Appendix D:
Geotechnical Design Report

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GEOTECHNICAL DESIGN REPORT

FAIRFIELD JUSTICE CAMPUS ASSET PROTECTION PROJECT

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1. INTRODUCTION

1.1 GENERAL

Cal Engineering & Geology (CE&G) is providing geotechnical engineering services to Mead & Hunt in support of the Solano County Justice Center Asset Protection Project. The project, located in the City of Fairfield, consists of providing flood protection to the Solano County Justice Center and is currently in Phase III. Our scope of services included a field investigation, laboratory testing, geotechnical analysis, and design recommendations.

1.2 PROJECT DESCRIPTION

The project, located in the City of Fairfield, consists of providing flood protection to the Solano County Justice Center by constructing concrete floodwalls, flood gates, and berms to protect the center from flood events. Figure 1 shows the approximate location of the project site and vicinity. Flood control structures are planned to be founded on spread footings and/or pier foundations. A pump station sump is also planned at the southeastern corner of Clay Street. The proposed locations of these flood protection improvements are shown on Figure 2, Site Plan.

1.3 PURPOSE AND SCOPE OF SERVICES

The geotechnical investigation completed by CE&G was undertaken to assess the existing surface and subsurface conditions in the vicinity of the project area and to develop geotechnical design recommendations for the proposed flood control infrastructure.

The scope of work completed for the geotechnical investigation and report included:

- Completion of an office study to identify and evaluate relevant geologic and geotechnical information available for the site, including published geologic maps, and previous geotechnical reports completed by others.
- Site reconnaissance to observe current site conditions and to mark the proposed boring locations for USA (Underground Service Alert).
- Completion of a subsurface exploration program using a truck-mounted drill rig, in accordance with Solano County Department of Resource Management (SCDRM) requirements.
- Laboratory testing to determine key engineering index properties of selected earth materials.

- Engineering analyses to develop recommendations for foundations, lateral earth pressures, shoring, earthwork, and pavement sections.
- Preparation of this geotechnical design report.

2. SITE DESCRIPTION

The Solano County Justice Center is located along the southern edge of Fairfield, CA, north of Highway 12. The Solano County Justice Center includes the sheriff's office, the coroner's office, the county superior court, the criminal court, and the county jail. The flat, paved area is bordered to the east and partially to the south by an earth levee. Suisun Marsh lies south of Highway 12.

The Justice Center buildings occupy the southern and western portions of the site, and a large at-grade paved parking lot is in the northeastern portion of the site. The site is bounded on the north by Texas Street, on the east by Clay Street, on the west by Union Street, and on the south by Delaware Street. There is a rail line located near the southeastern side of the Justice Center. The site elevations range from about 9 feet above sea level (asl) in the southern portion of the site to 14 feet asl in the northern portion of the site.

There is an unnamed drainage near the southeastern side of the site that drains to Suisun Slough.

3. GEOLOGIC CONDITIONS

3.1 REGIONAL SETTING

The project site lies within the Coast Ranges geomorphic province, which is a series of discontinuous northwest-trending mountain ranges, ridges, and intervening valleys characterized by complex folding and faulting. The greater San Francisco Bay was one of the intervening valleys within the province. The general geologic framework of the San Francisco Bay Area is illustrated in studies by Schlocker (1971), Wagner et al. (1991), Ellen and Wentworth (1999), and Graymer et al. (2006). Fairfield is located in the North Bay portion of the San Francisco Bay, directly north of the Suisun Bay and Suisun Marsh.

3.2 SITE GEOLOGY

The general vicinity of the project site has been mapped several times, with geologic mapping having different emphases: Bezore and others (1998) (Fairfield South Quadrangle); Wiegers and others (2006) (Fairfield North Quadrangle); Wentworth and others (1997); Knudsen and others (2000); Graymer and others (2006); and Witter and others (2006) (Figure 3, Regional Geologic Map).

These geologists are in general agreement that the site is underlain by Holocene alluvial deposits. Wentworth describes these materials as sand, gravel, silt, and mud. Wiegers describes these materials as predominately clay and silt.

3.3 SURFICIAL SOILS

The surficial soils at the project site have been mapped by the USDA National Resource Conservation Service and USDA Soil Conservation Service as the Capay clay for 0 percent slopes (NRCS, 2021). (Figure 4). The Capay clay is described as a moderately well-drained, flood basin silty and clayey alluvium, derived from metamorphic and sedimentary rock over fan alluvium (also derived from metamorphic and sedimentary rock).

The Capay clay is classified as a fat clay in the upper 40 inches with a plasticity index ranging from 29 to 44 percent and a liquid limit ranging from 53 to 72. From 40 inches to 81 inches below ground surface, it is classified as a lean clay with a plasticity index ranging from 25 to 33 percent and a liquid limit ranging from 45 to 56 percent.

3.4 ACTIVE FAULTS AND SEISMICITY

The project site is located within the North Bay, part of the greater San Francisco Bay Area, which is recognized as one of the more seismically active regions of California. The right-lateral strike-slip San Andreas fault system controls the northwest-southeast structural grain of the Coast Ranges and the Bay Area. The fault system marks the major boundary between two of earth's major tectonic plates, the Pacific Plate to the west and the North American Plate to the east. The Pacific Plate is moving north relative to the North American plate at approximately 40 mm/yr in the Bay Area (WGCEP, 2014).

The transform boundary between these two plates has resulted in a broad zone of multiple, subparallel faults within the North American Plate, along 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 plate motion in the greater San Francisco Bay Area is distributed across other faults such as the Hayward, Calaveras, Concord-Green Valley, Greenville, Rodgers Creek, and West Napa fault zones. Figure 5, Fault Activity Map, shows active faults in the vicinity of the Fairfield area.

An active fault is generally defined as experiencing fault offset in Holocene time (last approximately 11,000 years). According to the US Geological Survey Quaternary Fault and Fold Database (2006), no active faults are mapped as crossing through the site.

According to the 2018 California Department of Transportation ARS website, the closest mapped fault to the project site, the Great Valley Fault (Pittsburg-Kirby Hills section), can produce a magnitude 6.6 earthquake.

Since the project site is located in seismically active California, 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, 2014). Table 3-1 shows the approximate distances between the project site and various major surface fault traces within approximately 50 km of the site (Caltrans, 2018).

Table 3-1. Distances to Selected Active Fault Traces

Fault Name	Approximate Distance and Direction from Site to Surface Fault Traces
Great Valley (Pittsburg Kirby Hills)	6.4 km northeast
Cordelia	8.6 west
Great Valley (Gordon Valley)	9.0 km northeast
Vaca	9.1 km northeast
Great Valley	10.9 west
Los Medanos	20.0 km southwest
West Napa	22.7 km northwest
Concord	23.5 km southwest
Great Valley (Trout Creek)	32.3 km northeast
Clayton	32.6 km south/southeast
Great Valley (Midland)	35.5 km southeast
Rogers Creek	37.0 km southwest
Hayward (North)	39.2 km southwest
Great Valley (Dunnigan Hills)	44.8 km northeast
Calaveras	46.2 km southwest

A large magnitude earthquake on any of these faults or other active fault systems in the greater Bay Area has the potential to cause significant ground shaking at the site. The intensity of ground shaking that is likely to occur at the property is generally dependent upon the magnitude of the earthquake and the distance to the epicenter.

3.4.1 Liquefaction and Seismic Densification

Earthquake-induced soil liquefaction can be described as a significant loss of soil strength and stiffness caused by an increase in pore water pressure resulting from cyclic loading during shaking. The primary factors affecting soil liquefaction include: 1) intensity and duration of seismic shaking; 2) soil type and relative density; 3) overburden pressure; and 4) depth to groundwater. Liquefaction is associated primarily with loose (low density), saturated, fine- to medium-grained, cohesionless soils below the groundwater table, but can also occur in non-plastic to low-plasticity finer-grained soils. The potential consequences of liquefaction to engineered structures include loss of bearing capacity, buoyancy forces on underground structures, ground oscillations, or “cyclic mobility”, increased lateral earth pressures on retaining walls, liquefaction settlement, and lateral spreading or “flow failures” in slopes.

According to a liquefaction susceptibility study by Witter and others (2006), the project site is in an area mapped as having medium liquefaction susceptibility.

CE&G assessed the soil and groundwater conditions encountered in the exploratory borings. Based on subsurface information collected from our borings during this investigation, we judge the potential for liquefaction at the project site to be **low to nil** due to the presence of generally stiff cohesive soils with no significant loose natural granular soils.

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 the sites to be low due to the encountered soils above the groundwater table being primarily cohesive.

4. FIELD INVESTIGATION

4.1 SUBSURFACE EXPLORATION

4.1.1 Exploratory Borings

Five geotechnical borings were drilled as part of our investigation at the approximate locations shown in Figure 2. The borings were advanced 16.5 to 21.5 feet below the ground surface (bgs).

The geotechnical borings were drilled by Taber Drilling, Inc., on May 28, 2021, using a truck-mounted Diedrich D120 drill rig equipped with 4-inch-diameter solid-flight augers and an automatic hammer. Surface conditions at the boring locations consisted of asphalt pavement and gravel base.

Upon completion, the borings were backfilled with cement grout per SCDRM requirements. Drilling spoils were drummed and transported offsite.

4.1.2 Logging and Sampling

The materials encountered in the borings were logged in the field by a CE&G geologist. The soils were 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 hammer using an automatic trip hammer, dropping 30 inches. The number of blows required to drive the 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 taken to CE&G's local office for further analysis and storage.

4.1.3 Soil Conditions Encountered

The soils encountered in the borings generally consisted of artificial fill within the upper 2.5 to 5 feet, and alluvial soils to depths explored. The artificial fill was mostly comprised of fat clay. Concrete debris was encountered in the artificial fill in borings B-02 and B-03.

The alluvial material was generally comprised of mixtures of fat clay, lean clay, and silt of soft to firm consistencies. Poorly-graded gravel with clay and sand was encountered at the bottom 3 feet of boring B-03. The fine-grained materials were generally stiffer with depth.

For a more detailed description of the soils encountered in the borings, please see the boring logs and laboratory test results included in Appendix A.

4.1.4 Groundwater Conditions Encountered

Groundwater was encountered in the borings at depths ranging from 4.5 to 13 feet bgs. Approximate groundwater levels are included in the boring logs in Appendix A. Note that fluctuations in rainfall, tides, and other factors not apparent at the time of exploration, can influence groundwater levels.

4.2 GEOTECHNICAL LABORATORY TESTING

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; dry method)
- Particle Size Analysis (ASTM D6913 and ASTM D1140)
- Unconsolidated Undrained Triaxial Compression (ASTM D2850)
- Resistivity (Minimum) (Caltrans 643)
- pH (Caltrans 643)
- Sulfate Content (Caltrans 417-mod)
- Chloride Content (Caltrans 422-mod)

The results of the laboratory testing program are presented in Appendix B and are summarized below.

4.2.1 Index Tests

Moisture and density tests were performed on select samples at various depths from the borings. The soil and bedrock samples tested had moisture contents between 9 and 27 percent with dry densities between 96 and 116 pcf.

4.2.2 Atterberg Limits

Atterberg Limits testing was performed on three samples to determine the plasticity of fine-grained materials. Liquid limits were 26, 37, and 44, with plasticity indices of 7, 15, and 24 percent. A figure plotting liquid limit versus plasticity index is presented in Appendix B.

4.2.3 Shear Strength Testing

Shear strength testing was performed on three clay soil samples for unconsolidated-undrained triaxial strength with back-pressure saturation. Strength testing produced reasonable shear strengths for the soils encountered. The results including the shear stress plots are presented in Appendix B.

5. DISCUSSION AND CONCLUSIONS

The proposed flood protection improvements for the Fairfield Justice Center campus include:

- Raised roadway berms/ramps with retaining walls
- Flood barrier walls
- Passive flood gates
- Interior drainage improvements, including a new pump station

Except for the proposed sumps, the proposed structures will be supported on the existing artificial fill and/or native soils that are potentially expansive. The proposed sumps are deep enough that they will be supported on native soils. The soils encountered consist of fine-grained colluvium and residual soils. It is our opinion that the site can support the proposed improvements provided the design recommendations presented below are incorporated into the design.

5.1 EXPANSIVE SOILS

Near-surface soils are moderately to highly expansive. The shrink/swell effects of expansive soils are most common on pavements and lightly loaded slabs, as opposed to more heavily loaded foundations or mats. The impacts of expansive soils can be mitigated/reduced by proper moisture conditioning during site preparation and grading, and by placing non-expansive fill over the potentially expansive soils.

5.2 CORROSION

Corrosion testing was performed on two samples of near-surface soils and one sample of deeper soil from the project site within the expected depth of work in general accordance with Caltrans methods. Test results are presented below:

Table 5-1. Corrosion Testing Results

Boring (depth in feet)	Minimum Resistivity (Ohm-cm)	Chloride (mg/kg)	Sulfate (mg/kg)	pH
B-03 (6.5)	1,367	9	64 (0.0064%)	9.0
B-04 (5.5)	2,844	10	36 (0.0036)	9.1
B-05 (11.0)	1,266	27	95 (0.0095)	8.8

Caltrans Corrosion Guidelines, January 2015, identifies a site as being corrosive for structural elements if one or more of the following conditions exist:

- Chloride concentration is 500 ppm or greater;
- Sulfate concentration is 2000 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 propensity for corrosion. Based on the results of the laboratory testing performed, the soil samples tested had values for Chloride, Sulfate, pH that do not meet the Caltrans criteria for a corrosive site. The minimum resistivity of the tested soil samples was above the 1000 ohm-cm threshold defined.

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)
- The 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 samples tested had values for Sulfate and Chloride that do not meet ACI criteria and are 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.

5.3 LIQUEFACTION POTENTIAL

CE&G performed a qualitative assessment of the liquefaction potential of the soils encountered beneath the project area. The liquefaction assessment was performed by reviewing the soil types encountered, SPT blow counts, and fines content from our boring logs and laboratory testing. Based on the apparent absence of granular materials in the borings, the liquefaction potential is deemed to be **low to nil** due to the presence of generally stiff cohesive soils.

6. DESIGN AND CONSTRUCTION CONSIDERATIONS

Detailed recommendations for the geotechnical aspects of the proposed flood protection improvements are presented in subsequent sections of this report. Our evaluations and recommendations are based upon the previously discussed information collected for this investigation. The following recommendations may need to be modified if there are any changes in the proposed improvements, their layout or location, or the proposed grading.

6.1 DESIGN GROUNDWATER LEVEL

Groundwater was encountered in t our borings at depths between 4.5 and 13 feet below ground surface (approximately El. 1 to 6.5 feet). Groundwater may fluctuate depending on rainfall, groundwater pumping, proximity to adjacent ditches/channels, and landscape irrigation or other activities.

6.2 EARTHWORK

6.2.1 Clearing

Clearing will include selective removal of existing pavements and flatwork within the project areas along with existing landscaping to facilitate the new construction. Existing and abandoned underground utilities that may be present in areas of the planned improvements will also require removal or relocation.

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

6.2.2 Excavations

Excavations for this project will include excavation for subgrade preparation, removing existing underground facilities designated for removal, trenching for underground utilities, excavations for sump structures, and foundation excavations.

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, are the responsibility of the contractor.

Trench excavations adjacent to existing or proposed 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.

6.2.3 Site Preparation

As discussed in Section 5.1, the site is underlain by expansive soils to depths of up to about 4 to 5 feet below the existing grade. Expansive soils beneath slabs or flatwork should be removed to one foot below subgrade and replaced with non-expansive engineered fill and base rock.

After site preparation and before placement of compacted fills, the excavation bottom should be observed and approved by the geotechnical engineer or their representative. After approval, the subgrade should be scarified to a minimum depth of 8 inches, moisture conditioned to at least 3 percent of optimum moisture content, and compacted to between 88 and 92 percent of the maximum dry unit weight as measured by ASTM D1557. to the recommendations given under Section 6.2.5, Engineered Fill Placement and Compaction.

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.

6.2.4 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 aggregate base or subbase material) is required.

In general, engineered fill material should not contain rocks or lumps larger than 3 inches in greatest dimension, should not contain more than 15 percent of the material larger than

1½ inches, and should contain at least 20 percent passing the No. 200 sieve. In addition to these requirements, import fill should have a low expansion potential as indicated by a Plasticity Index of 15 or less, or an Expansion Index of less than 20. Structure backfill material should not contain rocks or lumps larger than 2 inches in greatest dimension, should not contain more than 15 percent of the material larger than 1½ inches, and should contain at least 30 percent passing the No. 200 sieve, a Liquid Limit less than 30, and a Plasticity Index between 8 and 15.

Engineered fill materials for the proposed flood berms not constrained between retaining walls should not contain rocks or lumps larger than 2 inches in greatest dimension, should not contain more than 10 percent of the material larger than 1½ inches, and should contain at least 30 percent passing the No. 200 sieve. In addition to these requirements, import fill should have a Liquid Limit less than 40 and a Plasticity Index between 8 and 30.

All import fills must be approved by the project geotechnical engineer, before delivery to the site, by providing representative samples of proposed import fills to the engineer for evaluation.

6.2.5 Engineered Fill Placement and Compaction

Engineered fill should be placed on soil subgrades that are prepared as recommended in this report. 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 determined 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 at least 2 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 slightly 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.

6.2.6 Utility Trench Excavation and Backfill

Utility trenches less than 5 feet in depth in the near-surface soil materials should be able to stand near vertical with minimal bracing. Sandy soils, where encountered, may need bracing to prevent the caving of the granular soils. We estimate that excavations should be able to be accomplished with conventional excavating equipment, such as backhoes and excavators. 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, are the responsibility of the contractor.

Pipe zone backfill, extending from the bottom of the trench to about 1 foot above the top of the 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, or City of Fairfield or Solano County standard specifications dictate otherwise.

Above the pipe zone, underground utility trenches may be backfilled with free-draining sand, on-site soil, or imported soil that is free of deleterious and hazardous material. The trench backfill should be compacted to the requirements given in Section 6.2.5, "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 should not be permitted.

Trench excavations that extend below an imaginary plane inclined at 1½:1 (h: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.

6.2.7 Wet Weather Construction

If site grading and construction are 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 delays 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. We recommend the grading contractor submit a wet weather construction plan outlining procedures they will employ to protect their work and to minimize damage to their work by rainstorms.

6.3 FOUNDATIONS

Based on our understanding of the project components, they can be supported on spread footings. We expect that the imposed loads for the various project components will generally be light to moderate. Recommendations for conventional spread footing foundations are presented in the following paragraphs.

Continuous and isolated footings may be designed to impose a maximum allowable bearing pressure of 2,000 pounds per square foot on foundation soils when considering dead loads and 3,000 psf 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 spread footings have a minimum width of 15 inches and 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 0.32 may be used for design. In addition, an allowable passive lateral bearing pressure equal to an equivalent fluid pressure of 300 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.

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 before the 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 is anticipated to be less than ¾ inch with differential settlements on the order of ¼ inch or less for a distance of 25 feet.

We request the opportunity to review the foundation plans and to provide supplemental recommendations as necessary.

6.4 LATERAL EARTH PRESSURES

Static lateral earth pressure will be imposed on retaining walls and shored excavations. Table 6 summarizes the lateral earth pressures recommended for use in the design of retaining walls and unbraced temporary shoring. Active pressure should be assumed for conditions where the top of the wall is free to deflect up to ½ inch. Passive pressure should be ignored for a depth of 18 inches in unpaved areas 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-.

Table 6-1 – Lateral Earth Pressures

Pressure Type	Above Groundwater Level (Equiv. Fluid Pressure)	Below Groundwater Level (Equiv. Fluid Pressure + Hydrostatic)
Active	45 pcf	84 pcf
At-Rest	65 pcf	94 pcf
Passive	300 pcf	200 pcf

The design team should also consider designing retaining wall systems for a temporary increase in lateral earth pressures due to dynamic earth pressures during seismic events. For seismic design considerations, the active earth pressure equivalent fluid weight of 45 pcf may be used in conjunction with a seismic pressure increment based on an equivalent fluid weight of 38 pcf. All pressure distributions may be taken as triangular, and all resultant forces may be assumed to apply at a distance of H/3 above the bottom of the wall (where H=wall height). The walls should be designed for the maximum of either the active plus seismic pressures, or the at-rest pressure (of 65 pcf).

Retaining walls should also be designed to resist an additional uniform surcharge pressure acting within the upper 5 feet of the wall, equivalent to one-half of any surcharge pressure applied at the surface. For light traffic loads (e.g., passenger vehicles) applied within 2 feet of the walls, an additional design load of 100 pounds per linear foot applied 1 foot below top of wall, should be added to earth pressures.

The above pressures are based on the assumption that sufficient drainage will be provided behind the walls to prevent the build-up of hydrostatic pressures from surface and subsurface water infiltration. Adequate drainage can be provided by a back drain system consisting of a 4-inch diameter perforated pipe bedded in a 12-inch-wide zone of 3/4-inch clean, open-graded rock at the base of the wall. The entire rock/pipe unit should be wrapped in filter fabric. The rock and fabric placed behind the wall should be at least one foot in width and should extend to within one foot of finished grade. The upper one foot of backfill should consist of on-site, compacted soils. Alternatively, prefabricated drainage panels can be used instead of drain rock, with the drainage panels connected to a 4-inch-diameter perforated pipe at the base of the wall. The back drainpipe should be sloped to drain by gravity and be connected to a system of closed pipes that lead to suitable discharge facilities. In addition, the "high" end and all 90-degree bends of the subdrain pipe should be connected to a riser which extends to the surface and acts as a cleanout. Where migration of moisture through retaining walls would be detrimental or undesirable, retaining walls should be waterproofed.

As noted previously, the design of unbraced shoring will likely be controlled by deflections and the shoring is anticipated to require bracing.

If the temporary shoring will be braced, a rectangular or trapezoidal loading diagram such as those recommended by Terzaghi & Peck, Tschebortarioff, and others (Caltrans Trenching and Shoring Manual and FHWA GEC No. 4) should be used. These methods generally correlate the earth pressure load to a percentage of the unit weight of the soil times the height of the excavation. The method and loading should be determined by the contractor and provided to the Engineer for review.

If a rectangular or trapezoidal loading is used, the native sandy deposits can be assumed to have a uniform lateral load of 30H psf for the full height (H) (in feet) of the excavation shoring plus a lateral fluid pressure of 62.4 pcf (unit weight of water) starting at the design groundwater elevation to account for groundwater. It is recommended that the contractor's shoring design engineer evaluate high and low groundwater cases to confirm which case governs the design. Shoring for excavations that penetrate below an imaginary plane having an inclination of 1½:1 (horizontal to vertical) extending down from the bottom edge of the foundations must be designed for foundation surcharge pressures and must limit deflections to less than ½ inch. Surcharge loading from traffic on the adjacent areas and construction equipment adjacent to excavations should be considered in shoring design.

6.5 SEISMIC DESIGN PARAMETERS

Due to the proximity of the site to the numerous active fault systems which traverse the greater San Francisco Bay Area, the project site will likely 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 shaking and 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 under the requirements of Chapter 16 of the 2019 California Building Code (CBC) and ASCE 7-16 for Site Class D type soils. The CBC seismic design parameters for the site are included in Table 6-1. The design parameters utilize a PGA of 0.657g.

Table 6-2. 2019 CBC Design Parameters

Item	Design Value	Source
Site Soil Class Definition	D	Table 1613.5.2
PGA	0.657	
PGA_M	0.723	
0.2 Second Spectral Response Acceleration, Ss	1.608	Figure 1613.5(3)
1.0 Second Spectral Response Acceleration, S_1	0.563	Figure 1613.5(4)
Values of Site Coefficient, Fa	1.0	Table 1613.5.3(1)
Value of Site Coefficient, Fv	1.7	Table 1613.5.3(2)
Designed Spectral Response Acceleration for Short Periods, S_{DS}	1.072	Equation 16-38 (S_{DS}=2/3(Fa Ss)
Designed Spectral Response Acceleration for 1-Sec Periods, S_{D1}	0.638	Equation 16-39 (S_{DS}=2/3(Fv S_1)
Long-period transition period, T_L, sec	8	

Note: The above parameters assume structures are not seismically isolated and do not incorporate a damping system. If this is not the case, a ground motion hazard analysis may be required. Reference: https://asce7hazardtool.online/ and https://hazards.atcouncil.org/#/

In place of the seismic lateral earth pressure parameters provided in Section 6.4, the following parameters may be used for a more rigorous design assuming a design lateral acceleration, k_h = 0.5xPGA, and the backfill parameters in Table 6-3 below.

T Table 6-3. Backfill Parameters

Parameter	Native Soil	Aggregate Base
Unit Weight, pcf	125	129
Saturated Unit Weight, pcf	128	132
Angel of Internal Friction, degrees	30	38

6.6 APPURTENANT SLABS

The use of concrete slab-on-grade construction is anticipated for exterior flatwork, walkways, and similar items. We recommend that a minimum of 12 inches of non-expansive engineered fill be placed beneath exterior concrete slabs-on-grade, prepared as recommended in the Site Preparation Section of this report. Soil subgrade should be maintained in a moist condition before pouring the concrete slab. If expansive soil has not been over-excavated, moisture-conditioned, and engineered, then the non-expansive fill or base rock should be underlain by a woven geotextile such as Mirafi 500x, or equal.

To reduce the potential for cracking of the appurtenant 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 should 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 should 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.

In the areas where flatwork and other improvements are to be made, we recommend that any underlying loose soil be removed and re-compacted as engineered fill to provide a more stable base for support of the structures. These soils should be compacted to the relative compaction specified in Section 6.2.5 as determined by the ASTM D-1557 test procedure. Due to the high expansion potential of the site soils, removal and re-compaction of the loose soils will reduce, but not eliminate, differential movement of shallow-founded structures and some cracking and heaving of the slabs may occur and should be expected.

To reduce the swell potential of the subgrade soils where slab-on-grade construction is planned, a near-saturation condition of the subgrade soil should be attained before the concrete slab is placed. This should be done to the satisfaction of the engineer or geologist from Cal Engineering & Geology, Inc. observing the work.

Pavements should slope to appropriate stormwater facilities.

6.7 PAVEMENTS

New and replacement pavements will be constructed as part of this project. We understand a design Traffic Indices (TIs) have not yet been established for the subject site, but we estimate that they will be in the range of between 4 and 6. We have developed alternate pavement sections for these TIs. The actual pavement sections for the proposed improvements should be determined by the project Civil Engineer. Soils at the site have an assumed R-value of 5. Based on this R-value, the following alternative pavement sections were developed:

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	7.5	10.0
5.0	2.5	11.0	13.5
6.0	3.0	13.5	16.5

Pavement sections should be placed on soil surfaces that have been prepared as outlined in the Grading 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 ½- or ¾-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.

6.8 TECHNICAL REVIEW AND CONSTRUCTION OBSERVATION

Before 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. LIMITATIONS

The conclusions and recommendations presented in this report are based on the information provided regarding the proposed project, and the results of the site reconnaissance, subsurface exploration, and laboratory 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 Owner's/Client'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.

The findings of this report should be considered valid for three years unless the conditions of the site change. After 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 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. REFERENCES

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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), 2019.

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0 0.25 0.5 1
MILES

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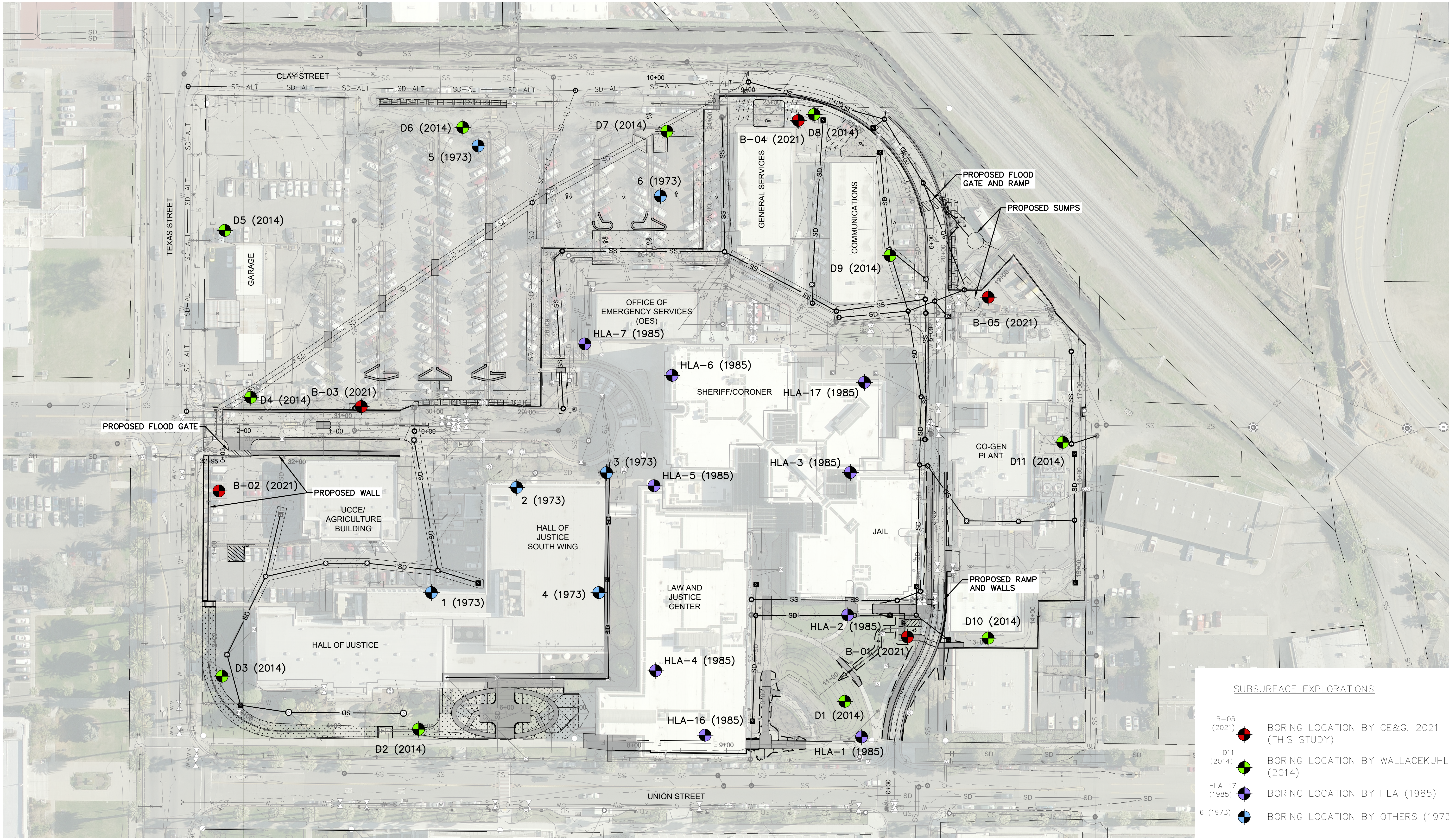
FLOOD PROTECTION IMPROVEMENTS
FAIRFIELD
SOLANO COUNTY, CALIFORNIA

SITE LOCATION MAP

190841	SEPTEMBER 2021	FIGURE 1



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1. BASEMAP FROM MEAD&HUNT; CAD FILES TITLED "1943400-190824.01 MH SHEET SET_V2020 - MH STANDARD.ZIP" RECEIVED ON 7/16/2021.
2. ORTHOIMAGERY FROM SOLANO COUNTY, 2008.

50 0 50 100
SCALE FEET

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FLOOD PROTECTION IMPROVEMENTS
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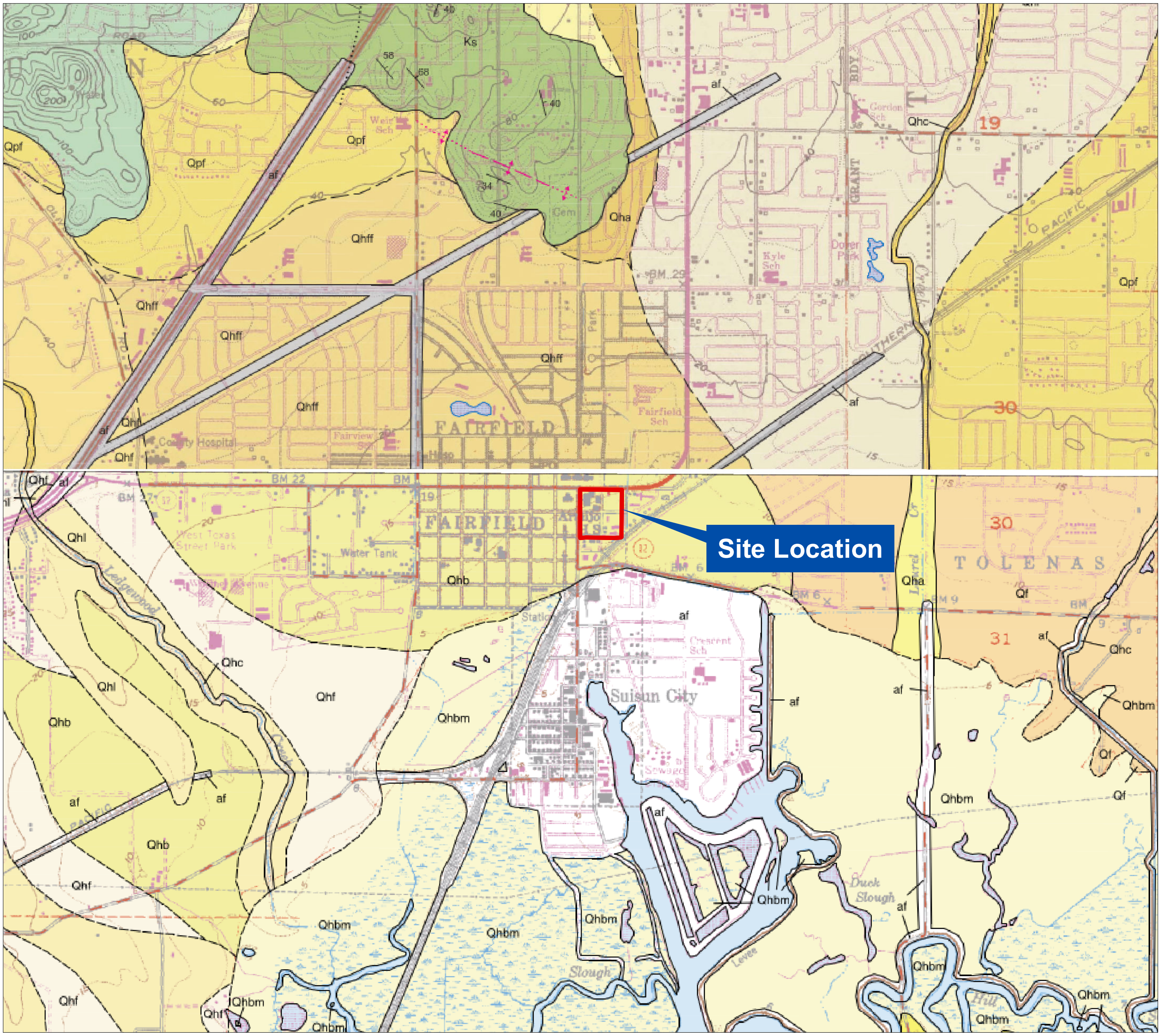
SITE PLAN

190841	SEPTEMBER 2021	FIGURE 2



BASEMAP REFERENCE

1. FAIRFIELD NORTH QUAD REGIONAL GEOLOGY FROM WIEGERS ET AL. 2006.
2. FAIRFIELD SOUTH QUAD REGIONAL GEOLOGY FROM BEZORE ET AL. 1998.

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MILES

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MAP UNIT DESCRIPTION FAIRFIELD NORTH

Af	ARTIFICIAL FILL (HOLOCENE, HISTORIC)
Qhc	MODERN STREAM CHANNEL DEPOSITS (HOLOCENE < 150 YEARS)
Qha	ALLUVIUM, UNDIVIDED (HOLOCENE)
Qhf	ALLUVIAL FAN DEPOSITS (HOLOCENE)
Qhff	ALLUVIAL FAN DEPOSITS, FINE FACIES (HOLOCENE)
Qa	ALLUVIUM, UNDIVIDED (LATEST PLEISTOCENE TO HOLOCENE)
Qpf	ALLUVIAL FAN DEPOSITS (LATE PLEISTOCENE)
Ks	SITES FORMATION (LATE CRETACEOUS)

MAP UNIT DESCRIPTION FAIRFIELD SOUTH

Af	ARTIFICIAL FILL
Qhbm	HOLOCENE ESTUARINE DEPOSITS (BAY MUD)
Qhc	MODERN STREAM CHANNEL DEPOSITS
Qhb	HOLOCENE BASIN DEPOSITS
Qhf	HOLOCENE FAN DEPOSITS
Qhl	HOLOCENE FAN LEVEE DEPOSITS
Qha	HOLOCENE ALLUVIUM, UNDIVIDED
Qf	LATE PLEISTOCENE TO HOLOCENE FAN DEPOSITS
Qa	LATE PLEISTOCENE TO HOLOLCENE ALLUVIUM, UNDIVIDED

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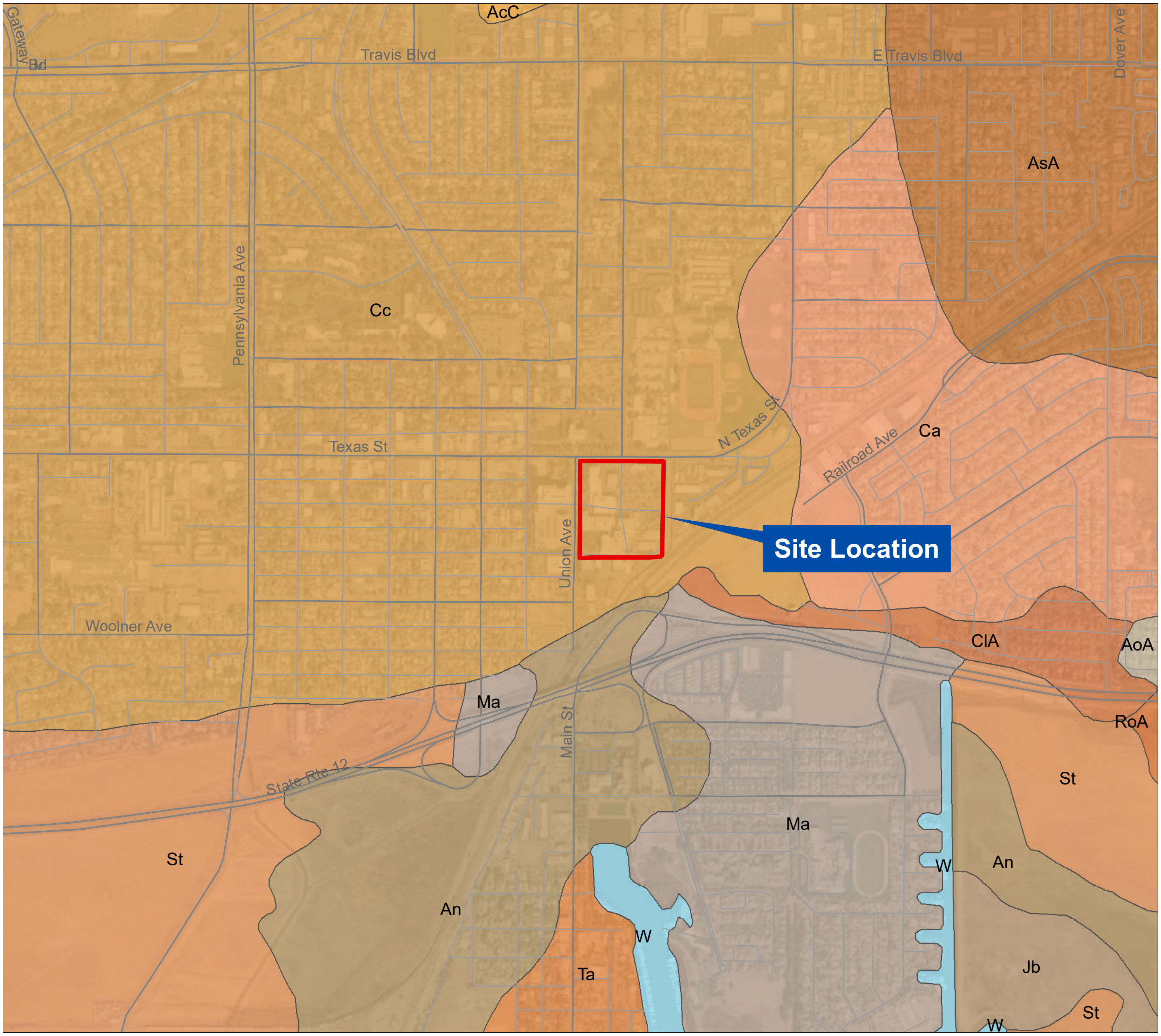
REGIONAL GEOLOGY MAP

190841	SEPTEMBER 2021	FIGURE 3

M:\2019\190841-MeadHunt_FairfieldJustice_Phase3\GIS\ArcGIS\Fig3-Regional-Geology.mxd; 9/2/2021; kdrozynska



BASEMAP REFERENCE

1. SOIL DATA DOWNLOADED ONLINE FROM NRCS SOIL SURVEY ON 07 JULY, 2021.

0 0.125 0.25 0.5
MILES

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MAP UNIT DESCRIPTION

AcC	ALTAMONT CLAY, 2 TO 9 PERCENT SLOPES	Ca	CAPAY SILTY CLAY LOAM, 0 PERCENT SLOPES, MLRA 17	Ma	MADE LAND
An	ALVISO SILTY CLAY LOAM	Cc	CAPAY CLAY, 0 PERCENT SLOPES, MLRA 17	RoA	RINCON CLAY LOAM, 0 TO 2 PERCENT SLOPE
AoA	ANTIOCH-SAN YSIDRO COMPLEX, 0 TO 2 PERCENT SLOPES	ClA	CLEAR LAKE CLAY, SALINE, DRAINED, 0 TO 2 PERCENT SLOPES, MLRA 14	St	SYCAMORE SILTY CLAY LOAM, SALINE
AsA	ANTIOCH-SAN YSIDRO COMPLEX, THICK SURFACE, 0 TO 2 PERCENT SLOPES	Jb	JOICE MUCK, CLAYEY SUBSOIL, 0 TO 2 PERCENT SLOPES, MLRA 16	Ta	TAMBA MUCKY CLAY, MLRA 16
				W	WATER

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NRCS SOILS MAP

190841	SEPTEMBER 2021	FIGURE 4

M:\2019\190841-MeadHunt_FairfieldJustice_Phase3\GIS\ArcGIS\Fig4-NRCS-Soils.mxd; 9/2/2021; kdrozynska

BASEMAP REFERENCE

1. FAULT LOCATIONS FROM US GEOLOGICAL SURVEY QUATERNARY FAULTS AND FOLDS DATABASE, ACCESSED ONLINE ON 12 DECEMBER 2017.

0 1.5 3 6
MILES

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MAP UNIT DESCRIPTION

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), Moderately 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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FLOOD PROTECTION IMPROVEMENTS
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FAULT ACTIVITY MAP

190841	SEPTEMBER 2021	FIGURE 5

Appendix A. Boring Logs

BORING NUMBER B-01

CAL ENGINEERING & GEOLOGY

CLIENT Mead & Hunt, Inc.
PROJECT NAME Fairfield Justice Campus Asset Protection Phase III
PROJECT NUMBER 190841
PROJECT LOCATION Fairfield, CA
DATE STARTED 5/28/2021 **COMPLETED** 5/28/2021
GROUND ELEVATION 12 ft **DATUM** WGS84 **HOLE SIZE** 4 in.
DRILLING CONTRACTOR Taber Drilling
COORDINATES: LATITUDE 38.247015 **LONGITUDE** -122.039981
DRILLING RIG/METHOD 4-in. Solid Flight Auger
GROUNDWATER AT TIME OF DRILLING 7.5 ft / Elev 4.5 ft
LOGGED BY R. Briseno **CHECKED BY**
GROUNDWATER AT END OF DRILLING ---
HAMMER TYPE 140 lb hammer with 30 in. autotrip
GROUNDWATER AFTER DRILLING ---

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTICITY INDEX (%)	FINES CONTENT (%)
0	ASPHALT-3.5 in.									
	BASEROCK-4.5 in.									
	Poorly Graded SAND (SP): brown, moist, loose, medium sand (ARTIFICIAL FILL)									
	Fat CLAY with Sand (CH): black, moist, firm, medium sand	CM	5-4-5							
	Fat CLAY (CH): black, moist, firm									
	Fat CLAY (CH): bluish gray and brown, moist, firm, iron stained	SPT	3-3-5	1						
5	Lean CLAY (CL): brown, moist, firm, well consolidated nodule at 6.5 ft.	CM	2-2-4	0	106	25	37	22	15	
	∇	SPT	0-1-2	0.5						
	wet, soft, iron and manganese stains									
10	Lean CLAY with Sand (CL): brown and dark brown, wet, firm, medium sand, iron and manganese stains	CM	2-2-4	0						
		SPT	5-5-7	0	114	18				
15	Lean CLAY with Sand (CL): gray, moist, firm, fine sand, iron and manganese stains	SPT	6-8-12							75

Bottom of borehole at 16.5 ft. Borehole backfilled with neat cement grout.

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BORING NUMBER B-02

CLIENT Mead & Hunt, Inc.
PROJECT NAME Fairfield Justice Campus Asset Protection Phase III
PROJECT NUMBER 190841
PROJECT LOCATION Fairfield, CA
DATE STARTED 5/28/2021 **COMPLETED** 5/28/2021
GROUND ELEVATION 14 ft **DATUM** WGS84 **HOLE SIZE** 4 in.
DRILLING CONTRACTOR Taber Drilling
COORDINATES: LATITUDE 38.249062 **LONGITUDE** -122.039432
DRILLING RIG/METHOD 4-in. Solid Flight Auger
▽ **GROUNDWATER AT TIME OF DRILLING** 12.5 ft / Elev 1.5 ft
LOGGED BY R. Briseno **CHECKED BY**
GROUNDWATER AT END OF DRILLING ---
HAMMER TYPE 140 lb hammer with 30 in. autotrip
▼ **GROUNDWATER AFTER DRILLING** 6.0 ft / Elev 8.0 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 (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
0		ASPHALT-3 in.									
		BASEROCK-1 in.									
		Fat CLAY (CH): black, moist, firm, concrete fragments in upper 3 feet (ARTIFICIAL FILL)	CM	8-8-9			15				
			SPT	4-4-5							
5		▼ Lean CLAY (CL): brown, moist, soft, few organic matter TXUU @ 6 ft	CM	4-5-6	0.5	98	26				
		Lean CLAY (CL): brown, moist, soft, iron and manganese stains	SPT	2-3-4	0.5						
10		Lean CLAY with Sand (CL): grayish brown, moist, soft	CM	6-9-11							
		▽ wet, 4 in. section of soft, some very fine sand, iron and manganese stains	SPT	3-4-5							
15		Lean CLAY with Sand (CL): grayish brown, wet, firm, iron and manganese stains, some fine sand	SPT	5-5-6							55
20		Lean CLAY with Sand (CL): grayish brown, wet, soft, iron and manganese stains, grades to firm at 21 ft.	SPT	4-5-9							

Bottom of borehole at 21.5 ft. Borehole backfilled with neat cement grout.

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BORING NUMBER B-03

CLIENT Mead & Hunt, Inc.
PROJECT NAME Fairfield Justice Campus Asset Protection Phase III
PROJECT NUMBER 190841
PROJECT LOCATION Fairfield, CA
DATE STARTED 5/28/2021 **COMPLETED** 5/28/2021
GROUND ELEVATION 14 ft **DATUM** WGS84 **HOLE SIZE** 4 in.
DRILLING CONTRACTOR Taber Drilling
COORDINATES: LATITUDE 38.248639 **LONGITUDE** -122.039114
DRILLING RIG/METHOD 4-in. Solid Flight Auger
▽ **GROUNDWATER AT TIME OF DRILLING** 9.3 ft / Elev 4.8 ft
LOGGED BY R. Briseno **CHECKED BY**
GROUNDWATER AT END OF DRILLING ---
HAMMER TYPE 140 lb hammer with 30 in. autotrip
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		ASPHALT- 1 in.									
		BASEROCK-2 in.									
		Concrete fragments	CM	24-14-15							
		Fat CLAY (CH): very dark gray, moist, firm (ARTIFICIAL FILL)	SPT	3-4-5							
5		Fat CLAY (CH): bluish gray mottled with yellowish brown (iron stains), moist, firm, few organic matter, increase in iron stains near 8 ft. TXUU @ 6 ft	CM	4-8-11		99	26				
			SPT	2-4-4							
	▽										
10		Lean Clay (CL): brown, wet, firm, caliche, iron and manganese stains grades with very fine sand	CM	5-8-12	2.25	110	19				
			SPT	5-6-6	1.75						
15		Lean CLAY with Silt and Sand (CL): brown, wet, soft to firm, iron and manganese stains, few very fine to medium sand	SPT	7-7-10				26	19	7	
20		Poorly Graded GRAVEL with Clay and Sand (GP-GC): dark brown, wet, medium dense, rounded gravel up to 1.0 in.	SPT	7-8-8							11

Bottom of borehole at 21.5 ft. Borehole backfilled with neat cement grout.

BORING NUMBER B-04

CE&G
CAL ENGINEERING & GEOLOGY

CLIENT Mead & Hunt, Inc.
PROJECT NAME Fairfield Justice Campus Asset Protection Phase III
PROJECT NUMBER 190841
PROJECT LOCATION Fairfield, CA
DATE STARTED 5/28/2021 **COMPLETED** 5/28/2021
GROUND ELEVATION 11 ft **DATUM** WGS84 **HOLE SIZE** 4 in.
DRILLING CONTRACTOR Taber Drilling
COORDINATES: LATITUDE 38.24734 **LONGITUDE** -122.038035
DRILLING RIG/METHOD 4-in. Solid Flight Auger
∇ **GROUNDWATER AT TIME OF DRILLING** 4.5 ft / Elev 6.5 ft
LOGGED BY R. Briseno **CHECKED BY**
GROUNDWATER AT END OF DRILLING ---
HAMMER TYPE 140 lb hammer with 30 in. autotrip
GROUNDWATER AFTER DRILLING ---

DEPTH (ft)	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	ASPHALT- 4 in.									
	BASEROCK-4.5 in.									
	Fat CLAY (CH): very dark gray, moist, firm, iron stained, trace organics (ARTIFICIAL FILL)	CM	5-7-9	1.25	96	27				
		SPT	6-7-10	1						82
	∇									
5	Well Graded SAND with Gravel (SW): brown, wet, very loose, fine to coarse sand, fine gravel	CM	2-2-4	0.75						
	SILT with Sand (ML): brown, wet, very loose/soft, fine sand									
	Lean CLAY with Sand to Clayey SAND with Silt (CL/SC): gray, moist, grades to firm by 8 ft., iron and manganese stains	SPT	0-0-2	0						
10	Silty CLAY (CL-ML): brown, moist, firm, iron and manganese stains	CM	8-10-14							
	Silty CLAY (CL-ML): brown, moist, firm, iron and manganese stains	SPT	5-7-9	2.25	113	19				
15	Lean CLAY (CL): gray, moist, hard, iron and manganese stains, caliche nodules	CM	9-13-21	4.25 2.5	112	19				
20	SILT (ML): gray, moist, medium dense/soft, iron stains	CM	7-11-13	0.5	108	22				

Bottom of borehole at 21.5 ft. Borehole backfilled with neat cement grout.

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BORING NUMBER B-05

CLIENT Mead & Hunt, Inc.
PROJECT NAME Fairfield Justice Campus Asset Protection Phase III
PROJECT NUMBER 190841
PROJECT LOCATION Fairfield, CA
DATE STARTED 5/28/2021 **COMPLETED** 5/28/2021
GROUND ELEVATION 14 ft **DATUM** WGS84 **HOLE SIZE** 4 in.
DRILLING CONTRACTOR Taber Drilling
COORDINATES: LATITUDE 38.246775 **LONGITUDE** -122.038701
DRILLING RIG/METHOD 4-in. Solid Flight Auger
GROUNDWATER AT TIME OF DRILLING 13.0 ft / Elev 1.0 ft
LOGGED BY R. Briseno **CHECKED BY**
GROUNDWATER AT END OF DRILLING ---
HAMMER TYPE 140 lb hammer with 30 in. autotrip
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		Fat CLAY (CH): black and dark brown, moist, hard, rock fragments, subangular gravel gravel, few sand (ARTIFICIAL FILL)	CM	16-11-16			9				
			SPT	6-5-7	4.25						
5		Fat CLAY and SILT (CH): light brown and very dark brown, moist, firm, gravel, iron stained	CM	4-4-5		99	21				
			SPT	2-3-3	1.25						
10		Lean CLAY (CL): dark bluish gray and greenish gray, moist, soft TXUU @ 11 ft	CM	3-3-5	0.5	105	20				
		Lean CLAY (CL): gray, soft grades to very soft, wet at 13 ft., iron and manganese stains	SPT	3-4-5							
15		Lean CLAY (CL): brown, wet, very soft/loose, iron and manganese stains	CM	4-7-8	0.5	116	20				
20		Lean CLAY (CL): brown, moist, hard, iron and manganese stains, caliche	SPT	6-7-14				44	20	24	

Bottom of borehole at 21.5 ft. Borehole backfilled with neat cement grout.

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

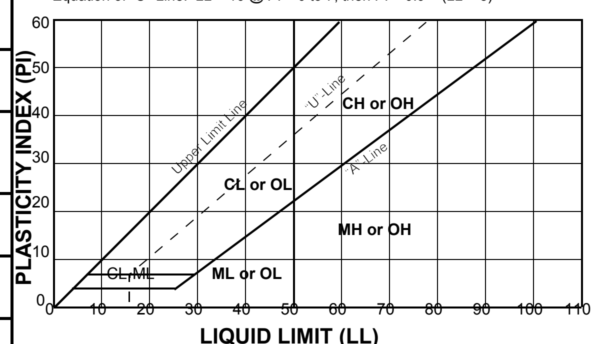
Field Identification			Group Symbols	Typical Names	Laboratory Classification Criteria		
Coarse-Grained Soils More than 50% of material is retained on the No. 200 sieve.	**Gravels** More than 50% coarse fraction retained on the No. 4 sieve	**Clean Gravels** < 5% Fines	**GW**	Well-graded gravels, gravel-sand mixtures, little or no fines	**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	$C_U = D_{60} \div D_{10} \geq 4$ 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	

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 Mead & Hunt, Inc.
PROJECT NUMBER 190841
PROJECT NAME Fairfield Justice Campus Asset Protection Phase III
PROJECT LOCATION Fairfield, CA

LITHOLOGIC SYMBOLS (Unified Soil Classification System)

ASPHALT: Asphalt

CH: USCS High Plasticity Clay

CL: USCS Low Plasticity Clay

CONCRETE: Concrete

GC: USCS Clayey Gravel

GW-GC: USCS Well-graded Gravel with Clay

MH: USCS Elastic Silt

ML: USCS Silt

SP: USCS Poorly-graded Sand

SW: USCS Well-graded Sand

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

Appendix B. Laboratory Testing

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SUMMARY OF LABORATORY RESULTS

CLIENT Mead & Hunt, Inc.
PROJECT NAME Fairfield Justice Campus Asset Protection Phase III
PROJECT NUMBER 190841
PROJECT LOCATION Fairfield, 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	6.0	6/18/2021	37	22	15				25.1	106.4		
B-01	11.5	6/18/2021							18.2	114.5		
B-01	15.0	6/18/2021				0.106	75					
B-02	2.0	6/18/2021							15.4			
B-02	15.0	6/18/2021				0.106	55					
B-03	11.0	6/18/2021							18.8	110.2		
B-03	15.0	6/18/2021	26	19	7							
B-03	20.0	6/18/2021				25	11					
B-04	2.0	6/18/2021							27.0	95.8		
B-04	2.5	6/18/2021				0.106	82					
B-04	11.5	6/18/2021							18.5	112.6		
B-04	16.0	6/18/2021							18.7	112.2		
B-04	20.0	6/18/2021							22.4	107.5		
B-05	1.5	6/18/2021							9.1			
B-05	6.0	6/18/2021							20.9	98.8		
B-05	16.0	6/18/2021							20.0	115.7		
B-05	20.0	6/18/2021	44	20	24							

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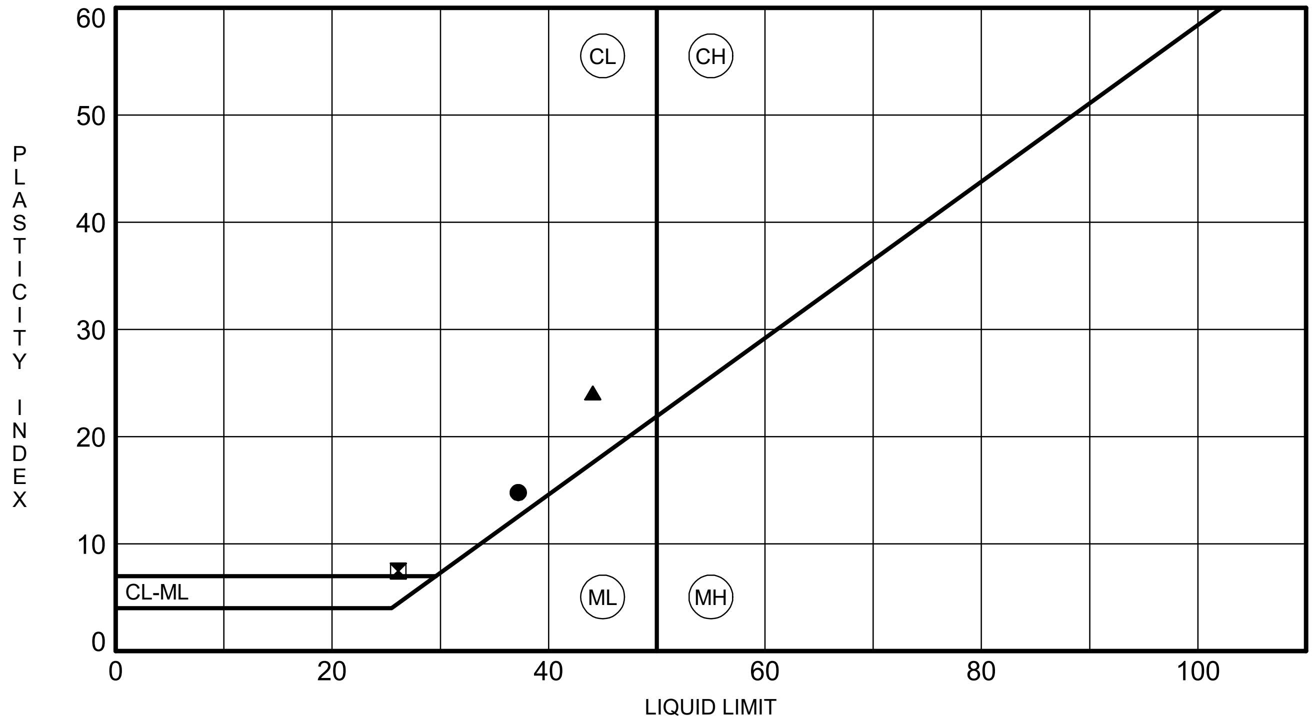
ATTERBERG LIMITS RESULTS

CLIENT Mead & Hunt, Inc.
PROJECT NAME Fairfield Justice Campus Asset Protection Phase III
PROJECT NUMBER 190841
PROJECT LOCATION Fairfield, CA



	BOREHOLE	DEPTH	LL	PL	PI	Fines	Classification	Date Tested
●	**B-01**	**6.0**	**37**	**22**	**15**			**6/18/2021**
⊠	**B-03**	**15.0**	**26**	**19**	**7**			**6/18/2021**
▲	**B-05**	**20.0**	**44**	**20**	**24**			**6/18/2021**

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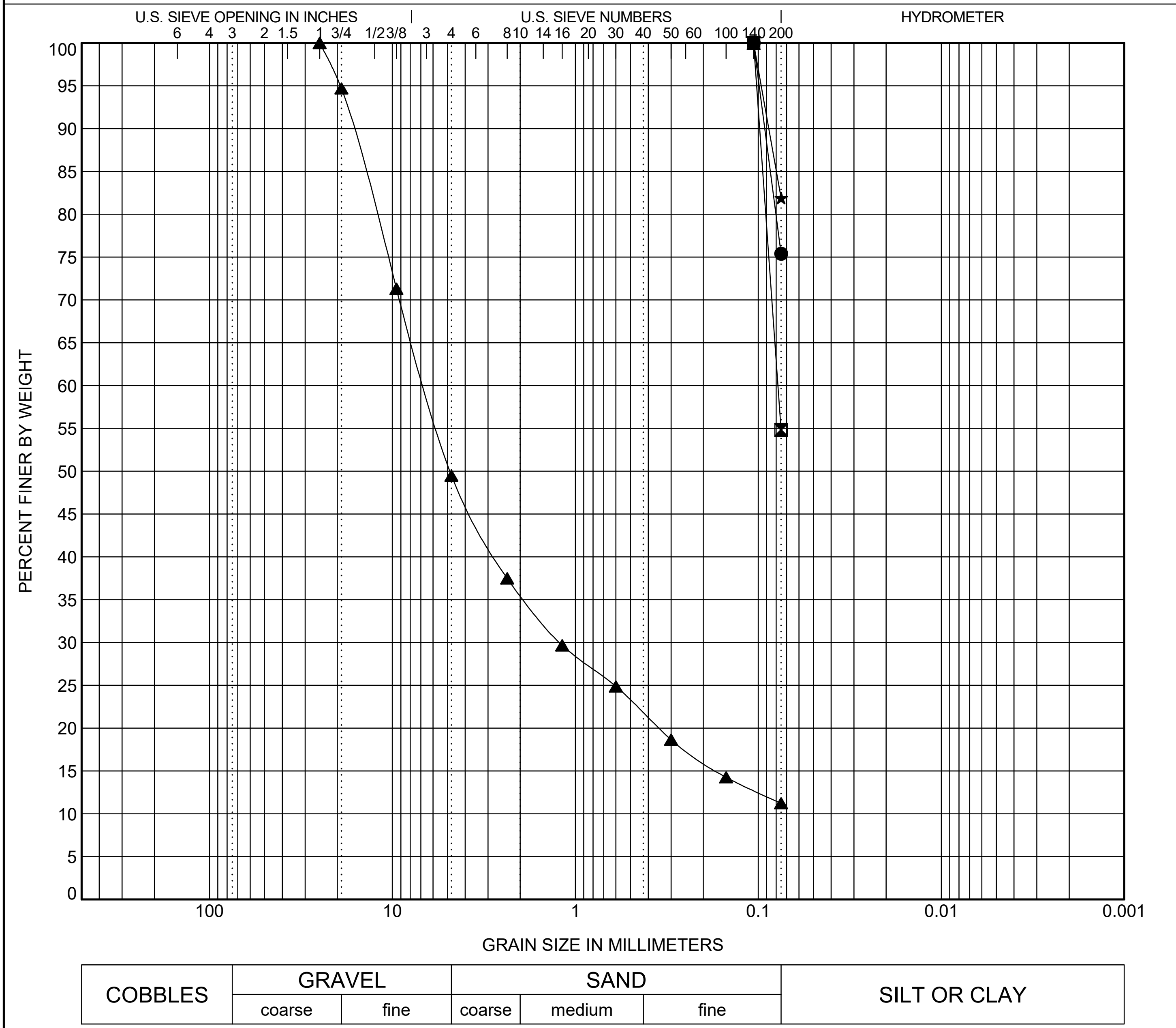
GRAIN SIZE DISTRIBUTION

CLIENT Mead & Hunt, Inc.

PROJECT NAME Fairfield Justice Campus Asset Protection Phase III

PROJECT NUMBER 190841

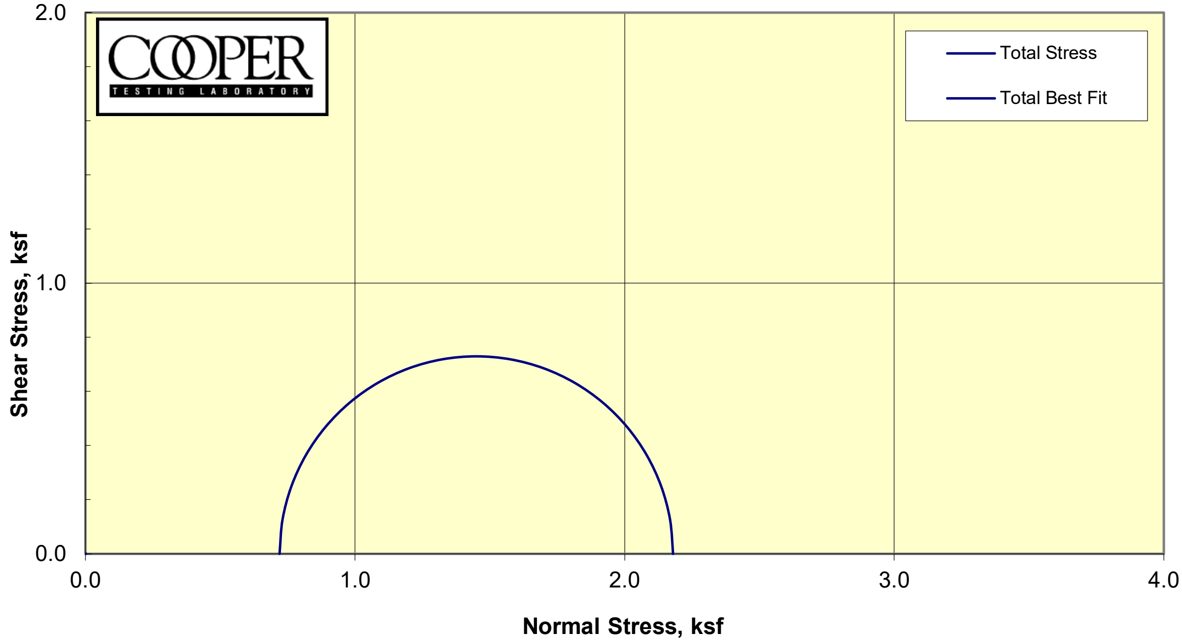
PROJECT LOCATION Fairfield, CA

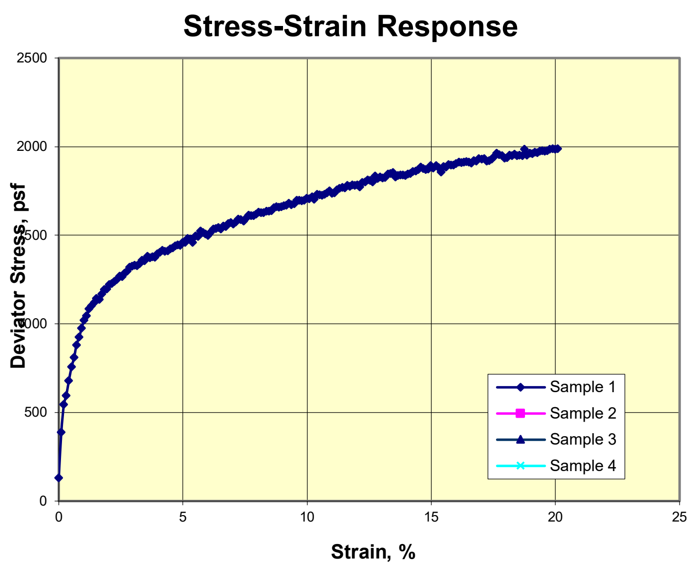


	BOREHOLE	DEPTH	DATE TESTED	Classification	LL	PL	PI	Cc	Cu
●	B-01	15.0	6/18/2021						
⧗	B-02	15.0	6/18/2021						
▲	B-03	20.0	6/18/2021					3.93	117.07
★	B-04	2.5	6/18/2021						

	BOREHOLE	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-01	15.0	0.106				0.0	24.6	75.4	
⧗	B-02	15.0	0.106	0.078			0.0	45.2	54.8	
▲	B-03	20.0	25	6.636	1.216	0.057	50.5	38.3	11.2	
★	B-04	2.5	0.106				0.0	18.1	81.9	

Triaxial Unconsolidated-Undrained
(ASTM D2850m)





Sample:	1	2	3	4
MC, %	26.3			
Dry Dens, pcf	98.4			
Sat. %	97.0			
Void Ratio	0.744			
Diameter in	2.38			
Height, in	4.98			
	Final			
MC, %	28.2			
Dry Dens, pcf	96.6			
Sat. %	100.0			
Void Ratio	0.776			
Diameter, in	2.40			
Height, in	4.98			
Cell, psi	53.5			
BP, psi	48.5			
	Effective Stresses At:			
Strain, %	5.0			
Deviator ksf	1.460			
Excess PP	0.000			
Sigma 1	2.180			
Sigma 3	0.720			
P, ksf	1.450			
Q, ksf	0.730			
Stress Ratio	3.027			
Rate in/min	0.0491			
Total C	N/A	ksf		
Total Phi	N/A	Degrees		
Eff. C	N/A	ksf		
Eff. Phi	N/A	Degrees		

Job No.: 471-346 **Date:** 6/23/2021

Client: Cal Engineering & Geology BY:MD/DC

Project: 190841

Sample 1) B-02_2-4 @ 6' Olive Brown CLAY

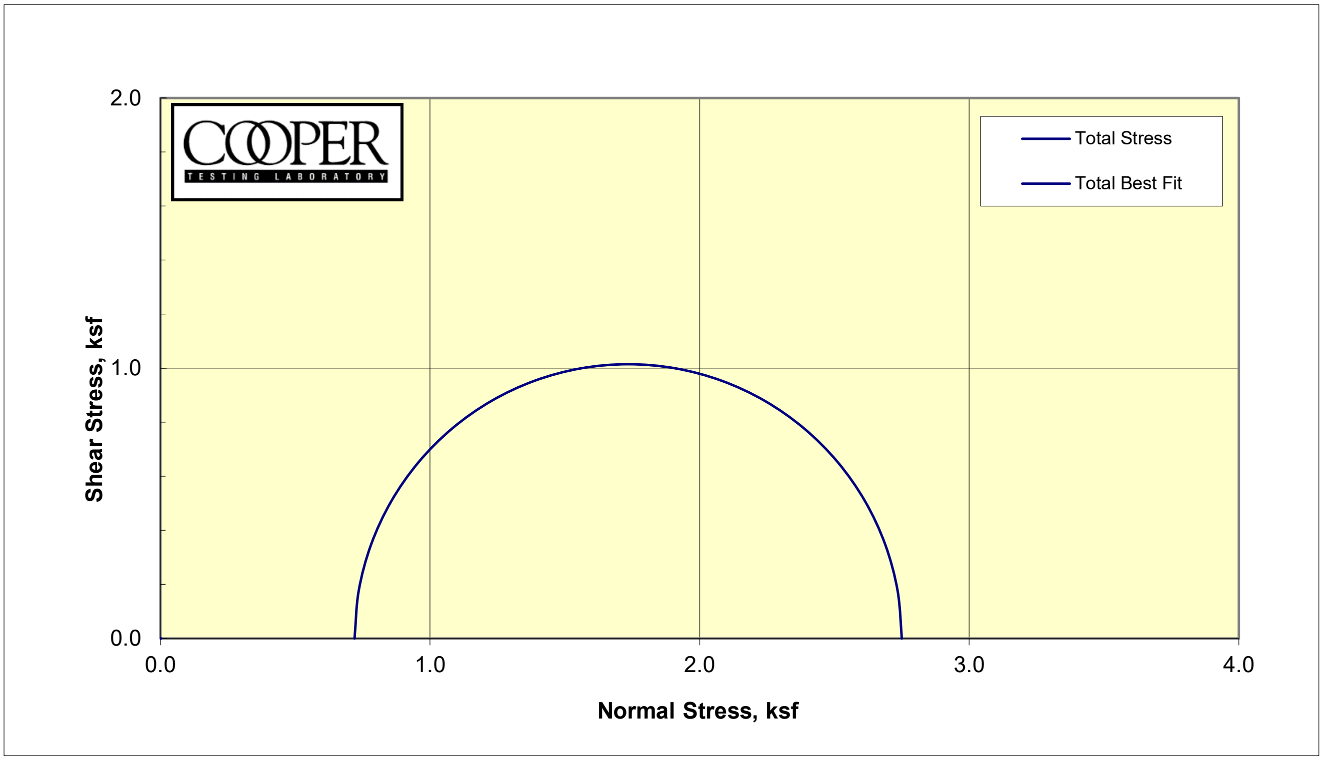
Sample 2)

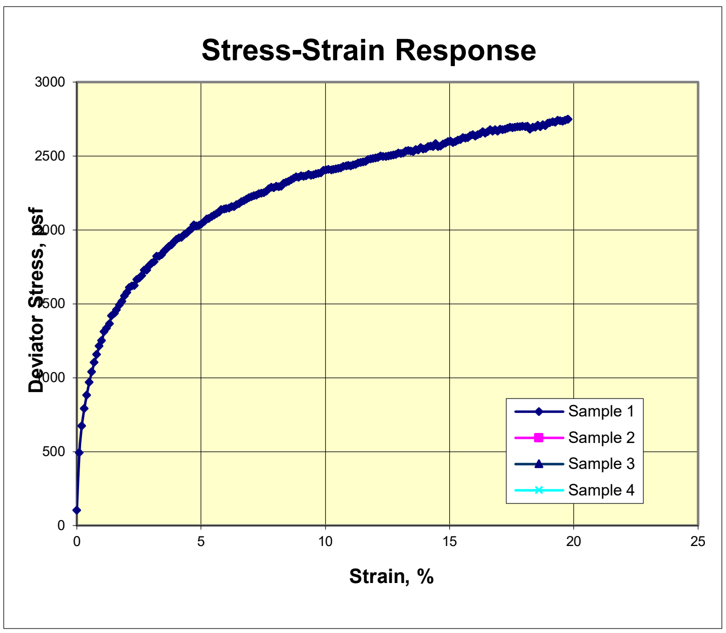
Sample 3)

Sample 4)

REMARKS: Strengths picked at 5% strain.
*Sample was back-pressure saturated prior to shear.

Triaxial Unconsolidated-Undrained
(ASTM D2850m)



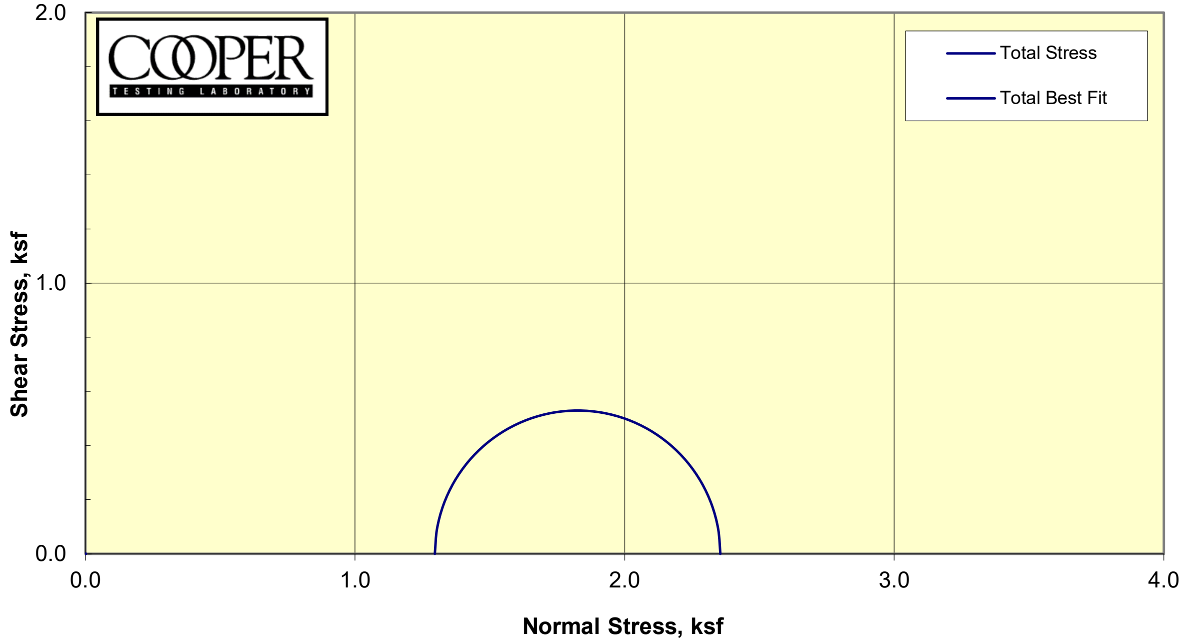


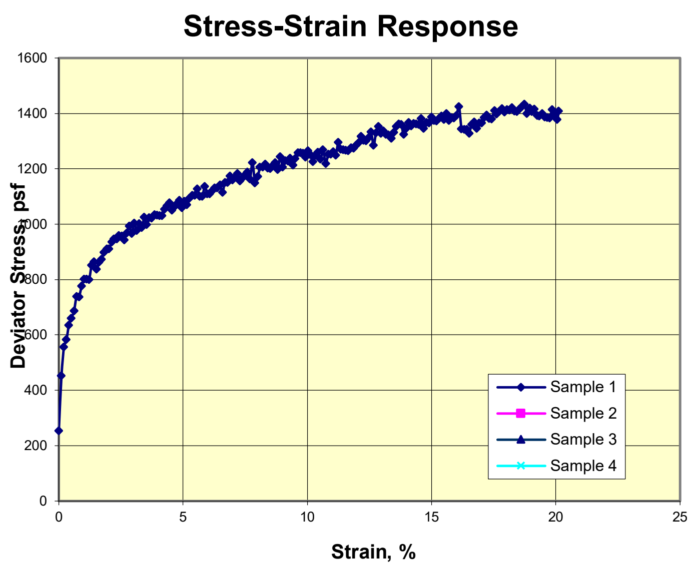
Sample:	1	2	3	4
MC, %	25.9			
Dry Dens, pcf	98.5			
Sat. %	93.8			
Void Ratio	0.774			
Diameter in	2.38			
Height, in	5.05			
	Final			
MC, %	28.1			
Dry Dens, pcf	97.8			
Sat. %	100.0			
Void Ratio	0.787			
Diameter, in	2.39			
Height, in	5.06			
Cell, psi	63.5			
BP, psi	58.5			
	Effective Stresses At:			
Strain, %	5.0			
Deviator ksf	2.030			
Excess PP	0.000			
Sigma 1	2.750			
Sigma 3	0.720			
P, ksf	1.735			
Q, ksf	1.015			
Stress Ratio	3.819			
Rate in/min	0.0489			
Total C	N/A	ksf		
Total Phi	N/A	Degrees		
Eff. C	N/A	ksf		
Eff. Phi	N/A	Degrees		

Job No.:	471-346	**Date:**	6/23/2021
Client:	Cal Engineering & Geology		BY:MD/DC
Project:	190841		
Sample 1)	B-03_3-2 @ 6'	Greenish Gray CLAY	
Sample 2)			
Sample 3)			
Sample 4)			

REMARKS: Strengths picked at 5% strain.
*Sample was back-pressure saturated prior to shear.

Triaxial Unconsolidated-Undrained
(ASTM D2850m)





Sample:	1	2	3	4
MC, %	19.8			
Dry Dens, pcf	105.4			
Sat. %	89.4			
Void Ratio	0.598			
Diameter in	2.40			
Height, in	5.00			
	Final			
MC, %	21.0			
Dry Dens, pcf	107.5			
Sat. %	100.0			
Void Ratio	0.567			
Diameter, in	2.38			
Height, in	4.97			
Cell, psi	67.5			
BP, psi	58.5			
	Effective Stresses At:			
Strain, %	5.0			
Deviator ksf	1.059			
Excess PP	0.000			
Sigma 1	2.355			
Sigma 3	1.296			
P, ksf	1.825			
Q, ksf	0.529			
Stress Ratio	1.817			
Rate in/min	0.0504			
Total C	N/A	ksf		
Total Phi	N/A	Degrees		
Eff. C	N/A	ksf		
Eff. Phi	N/A	Degrees		

Job No.:	471-346	**Date:**	6/23/2021
Client:	Cal Engineering & Geology		BY:MD/DC
Project:	190841		
Sample 1)	B-05_5-8 @ 11'	Olive Brown Mottled Dark Gray Sandy CLAY	
Sample 2)			
Sample 3)			
Sample 4)			

REMARKS: Strengths picked at 5% strain.
*Sample was back-pressure saturated prior to shear.

COOPER TESTING LABORATORY

Corrosivity Test Summary

CTL # 471-352 **Date:** 7/21/2021 **Tested By:** PJ **Checked:** PJ
Client: CEG **Project:** Fairfield Justice **Proj. No:** 190841
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-03	3-3	6.5	-	1367	-	9	64	0.0064	9.0	-	4.4	Olive Brown CLAY w/ Sand & Gravel
B-04	4-4	5.5	-	2844	-	10	36	0.0036	9.1	-	3.7	Yellowish Brown Clayey SAND w/ Gravel
B-05	5-7	11	-	1266	-	27	95	0.0095	8.8	-	4.5	Olive Gray Sandy CLAY w/ Gravel

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