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GEOTECHNICAL REPORT ON
CESAR CHAVEZ PARK RESTROOM PROJECT
SPINNAKER WAY
BERKELEY, CALIFORNIA

by
Haley & Aldrich, Inc.
Walnut Creek, California

for
TranSystems
Berkeley, California

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PREPARED FOR
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BERKELEY, CALIFORNIA

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HALEY ALDRICH

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1. Introduction

1.1 GENERAL

Haley & Aldrich, Inc. (Haley & Aldrich) has provided geotechnical services to TranSystems for the Cesar Chavez Park Restroom (the Project) located in Berkeley, California. The work has been completed to provide geotechnical design recommendations for design of a new restroom building.

1.2 PROJECT AND SITE DESCRIPTION

The project site is located at the southern end of Cesar Chavez Park, just north of the intersection of Spinnaker Way and Breakwater Drive, in Berkeley, California (Figure 1). The area of the proposed restroom building currently consists of a partially-fenced concrete pad with a portable toilet bounded by a grassy area with one large tree to the north and west and asphalt pavement to the east and south. There are also multiple utility vaults in the vicinity of the proposed improvements. The City of Berkeley plans to design and construct a permanent restroom structure at the location of the concrete slab and portable toilet.

The site topography is generally flat in the area of the planned building footprint; however, topography of the area immediately north of the planned restroom is gently sloped due to the presence of a closed landfill. The elevation at the proposed building location is about 19 feet above sea level (WGS84). Key features are shown in Figure 2.

1.3 PURPOSE AND SCOPE OF SERVICES

The purpose of Haley & Aldrich'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 and construction. 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 Underground Service Alert (USA);
- a subsurface exploration program consisting of two 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. Site Geology

2.1 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.

2.2 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). Bay Mud deposits are estimated to be greater than 10 feet in the project area; however, Bay Mud beneath the site has been altered due to the placement of landfill debris (McDonald and others, 1978).

2.3 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 millimeters per year (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 (Figure 4).

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-year Mw≥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 Mw≥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 1.

Table 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

2.4 GEOHAZARD MAPPING

2.4.1 Active Faulting

According to the California Geological Survey (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 (2003b), the site does not cross through an Alquist-Priolo Earthquake Fault Zone.

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

2.4.2 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 soil underlying the project is mapped as having very high liquefaction susceptibility (Figure 5).

The project area is shown within a liquefaction hazard zone on the State of California Seismic Hazard Map produced by the CGS for the Oakland West 7.5-minute quadrangle (CGS, 2003a). 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.

2.5 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 (2003a) showing the depth of historically high groundwater levels for the West Oakland 7.5-minute quadrangle shows groundwater depths of less than 5 feet for the project site due to its close proximity to San Francisco Bay. Groundwater depths beneath the site likely fluctuate due to tidal influences.

Site-specific groundwater data from our investigation is discussed in Section 3.2.4.

3. Field Investigations

3.1 SITE RECONNAISSANCE

Haley & Aldrich performed a field reconnaissance of the site on 31 July 2024, 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 USA.

3.2 SUBSURFACE EXPLORATIONS

3.2.1 Exploratory Boring

Two 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 borings were drilled by Exploration Geoservices, Inc. on 19 August 2024, using a truck-mounted Mobile B-53 drill rig equipped with 8-inch-diameter hollow-stem-augers and a 140-pound cable-drop hammer. The surface conditions at the boring locations consisted of grass.

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 11 October 2024.

3.2.2 Logging and Sampling

The materials encountered in the boring were logged in the field by a Haley & Aldrich engineer. 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 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).

The CM and SPT samplers were driven 18 inches (unless otherwise noted on the boring logs) with a 140-pound cable-drop 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 A. The blow counts included on the boring logs represent the field values and are uncorrected.

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 Cooper Testing Lab, in East Palo Alto, for testing and storage, respectively.

3.2.3 Soil Conditions Encountered

The upper 4 to 10 feet of fill encountered in borings B-1 and B-2 consisted of dry to moist, stiff to very stiff, lean clay with varying amounts of gravel and sand. Due to the site's proximity to a closed landfill, traces of glass, wood, and organics were also encountered in these soils. At approximately 10 feet below ground surface (bgs) in boring B-1, the material consisted of wet, stiff to very hard fat clay with increasing amounts of landfill debris, including glass, wood, and rubber fragments. Encountered materials from 10 to 36.5 feet bgs regularly fell out of the sampler and had to be retrieved using a catcher, indicating the materials had little structure and were likely saturated.

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.

3.2.4 Groundwater Conditions Encountered

Groundwater was encountered at approximately 10 feet bgs in Boring B-1 but was not encountered in boring B-2.

3.3 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, 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 Test (ASTM D2850)

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

4. Discussion and Conclusions

The planned improvements consist of minor grading and excavations to construct a foundation for the placement of a prefabricated public restroom building and its utilities. Other project features include the installation of drinking fountains, a bike rack, and other site-related improvements. The soils encountered consisted predominantly of silty and clayey sands with varying amounts of gravel in the upper 5 feet, underlain by fat clay with various types and amounts of landfill debris. These materials represent the landfill cover and underlying landfill debris and native soils. 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 landfill material encountered beneath the proposed foundation location;
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.

In addition, due the proximity to or location on a closed landfill, there may be methane mitigation measures that will need to be implemented.

4.1 LIQUEFACTION

4.1.1 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.

Regional mapping by the USGS suggests granular soils in the area are considered to have very high liquefaction susceptibility. 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 groundwater may be subject to such phenomenon. Based on the subsurface materials encountered 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.

4.1.2 Seismically Induced Settlement

Seismic densification is the densification of unsaturated, loose to medium-dense granular soils due to strong vibrations resulting from earthquake shaking. We judge the potential for seismic densification at this site to be low based on the nature of the materials beneath the site.

4.1.3 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 there are no significant sandy layers underlying the site. 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.

4.2 SETTLEMENT ANALYSIS

Under normal conditions, the site's location adjacent to San Francisco Bay would indicate the potential for consolidation settlement under applied loads. The site has been significantly altered by historical landfilling activities. The borings for this investigation generally encountered stiff to very stiff clays based on the blow counts shown on the borings. As such, based on the anticipated structure loads, the proposed foundation type and the soils encountered, we expect settlement to be within acceptable limits.

4.3 FOUNDATIONS

The proposed plans show the prefabricated restroom building footprint to be approximately 8 feet by 18 feet in plan. Because of the variability of the subsurface materials and the size of the proposed building, we recommend using a mat type foundation to support the anticipated loads. The mat foundation should be supported on a minimum of 12 inches of engineered fill consisting of ¾-inch Caltrans Class 2 aggregate base rock, or ¾-inch drain rock if methane mitigation will be required.

4.4 HIGH GROUNDWATER LEVEL

An elevated groundwater level is present at the site, which is typically about 10 feet below the ground surface (~El. 10) in the vicinity of the site. There may be significant groundwater expected in utility trench excavations.

4.5 MOISTURE SENSITIVE SOILS AND WET WEATHER CONSTRUCTION

The soils anticipated to be encountered at the foundation level are expected to consist of moist and stiff lean clay with varying amounts of gravel and sand which may be sensitive to 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 5.1 of this report for recommendations pertaining to moisture-sensitive soils.

5. Recommendations

Detailed recommendations for the geotechnical aspects of the proposed project components 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.

5.1 EARTHWORK

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

5.1.2 Excavations

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 landfill debris. This may require additional personal protective equipment for construction personnel.

5.1.3 Subgrade Preparation

To mitigate disturbance of soil at the site from demolition and removal of existing site improvements and to provide for more uniform slab support, 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 5.1.5. Engineered Fill Placement and Compaction. Depressions or holes created by the removal of deeper features should be cleared of loose material and prepared as noted above prior to placing engineered fill.

Subgrade preparation should extend to 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.

5.1.4 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, i.e. landfill, materials, and meeting the gradation requirements below may be used as general engineered fill to achieve project grades, except when special material (such as 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 the Plasticity Index of 15 or less, or the Expansion Index of less than 20. In addition, if any landfill cover soil is removed, it should be replaced in kind to maintain the integrity of the cover.

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.

5.1.5 Engineered Fill Placement and Compaction

The size of the project area will impact the type of compaction equipment that can be used. Engineered fill should be placed in horizontal lifts each not exceeding 6 inches in thickness and mechanically compacted to the recommendations below at the recommended moisture content. Depending on the equipment that is used the lift thickness may need to be thinner. Engineered fill will be required for foundation support and potentially for raising site grades.

Engineered fills consisting of on-site soils and imported soils should be compacted to a minimum of 90 percent relative compaction for soil materials, or 95 percent relative compaction for aggregate base materials, 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.

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.

5.1.6 Considerations for Soil Moisture and Seepage Control

Subgrade soil and engineered fill should be compacted at a moisture content meeting our recommendations. Once compacted, soils should be protected from drying and wetting. This may be accomplished by regular watering with a water truck, or other means, 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 should be protected against saturation. Water, if allowed to seep into the subgrade soil or pavement section, may 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.

5.1.7 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 should submit a wet weather construction plan outlining procedures they will employ to protect their work and to minimize damage to their work by rainstorms.

5.1.8 Utility Trench Excavation and Backfill

We estimate that excavations within the encountered soil should be able to be accomplished with conventional digging equipment, such as backhoes and excavators, and that jackhammers and/or blasting should not be necessary. 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.

Pipe-zone backfill, extending from the bottom of the trench to about 1 foot above the top of pipe, should consist of free-draining sand (at least 90% passing a No. 4 sieve and less than 5% passing a No. 200 sieve) compacted to a minimum of 90 percent relative compaction unless concrete or cement slurry is specified.

Above the pipe zone, underground utility trenches may be backfilled with free-draining sand, on-site soil or imported soil. The trench backfill should be compacted to the requirements given in the section on "Engineered Fill Placement and Compaction." Trench backfill should be capped with at least 12 inches of compacted, on-site soil similar to that of the adjoining subgrade. The upper 12 inches of trench backfill in areas to be paved should be compacted to a minimum of 95 percent relative compaction. Compaction should be performed by mechanical means only. Water jetting or flooding to attain compaction of backfill is not permitted. These recommendations should be compared for consistency with any City of Berkeley Standard Details.

Trench excavations that extend below an imaginary plane inclined at 1½ horizontal to 1 vertical, 1½(h):1(v) below the bottom edge of foundations should be properly shored to maintain support of the existing facilities. Trenches that run parallel to the proposed foundations should not be excavated within the imaginary plane inclined at 1½:1 (h:v) below the bottom of the footing.

5.2 TEMPORARY EXCAVATIONS

5.2.1 General

The Contractor should be aware that excavation depths (including utility trench excavations), slope height or slope inclination, should in no case exceed those specified in local, state, and/or federal safety regulations (e.g., OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, or successor regulations). Such regulations are strictly enforced and, if they are not followed, the Owner, Contractor, and/or earthwork and utility subcontractors could be liable for substantial penalties. The stability and safety of excavations, braced or unbraced, is the responsibility of the contractor.

The walls of excavations in the mixed near-surface soil (<5 feet depth) may not stand vertically for extended periods and may require shoring or sloping. The near-surface soils should be considered Type C soils when applying OSHA regulations. For this soil type OSHA recommends a maximum slope inclination of 1½:1 (h:v), or flatter for excavations 20 feet or less in depth.

5.2.2 Shoring Design

Temporary shoring should be made the responsibility of the contractor. Shoring walls should be braced to limit deflections. Shoring and bracing should be designed by a registered engineer and then submitted to the Engineer for review prior to construction. It is recommended that all temporary shoring be designed in conformance with the State of California, Department of Transportation, Trenching and Shoring Manual.

Unbraced shoring may be considered if the deflections and related ground deformation can be demonstrated to be less than ½-inch. The low allowable deflection for temporary shoring is to reduce the potential for surface displacement of adjacent areas and structures.

Internally braced shoring can also be utilized, if preferred. For either excavation shoring system, project specifications should include location specific requirements for limiting shoring induced ground deformation during and following construction.

Shoring and bracing should be designed by a registered engineer as water-tight to facilitate excavation dewatering and to limit dewatering of the area surrounding the excavation. Shoring should be designed to resist hydrostatic pressures in combination with static (braced) earth pressures. Construction induced vibrations should be minimized during shoring placement.

5.3 FOUNDATION

5.3.1 Mat Foundation

We recommend the restroom building be supported on a thickened reinforced concrete slab or mat foundation, designed and constructed in accordance with the following recommendations. The mat slab may be designed using a net allowable soil bearing pressure of 1,000 pounds per square foot (psf) for dead plus live loads. This value may be increased by one-third when considering short-term loads such as wind and seismic forces. The mat foundation may be designed using a modulus of subgrade reaction (kv1) of 100 kips per cubic foot, for static and seismic loading, where the mat is supported on 12 inches of Class 2 Aggregate Base rock, or ¾-inch drain rock as a capillary moisture break. Reinforcement for the foundations should be determined by the project structural engineer. The aggregate base or drain rock supporting the mat foundation should extend about 1 foot beyond the limits of the mat and the mat should match the building footprint unless the manufacturer recommends otherwise.

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 is anticipated to be less than 1 inch with differential settlements on the order of ¼-inch or less over the length of the slab assuming a contact pressure of <1,000 psf from the mat foundation.

Soil resistance to lateral loads will be provided by a combination of frictional resistance between the bottom of the slab and underlying soils and by passive pressures acting against the embedded sides of the foundation. 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 300 psf/foot 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.

Moisture vapor may condense on the underside of concrete slabs-on-grade. If such condensation would be undesirable, a minimum 15-mil synthetic membrane, such as Stego Wrap or equal, should be placed beneath the slab. To help provide puncture protection and to aid in slab curing, the membrane, if overlying crushed rock, can be covered with about 2 inches of clean-washed sand.

If methane mitigation is required, consideration should be given to upgrading the vapor barrier to Stego's Drago Wrap product, or equal. This material is more chemical resistant and will also provide protection from moisture vapor intrusion. The crushed rock can also provide a medium for piping for vapor collection. The crushed rock should be encapsulated within an appropriate non-woven geotextile.

It should be emphasized that we are not floor moisture-proofing experts. While the current industry standard is to place a vapor retarder over a compacted gravel layer as described above, this system may not be completely effective in preventing floor slab moisture problems. These systems typically will not necessarily assure that floor slab moisture transmission rates will meet floor-covering manufacturing standards and that indoor humidity levels be appropriate to inhibit mold growth. The design and construction of such systems are totally dependent on the proposed use and design of the proposed building. All elements of building design and function should be considered in the slab-on-grade floor design. Building design and construction may have a greater role in perceived moisture problems since sealed buildings/rooms or inadequate ventilation may produce excess moisture in a building and affect indoor air quality.

5.3.2 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 D type soils and Risk Category 2. The California Building Code seismic design parameters for the site are included in Table 2.

Table 2. 2022 CBC Seismic Design Parameters

Item	Design Value
Site Soil Class Definition	D
Ss – 0.2 Second Spectral Response Acceleration	1.761
S_1 – 1.0 Second Spectral Response Acceleration	0.669
Fa – Values of Site Coefficient	1.2
Fv[1] – Value of Site Coefficient	--
S_{DS} – Designed Spectral Response Acceleration for Short Periods	1.408
S_{D1}[1] – Designed Spectral Response Acceleration for 1-Sec Periods	--
S_{MS}[1] – MCE_R Spectral Response Acceleration Parameter (g)[2]	2.113
S_{M1}[1] – MCE_R Spectral Response Acceleration Parameter (g)[2]	--
PGA	0.74
PGA_M	0.888

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 = 37.86993 / -122.31884.*

5.4 SURFACE DRAINAGE

Engineering design of grading and drainage at the site is the responsibility of the project Civil Engineer. We recommend that the following points be considered by the project Civil Engineer and incorporated into the project plans where appropriate.

Generally, surface drainage should be directed away from building foundations, concrete slabs-on-grade, and pavements and directed towards suitable discharge locations. Ponding of surface water should be avoided by establishing positive drainage away from all improvements. Collected surface water and discharge from roof downspouts should be discharged into a pipe or towards drainage structures and the water carried to a suitable discharge point.

5.5 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.

6. 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, Haley & Aldrich, Inc. should be notified promptly so that conditions can be reviewed and recommendations reconsidered, as appropriate.

It is the City of Berkeley'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, Haley & Aldrich, 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 3 years unless the conditions of the site change. After a period of 3 years, Haley & Aldrich 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.

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FIGURES

BASEMAP REFERENCE

1. STREET CENTERLINES FROM CALTRANS CALIFORNIA ROAD SYSTEM, DOWNLOADED ON 18 FEB 2020.
2. ORTHOIMAGERY FROM ESRI (MAXAR), 2022.

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CESAR CHAVEZ PARK RESTROOM
NORTH OF SPINNAKER WAY
BERKELEY, CALIFORNIA

SITE LOCATION MAP

0207774	JANUARY 2025	FIGURE 1

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BASEMAP REFERENCE

1. CONTOURS FROM LIDAR-DERIVED DIGITAL ELEVATION MODEL.
2. CONTOUR INTERVAL = 2FT.
3. ORTHOIMAGERY FROM MAXAR, 2022.
4. PARCEL DATA FROM ALAMEDA COUNTY DATABASE, ACCESSED IN 2022.

APPROXIMATE SITE BOUNDARY

PROPOSED BORING LOCATIONS

N

0 7.5 15 30

FEET

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SITE PLAN

0207774	JANUARY 2025	FIGURE 2

BASEMAP REFERENCE

1. STREET CENTERLINES FROM CALTRANS CALIFORNIA ROAD SYSTEM, DOWNLOADED ON 18 FEB 2020.
2. ORTHOIMAGERY FROM ESRI (MAXAR), 2022.
3. REGIONAL GEOLOGY FROM WITTER, ET AL. 2006.

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MAP UNIT REFERENCE

af	ARTIFICIAL FILL	Qhbs	BEACH SAND	Qop	PEDIMENT DEPOSITS
afem	ARTIFICIAL FILL OVER ESTUARINE MUD	Qhf	ALLUVIAL FAN DEPOSITS	br	BEDROCK
acf	ARTIFICIAL CHANNEL FILL	Qhl	ALLUVIAL FAN LEVEE DEPOSITS		
ac	ARTIFICIAL STREAM CHANNEL	Qf	HOLOCENE TO LATE PLEISTOCENE ALLUVIAL FAN LEVEE DEPOSITS		
Qhc	HISTORICAL STREAM CHANNEL DEPOSITS	Qpf	LATEST PLEISTOCENE ALLUVIAL FAN LEVEE DEPOSITS		

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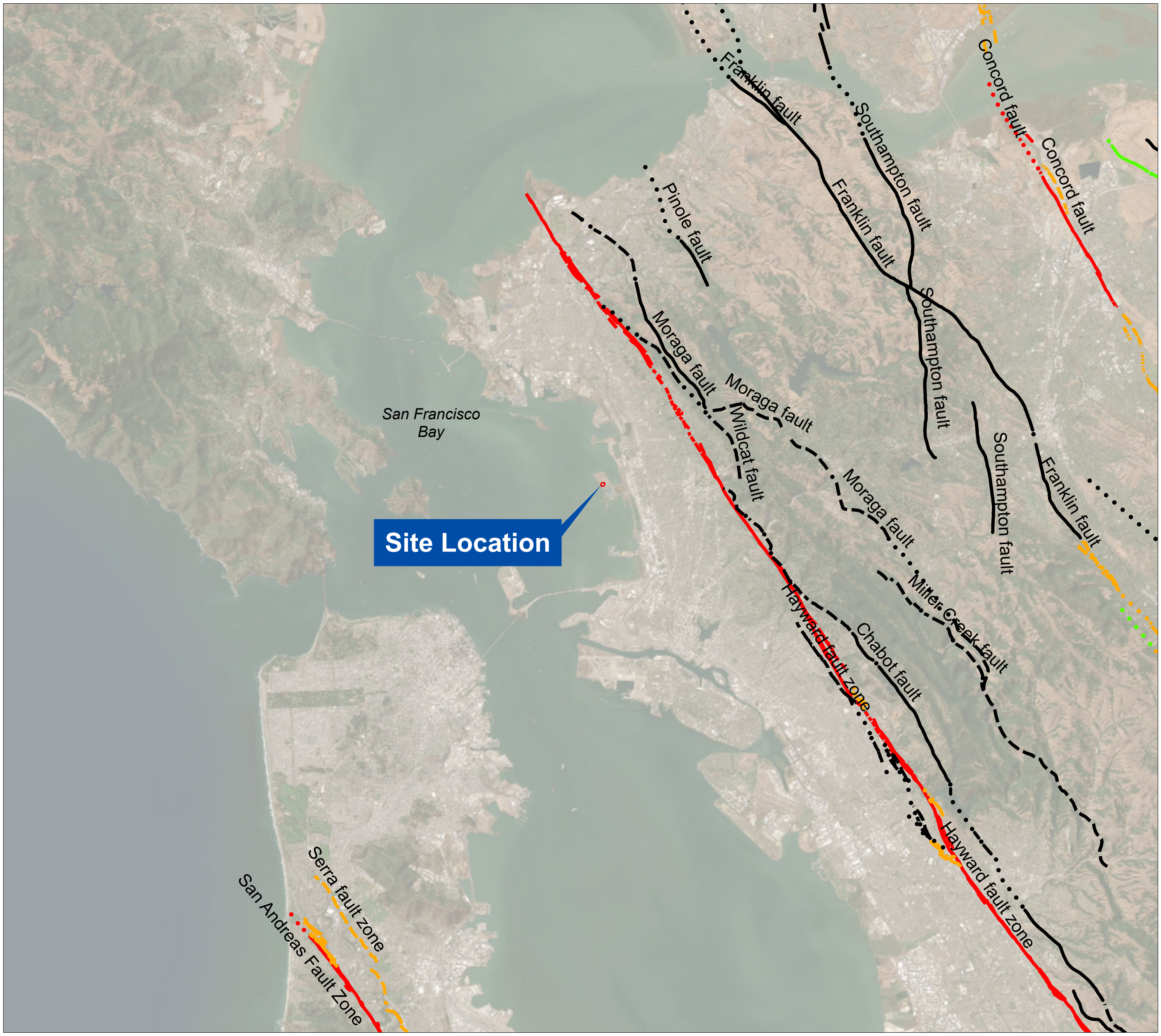
REGIONAL GEOLOGY MAP

0207774 | JANUARY 2025 | FIGURE 3



BASEMAP REFERENCE

1. STREET CENTERLINES FROM CALTRANS CALIFORNIA ROAD SYSTEM, DOWNLOADED ON 18 FEB 2020.
2. ORTHOIMAGERY FROM ESRI (MAXAR), 2022.
3. FAULT ACTIVITY FROM USGS DATABASE, ACCESSED IN 2020.

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MAP UNIT REFERENCE

- 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
- late Quaternary (<130,000 years), well constrained location
- late Quaternary (<130,000 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

0207774	JANUARY 2025	FIGURE 4

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BASEMAP REFERENCE

1. STREET CENTERLINES FROM CALTRANS CALIFORNIA ROAD SYSTEM, DOWNLOADED ON 18 FEB 2020.
2. ORTHOIMAGERY FROM ESRI (MAXAR), 2022.
3. LIQUEFACTION SUSCEPTIBILITY FROM CGS DATABASE, ACCESSED IN 2020.

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LIQUEFACTION SUSCEPTIBILITY

Very High
Moderate
Low
Very Low

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LIQUEFACTION POTENTIAL MAP

0207774	JANUARY 2025	FIGURE 5

APPENDIX A
Boring Logs

BORING NUMBER B-1

CLIENT TranSystems
PROJECT NAME Cesar Chavez Park Restroom Project
PROJECT NUMBER 0207774
PROJECT LOCATION Berkeley, California
DATE STARTED 8/19/2024 **COMPLETED** 8/19/2024
GROUND ELEVATION 20 ft **DATUM** WGS84 **HOLE SIZE** 8 in.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
COORDINATES: **LATITUDE** 37.869977 **LONGITUDE** -122.318829
DRILLING RIG/METHOD Mobile B-53/8-in. Hollowstem Auger
GROUNDWATER AT TIME OF DRILLING --- Not Measured
LOGGED BY C. Rodil **CHECKED BY** K. Loeb
GROUNDWATER AT END OF DRILLING --- Not Measured
HAMMER TYPE 140 lb hammer with 30 in. cathead
GROUNDWATER AFTER DRILLING 10.0 ft / Elev 10.0 ft

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		TOPSOIL									
		Gravely Lean CLAY w/ Sand (CL): dark grayish brown (10YR 4/2), dry, dense, fine to coarse sand, angular gravel up to 1.0", trace roots, trace glass shards	CM	11-20-36			7				22
		Sandy Lean CLAY (CL): dark grayish brown (10YR 4/2), moist, medium stiff, low plasticity, fine to medium sand, trace organics	SPT	23-11-11				43	18	25	
5		-switch to mud rotary at 5.0', gravels in sample up to 1.0"	CM	5-5-7	1.75						54
		-TXUU test at 6'									
		-No Recovery, catcher added, no recovery, driller noted loss of drilling fluid up to 50 gallons, switch back to hallow-stem	SPT	5-7-5							
10		Fat CLAY (CH): greenish black (GLEY 1 2.5/1) to black (GLEY 1 2.5/), wet, very hard, high plasticity, silty, some fine sand	CM	27-50/0"							
		-No Recovery, catcher used, wood retrieved in catcher in satruated zone, glass shards	CM	12-14-21							
15		-large wood debris, black, intermixed with sand	CM	25-40-50/5"							
20		Fat CLAY (CH): greenish black (GLEY 1 2.5/1), moist, very stiff, medium to high plasticity, few fine to medium sand, some wood fibers	CM	23-46-24	1.75 1.25	92	29				
25			CM	13-7-50/5"							
		-random landfill waste including wood chips, glass, and rubber, wet, black sand and clay matrix									
30		-No Recovery	CM	13-12-10							
35											

(Continued Next Page)

CLIENT TranSystems

PROJECT NAME Cesar Chavez Park Restroom Project

PROJECT NUMBER 0207774

PROJECT LOCATION Berkeley, California

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		Fat CLAY (CH): greenish black (GLEY 1 2.5/1), moist, very stiff, medium to high plasticity, few fine to medium sand, some wood fibers *(continued)* -No Recovery, catcher used, no recovery	CM	16-15-20							

Bottom of borehole at 36.5 ft. Borehole backfilled with neat cement grout.

BORING NUMBER B-2

CLIENT TranSystems

PROJECT NAME Cesar Chavez Park Restroom Project

PROJECT NUMBER 0207774

PROJECT LOCATION Berkeley, California

DATE STARTED 8/19/2024 **COMPLETED** 8/19/2024

GROUND ELEVATION 20 ft **DATUM** WGS84 **HOLE SIZE** 8 in.

DRILLING CONTRACTOR Exploration Geoservices, Inc.

COORDINATES: LATITUDE 37.869917 **LONGITUDE** -122.318953

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

GROUNDWATER AT TIME OF DRILLING --- Not Encountered

LOGGED BY C. Rodil **CHECKED BY** K. Loeb

GROUNDWATER AT END OF DRILLING --- Not Encountered

HAMMER TYPE 140 lb hammer with 30 in. cathead

GROUNDWATER AFTER DRILLING --- Not Encountered

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		TOPSOIL									
		SAND Lean CLAY w/ Gravel (CL): dark grayish brown (10YR 4/2), dry, very dense, fine to coarse sand, angular gravel up to 1.5", glass shards, some oxidation, trace organics	CM	26-40-50		111	8				
		-becomes medium dense	SPT	17-11-9							
5		Sandy Lean CLAY w/ Gravel (CL): very dark brown (10YR 2/2), moist, low plasticity, stiff to very stiff, fine to medium sand, fine gravel, debris, glass shards, wood chips	CM	12-20-16			31				
			SPT	3-8-10							
10		-becomes black, increases in moisture -larger wood debris, sand and clay matrix	CM	6-8-9							

Bottom of borehole at 10.5 ft. Borehole backfilled with neat cement grout.

KEY TO SYMBOLS

CLIENT TranSystems
PROJECT NAME Cesar Chavez Park Restroom Project
PROJECT NUMBER 0207774
PROJECT LOCATION Berkeley, California

LITHOLOGIC SYMBOLS (Unified Soil Classification System)

CH: USCS High Plasticity Clay

CL: USCS Low Plasticity Clay

SM: USCS Silty Sand

TOPSOIL: Topsoil

SAMPLER SYMBOLS

California Modified Sampler

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

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

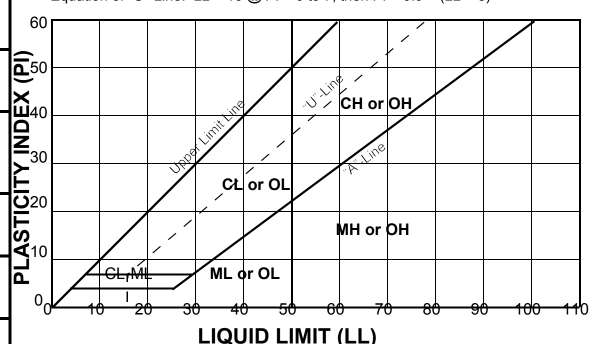
GW/GM or GP/GM: Gravel/Silty Gravel
GW/GC or GP/GC: Gravel/Clayey Gravel
SW/SM or SP/SM: Sand/Silty Sand
SW/SC or SP/SC: Sand/Clayey Sand

Field Identification		Group Symbols	Typical Names
	Identification Procedures on Percentage Passing the No. 40 Sieve		
Fine-Grained Soils More than 50% of material passes the No. 200 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	▽	Depth at which Groundwater was Encountered During Drilling
CM	California Modified Sampler	▼	Depth at which Groundwater was Measured After Drilling
SPT	Standard Penetration Test Sampler	**PP**	Pocket Penetrometer Test
SHL	Shelby Tube Sampler	**PTV**	Pocket Torvane Test
BU	Bulk Sample	**-#200**	% of Material Passing the No. 200 Sieve Test (ASTM D-1140)
LL	Liquid Limit of Sample (ASTM D-4318)	**PSA**	Particle-Size Analysis (ASTM D-422 & D-1140)
PI	Plasticity Index of Sample (ASTM D-4318)	**C**	Consolidation Test (ASTM D-2435)
Q_U	Unconfined Compression Test (ASTM D-2166)	**TXUU**	Unconsolidated Undrained Compression Test (ASTM D-2850)

KEY TO SAMPLE INTERVALS

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

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UNIFIED SOIL CLASSIFICATION SYSTEM AND KEY TO BORING LOG

Rock Hardness Descriptions

Very Hard	Cannot be scratched with knife or sharp pick. Breaking of hand specimen requires several hard blows of geologist's pick.
Hard	Can be scratched with knife or pick only with difficulty. Hard blow of hammer required to detach hand specimen.
Moderately Hard	Can be scratched with knife or pick. Gouges or grooves to 1/4-inch deep can be excavated by hard blow of geologist's pick. Hand specimens can be detached by moderate blow.
Medium	Can be grooved or gouged 1/16-inch deep by firm pressure of knife or pick point. Can be excavated in small chips to pieces about 1-inch maximum size by hard blows of the point of a geologist's pick.
Soft	Can be gouged or grooved readily with knife or pick point. Can be excavated in chips to pieces several inches in size by moderate blows of a pick point. Small tin pieces can be broken by finger pressure.
Very Soft	Can be carved with knife. Can be excavated readily with point of pick. Pieces 1-inch or more in thickness can be broken with finger pressure. Can be scratched readily by fingernail.

Bedding Thickness & Joint/Fracture Spacing Descriptions

Centimeters	Inches	Bedding	Joints/Fractures
< 2	< ¾	Laminated	Extremely Close
2-5	¾-2	Very Thin	Very Close
5-30	2-12	Thin	Close
30-90	12-36	Medium	Moderate
90-300	36-120	Thick	Wide
> 300	> 120	Very Thick	Very Wide

Rock Weathering Descriptions

Fresh	Rock fresh, crystals bright, few joints may show slight staining. Rock rings under hammer if crystalline.
Very Slight	Rock generally fresh, joints may show thin clay coatings, crystals in broken face show bright. Rock rings under hammer if crystalline.
Slight	Rock generally fresh, joints stained, and discoloration extends into rock up to 1 inch. Joints may contain clay. In granitoid rocks some occasional feldspar crystals are dulled and discolored. Crystalline rocks ring under hammer.
Moderate	Significant portions of rock show discoloration and weathering effects. In granitoid rocks, most feldspars are dull and discolored; some show clayey. Rock has dull sound under hammer and shows significant loss of strength as compared with fresh rock.
Moderately Severe	All rock except quartz discolored or stained. In granitoid rocks, all feldspars dull and discolored and majority show kaolinization. Rock shows severe loss of strength and can be excavated with geologist's pick. Rock goes "clunk" when struck.
Severe	All rock except quartz discolored or stained. Rock "fabric" clear and evident, but reduced in strength to strong soil. In granitoid rocks, all feldspars kaolinized to some extent. Some fragments of strong rock usually left.
Very Severe	All rock except quartz discolored or stained. Rock "fabric" discernible. But mass effectively reduced to "soil" with only fragments of strong rock remaining.
Complete	Rock reduced to "soil." Rock "fabric" not discernible or discernible only in small scattered locations. Quartz may be present as dikes or stringers.

The above Bedrock Characteristics are based on the ASCE Manual No. 56, "Subsrface Investigation For Design And Construction Of Foundations Of Buildings," 1976.

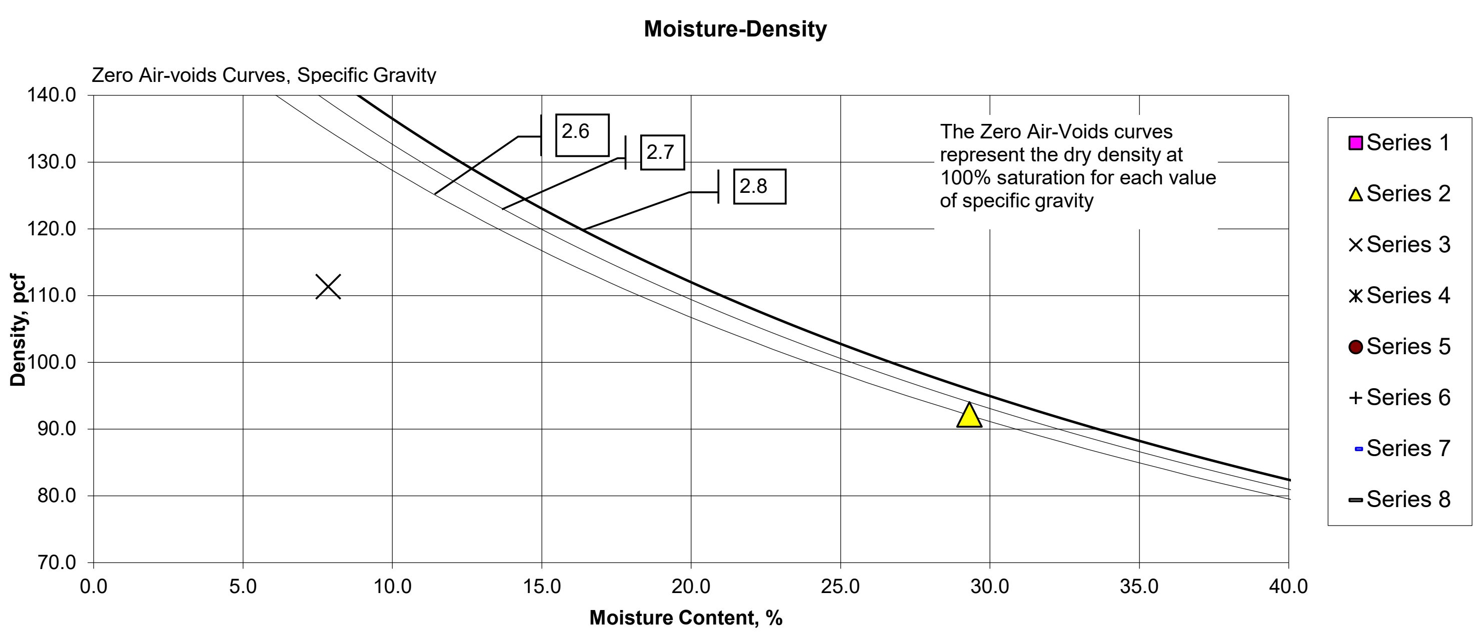
APPENDIX B
Laboratory Testing

Moisture-Density-Porosity Report
Cooper Testing Labs, Inc. (ASTM D7263b)

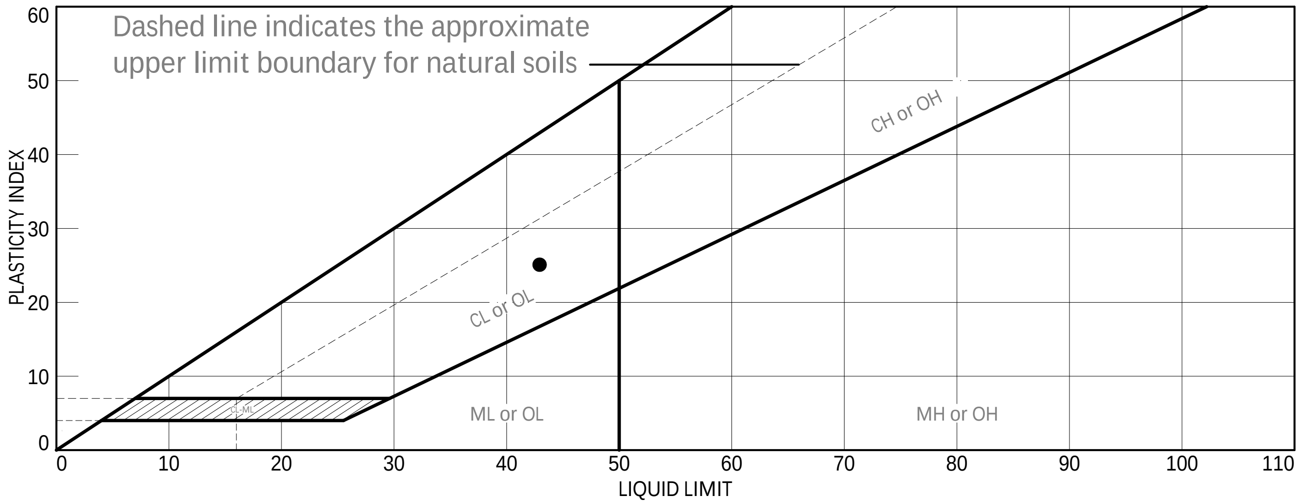
CTL Job No:	715-113	**Project No.**	0207774	**By:**	RU
Client:	Haley & Aldrich	**Date:**	09/09/24		
Project Name:	Cesar Chavez Restroom	**Remarks:**			

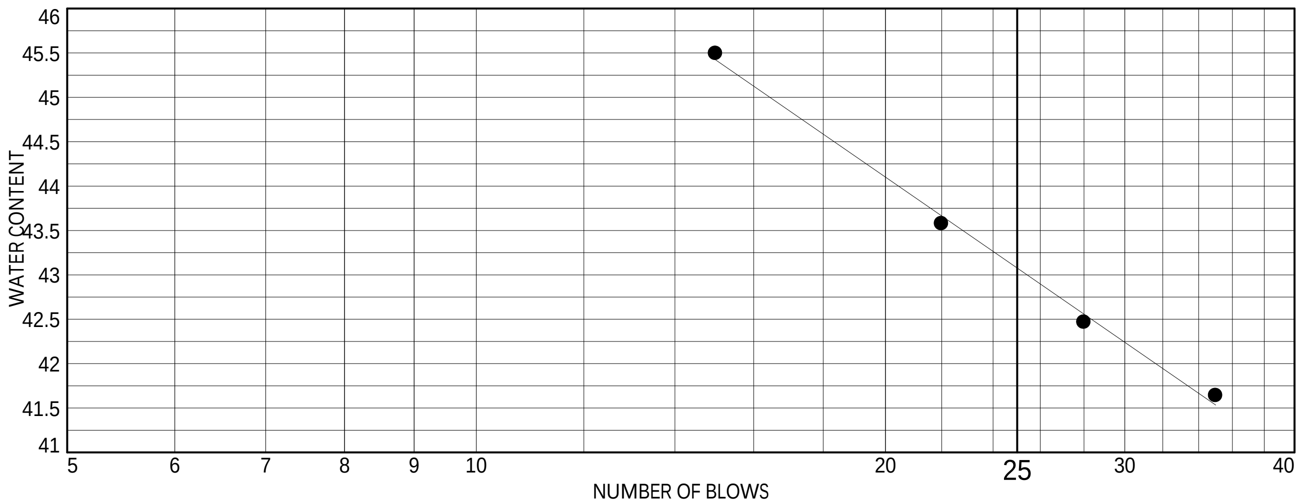
Boring:	B-1	B-1	B-2	B-2				
Sample:	1-2	1-8	2-2	2-4				
Depth, ft:	2	21	2	7				
Visual Description:	Brown Sandy CLAY	Dark Gray Sandy CLAY w/ Gravel	Brown Sandy CLAY w/ Gravel (Weathered Rock)	Brown Clayey SAND w/ organics & glass				
Actual G_s								
Assumed G_s		2.70	2.70					
Moisture, %	6.9	29.3	7.8	30.5				
Wet Unit wt, pcf		119.2	120.1					
Dry Unit wt, pcf		92.2	111.3					
Dry Bulk Dens.pb, (g/cc)		1.48	1.78					
Saturation, %		95.6	41.2					
Total Porosity, %		45.3	33.9					
Volumetric Water Cont,Θw,%		43.3	14.0					
Volumetric Air Cont., Θa,%		2.0	19.9					
Void Ratio		0.83	0.51					
Series	1	2	3	4	5	6	7	8

Note: All reported parameters are from the as-received sample condition unless otherwise noted. If an assumed specific gravity (Gs) was used then the saturation, porosities, and void ratio should be considered approximate.



LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Brown Sandy Lean CLAY w/ Gravel	43	18	25			

Project No. 715-113 Client: Haley & Aldrich

Project: Cesar Chavez Restroom - 0207774

● Source of Sample: B-1 Depth: 2.5' Sample Number: 1-3

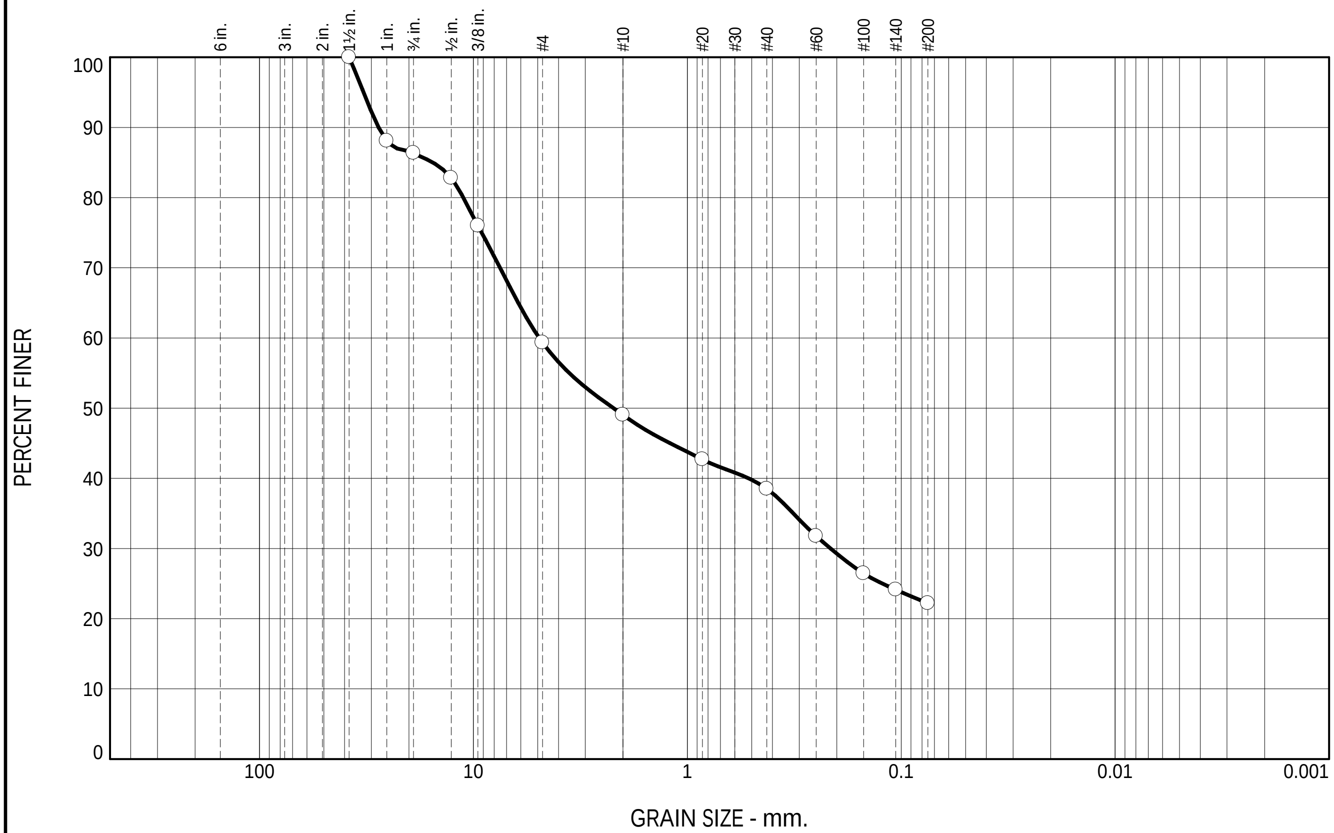
Remarks:

● Sample was prepared using the wet prep method.

COOPER TESTING LABORATORY

Figure

Particle Size Distribution Report



	+3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PL	PI
○	0.0	40.7	37.1	22.2					

SIEVE inches size	PERCENT FINER ○		
1.5	100.0		
1"	88.1		
3/4"	86.4		
1/2"	82.8		
3/8"	76.0		
	GRAIN SIZE		
D_{60}	4.9235		
D_{30}	0.2138		
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ○		
#4	59.3		
#10	49.0		
#20	42.7		
#40	38.5		
#60	31.8		
#100	26.4		
#140	24.1		
#200	22.2		

Material Description

○ Yellowish Brown Clayey GRAVEL w/ Sand

REMARKS:

○

○ Source of Sample: B-1 Depth: 1.5' Sample Number: 1-1

COOPER TESTING LABORATORY

Client: Haley & Aldrich

Project: Cesar Chavez Restroom - 0207774

Project No.: 715-113 Figure

#200 Sieve Wash Analysis
ASTM D 1140

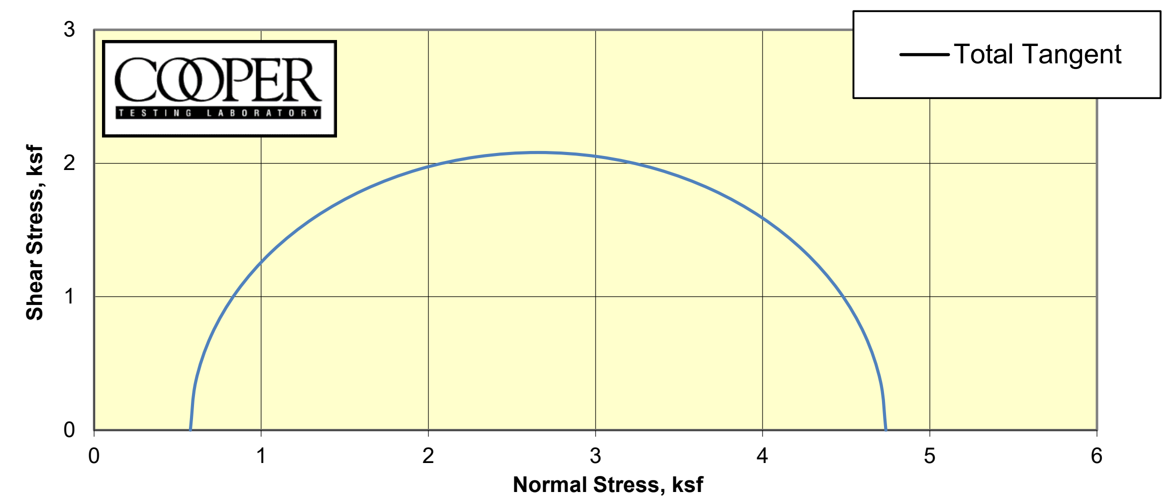
COOPER TESTING LABORATORY

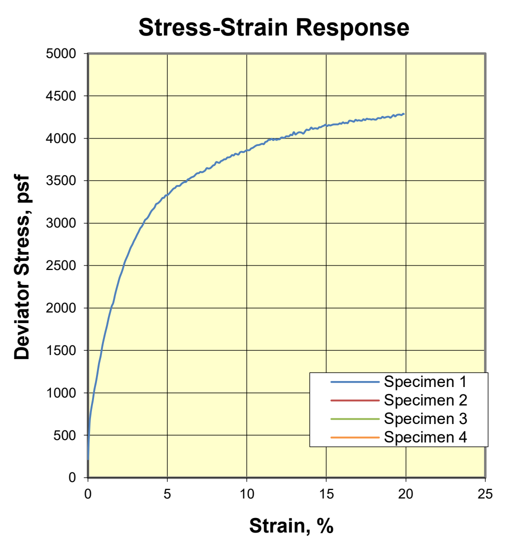
Job No.: 715-113	**Project No.:** 0207774	**Run By:** MD	
Client: Haley & Aldrich	**Date:** 9/17/2024	**Checked By:** DC	
Project: Cesar Chavez Restroom			

Boring:	B-1							
Sample:	1-4							
Depth, ft.:	6							
Soil Type:	Gray Sandy CLAY							
Wt of Dish & Dry Soil, gm	857.0							
Weight of Dish, gm	174.7							
Weight of Dry Soil, gm	682.3							
Wt. Ret. on #4 Sieve, gm	49.2							
Wt. Ret. on #200 Sieve, gm	314.6							
% Gravel	**7.2**							
% Sand	**38.9**							
% Silt & Clay	**53.9**							

Remarks: As an added benefit to our clients, the gravel fraction may be included in this report. Whether or not it is included is dependent upon both the technician's time available and if there is a significant enough amount of gravel. The gravel is always included in the percent retained on the #200 sieve but may not be weighed separately to determine the percentage, especially if there is only a trace amount, (5% or less).

Unconsolidated Undrained Triaxial Compression
ASTM D2850





CTL Number:	715-113
Client Name:	Haley & Aldrich
Project Name:	Cesar Chavez Restroom
Project Number:	0207774
Date:	9/6/2024 By: MD/DC
Total C	ksf
Total phi	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-1			
Sample	1-4			
Depth	6			
Visual Description	Gray Sandy CLAY			
MC (%)	16.2			
Dry Density (pcf)	113.6			
Saturation (%)	86.9			
Void Ratio	0.511			
Diameter (in)	2.42			
Height (in)	5.00			
Final				
MC (%)	19.2			
Dry Density (pcf)	112.4			
Saturation (%)	100.0			
Void Ratio	0.528			
Diameter (in)	2.42			
Height (in)	5.03			
Cell Pressure (psi)	63.0			
Back Pressure (psi)	59.0			
Total Stresses At:				
Strain (%)	15.0			
Deviator (ksf)	4.161			
Excess PP (psi)				
Sigma 1 (ksf)	4.737			
Sigma 3 (ksf)	0.576			
P (ksf)	2.657			
Q (ksf)	2.081			
Stress Ratio	8.224			
Rate (in/min)	0.0246			