	35 km north-northwest

3.4 GEOHAZARD MAPPING

3.4.1 Active Faulting

According to CGS (2018), a Holocene-active fault is defined as a fault that has had surface displacement within Holocene time (the last 11,700 years), and a pre-Holocene fault is defined as a fault whose recency of past movement is older than 11,700 years. The Alquist-Priolo Earthquake Fault Zoning Act only addresses the hazard of surface fault rupture for Holocene-active faults, although pre-Holocene-active faults may also have the potential for future surface fault rupture (CGS, 2018). The Alquist-Priolo Earthquake Fault Zoning Act's main purpose is to prevent the construction of buildings used for human occupancy on the surface trace of active faults. Before a new project is permitted, cities and counties require a geologic investigation to demonstrate that proposed buildings will not be constructed on active faults. According to CGS (2018), the site does not cross through an Alquist-Priolo Earthquake Fault Zone.

According to the USGS Quaternary Fault and Fold Database (2017), there are no quaternary active faults mapped as crossing the project site.

3.4.2 Liquefaction Hazards

Witter and others (2006) have generated a map showing liquefaction susceptibility for the San Francisco Bay Area with a 5-class scale that includes very low (essentially bedrock areas), low, moderate, high, and very high liquefaction susceptibility classes. The soils underlying the project is mapped as having very high liquefaction susceptibility.

The project area is shown within a liquefaction hazard zone on the State of California Seismic Hazard Map produced by the California Geological Survey (CGS) for the Oakland West 7.5-minute quadrangle (CGS, 2003). This regulatory map relied extensively on the geologic mapping by Knudsen and others (2000) described above, which was refined by Witter without major changes in the project area.

3.5 REGIONAL GROUNDWATER

The California Department of Water Resources identifies the site as lying within the Santa Clara Valley – East Bay Plain subbasin, which is one of several groundwater subbasins within the Santa Clara Valley groundwater basin.

A map prepared by CGS (2018) showing the depth of historically high groundwater levels for the West Oakland 7.5-minute quadrangle shows groundwater data between 0 and 5 feet for the project site due to its close proximity to San Francisco Bay.

Site-specific groundwater data from our investigation is discussed in Section 3.5.

4.0 FIELD INVESTIGATIONS

4.1 SITE RECONNAISSANCE

CE&G performed a field reconnaissance of the site on 19 October 2022, in advance of performing a subsurface boring program. Site reconnaissance included meeting with a private utility locator (Geotech Utility Locating), determining site access for drilling equipment, photographic documentation of the project site, and identifying and marking proposed boring locations. These markings were also used for utility clearance by Underground Service Alert (USA).

4.2 SUBSURFACE EXPLORATIONS

4.2.1 Exploratory Boring

Five geotechnical borings were drilled in the vicinity of the planned improvements as part of our investigation. The approximate boring locations are shown in Figure 2.

The geotechnical boring was drilled by Exploration Geoservices, Inc. on 1 November 2022, using a truck-mounted a truck-mounted Mobile B-53 drill rig equipped with 8-inch-diameter hollow-stem-augers and a 140 pounds auto-trip hammer. The surface conditions at the boring locations consisted of dry vegetation or pavement.

Upon completion, the borings were backfilled with cement grout in accordance with City of Berkeley requirements. Drilling spoils were drummed and stored on site until properly characterized and removed and disposed of at a permitted disposal facility on December 12, 2022.

4.2.2 Logging and Sampling

The materials encountered in the boring were logged in the field by a CE&G geologist. The soil was visually classified in the field, office, and laboratory according to the Unified Soil Classification System (USCS) in general accordance with ASTM D2487 and D2488.

During the drilling operations, soil samples were obtained using one of the following sampling methods:

- California Modified (CM) Sampler; 3.0-inch outer diameter (O.D.), 2.5-inch inner diameter (I.D.) (ASTM D1586)
- Standard Penetration Test (SPT) Split Spoon Sampler; 2.0-inch O.D., 1.375-inch I.D. (ASTM D1586)

- Thin-Walled Tube (Shelby Tube) Sampler; 3.0-inch O.D, 2.935-inch I.D. (ASTM D1587)

The CM and SPT samplers were driven 18 inches (unless otherwise noted on the boring logs) with a 140-pound auto-trip hammer, dropping 30 inches. The number of blows required to drive the SPT or CM samplers through each 6-inch interval was recorded for each sample. The results are included on the boring logs in Appendix B. The blow counts included on the boring logs represent the field values and are uncorrected.

Thin-Walled Shelby Tubes were pushed 30 inches (unless otherwise noted in the boring logs). The approximate push force to push each tube was recorded in pounds per square inch (psi) for each sample, which varies as the sampler is pushed to its full length. The recorded forces are included on the boring logs (Appendix B).

Soil samples obtained from the borings were packaged and sealed in the field to reduce the potential for moisture loss and disturbance. The samples were then taken to CE&G's laboratory, in Hayward, Cooper Testing Lab, in Palo Alto, and McCampbell Analytical, in Concord, for testing and storage.

4.2.3 Soil Conditions Encountered

Subsurface soil conditions encountered in our borings were generally consistent with regional geologic mapping.

Artificial Fill

Artificial fill was encountered in each boring and ranged in thickness from approximately 2 to 11 feet in the vicinity of the borings. The encountered fill material generally consisted of moist clays that are soft to hard.

A second layer of more heterogeneous fill was encountered between the surface fill and underlying bay sediments. This soil was highly variable containing silt, clay and organic matter. In general the material was loos where granular in nature and soft to medium stiff where fine grained. The layer was up to 11 to 12 feet thick.

Young Bay Mud:

Young estuarine deposits (Bay mud) were encountered beneath the fill in borings B-1 through B-3 and ranged in thickness between approximately 7 and 13 feet. The encountered Young Bay Mud consisted of very soft, medium to high plasticity clays and silts.

Older Alluvium:

Underlying the encountered fill and/or Bay mud is alluvium, which was encountered to the maximum depth explored of 50 feet bgs. The encountered alluvium generally consisted of medium stiff to very stiff lean and fat clays, with alternating layers of wet, loose to medium dense sandy silt, silty sand, and poorly graded sand.

For a more detailed description of the soil encountered in the borings, the boring logs and laboratory test results are included in Appendices A and B, respectively.

4.2.4 Groundwater Conditions Encountered

Groundwater was encountered at approximately 3 feet bgs in Borings B-1 and B-2, and at approximately 11 feet in Boring B-3 during this investigation.

4.3 GEOTECHNICAL LABORATORY TESTING

Laboratory testing was performed to obtain information concerning the qualitative and quantitative physical properties of the samples recovered during the subsurface exploration program. Tests were performed by Cooper Testing Laboratory in Palo Alto, California, and the CE&G Testing Laboratory in Hayward, California, in general conformance with applicable ASTM standards. The following tests were performed:

- Moisture Content and Dry Unit Weight (ASTM D2216)
- Atterberg Limits (ASTM D4318)
- Wash Over #200 Sieve (ASTM 1140)
- Unconsolidated-Undrained Triaxial Compression Test (ASTM D2850)
- Corrosion Caltrans Package includes:
  - Resistivity (Minimum) (Caltrans 643)
  - pH (Caltrans 643)
  - Chloride (Caltrans 422m)
  - Sulfate (Caltrans 417m)
- R-Value (ASTM D2844)

The laboratory testing results are presented on the boring logs and presented in Appendix B.

5.0 DISCUSSION AND CONCLUSIONS

The planned improvements consist of constructing a fieldhouse/storage building, a U10 soccer field, six new pickleball courts, and a new parking lot, . The soils encountered consisted predominantly of clayey soils, but there also appears to be a significant thickness of undocumented fill overlying and/or mixed with Bay Mud. It is our professional opinion that the planned improvements are feasible from a geotechnical standpoint, provided the recommendations presented in this report are followed.

The most important geotechnical issues to note during project design and construction are:

1. Seismic hazards such as ground shaking and liquefaction potential
2. The existence of a significant grade differential between different parts of the project and the presence of undocumented heterogeneous fill and potentially compressible soils between about 3 and 21 feet below the ground surface;
3. The moderate compressibility of fine-grained soils underlying the project area;
4. High groundwater level affecting foundation and underground utility construction; and
5. Moisture sensitive soils and wet weather construction.

These issues are discussed further in the following sections.

5.1 LIQUEFACTION

5.1.1 Liquefaction and Densification Susceptibility

Liquefaction is a soil behavior phenomenon in which a soil located below the groundwater surface loses a substantial amount of strength due to high excess pore-water pressure generated and accumulated during strong earthquake ground shaking. During and immediately following earthquake ground shaking, induced cyclic shear creates a tendency in most soils to change volume by rearrangement of the soil-particle structure. The potential for excess pore-water pressure generation and strength loss associated with this volume change tendency is highly dependent on the gradation and density of the soil, with greater potential in looser generally cohesionless (sandy) soils. Recently deposited (i.e., geologically young) and relatively loose natural soils, and uncompacted or poorly compacted artificial fills located below the groundwater table, are potentially susceptible to liquefaction.

The granular soils below the site vicinity are considered to have very high liquefaction susceptibility based on independent mapping and analysis by the US Geological Survey. However, the soils encountered during our investigation below the groundwater table included layers of clays and clayey sands that typically have low liquefaction susceptibility.

Dynamic densification is the densification of unsaturated, loose granular soils due to strong vibration such as that resulting from earthquake shaking. Granular soils and loose fills above ground water may be subject to such phenomenon. Based on the subsurface materials encountered during previous geotechnical investigations and the subsurface exploration completed as part of this investigation, we judge the potential for seismic densification to be low due to the relatively high groundwater table and lack of granular soils above the groundwater table.

5.1.2 Probabilistic Ground Motions

The site ground motions used in our liquefaction assessment were evaluated based on geotechnical investigation guidelines in ASCE 7-16 and USGS seismic hazard deaggregation (2008). Using tools contained on the USGS website, we completed a probabilistic assessment of earthquake shaking hazard at the project site. Liquefaction analyses were performed on the geotechnical boring SPT data using these ground motion parameters.

5.1.3 SPT Analysis Methodology

The liquefaction analysis was completed using spreadsheet and software-based calculations. Both calculation methods utilize empirical methods developed using field observations and laboratory test data in conjunction with results from SPT $(N_1)_{60}$ values. The measured SPT N-values were corrected to $(N_1)_{60CS}$ to account for the presence of cohesive/plastic (fine-grained) soils. Further corrections were made for the reported SPT N-values for the effect of overburden pressure, short rod length, standardized sampler configuration and borehole diameter. All the correction factors used in the liquefaction analysis are listed below:

$$(N_1)_{60} = N \cdot C_N \cdot C_R \cdot C_S \cdot C_B \cdot C_E$$

where:

C_N = correction for overburden pressure

C_R = correction for short rod length

C_S = correction for non-standardized sampler configuration

C_B = correction for borehole diameter and

C_E = correction for hammer energy efficiency

The index properties of the soil layers including soil classification, unit weight, and percent fines of soil samples obtained from each of the borings (B-1 through B-5) were used to complete the liquefaction analysis. In cases where lab tests were not performed, the soil characteristics were estimated based on lab tests on same or similar soil material at the same depth in nearby borings and using correlations provided in Caltrans Geotechnical Manual (2014).

For the purposes of SPT-based liquefaction analysis it was assumed that groundwater will be at or near a depth of 3 feet below the ground surface.

Liquefaction susceptibility at each boring location was analyzed using LiqSV software developed by Geologismiki. The software input values include measured field SPT data and assesses liquefaction potential, and post-earthquake vertical settlement given a user-defined earthquake magnitude and PGA. The seismic magnitude factor, stress reduction factor, and SPT liquefaction calculations are completed using the methods recommended by Idris & Boulanger (2014). The fines correction factors are determined according to Seed & Idriss (NCEER workshop).

The analysis concluded that the project site does not show significant settlement based on the SPT N-values and fines content of the soil layers observed d

5.1.4 Design Earthquake Events for Liquefaction Evaluation

CE&G evaluated liquefaction for two cases. One case was an earthquake resulting in a PGA of 0.79g which matches the event shown on the seismic design parameters in Table 6-1. Using the seismic deaggregation tool on the USGS website, this event has about a 5% chance of exceedance in 50 years (0.65g) and is associate with a 6.9 Magnitude earthquake at a distance of 10.18 km from the site.

The other event evaluated is an earthquake having a 10% chance of exceedance in 50 years. This recurrence interval is consistent with the guidelines of the California Department of Conservation Special Publication 117A (2008). This event is associated with a similar magnitude event occurring at a distance of 12.3 km from the site and has a PGA of 0.52g.

5.1.5 Seismically Induced Settlement

Seismic densification is the densification of unsaturated, loose to medium dense granular soils due to strong vibration such as that resulting from earthquake shaking. We judge the potential for seismic densification at this site to be low because the granular soils at the site are generally saturated.

5.1.6 Lateral Spreading

Lateral spreading is a phenomenon associated with strength loss following liquefaction and lateral movement of a liquefied layer toward a nearby free face. The project site is relatively flat and the sandy layers below the site are significantly lower than the nearby bay. Based on our qualitative review of the project site and pertinent information, the potential for lateral spreading at the site is judged to be low.

5.2 SETTLEMENT ANALYSIS

A significant geotechnical consideration for the project site is the presence of a soft, saturated clay and silt layer consisting of mixtures of debris and silt between depths of 3 to 15 feet bgs. These soils are assumed to be normally to slightly over-consolidated. These fine grained, compressible soils also have a relatively low undrained shear strength, which has a very significant influence on the estimated settlement from new fill and structural loads on the site.

Settlement analyses were completed using the computer program SETTLE3D, v.4 by Rocscience to consider the total and long-term consolidation of the clay layers below the bottom of the project site. Our analysis concluded that approximately 8 inches of settlement would result from the “gross” new structural loads.

5.3 FOUNDATIONS

The proposed plans show the modular buildings and access ramps supported on ground that has up to a 6-foot difference in existing fill thickness. Shallow foundations supported on grade would experience significant differential settlement from west to east due consolidation effects. Deep foundations, i.e. drilled piers or auger cast piles, are recommended in these conditions. Shallow foundations should only be considered for lightly loaded structures founded on similar fill thicknesses or where differential settlement can otherwise be mitigated.

5.4 SURCHARGE FILL

At your request, we provided a limited discussion of a potential surcharge program under separate cover. While a surcharge program could reduce the potential for long-term settlement, the time and expense required, and potential impacts to existing facilities made this not potentially viable.

5.5 HIGH GROUNDWATER LEVEL

Due to the high groundwater level at the site, which is typically about 3 feet below the ground surface (~El. 7) in the lower portions of the site, there will be significant groundwater expected in deeper foundation or utility trench excavations. Consideration should be made using deep foundations to mitigate the high groundwater table.

5.6 MOISTURE SENSITIVE SOILS AND WET WEATHER CONSTRUCTION

The soils anticipated to be encountered at the base of the excavation are expected to consist of wet and loose to medium dense sands and silt that are very sensitive to minor changes in moisture. Compaction difficulties will quickly occur if too much or too little moisture is added to the soil. Refer to the recommendations in Section 6.1 of this report for recommendations pertaining to moisture sensitive soils.

5.7 CORROSION TESTING

Corrosion testing was performed to estimate the corrosivity of the soil. Corrosion testing was performed using the Caltrans standard method of tests. One sample was tested for resistivity, chloride, sulfate, and pH. The results of the test are presented in Table 4-2 and in Appendix B.

Table 4-2. Corrosion Testing Results

Boring (depth in feet)	Resistivity (Ohm-cm)	Chloride (mg/kg)	Sulfate (mg/kg)	pH
B-2 (1.5-2.0)	1,435	290	54	6.6

Caltrans Corrosion Guidelines, May 2021, identifies a site to be corrosive for structural elements if one or more of the following conditions exist:

- Chloride concentration is 500 ppm or greater;
- Sulfate concentration is 1,500 ppm or greater;
- pH is 5.5 or less.

A minimum resistivity value for soil and/or water less than 1000 ohm-cm indicates the presence of high quantities of soluble salts and a higher susceptibility to being corrosive. Based on the results of the laboratory testing performed, the soil samples tested are not considered to be corrosive, based on the Caltrans criteria listed above.

According to ACI 318 Section 4.3, Table 4.3.1:

- Sulfate concentration below 0.10 percent by weight (1,000 ppm) is negligible (no restrictions on concrete type)
- Water-soluble chloride content of less than 500 ppm is generally considered non-corrosive to concrete.

Based on the results of the laboratory testing performed, the soil sample tested had values for Sulfate and Chloride that do not meet ACI criteria and is considered non-corrosive to concrete.

Corrosion results are to be considered preliminary and are an indicator of potential soil corrosivity for the sample tested. Other soils found on-site may be more, less, or of similar corrosive nature. Our scope of services does not include corrosion engineering; therefore, a detailed analysis of the corrosion tests is not included.

6.0 DESIGN RECOMMENDATIONS AND CONSTRUCTION CONSIDERATIONS

Detailed recommendations for the geotechnical aspects of the proposed improvements are presented in the subsequent sections of this report. Our evaluations and recommendations are based upon our analyses and evaluation of the previously discussed information that has been provided to us. The following recommendations may need to be modified if there are changes in the proposed improvements, their layout or location, or the proposed grading.

6.1 EARTHWORK

6.1.1 Demolition, Clearing, Stripping

Site clearing should include removal of existing structures and foundations, deleterious materials, debris, and obstructions that are designated for removal. Depressions, voids, and holes that extend below proposed finish grade should be cleaned and backfilled with engineered fill compacted to the recommendations in this report.

The designated building areas and associated improvements are currently heavily vegetated with shrubs and trees that will be removed from the project site prior to development. In addition, existing underground utilities will require removal, relocation, or protection during construction.

6.1.2 Excavations

Excavations for this site are anticipated to be up to 25 feet in depth and will include the following:

- Excavation for project foundations (25 ft),
- Excavation for the proposed lift station (14 ft), and
- Trenching for proposed utility lines (TBD).

The walls of excavations in the near-surface soil (<5 feet deep) should be able to stand near-vertical with minimal bracing, provided proper moisture content in the soil is maintained. Deeper trenches and excavations will likely require temporary shoring and dewatering. Excavations should be constructed in accordance with the current CAL-OSHA safety standards and local jurisdiction. The stability and safety of excavations, braced or unbraced, is the responsibility of the contractor.

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

Excavations deeper than 3 feet may encounter soil containing petroleum products or other constituents of concern. This may require additional personal protective equipment for construction personnel.

6.1.3 Subgrade Preparation

To mitigate disturbance of soil at the site from demolition and removal of existing site improvements, subgrade soil in areas to receive engineered fill, concrete slabs-on-grade, foundations or pavements should be scarified 12 inches, moisture conditioned and compacted to the recommendations given under Section 6.1.5. Engineered Fill Placement and Compaction. Depressions or holes created by removal of deeper features should be cleared of loose material and prepared as noted above prior to placing engineered fill.

Subgrade preparation should extend a minimum of 5 feet beyond the outermost limits of the fills, foundations, slabs, or pavements unless it is restricted. Prepared soil subgrades should be non-yielding when proof-rolled by a fully loaded water truck or equipment of similar weight. Moisture conditioning of subgrade soils should consist of adding water if the soils are too dry and allowing the soils to dry if the soils are too wet. After the subgrades have been prepared, the areas may be raised to design grades by the placement of engineered fill.

If unstable, wet, or soft soil is encountered, the soil will require processing before compaction can be achieved. When the construction schedule does not allow for air-drying, other means such as lime or cement treatment, over-excavation and replacement, geotextile fabrics, etc. may be considered to help stabilize the subgrade. The method to be used should be determined at the time of construction based on the actual site conditions. We recommend obtaining unit prices for subgrade stabilization during the construction bid process.

6.1.4 Material for Engineered Fill

In general, on-site soils with an organic content of less than 3 percent by weight, free of any hazardous or deleterious materials, and meeting the gradation requirements below may be used as general engineered fill to achieve project grades, except when special material (such as or capillary break material) is required.

In general, engineered fill material shall not contain rocks or lumps larger than 3 inches in greatest dimension, shall not contain more than 15 percent of the material larger than 1½ inches, and shall contain at least 20 percent passing the No. 200 sieve. In addition to these requirements, import fill shall have a low expansion potential as indicated by Plasticity Index of 15 or less, or Expansion Index of less than 20.

All import fills must be approved by the project geotechnical engineer prior to delivery to the site. At least five (5) working days prior to importing to the site, a representative sample of the proposed import fill should be delivered to our laboratory for evaluation.

6.1.5 Engineered Fill Placement and Compaction

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

Engineered fills consisting of on-site soils and imported soils should be compacted to a minimum of 90 percent relative compaction with moisture content between about 1 and 3 percent above the laboratory optimum value. In pavement areas, the upper 12 inches of subgrade soil and the full section of aggregate base should be compacted to a minimum of 95 percent relative compaction with moisture content 0 to 3% above the optimum value. Aggregate base in vehicle pavement areas should be compacted at slightly above the optimum moisture content to a minimum of 95 percent relative compaction.

6.1.6 Considerations for Soil Moisture and Seepage Control

Subgrade soil and engineered fill shall be compacted at moisture content meeting our recommendations. Once compacted, soils shall be protected from drying and wetting. This may be accomplished by regular watering with a water truck to prevent excessive drying or covering with plastic sheeting to prevent excessive wetting from rainfall.

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

6.1.7 Wet Weather Construction

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

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

6.2 SEISMIC DESIGN PARAMETERS

Due to the proximity of the site to the numerous active fault systems which traverse the greater San Francisco Bay Area, it is likely that the project site will be subjected to the effects of a major earthquake during the design life of the proposed improvements. The effects are likely to consist of significant ground accelerations. These ground-type movements may cause damage to the proposed improvements. We, therefore, recommend that at a minimum the structural systems for the proposed improvements be designed in accordance with the requirements of Chapter 16 of the 2022 California Building Code and ASCE 7-16, Supplement 3, for Site Class E type soils and Risk Category 2. The California Building Code seismic design parameters for the site are included in Table 6-1.

Table 6-1. 2022 CBC Seismic Design Parameters

Item	Design Value
Site Soil Class Definition	E
S_s – 0.2 Second Spectral Response Acceleration	1.879
S_1 – 1.0 Second Spectral Response Acceleration	0.717
Fa – Values of Site Coefficient	1.2
Fv[1] – Value of Site Coefficient	--
S_{DS} – Designed Spectral Response Acceleration for Short Periods	1.504
S_{D1}[1] – Designed Spectral Response Acceleration for 1-Sec Periods	--
S_{MS}[1] – MCE_R Spectral Response Acceleration Parameter (g)[2]	--
S_{M1}[1] – MCE_R Spectral Response Acceleration Parameter (g)[2]	--
PGA	0.789
PGA_M	0.868

Notes:

1) *Values of Fv, S_{M1}, and S_{D1} are undefined for this site class without performance of a site-specific ground motion hazard analysis. See ASCE 7-16 Section 11.*
2) *g = acceleration of gravity*
3) *Design values presented above are based on a site latitude / longitude = 37.87310 / -122.30739.*

6.3 CIDH PILES AND AUGER CAST PILES

To mitigate consolidation settlement due to loads, we recommend a deep foundation system consisting of CIDH piles or ACP be used for support of the field house building and adjacent retaining walls. The structural designer may consider preparing a reference design and having the final design prepared by a Design-Build specialty contractor that is experienced with these foundation types.

We recommend these structures be supported on a foundation system using 18- or 24-inch CIDH or ACP with a minimum concrete compressive strength of 5,000 pounds per square inch (psi). A reinforced concrete mat slab with grade beams may be used as a pile cap to structurally connect the piles and support the modular buildings. The thickness of this mat will be determined by the project structural engineer.

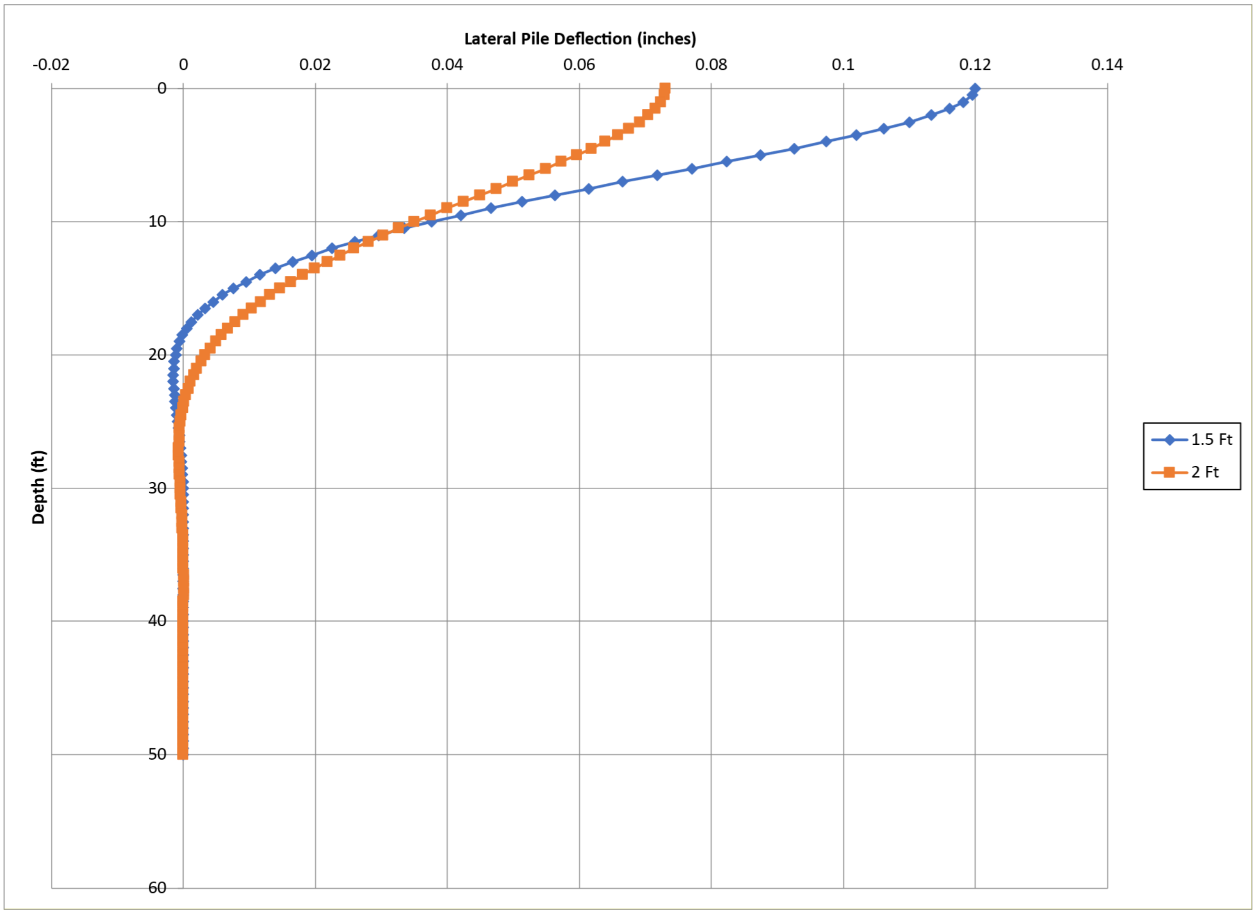
We estimated that the piles will need to carry approximately 15 kips each and will be installed in a linear pattern with a minimum tip elevation of -12 feet (NAVD88) and a spacing between piles of approximately 8 to 10 feet in both directions. We should review these estimates as the design progresses to adjust the size and depth if necessary.

6.3.1 Vertical Capacity

The vertical load capacities include both side friction on the pile shaft as well as end bearing on the pile tip for ACP. This minimum tip elevation is intended for the piles to be embedded sufficiently to bear at the same elevation, achieve the minimum capacity and achieve fixity for lateral loads. Piles should be spaced a minimum of 3 pile diameters apart, measured center-to-center. We should be contacted regarding a capacity reduction if piles are more closely spaced. A reinforced concrete pile cap or mat designed by the project structural engineer should provide structural connection between the piles.

6.3.2 Lateral Load Resistance

Resistance to lateral loading has been estimated for the pile foundations. The ultimate lateral load resistance of pile foundations is a function of the stiffness of the surrounding soil, the stiffness of the pile, the resulting deflection at the top of the pile, and the moment capacity of the pile cross-section. The lateral load response for individual 18-inch and 24-inch CIDH and ACP was assessed using “fixed head” and “free head” conditions using the computer program Lpile v.2019 by Ensoft. A representative soil profile was determined from the borings to estimate the passive resistance. Pile head deflection vs. depth is presented in the following chart based on gross cross-sectional properties and a 10-kip lateral load for a free-head condition. The structural engineer should decide on the use of a free- or fixed-head condition depending on how lateral loads are accounted for in design and what the tolerance for deflection is.



6.4 SPREAD FOOTINGS

Continuous and isolated spread footings may be used to support the ramp foundation that leads to the field house if differential settlements can be mitigated. The ramp is anticipated to impose a maximum allowable bearing pressure of 1,500 pounds per square foot on foundation soils when considering dead plus normal live loading. This allowable foundation soil pressure may be increased by one-third when considering short term wind or seismic loading. We recommend that the footings be embedded a minimum of 24 inches below the lowest adjacent finished grade.

Soil resistance to lateral loads will be provided by a combination of frictional resistance between the bottom of the footings and underlying soils and by passive pressures acting against the embedded sides of the footings. For frictional resistance, an ultimate coefficient of friction of 0.30 may be used for design. In addition, an allowable passive lateral bearing pressure equal to an equivalent fluid pressure of 235 psf/ft may be used provided the footings are poured tight against undisturbed native or compacted soils. These values may be used in combination without reduction. The upper 12 inches of soil should be neglected when calculating passive resistance unless covered by concrete slabs or pavement.

Concrete should be placed only in excavations that are clean and free of loose soils or debris. Foundation excavations should be maintained in a moist condition prior to placement of concrete. A member of our staff should observe foundation excavations to verify that adequate foundation bearing soils have been reached. The project structural engineer should determine the foundation reinforcement.

Settlements are expected to be primarily elastic with most of the settlement occurring immediately upon application of load. Long term settlement of the foundation system are anticipated to be less than 1/2-inch with differential settlements on the order of 1/4-inch or less for a distance of 25 feet.

6.5 CONCRETE SLABS-ON-GRADE

The use of concrete slabs-on-grade are anticipated for exterior patios, walkways, etc. Soil subgrade shall be maintained in a moist condition prior to pouring the concrete slab. Subgrades beneath concrete slabs should consist of a minimum of 6 inches of Class 2 aggregate base compacted to 95 percent relative compaction (ASTM D1557, latest edition). The Class 2 aggregate base material should conform to Section 26 of the Caltrans Standard Specifications

To reduce the potential for cracking of the concrete slabs, we recommend that the slabs be a minimum of 5 inches thick. The slabs should include minimum reinforcement of #3 bars in both directions at 12-inch centers or #4 bars in both directions at 18-inch centers. The steel should be placed in the middle of the slab and should be held in place by dobie blocks or other suitable means. Actual dimensions and reinforcement shall be determined by the project Structural Engineer.

Even with the steel reinforcement and base rock, it should be recognized that some cracking and differential movement of the slabs will likely occur and should be expected. Exterior concrete slabs-on-grade shall be cast free from adjacent footings or other non-heaving edge restraints. This may be accomplished by using a strip of 1/2-inch asphalt-impregnated felt divider material between the slab edges and the adjacent structure. Construction and/or control joints should be provided in concrete slabs as recommended by the structural engineer.

6.6 SITE RETAINING WALLS

Retaining walls are anticipated to be used to support the field house building and access ramp and should be designed to resist lateral earth pressures recommended below.

6.6.1 Lateral Earth Pressures

Static lateral earth pressure will be imposed on all retaining structures, including retaining walls and temporary shoring. Table 6-2 summarizes the lateral earth pressures recommended for use in design. Active pressure should be assumed for conditions where the top of the wall is free to deflect up to ½ inch, otherwise at-rest pressure should be used. Passive pressure should be ignored for a depth of 12 inches and may be utilized to resist overturning and sliding. Where structures will be located below groundwater, hydrostatic pressures are already considered in the passive lateral earth pressure values shown in Table 6-2.

Table 6-2. Lateral Earth Pressures

Pressure Type	Above Groundwater Level (Equiv. Fluid Pressure)	Below Groundwater Level (Buoyant Equiv. Fluid Pressure + Hydrostatic)
Active	45 pcf	85 pcf
At-Rest	65 pcf	90 pcf
Passive	215 pcf	145 pcf

6.6.2 Drainage

Drainage for retaining structures may be provided by a subdrain system behind the retaining walls. The system should consist of a minimum 4-inch diameter perforated pipe, placed with the perforations facing downward, and embedded in a 12-inch-wide layer of Caltrans Class 2 permeable material. Native clayey soil or aggregate base and asphalt pavement should be used for the upper foot of wall backfill in order to cap the drainage material from infiltrating surface water. As an alternative to the Class 2 Permeable drainage material, a clean coarse open-graded gravel or drain rock may be used. If coarse gravel or drain rock is selected as a drainage material it should be separated from all adjacent soil by an engineering filter fabric such as Mirafi 140N, or a similar geotextile. The subdrain pipe shall be connected to a free-draining outlet.

6.7 VEHICLE PAVEMENTS

New pavements will be constructed as part of this project to provide vehicular access proposed improvements. We understand a design Traffic Index ranging from 4 to 7 is to be used for design. For pedestrian pavements that may receive occasional service vehicle traffic, a Traffic Index of 4 may be assumed.

Laboratory testing indicates a subgrade R-value of 8 for the project site. Flexible pavement sections have been developed given these assumptions and are presented in Table 6-3. The actual pavement section for the proposed driveways shall be determined by the project Civil Engineer.

Table 6-3: Flexible Pavement Section Design for R-Value = 5

Traffic Index	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	Total Pavement (inches)
4.0	2.5	7.0	9.5
4.5	2.5	7.0	9.5
5.0	2.5	10.5	13.0
5.5	3.0	11.5	14.5
6.0	3.0	13.0	16.0
6.5	3.5	14.0	17.5
7.0	4.0	14.5	18.5

Pavement sections shall be placed on soil surfaces that have been prepared as outlined in the Earthwork section of this report. The full section of aggregate base should be compacted to a minimum of 95 percent relative compaction (ASTM D1557, latest edition).

Asphalt concrete should meet the requirements for 1/2- or 3/4-inch maximum, medium Type A Hot Mix Asphalt (asphalt concrete), Section 39, Caltrans Standard Specifications, latest edition. The Class 2 aggregate base material should conform to Section 26 of the Caltrans Standard Specifications.

If concrete slabs are used for pavements (rigid pavement section) for Traffic Indices less than 9, we recommend the slabs consist of a minimum of 9 inches of concrete and be underlain by a minimum of 12 inches of Class 2 aggregate base compacted to a minimum of 95 percent relative compaction.

6.8 PICKLEBALL COURTS

Pickleball ball courts will be constructed with either an asphalt concrete or concrete surface. We understand that post-tensioned concrete will be used if a concrete surface is selected in design. For an asphalt surface, we recommend a section consisting of 3 inches of asphalt concrete over 9 inches of Class 2 aggregate base rock. Construction should follow the recommendations in Section 6.7 Vehicle Pavements.

Post-tensioned slab-on-grade sections should be designed based on the parameters contained in Table 6-4 should be used as input to develop geotechnical design criteria for a post-tensioned slab-on-grade system. These input parameters are based on our evaluation of the site conditions and laboratory test data.

Table 6-4. Geotechnical Input Parameters

Parameter	Value
Liquid Limit (LL)	32
Plastic Limit (PL)	18
Plasticity Index (PI)	14
Percent Passing #200 Sieve	70
Percent finer than 2 microns	35
Dry Density	115 pcf

The slab system should be designed by the structural engineer based on the latest Post Tensioning Institute guidelines for slab-on-grade sport courts. We recommend that geotechnical design criteria presented in Table 6-5 be incorporated into the design of the pickleball court system.

Table 6-5. Post Tensioned Slab-On-Grade Geotechnical Design Parameters

Parameters	Value
Allowable Bearing Capacity[1]	1,500 psf
Friction Coefficient	0.32
Passive Equivalent Fluid Pressure[2]	300 pcf
Differential Soil Movement (y_m)	
Center Lift	1.05 inches
Edge Lift	0.6 inches
Edge Variation Distance (e_m)	
Center Lift	5 feet
Edge Lift	4.3 feet
Minimum Slab Thickness	10 inches
Cantilever Length (I_c)	6 feet

[1] Allowable bearing capacity may be increased by 1/3 for wind and seismic loading.
[2] Neglect the upper 12 inches unless the ground is confined by the slab and/or pavement.

6.9 TECHNICAL REVIEW AND CONSTRUCTION OBSERVATION

Prior to construction the geotechnical engineer should review the project plans and specifications for conformance with the intent of the recommendations presented in this report. The geotechnical engineer should be contacted a minimum of 48 hours in advance of excavation operations to observe the subsurface conditions

7.0 LIMITATIONS

The conclusions and recommendations presented in this report are based on the information provided during the development of this report regarding the planned design and construction, and the results of the geologic mapping, subsurface exploration, and testing, combined with interpolation of the subsurface conditions between boring locations. Site conditions described in the text of this report are those existing at the time of our last field reconnaissance and are not necessarily representative of the site conditions at other times or locations. This information notwithstanding, the nature and extent of subsurface variations between borings may not become evident until construction. If variations are encountered during construction, Cal Engineering & Geology, Inc. should be notified promptly so that conditions can be reviewed and recommendations reconsidered, as appropriate.

It is the City's responsibility to ensure that recommendations contained in this report are carried out during the construction phases of the project. This report was prepared based on preliminary design information provided which is subject to change during the design process. At approximately the 90 percent design level, Cal Engineering & Geology, Inc. should review the design assumptions made in this report and prepare addenda or memoranda as appropriate. Any modifications included in these addenda or memoranda should be carefully reviewed by the project designers to make sure that any conclusions or recommendations that are modified are accounted for in the final design of the project.

The findings of this report should be considered valid for a period of three years unless the conditions of the site change. After a period of three years, CE&G should be contacted to review the site conditions and prepare a letter regarding the applicability of this report.

This report presents the results of a geotechnical and geologic investigation only and should not be construed as an environmental audit or study. The evaluation or identification of the potential presence of hazardous materials at the site was not requested and was beyond the scope of this investigation and report.

The conclusions and recommendations contained in this report are valid only for the project described in this report. We have employed accepted geotechnical engineering procedures, and our professional opinions and conclusions are made in accordance with generally accepted geotechnical engineering principles and practices. This standard is in lieu of all other warranties, either expressed or implied.

8.0 REFERENCES

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California Geological Survey, 2003, Seismic Hazard Zone Report for the West Oakland 7.5-minute Quadrangle, Alameda County, California: California Geological Survey Seismic Hazard Zone Report 090 (rev).

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BASEMAP REFERENCE

1. STREET CENTERLINES FROM CALTRANS CALIFORNIA ROAD SYSTEM, DOWNLOADED ON 18 FEB 2020.
2. ORTHOIMAGERY FROM ESRI (MAXAR), 2019.

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MILES

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400 - 408 GILMAN ST
BERKELEY, CALIFORNIA

SITE LOCATION MAP

220960	APRIL 2024	FIGURE 1

REFERENCES

1. TOPOGRAPHIC SURVEY AND SITE PLAN FROM TRANSYSTEMS; CAD FILES RECEIVED ON 11/11/2022 (SURVEY) AND 05/10/2024 (SITE PLAN).
2. ORTHOIMAGERY FROM MAXAR, 2022.

SUBSURFACE EXPLORATIONS

B-5 BORING BY CE&G (11/11/2022)

B-7 BORING BY MPEG (9/8/2005)

CPT-1 CPT BY USGS (6/21/2001)

30 0 30 60
SCALE FEET

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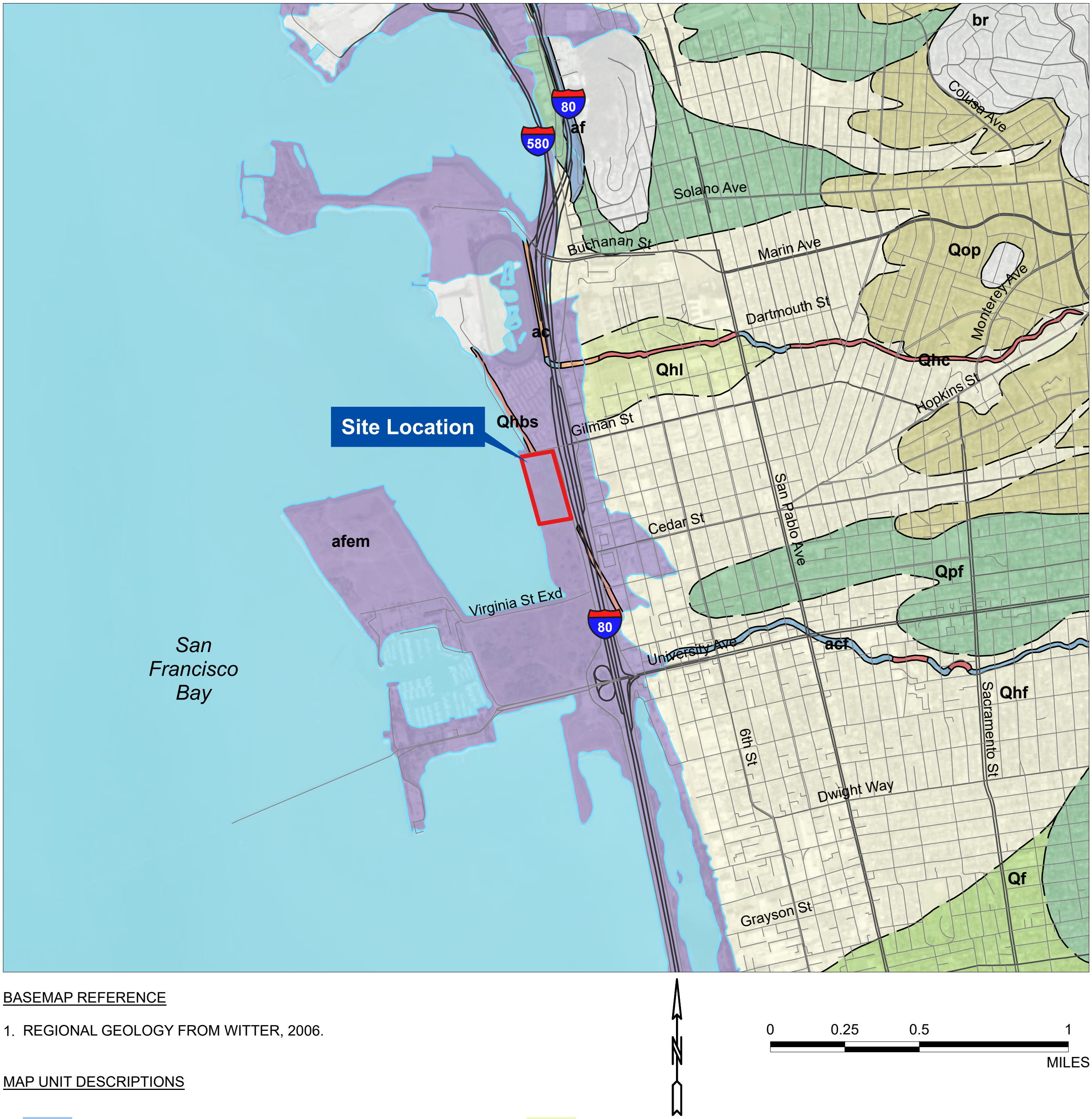
SITE PLAN

220960	APRIL 2024	FIGURE 2



BASEMAP REFERENCE

1. REGIONAL GEOLOGY FROM WITTER, 2006.

MAP UNIT DESCRIPTIONS

af	Artificial fill (Historical)
afem	Artificial fill over estuarine mud (Historical)
acf	Artificial channel fill (Historical)
ac	Artificial stream channel (Historical)
Qhc	Historical stream channel deposits (Historical)
Qhbs	Beach sand (Latest Holocene)
Qhf	Alluvial fan deposits (Holocene)
Qhl	Alluvial fan levee deposits (Holocene)
Qf	Alluvial fan deposits (Holocene to Latest Pleistocene)
Qpf	Alluvial fan deposits (Latest Pleistocene)
Qop	Pediment deposits (Early to Late Pleistocene)
br	Older deposit and bedrock (Early Quaternary and Older)
H2O	WATER

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REGIONAL GEOLOGY

220960 | APRIL 2024 | FIGURE 3

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MILES

BASEMAP REFERENCE

1. LIQUEFACTOIN SUSCEPTIBILITY FROM WITTER ET AL., 2006.

MAP UNIT DESCRIPTIONS

- VERY HIGH
- HIGH
- MODERATE
- LOW
- VERY LOW
- NOT MAPPED

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BERKELEY, CALIFORNIA

LIQUEFACTION SUSCEPTIBILITY

220960	APRIL 2024	FIGURE 4

BASEMAP REFERENCE

1. FAULT LOCATIONS FROM US GEOLOGICAL SURVEY QUATERNARY FAULTS AND FOLDS DATABASE, ACCESSED ONLINE ON 30 JULY 2021.
2. ORTHOIMAGERY FROM ESRI (MAXAR), 2019.

N

0 1.5 3 6
MILES

MAP UNIT DESCRIPTIONS

HISTORICAL (<150 YEARS), WELL CONSTRAINED LOCATION

HISTORICAL (<150 YEARS), MODERATELY CONSTRAINED LOCATION

HISTORICAL (<150 YEARS), INFERRED LOCATION

LATEST QUATERNARY (<15,000 YEARS), WELL CONSTRAINED LOCATION

LATEST QUATERNARY (<15,000 YEARS), MODERATELY CONSTRAINED LOCATION

LATEST QUATERNARY (<15,000 YEARS), INFERRED LOCATION

UNDIFFERENTIATED QUATERNARY(<1.6 MILLION YEARS), WELL CONSTRAINED LOCATION

UNDIFFERENTIATED QUATERNARY(<1.6 MILLION YEARS), MODERATELY CONSTRAINED LOCATION

UNDIFFERENTIATED QUATERNARY(<1.6 MILLION YEARS), INFERRED LOCATION

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FAULT ACTIVITY MAP

220960	APRIL 2024	FIGURE 5

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Appendix A. Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

Field Identification			Group Symbols	Typical Names	Laboratory Classification Criteria	
Coarse-Grained Soils More than 50% of material is retained on the No. 200 sieve.	**Gravels** More than 50% coarse fraction retained on the No. 4 sieve	**Clean Gravels** < 5% Fines	**GW**	Well-graded gravels, gravel-sand mixtures, little or no fines	$C_U = D_{60} \div D_{10} \geq 4$ and $C_C = (D_{30})^2 \div (D_{10} \times D_{60}) \geq 1$ & ≤ 3	
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines	$C_U = D_{60} \div D_{10} < 4$ and/or $C_C = (D_{30})^2 \div (D_{10} \times D_{60}) < 1$ & > 3	
		Gravels with Fines >12% Fines	**GM**	Silty gravels, poorly graded gravel-sand-silt mixtures	Fines classify as ML or MH	If fines classify as CL-ML, use dual symbol GC/GM
			GC	Clayey gravels, poorly graded gravel-sand-clay mixtures	Fines classify as CL or CH	
	Sands More than 50% coarse fraction passes the No. 4 sieve	**Clean Sands** < 5% Fines	**SW**	Well-graded sands, gravelly sands, little or no fines	$C_U = D_{60} \div D_{10} \geq 6$ and $C_C = (D_{30})^2 \div (D_{10} \times D_{60}) \geq 1$ & ≤ 3	
			SP	Poorly graded sands, gravelly sands, little or no fines	$C_U = D_{60} \div D_{10} < 6$ and/or $C_C = (D_{30})^2 \div (D_{10} \times D_{60}) < 1$ & > 3	
		Sands with Fines >12% Fines	**SM**	Silty sands, poorly graded sand-silt mixtures	Fines classify as ML or MH	If fines classify as CL-ML, use dual symbol SC/SM
			SC	Clayey sands, poorly graded sand-clay mixtures	Fines classify as CL or CH	

CLASSIFICATION OF GRAVELS & SANDS WITH 5% TO 12% FINES REQUIRES DUAL SYMBOLS

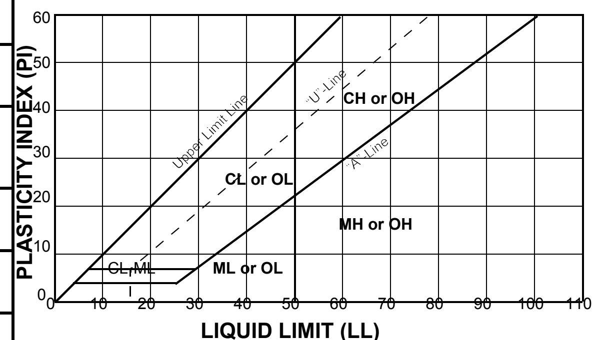
GW/GM or GP/GM: Gravel/Silty Gravel
GW/GC or GP/GC: Gravel/Clayey Gravel
SW/SM or SP/SM: Sand/Silty Sand
SW/SC or SP/SC: Sand/Clayey Sand

Field Identification		Group Symbols	Typical Names
Fine-Grained Soils More than 50% of material passes the No. 200 sieve.	**Identification Procedures on Percentage Passing the No. 40 Sieve**		
	Silts & Clays Liquid Limit less than 50%	**ML**	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands with slight plasticity
		CL	Inorganic clays of low to medium plasticity, gravelly, sandy, and/or silty clays, lean clays
		OL	Organic silts, organic silty clays of low plasticity
	Silts & Clays Liquid Limit greater than 50%	**MH**	Inorganic silts, micaceous or diatomaceous fine sandy/-silty soil, elastic silts
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic clays of medium to high plasticity
HIGHLY ORGANIC SOILS		**PT**	Peat and other highly organic soils

PLASTICITY CHART

For Classification of Fine-Grained Soils and Fine-Grained Fraction of Coarse-Grained Soils

Equation of "A"-Line: PI = 4 @ LL = 4 to 25.5, then PI = 0.73 × (LL − 20)
Equation of "U"-Line: LL = 16 @ PI = 0 to 7, then PI = 0.9 × (LL − 8)



KEY TO SAMPLER TYPES AND OTHER LOG SYMBOLS

CS	California Standard Sampler
CM	California Modified Sampler
SPT	Standard Penetration Test Sampler
SHL	Shelby Tube Sampler
BU	Bulk Sample
LL	Liquid Limit of Sample (ASTM D-4318)
PI	Plasticity Index of Sample (ASTM D-4318)
Q_U	Unconfined Compression Test (ASTM D-2166)
▽	Depth at which Groundwater was Encountered During Drilling
▼	Depth at which Groundwater was Measured After Drilling
PP	Pocket Penetrometer Test
PTV	Pocket Torvane Test
-#200	% of Material Passing the No. 200 Sieve Test (ASTM D-1140)
PSA	Particle-Size Analysis (ASTM D-422 & D-1140)
C	Consolidation Test (ASTM D-2435)
TXUU	Unconsolidated Undrained Compression Test (ASTM D-2850)

KEY TO SAMPLE INTERVALS

CS	Length of Sampler Interval with a CS Sampler
☒	Length of Sampler Interval with a CM Sampler
	Length of Sampler Interval with a SPT Sampler
SHL	Length of Sampler Interval with a SHL Sampler
BU	Bulk Sample Recovered for Interval Shown (i.e., cuttings)
	Length of Coring Run with Core Barrel Type Sampler
NR	No Sample Recovered for Interval Shown

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UNIFIED SOIL CLASSIFICATION SYSTEM AND KEY TO BORING LOG

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KEY TO SYMBOLS

CLIENT Transystems
PROJECT NAME Tom Bates Regional Sports Complex
PROJECT NUMBER 220960
PROJECT LOCATION 400 Gilman Street, Berkeley CA

LITHOLOGIC SYMBOLS (Unified Soil Classification System)

ASPHALT: Asphalt

CH: USCS High Plasticity Clay

CL: USCS Low Plasticity Clay

CL-CH: USCS Low to High Plasticity Clay

GW: USCS Well-graded Gravel

ML: USCS Silt

OH: USCS High Plasticity Organic silt or clay

PT: USCS Peat

SC: USCS Clayey Sand

SAMPLER SYMBOLS

California Modified Sampler

Shelby Tube

Standard Penetration Test

WELL CONSTRUCTION SYMBOLS

ABBREVIATIONS

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

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

Water Level at Time Drilling, or as Shown

Water Level at End of Drilling, or as Shown

Water Level After 24 Hours, or as Shown

CE&G
CAL ENGINEERING & GEOLOGY

CLIENT Transystems		**PROJECT NAME** Tom Bates Regional Sports Complex	
PROJECT NUMBER 220960		**PROJECT LOCATION** 400 Gilman Street, Berkeley CA	
DATE STARTED 11/1/2022	**COMPLETED** 11/1/2022	**GROUND ELEVATION** 10 ft **DATUM** WGS84	**HOLE SIZE** 8 in.
DRILLING CONTRACTOR Exploration Geoservices		**COORDINATES: LATITUDE** 37.877122	**LONGITUDE** -122.308872
DRILLING RIG/METHOD Moble B-53/8-in. Hollowstem Auger		∇ **GROUNDWATER AT TIME OF DRILLING** 3.0 ft / Elev 7.0 ft	
LOGGED BY R. Briseno	**CHECKED BY** C. Nardi	**GROUNDWATER AT END OF DRILLING** ---	
HAMMER TYPE 140 lb hammer with 30 in. cable drop		**GROUNDWATER AFTER DRILLING** ---	

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
0		3 in. ASPHALT									
		4 in. baserock									
		Lean to Fat CLAY (CL-CH): brown, bluish gray, greenish gray, gray, (heterogenous coloring), moist, soft to stiff, trace very fine sand, claystone rock fragments (ARTIFICIAL FILL)	CM	5-7-10	.5	108	20				
		∇ TXUU @ 2.0 ft	SPT	5-7-8	3						
5		PEAT (PT): black, moist, soft, organic odor, fibrous material	CM	7-6-4							
		Fat CLAY (CL): black, wet, very soft, organic odor	SPT	2-3-8							
		SILT (ML): very dark gray, moist, soft, odorous									
10		Organic CLAY (OH): black, wet, very soft, odorous	SH								
			CM	3-6-11							
15			SPT	5-10-12							
		SILT (CL): bluish gray, wet, very soft									
20		SILT (CL): bluish gray, wet, very soft	CM	11-15-17		110	23				
		Change indicated by driller									
25		Clayey SAND with Gravel (SC): brown, wet, medium dense, fine rounded gravel, fine to coarse sand, iron and manganese stains	SPT	18-13-18							28
30		Clayey SAND (SC): brown, wet, medium dense, fine to coarse sand, iron and manganese stains	CM	9-18-40							
35		Lean CLAY (CL): brown, moist, stiff, trace coarse rounded sand, iron and	SPT	5-12-19							

(Continued Next Page)

BORING NUMBER B-01

CAL ENGINEERING & GEOLOGY

CLIENT Transystems

PROJECT NAME Tom Bates Regional Sports Complex

PROJECT NUMBER 220960

PROJECT LOCATION 400 Gilman Street, Berkeley CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
35		manganese stains									
40		Lean to Fat CLAY (CL-CH): gray, moist, stiff, iron stains TXUU @ 39.5 ft	CM	12-17-28	3 3.5	120	11				
45		Silty Lean CLAY (CL): brown, moist, medium stiff	CM	14-22-33	2.5 2.25						
50		Lean CLAY (CL): light greenish gray, moist, soft, trace rounded gravel up to 1 in., trace sand, iron stains	CM	12-15-21	1.5 1						

Bottom of borehole at 50.0 ft. Borehole backfilled with neat cement grout.

BORING NUMBER B-02

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CLIENT Transystems
PROJECT NAME Tom Bates Regional Sports Complex
PROJECT NUMBER 220960
PROJECT LOCATION 400 Gilman Street, Berkeley CA
DATE STARTED 11/1/2022 **COMPLETED** 11/1/2022
GROUND ELEVATION 10 ft **DATUM** WGS84 **HOLE SIZE** 8 in.
DRILLING CONTRACTOR Exploration Geoservices
COORDINATES: LATITUDE 37.876849 **LONGITUDE** -122.308805
DRILLING RIG/METHOD Moble B-53/8-in. Hollowstem Auger
∇ **GROUNDWATER AT TIME OF DRILLING** 3.0 ft / Elev 7.0 ft
LOGGED BY R. Briseno **CHECKED BY** C. Nardi
GROUNDWATER AT END OF DRILLING ---
HAMMER TYPE 140 lb hammer with 30 in. cable drop
GROUNDWATER AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
0		Lean CLAY (CL): brown, moist, hard (ARTIFICIAL FILL) Corrosion @ 1.5 to 2.0 ft	CM	7-11-14	4.5+						
		∇ Lean to Fat CLAY (CL-CH): brown, bluish gray, greenish gray, gray, (heterogenous coloring), moist, stiff, iron stains Corrosion @ 1.5 - 2.0 ft	SPT	11-17-25							
5		PEAT (PT): black, moist, very soft, organic odor, fibrous material	CM	15-6-11							
			SPT	7-5-10							
10		PEAT (PT): black, moist, very soft, organic odor, fibrous material	CM	7-10-9		73	49				
			SPT	6-3-5							
15			CM	5-9-12		90	34				
		Clayey SILT (ML): greenish gray, wet, very soft									
20		SILT (ML): bluish gray, wet, very soft	CM	9-14-18							

Bottom of borehole at 20.0 ft. Borehole backfilled with neat cement grout.

BORING NUMBER B-03

CE&G
CAL ENGINEERING & GEOLOGY

CLIENT Transystems
PROJECT NAME Tom Bates Regional Sports Complex
PROJECT NUMBER 220960
PROJECT LOCATION 400 Gilman Street, Berkeley CA
DATE STARTED 11/1/2022 **COMPLETED** 11/1/2022
GROUND ELEVATION **DATUM** WGS84 **HOLE SIZE** 8 in.
DRILLING CONTRACTOR Exploration Geoservices
COORDINATES: LATITUDE 37.876329 **LONGITUDE** -122.308692
DRILLING RIG/METHOD Moble B-53/8-in. Hollowstem Auger
GROUNDWATER AT TIME OF DRILLING 11.0 ft
LOGGED BY R. Briseno **CHECKED BY** C. Nardi
GROUNDWATER AT END OF DRILLING ---
HAMMER TYPE 140 lb hammer with 30 in. cable drop
GROUNDWATER AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
0		Lean CLAY (CL): heterogenous browns, moist, hard (ARTIFICIAL FILL)	CM	27-35-33		103	11				
			SPT	17-17-18							
5		color change to greenish gray	CM	17-50/4"							
			SPT								
10			CM	20-17-17							
			SPT	3-3-3							
15		PEAT (PT): black, moist, very soft, organic odor, fibrous material	SPT	6-8-9							
20		SILT (ML): black, wet, very soft	CM	4-5-6							

Bottom of borehole at 20.0 ft. Borehole backfilled with neat cement grout.

BORING NUMBER B-04

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CAL ENGINEERING & GEOLOGY

CLIENT Transystems
PROJECT NAME Tom Bates Regional Sports Complex
PROJECT NUMBER 220960
PROJECT LOCATION 400 Gilman Street, Berkeley CA
DATE STARTED 11/1/2022 **COMPLETED** 11/1/2022
GROUND ELEVATION **DATUM** WGS84 **HOLE SIZE** 8 in.
DRILLING CONTRACTOR Exploration Geoservices
COORDINATES: LATITUDE 37.876579 **LONGITUDE** -122.309204
DRILLING RIG/METHOD Moble B-53/8-in. Hollowstem Auger
GROUNDWATER AT TIME OF DRILLING ---
LOGGED BY R. Briseno **CHECKED BY** C. Nardi
GROUNDWATER AT END OF DRILLING ---
HAMMER TYPE 140 lb hammer with 30 in. cable drop
GROUNDWATER AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTICITY INDEX (%)	FINES CONTENT (%)
0		Lean CLAY (CL): heterogenous browns, moist, hard (ARTIFICIAL FILL) R-Value @ 1.0 to 5.0 ft	CM	10-13-16		98	11	32	18	14	
			SPT	12-11-13				32	18	14	
5			CM	13-15-16/0"		101	15				
		color change to dark gray to black color change to brown	SPT	7-9-9							
10			CM	10-12-39							
		Clayey SILT (ML): light green, moist, medium dense, trace sand	SPT	12-16-24							
15			CM	15-9-9							

Bottom of borehole at 15.0 ft. Borehole backfilled with neat cement grout.

BORING NUMBER B-05

CAL ENGINEERING & GEOLOGY

CLIENT Transystems

PROJECT NAME Tom Bates Regional Sports Complex

PROJECT NUMBER 220960

PROJECT LOCATION 400 Gilman Street, Berkeley CA

DATE STARTED 11/1/2022 **COMPLETED** 11/1/2022

GROUND ELEVATION **DATUM** WGS84 **HOLE SIZE** 8 in.

DRILLING CONTRACTOR Exploration Geoservices

COORDINATES: LATITUDE 37.876275 **LONGITUDE** -122.309066

DRILLING RIG/METHOD Moble B-53/8-in. Hollowstem Auger

GROUNDWATER AT TIME OF DRILLING ---

LOGGED BY R. Briseno **CHECKED BY** C. Nardi

GROUNDWATER AT END OF DRILLING ---

HAMMER TYPE 140 lb hammer with 30 in. cable drop

GROUNDWATER AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	PLASTICITY INDEX (%)	FINES CONTENT (%)
0		Lean CLAY (CL): heterogenous browns, moist, hard, few sand, trace rounded pea gravel, iron stains (ARTIFICIAL FILL)	CM	14-19-18		101	14				
			CM	11-18-19							
5			CM	10-12-18							

Bottom of borehole at 6.5 ft. Borehole backfilled with neat cement grout.

Appendix B. Laboratory Testing

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SUMMARY OF LABORATORY RESULTS

CLIENT Transystems

PROJECT NAME Tom Bates Regional Sports Complex

PROJECT NUMBER 220960

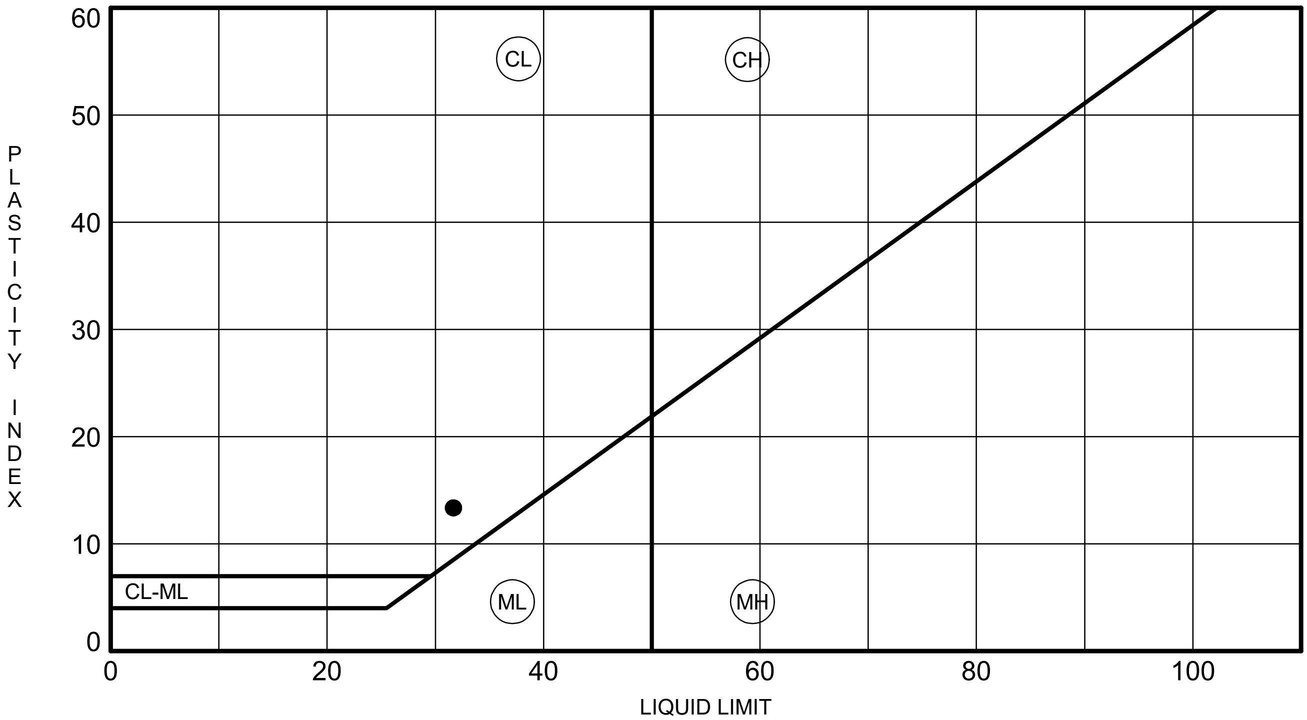
PROJECT LOCATION 400 Gilman Street, Berkeley CA

Borehole	Depth	Date Tested	Liquid Limit	Plastic Limit	Plasticity Index	Maximum Screen Size (mm)	%<#200 Sieve	Class-ification	Water Content (%)	Dry Density (pcf)	Satur-ation (%)	Void Ratio
B-01	2.0	12/2/2022							19.8	108.2		
B-01	19.5	11/11/2022							23.1	110.1		
B-01	23.5	11/11/2022				0.106	28					
B-01	39.5	12/2/2022							10.7	119.8		
B-02	9.5	11/11/2022							49.3	72.8		
B-02	14.5	11/11/2022							34.2	90.0		
B-03	2.0	11/11/2022							10.9	103.4		
B-04	1.5	11/11/2022	32	18	14							
B-04	2.0	11/11/2022							11.5	98.3		
B-04	2.5	11/11/2022	32	18	14							
B-04	6.0	11/11/2022							14.6	101.2		
B-05	2.0	11/11/2022							14.2	100.6		

ATTERBERG LIMITS RESULTS

CAL ENGINEERING & GEOLOGY

CLIENT TranSystem
PROJECT NAME Tom Bates Regional Sports Complex
PROJECT NUMBER 220960
PROJECT LOCATION 400 Gilman Street, Berkeley CA

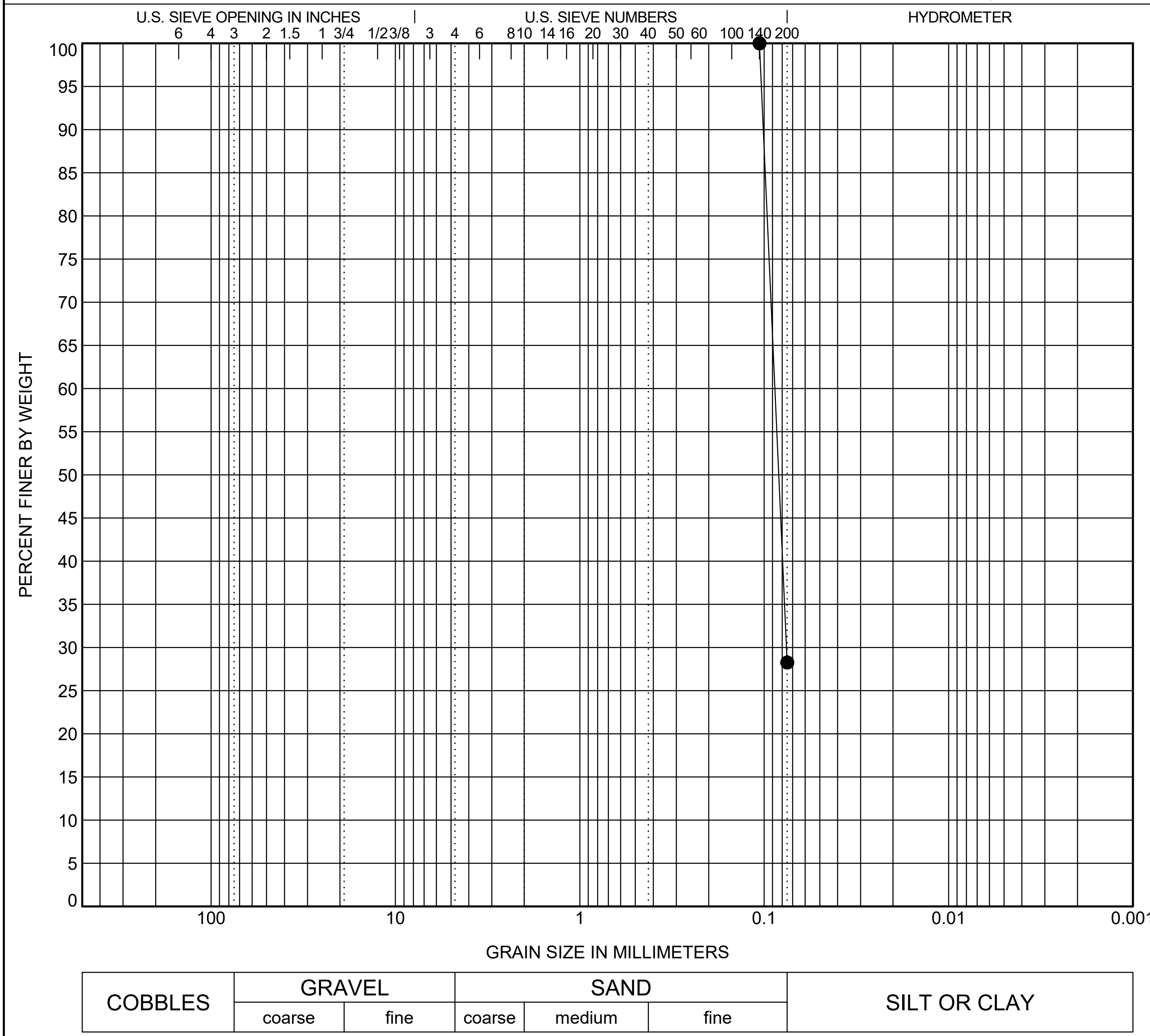


	BOREHOLE	DEPTH	LL	PL	PI	Fines	Classification	Date Tested
●	**B-04**	**1.5 to 2.5**	**32**	**18**	**14**			**11/11/2022**

GRAIN SIZE DISTRIBUTION

CE&G
CAL ENGINEERING & GEOLOGY

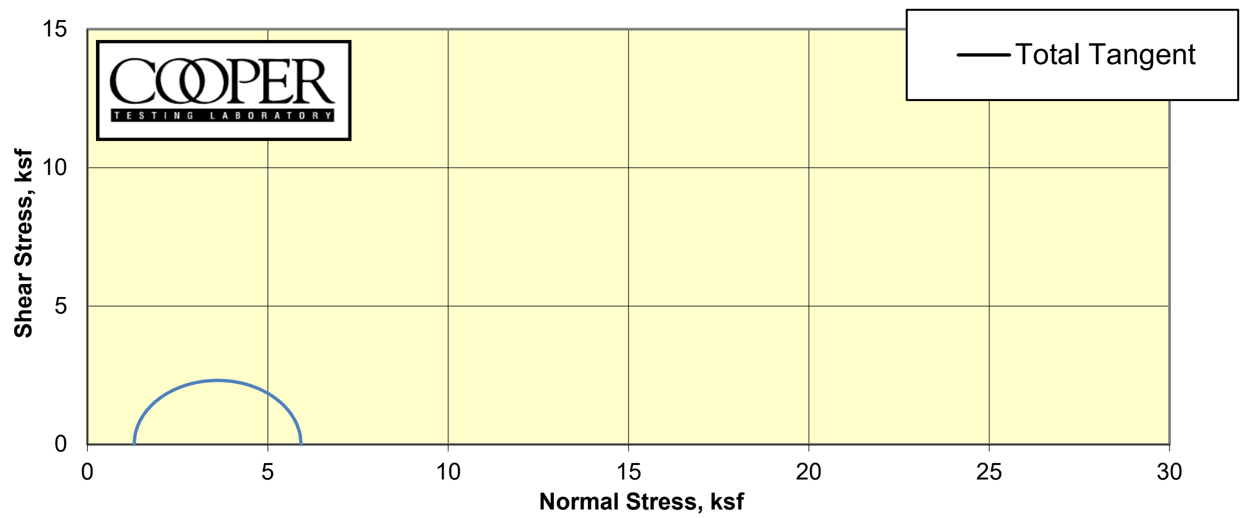
CLIENT TranSystem **PROJECT NAME** Tom Bates Regional Sports Complex
PROJECT NUMBER 220960 **PROJECT LOCATION** 400 Gilman Street, Berkeley CA

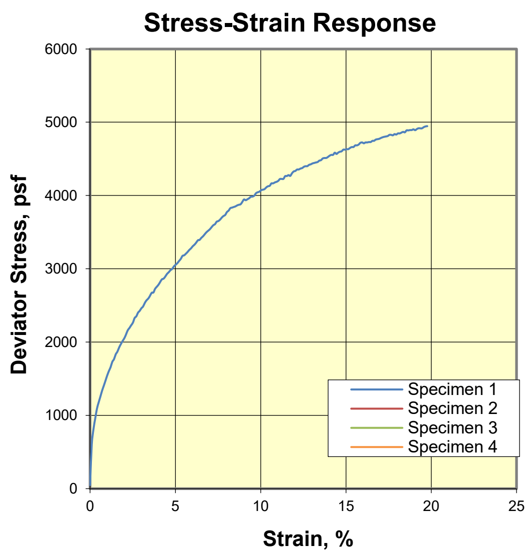


	BOREHOLE	DEPTH	DATE TESTED	Classification	LL	PL	PI	Cc	Cu
●	**B-01**	**23.5**	**11/11/2022**						

	BOREHOLE	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-01**	**23.5**	**0.106**	**0.087**	**0.076**		**0.0**	**71.7**	**28.3**	

Unconsolidated Undrained Triaxial Compression
ASTM D2850



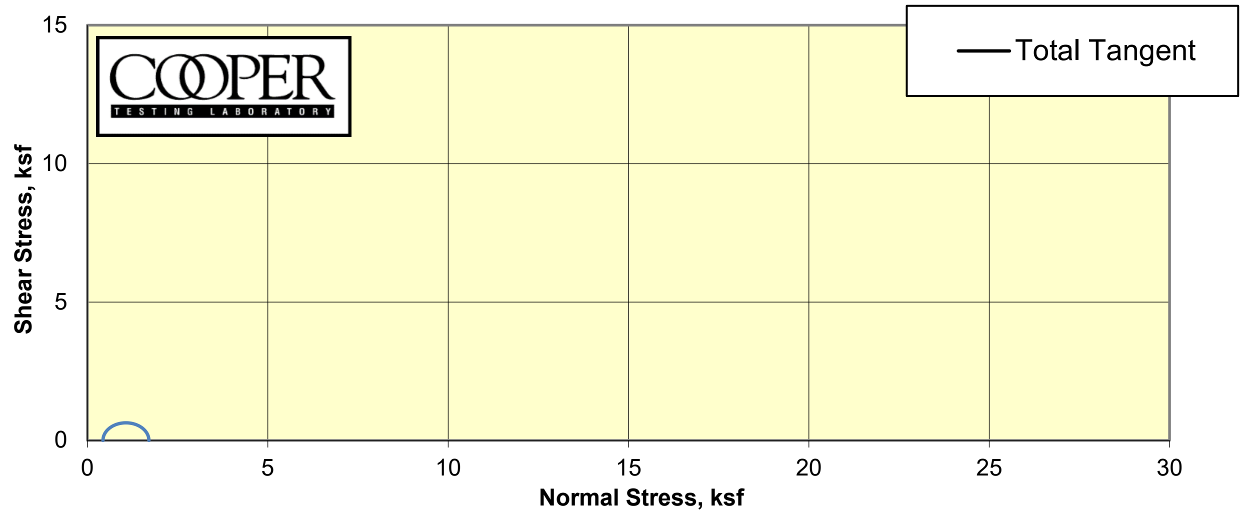


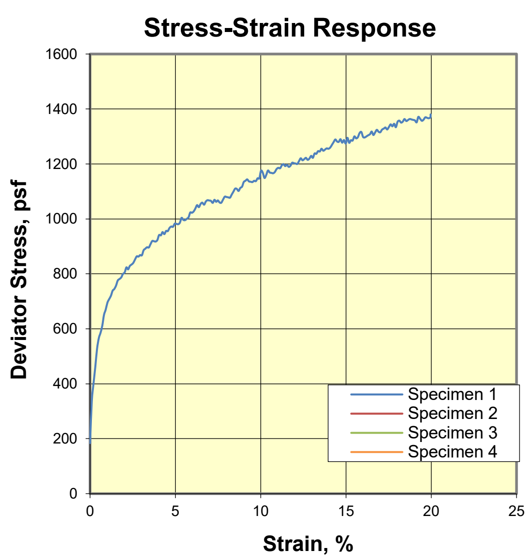
Specimen	1	2	3	4
Boring	B-01			
Sample	1-17			
Depth	39.5			
Visual Description	Grayish Brown CLAY			
MC (%)	10.7			
Dry Density (pcf)	119.8			
Saturation (%)	78.5			
Void Ratio	0.354			
Diameter (in)	2.41			
Height (in)	5.02			
	Final			
MC (%)	12.5			
Dry Density (pcf)	122.4			
Saturation (%)	100.0			
Void Ratio	0.326			
Diameter (in)	2.37			
Height (in)	5.06			
Cell Pressure (psi)	57.5			
Back Pressure (psi)	48.5			
	Total Stresses At:			
Strain (%)	15.0			
Deviator (ksf)	4.627			
Excess PP (psi)				
Sigma 1 (ksf)	5.923			
Sigma 3 (ksf)	1.296			
P (ksf)	3.609			
Q (ksf)	2.313			
Stress Ratio	4.570			
Rate (in/min)	0.0491			

CTL Number:	471-391		
Client Name:	Cal Engineering & Geology		
Project Name:	Tom Bates Sports Complex		
Project Number:	220960		
Date:	12/2/2022	By:	MD/DC
Total C	#DIV/0!	ksf	
Total phi	#DIV/0!	degrees	
Eff. C	N/A	ksf	
Eff. Phi	N/A	degrees	

Remarks: Sample was back-pressure saturated to a B parameter of 0.95 or greater prior to shear.

Unconsolidated Undrained Triaxial Compression
ASTM D2850





CTL Number:	471-391		
Client Name:	Cal Engineering & Geology		
Project Name:	Tom Bates Sports Complex		
Project Number:	220960		
Date:	12/2/2022	By:	MD/DC
Total C	#DIV/0!	ksf	
Total phi	#DIV/0!	degrees	
Eff. C	N/A	ksf	
Eff. Phi	N/A	degrees	©

Remarks: Sample was back-pressure saturated to a B parameter of 0.95 or greater prior to shear.

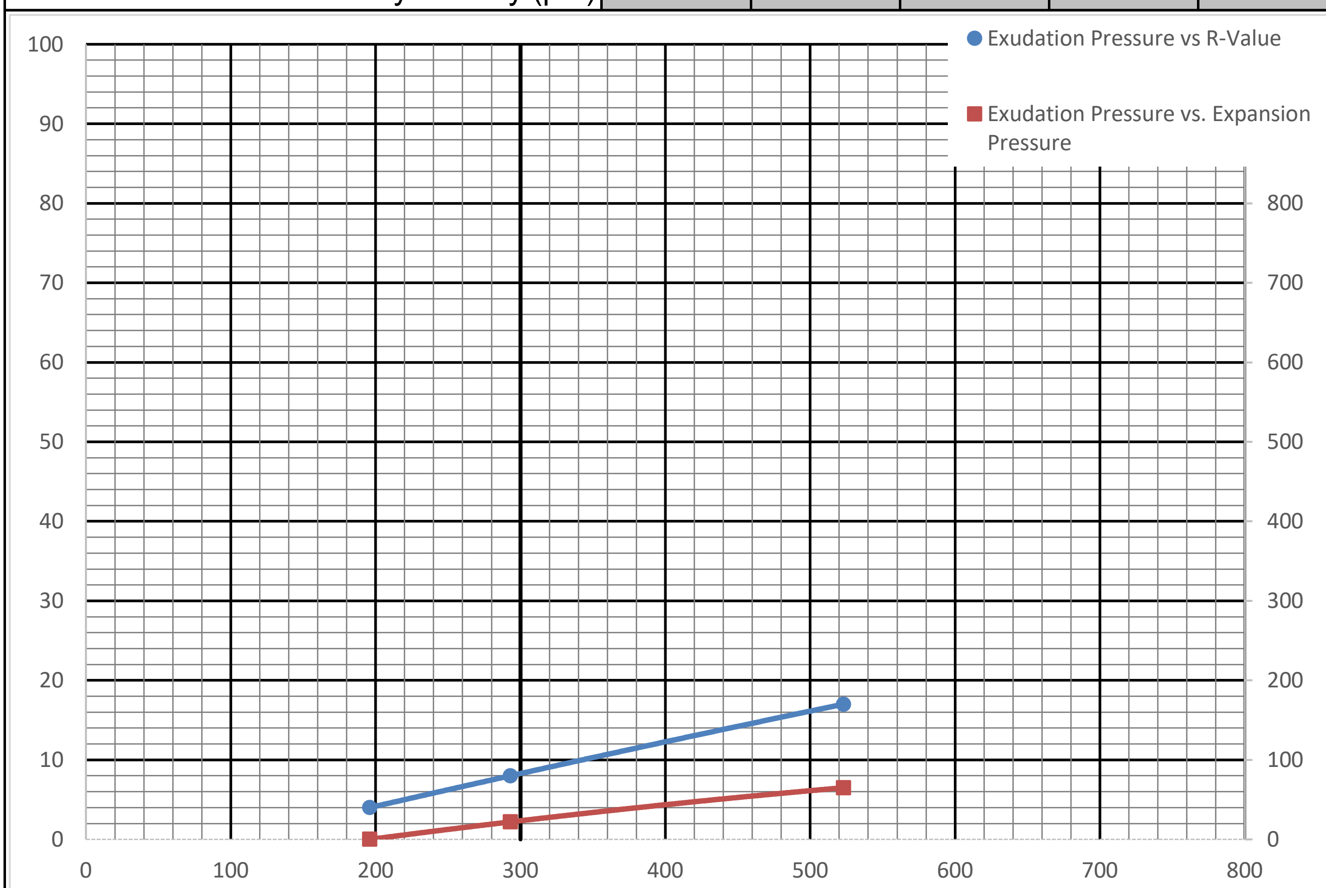
Specimen	1	2	3	4
Boring	B-01			
Sample	1-2			
Depth	2			
Visual Description	Dakr Gray Sandy CLAY w/ Gravel			
MC (%)	19.8			
Dry Density (pcf)	108.2			
Saturation (%)	93.0			
Void Ratio	0.586			
Diameter (in)	2.39			
Height (in)	5.00			
	Final			
MC (%)	21.3			
Dry Density (pcf)	108.3			
Saturation (%)	100.0			
Void Ratio	0.585			
Diameter (in)	2.39			
Height (in)	5.01			
Cell Pressure (psi)	51.5			
Back Pressure (psi)	48.5			
	Total Stresses At:			
Strain (%)	15.0			
Deviator (ksf)	1.274			
Excess PP (psi)				
Sigma 1 (ksf)	1.706			
Sigma 3 (ksf)	0.432			
P (ksf)	1.069			
Q (ksf)	0.637			
Stress Ratio	3.950			
Rate (in/min)	0.0474			

R-Value
CTM 301

COOPER TESTING LABORATORY

CTL Job No.:	471-391	Boring:	Bulk	Reduced By:	RU
Client:	Cal Engineering & Geology	Sample:	B4/5	Checked By:	PJ
Project Number:	220960	Depth:		Date:	11/29/2022
Project Name:	Tom Bates Sports Complex			**R-Value**	**8**
Soil Description:	Brown Sandy CLAY				
Remarks:				**Expansion Pressure**	**25**

Specimen Designation	A	B	C	D	E
Compactor Foot Pressure (psi)	70	45	30		
Exudation Pressure (psi)	523	293	196		
Exudation Load (lbf)	6572	3682	2463		
Height After Compaction (in)	2.70	2.50	2.53		
Expansion Pressure (psf)	65	22	0		
Stabilometer @ 2000	128	140	150		
Turns Displacement	3.46	4.12	4.44		
R-value	15	8	4		
Corrected R-Value	17	8	4		
Moisture Content (%)	20.7	23.6	17.9		
Wet Density (pcf)	129.6	123.0	118.9		
Dry Density (pcf)	107.3	99.5	100.8		



COOPER TESTING LABORATORY

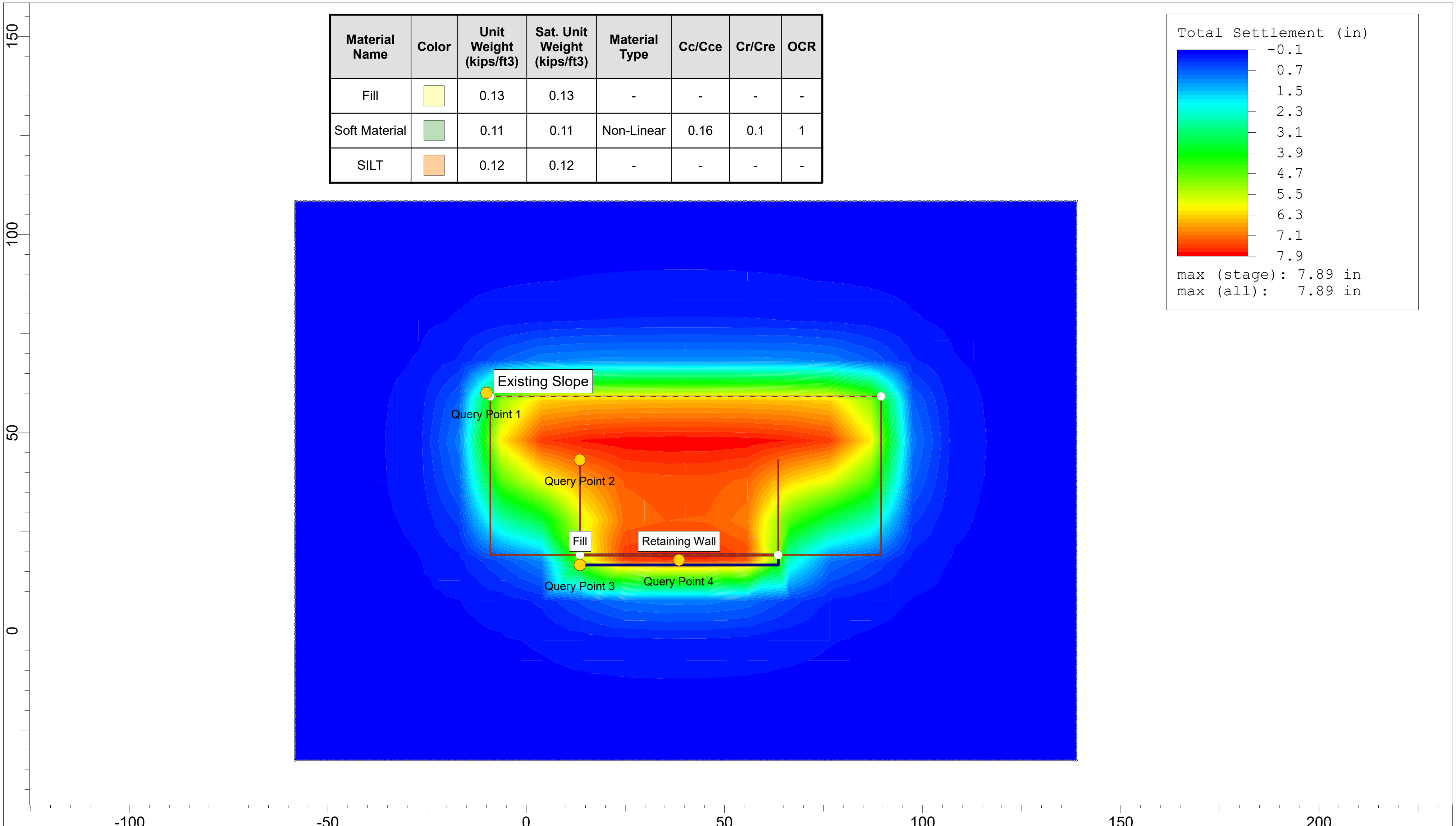
Corrosivity Test Summary

CTL # 471-391 **Date:** 11/18/2022 **Tested By:** PJ **Checked:** PJ
Client: Cal Engineering & Geology **Project:** Tom Bates Sports Complex **Proj. No:** 220960
Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP	Moisture	Soil Visual Description
Boring	Sample, No.	Depth, ft.	As Rec.	Minimum	Saturated	mg/kg	mg/kg	%		(Redox)	At Test	
						Dry Wt.	Dry Wt.	Dry Wt.		mv	%	
			ASTM G57	Cal 643	ASTM G57	Cal 422-mod.	Cal 417-mod.	Cal 417-mod.	Cal 643	SM 2580B	ASTM D2216	
B-02	2-1 & 2-2	1.5 & 2.0	-	1435	-	290	54	0.0054	6.3	-	2.6	Brown CLAY w/ Sand

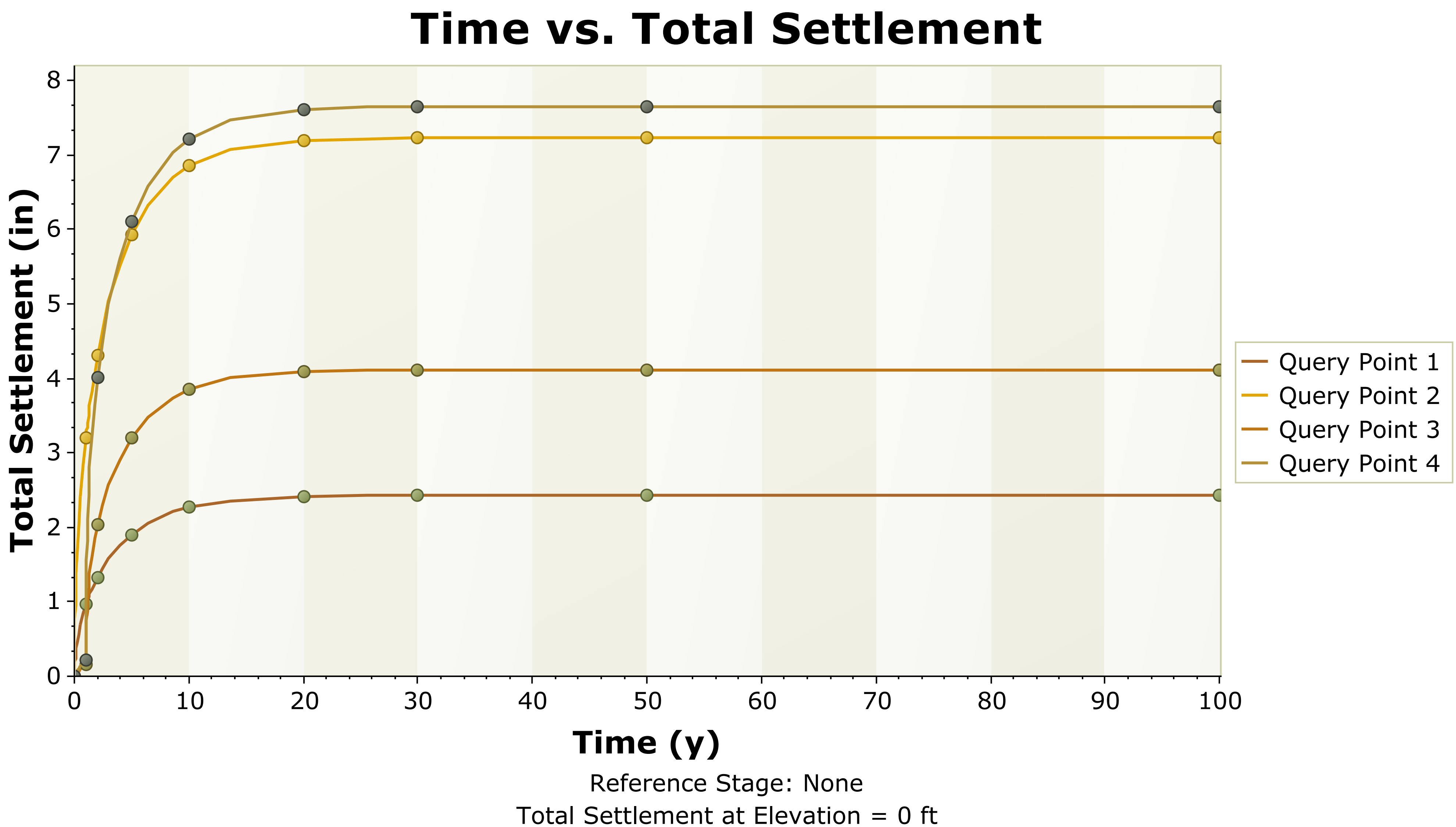
Appendix C. Calculations

Material Name	Color	Unit Weight (kips/ft3)	Sat. Unit Weight (kips/ft3)	Material Type	Cc/Cce	Cr/Cre	OCR
Fill		0.13	0.13	-	-	-	-
Soft Material		0.11	0.11	Non-Linear	0.16	0.1	1
SILT		0.12	0.12	-	-	-	-



CE&G
A division of Haley & Aldrich

Tom Bates Sports Complex	
Fill w/ Retaining Wall	
C. Rodil	CE&G (Haley & Aldrich)
2/21/2023	Fill-With-1500psf-2.5 Ft Footing.s3z



CE&G A division of Haley & Aldrich	Tom Bates Sports Complex	
	Fill w/ Retaining Wall	
	C. Rodil	CE&G (Haley & Aldrich)
	2/21/2023	Fill-With-1500psf-2.5 Ft Footing.s3z