
Appendix D

Geotechnical Evaluation

Geotechnical Design Report

JOHN SIMMS PARK STORMWATER CAPTURE AND STORAGE FACILITIES 16614 South Clark Avenue Bellflower, California



Prepared for:

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June 20, 2022
Project No. CWT 22-05E



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Mr. Oliver Galang
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Subject: **GEOTECHNICAL DESIGN REPORT
JOHN SIMMS PARK
STORMWATER CAPTURE AND STORAGE FACILITIES
16614 South Clark Avenue
Bellflower, California**

Dear Mr. Galang:

Presented herein is Tetra Tech's geotechnical design report for the proposed stormwater capture and storage facilities as well as an evaluation of the suitability of the near-surface soils for infiltration using permeable pavements within the existing parking lot stalls at the John Simms Park located at 16614 South Clark Avenue in the City of Bellflower, California. This report summarizes the results of our geotechnical investigation to characterize the soils at the site and provides geotechnical recommendations for the design and construction of the proposed facilities. It is noted that in our previous Preliminary Geotechnical Assessment Report dated May 19, 2020 it was assessed that the onsite soils at the depths where infiltration testing had been proposed were not suitable for infiltration. Thus, the design concept for the facilities was reassessed to include only the capture and storage components of the original project concept while expanding the infiltration assessment to include the surficial soils within the parking lot areas. The appendices of the report include logs of borings, results of laboratory testing and field infiltration tests, a summary of the seismic demand and liquefaction analyses.

We appreciate the opportunity to provide our professional services on this project. If you have any questions regarding this report or if we can be of further service, please do not hesitate to contact the undersigned.

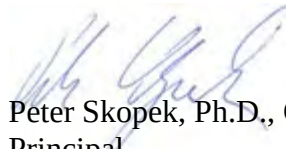
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1. INTRODUCTION AND PROJECT DESCRIPTION

The development of the John Simms Park Stormwater Capture Project represents a major step to achieve pollutant load reductions for the Los Cerritos Channel Watershed Management Program. Los Cerritos Channel Watershed is a largely built-out, urbanized watershed of approximately 17,711 acres in the Los Cerritos and Alamos Bay Watershed Management Area. The drainage treatment area for the project is about 758 acres.

This report presents the results of Tetra Tech BAS' (Tetra Tech) geotechnical engineering evaluation for the proposed stormwater capture and storage facilities as well as an evaluation of the suitability of the surficial soils for infiltration using permeable pavements within the existing parking lot stalls at the John Simms Park (see Figure 1 – Project Location Map) located at 16614 South Clark Avenue, in the City of Bellflower, California. It is noted that in our previous report Preliminary Geotechnical Assessment Report dated May 19, 2020 it was assessed that the onsite soils at depths where infiltration had been proposed were not suitable for infiltration. Thus, the design concept for the facilities was reassessed to include only the capture and storage components of the original project concept while adjusting the stormwater infiltration component to include the infiltration into surficial soils within the parking lot stall areas. The appendices to this report include logs of borings, results of laboratory and field infiltration tests for the preliminary as well as this current investigation effort, a summary of seismic demand and liquefaction analyses.

The project is intended to intercept a storm runoff flow of about 60 cubic feet per second (cfs) from a storm drain BI 0009 Unit 3 Line A located below Clark Avenue to the east of the park. The storm drain is a 90-inch reinforced concrete pipe (RCP) managed by the Los Angeles County Flood Control District (LACFCD). The diverted stormwater will be captured and pre-treated through a hydrodynamic separator and then conveyed to a best management practice (BMP) underground storage facility with a 4 million gallon, i.e., 12.2 acre-feet, capacity. The invert of the underground stormwater storage facility will be at a depth of about 20 feet and the facility itself will be about 144 feet wide (north-south direction) by 370 feet long (west-east direction). The stored pre-treated stormwater will be further treated through a 7.8 cfs filtration unit before being released into an existing 24-inch storm drain located within the park area.

It is understood that the main hydraulic components of the project include (depths measured from current grades):

- stormwater storage units with a bottom depth of 20 to 24 feet,
- storm drain diversion with an invert depth of about 16 to 18 feet,
- actuated valve with an invert depth of about 16 to 18 feet,
- pre-treatment unit with an invert depth of about 22 to 24 feet,
- post-treatment or filtration unit with an invert depth of about 10 feet,
- conveyance pipelines at varying depths,
- and pump station with an invert depth of about 20 to 24 feet.

It is also understood that the project involves constructing a building addition about 20 feet by 20 feet to an existing building which will be used as a stormwater harvesting facility. There is no

information regarding the foundation type for the existing at-grade building, but it is assumed that it is supported on shallow footings. If this assumption is not correct, this office should be contacted to review our recommendations.

The project also proposes to reduce the impervious surface area within the park by converting the existing parking lot stalls into a permeable pavement surface, adding bioretention areas between parking stalls and adding native landscaping areas.

2. SCOPE OF WORK

Tetra Tech's scope of services for this project consisted of the following tasks:

- Review readily available background data.
- Perform a reconnaissance site visit to observe ground conditions and mark boring locations.
- Obtain drilling permits from the Los Angeles County Department of Public Health (LACDPH).
- Coordinate with LACDPH engineering staff, park personnel, and Underground Service Alert (USA) for clearance of buried on-site utilities prior to drilling.
- Conduct a subsurface investigation within the general area where the proposed capture and storage facilities will be located, including excavating, logging, and geotechnical sampling of 5 soil exploratory borings to a maximum depth of 51.5 feet.
- Install 3 percolation borings to depths of approximately 3.5 feet within the parking lot area and 2 percolation borings to a depth of 22.5 feet within the field area, and conduct percolation testing in general accordance with the boring percolation test procedure described in the Los Angeles County Department of Public Works (LACDPW) GS200.1 Guidelines (2021).
- Perform laboratory testing of selected soil samples recovered from the borings to evaluate geotechnical engineering properties of the on-site soils.
- Conduct an evaluation of the collected data to develop geotechnical recommendations for the design and construction of the proposed structures including the following items:
 - ♦ An evaluation of subsurface conditions and description of types, distribution, and engineering characteristics of subsurface materials.
 - ♦ An evaluation of the liquefaction potential and dynamic settlement of the on-site granular materials.
 - ♦ An evaluation of the suitability of on-site soils for the support of structures.
 - ♦ An evaluation of the suitability of surficial soils in the parking lot area for infiltration utilizing permeable pavers.
 - ♦ Recommendations for design of foundation systems for proposed structures including allowable bearing capacity, lateral resistance, and settlement estimates.
 - ♦ Determination of seismic design parameters in accordance with the 2019 California Building Code (CBC).
 - ♦ Evaluation of lateral earth pressure for the design of the underground storage facility and for the design of temporary construction shoring.
 - ♦ An evaluation of the corrosion potential of on-site soils to buried concrete.
- Prepare this written report documenting the work performed, physical data acquired, and provide geotechnical recommendations.

3. SUBSURFACE EXPLORATION

The preliminary subsurface investigation was carried out on April 16, 2020 by Tetra Tech to provide preliminary recommendations for stormwater infiltration within the field area at the park and included the drilling, logging, and sampling of 2 hollow stem auger borings B-1 and B-2, and 2 associated percolation test borings denoted P-1 and P-2 drilled within 10 feet of the respective exploratory boreholes.

The hollow stem auger exploratory and percolation borings were excavated using a CME-75 truck-mounted drill rig equipped with an 8-inch diameter auger at the approximate locations indicated on Figure 2 – Site Layout and Boring Location Map. Borings B-1 and B-2 were advanced to a maximum depth of 51.5 feet and percolation boreholes P-1 and P-2 were advanced to a depth of approximately 22 and 20 feet, respectively. The approximate latitude and longitude of the soil explorations, the approximate elevations, and depths of the explorations are included in Table 1. The soil boring logs are presented in Appendix A1.

The supplementary subsurface investigation was carried out on May 3, 2022 by Tetra Tech and included the drilling, logging, and sampling of 3 hollow stem auger exploratory borings, B-3 through B-5, and 3 associated percolation test borings denoted P-3 through P-5.

The exploratory borings were excavated using a CME-75 truck-mounted drill rig equipped with 8-inch diameter augers at the approximate locations indicated on Figure 2. Boring B-3 was advanced to a depth of 51.5 feet and borings B-4 and B-5 were advanced to a depth of approximately 31.5 feet. Percolation boreholes P-3 through P-5 were advanced to a depth of approximately 3.5 feet. The approximate latitude and longitude of the current soil explorations, the approximate elevations, and depths of the current explorations are included in Table 1.

Prior to starting the field exploration program, a field reconnaissance was conducted to observe surface conditions and to mark the locations of the planned exploratory and percolation boreholes in agreement with park staff. A drilling permit was obtained from the Los Angeles County Department of Public Health (LACDPH) for all the subsurface explorations. Underground Service Alert, the LACDPH inspector, and park staff were also notified of the subsurface exploration schedule at least 48 hours prior to drilling.

Bulk, driven ring-type, and small bag samples were retrieved at selected depths during drilling of the exploratory borings. Standard Penetration Testing (SPT) was performed using an SPT sampler driven by an automatic 140-pound hammer with a drop of 30 inches in general accordance with ASTM D1586. The hammer calibration record indicated an energy transfer ratio of 80.4 percent. Ring-type samples were collected utilizing a California-type sampler driven by the same equipment used for the SPTs. Sampling was carried out at 2.5- and 5-foot intervals at all the borings.

Table 1
Boring Information

Boring Number	General Location	Approximate Latitude	Approximate Longitude	Approximate Ground Elevation (feet)*	Approximate Depth (feet)
Preliminary Investigation					
B-1/P-1	East end of open field area	33.883505	-118.130304	64	51.5 / 22
B-2/P-2	West end of open field area	33.883503	-118.131257	64	51.5 / 20
Supplementary Investigation					
B-3	Center of the field area	33.883502	-118.130882	64	51.5
B-4	East side of parking area	33.884096	-118.132389	67	31.5
B-5	West side of parking area	33.884034	-118.133736	66	31.5
P-3	Northeast portion of parking area	33.884476	-118.132289	67	3.5
P-4	Central portion of parking area	33.884128	-118.133090	66	3.5
P-5	Southwest portion of parking area	33.883631	-118.133359	65	3.5
*Estimated from Google Earth Cells highlighted in light blue correspond percolation borings from the supplementary investigation					

The soil borings were surface-logged by a California Professional Geologist in general accordance with the visual-manual procedure for description and identification of soils, ASTM D2488. The Geologist prepared the recovered samples for subsequent reference and laboratory testing. The soil boring logs are presented in Appendix A2.

At the completion of drilling, the borings were backfilled with a bentonite cement grout in accordance with LACDPH requirements. The excavated soils from the boreholes were disposed at locations specified by park personnel. The percolation boreholes were backfilled with tamped excavated soil cuttings and the asphalt surface was restored with an asphalt cold patch.

4. LABORATORY TESTING

The investigation included laboratory tests on selected samples recovered from the soil borings to aid in the classification of soils and to evaluate their pertinent engineering properties. The following tests were performed during the course of the preliminary investigation:

- Moisture Content of Soil, ASTM D2216;
- Density of Soil Specimens, ASTM D7263;
- Particle Size Analysis of Soils, ASTM D6913 and ASTM D7928;
- Atterberg Limits, ASTM D4318;
- Expansion Index, ASTM D4829;

The following tests were performed during the course of the supplementary investigation:

- Moisture Content of Soil, ASTM D2216;
- Density of Soil Specimens, ASTM D7263;
- Particle Size Analysis of Soils, ASTM D6913 and ASTM D7928;
- Atterberg Limits, ASTM D4318;
- Compaction Testing, ASTM D1557;
- Direct Shear Testing, ASTM D3080;
- Consolidation Testing, ASTM D2435;
- R-value, ASTM D2844;
- Corrosion Testing: Minimum electrical resistivity and pH, CTM 643;
Sulphate Content, CTM 417;
Chloride Content, CTM 422.

Results of all laboratory tests are presented in Appendices B1 and B2 for the preliminary and supplementary investigation, respectively. For ease of referral to the soil profile, most of the laboratory results are also included on the boring logs in respective Appendices A1 and A2.

5. FIELD PERCOLATION TESTING

During the preliminary investigation Tetra Tech performed 2 small diameter borehole percolation tests at depths of about 20 feet denoted P-1 and P-2 located within 15 feet of the geotechnical borings B-1 and B-2, respectively, using the test procedure described in the LACDPW infiltration guidelines GS200.2 (2019). P-1 and P-2, were installed to a depth of about 22 and 20 feet, respectively. Both percolation boreholes penetrated into a native stratum of silty sands with 30 to 40 percent of fines. The results of the percolation testing are summarized in Table 2 and presented in Appendix C1.

Table 2
Design Percolation Rates

Boring Percolation Test No.	Borehole Depth (feet)	Design Infiltration Rate (inches/hour)
Preliminary Percolation Testing		
P-1	22	0.05
P-2	20	0.04
Supplementary Percolation Testing		
P-3	3.5	0.006
P-4	3.5	0.012
P-5	3.5	0.043

During the supplementary investigation Tetra Tech performed 3 small diameter borehole percolation tests denoted P-3 through P-5 located within the parking lot area, using the test procedure described in the LACDPW infiltration guidelines GS200.1(2021). The percolation tests were performed at a depth of about 3.5 feet.

The approximate coordinates of the percolation tests, the approximate top of borehole elevations, and borehole depths are included in Table 1. The infiltration test borings were constructed by placing 2 to 6 inches of ¾-inch gravel at the bottom of each boring and then installing a 3-inch-diameter perforated pipe (casing) with 5/8-inch-diameter holes. The casings extended from the top of the gravel placed at the bottom of each boring to the ground surface and were wrapped in a protective cloth filter sock to limit the migration of soil particles into the pipe. The annular space of each boring was filled with ¾-inch gravel (filter pack) up to a few feet above the intended zone of infiltration testing. The ¾-inch gravel was selected as the filter pack material since it has a hydraulic conductivity significantly larger than that of the surrounding soil.

Percolation boreholes were presoaked for at least 1 hour before the test. For the percolation testing an initial constant water level of 1 through 3 feet was maintained above the bottom of the borehole. The readings to determine the water depth were taken with a well sounder every 30 minutes or more until a stabilized drop rate was obtained (per GS 200.1 a stabilized rate is obtained when the

highest and lowest readings are within 10 percent from each other for 3 consecutive readings), however testing was not completed until a 4-hour minimum testing period was completed.

The raw field percolation rate was converted, as indicated in the infiltration test logs in Appendix C2, to a corrected field infiltration rate considering the effect of porosity of the gravel pack estimated to be approximately 0.35. The corrected field infiltration rate expressed in inches per hour was then further adjusted as explained below and on the percolation logs to obtain the design infiltration rates. A testing method reduction factor RF_t of 2 was applied as required by the guidelines to account for the direction of flow during the test and the reliability of the method. To account for effects related to the limited number of tests and the large area considered for the project, a reduction factor RF_v of 1.5 was used (typical range between 1 and 3). Lastly, to account for long-term siltation, and plugging, a reduction factor RF_s of 3 was considered (typical range between 1 and 3). The results of the borehole percolation testing and calculation of the adjusted design infiltration rates are summarized in Table 2.

The percolation results from this exploration indicate that the design percolation rate within the parking lot area ranges between 0.006 and 0.01 inches/hour. The design infiltration rates are much lower than the acceptable minimum of 0.3 inches/hour required by the LACDPW Guidelines (2021).

6. SUBSURFACE CONDITIONS

6.1. Regional Geology

The project site is located in the central portion of the greater Los Angeles. The Los Angeles Basin is located within Peninsular Ranges geomorphic province which is characterized by generally northwest trending elevated ground, hills and mountain ranges with intervening valleys. Topographic relief across the basin includes a low-lying plain that rises gently from the Pacific Ocean inland to the surrounding Santa Monica and San Gabriel Mountains to the north, Puente Hills to the northeast, the Santa Ana Mountains to the southeast, and the San Joaquin hills and Palos Verdes Peninsula to the south. Structurally, the Peninsular Ranges province includes northwest-southeast trending structural blocks separated by northwest-southeast trending strike-slip faults. The coastal portion of the basin is filled with several thousand feet of recent to later Tertiary age sediments.

6.2. Site Geology

Regional geologic mapping published by Roffers and Bedrossian (2010) (Figure 3 – Geologic Map), shows the site to be underlain by young alluvial fan deposits (Qyf) of Holocene to late Pleistocene age consisting of gravel, sand, and silt. Generalized descriptions of subsurface materials reported in this investigation includes native alluvial soils. Minor amounts of artificial fill soil were encountered beneath asphalt concrete pavement in the exploratory borings conducted during this study. Reported subsurface conditions are described in the following sections. Copies of the boring logs are included in Appendixes A1 and A2 and the locations are shown on Figure 2.

6.3. Artificial Fill

The parking lot area of the project is underlain by 1 to 2 feet of artificial fill material, generally consisting of fine silty sand to sandy silt, medium dense to medium stiff, moist, and containing varying amounts of fine gravel. The material is similar to the underlying native alluvium, suggesting that the existing native soil was amended for use as subgrade for the parking lot.

6.4. Native Alluvium (Qyf)

Native young alluvial soils were encountered at the surface in the field area, and below artificial fill in the parking lot area, to the maximum explored depth of 51.5 feet and included both fine- and coarse-grained soils. The fine-grained materials consisted of light olive gray to dark gray, and brown sandy silts and a few interspersed layers of lean clays. Most of the encountered materials were coarse-grained and consisted of light olive gray to greenish gray, brown to olive brown silty sands to poorly-graded sands.

Uncorrected SPT blowcounts in the native alluvium for the fine-grained (clay, silt) soils ranged from 6 to 19 indicating stiff to very stiff consistencies. SPT blowcounts in the native alluvial coarse-grained (sandy) soils ranged from 9 to 39 indicating loose to dense materials.

6.5. Groundwater

According to the State of California Seismic Hazard Zone Report for the South Gate 7.5-minute Quadrangle (CDMG, 1998), the historic high groundwater level near the site has been mapped at a depth of about 8 feet (Figure 4 – Historic High Groundwater Map). It is noted that the CDMG groundwater contours are based on early last century water well logs (Mendenhall, 1905, Conkling, 1927) and also include water measurements from wells from the Central Basin Investigation (State Water Resources Board, 1952), Department of Water Resources (circa 1940's) which reflect conditions previous to massive infrastructure and urban development that has taken place in the last 50 years and therefore they do not likely correspond or apply to current conditions and circumstances.

Groundwater was encountered in boring B-3 during the supplementary field exploration at a depth of 34.5 feet, i.e., elevation 29.5 feet, and in borings B-1 and B-2 at depths 35 and 36.7 feet, i.e., elevation 29 and 27.3 feet, respectively during the supplementary exploration.

LACDPW groundwater well data obtained from their website for wells within a 2-mile radius of the site (<https://dpw.lacounty.gov/general/wells/>) between 1943 and 2018 indicated groundwater elevations ranging from - 120 to 20 feet over the last 70 years (see Diagram 1). Based on the assessment of the local stratigraphy and local topography, as well as the LACDPW well data, it is our conclusion that the high groundwater at the site during the last 60 years has been deeper than 29 feet i.e., elevation 35 feet.

Fluctuations of the groundwater level, localized zones of perched water, and increased soil moisture content should be anticipated during and following the rainy season. Irrigation of landscaped areas on or adjacent to the site can also cause a fluctuation of local groundwater levels. Evaluation of such factors is beyond the scope of our services.

Based on the research and observed conditions, groundwater is not expected to impact the construction of the proposed development, however the historic high groundwater depth of about 8 feet from the existing grade should be considered for the structural design. Therefore, any proposed structures embedded or buried deeper than 8 feet from the existing grade should consider hydrostatic lateral and hydraulic uplift forces. During construction the Contractor should expect groundwater depth of about 35 feet from existing grade.

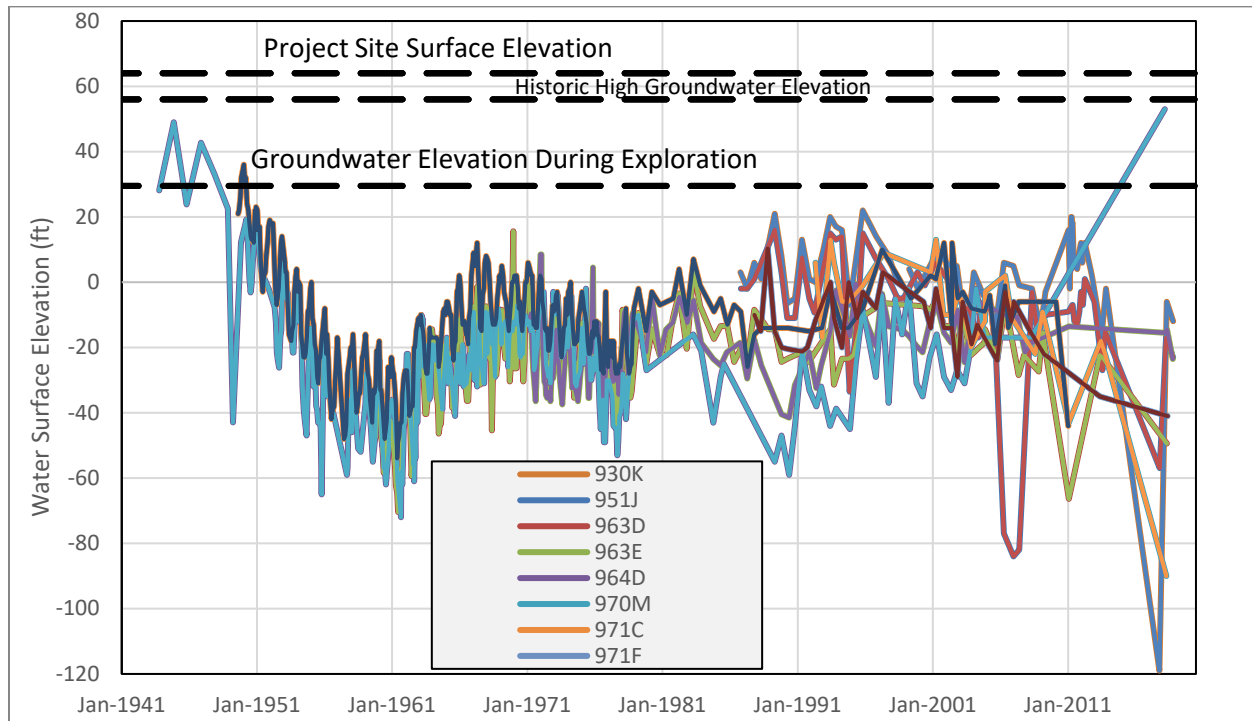


Diagram 1. Groundwater Elevations for Nearby LACDPW Groundwater Wells.

7. ENGINEERING SEISMOLOGY AND GEOLOGIC HAZARDS

7.1. General Seismic Setting

The Southern California region is known to be seismically active. Earthquakes occurring within approximately 60 miles of the site are considered capable of generating ground shaking of engineering significance to the proposed construction. The project is located in the general proximity of several active and potentially active faults, as shown on Figure 5 – Regional Faults and Seismicity Map. Active faults are defined as those that demonstrated evidence of surface displacement within the Holocene period (approximately the last 11,000 years).

Active faults within approximately 10 miles of the subject site include the Puente Hills Blind Thrust fault located 1.4 miles north of the site, the Los Alamitos fault located approximately 2.8 miles south of the site, the Newport-Inglewood fault located approximately 5.3 miles southwest of the site, the Whittier fault located approximately 9.2 miles northeast of the site, and the Compton Blind Thrust fault located about 10 miles southwest of the site. The San Andreas Fault is located about 41.6 miles to the northeast of the site.

Table 3 lists selected known active faults that may affect the subject site and their associated potential maximum moment magnitude (M_{max}) as published by Cao et al. (2003) for the California Geological Survey (CGS). The approximate distances to the site were calculated from Jennings (2010).

Superimposed on the area map in Figure 5 are earthquake epicenters of magnitude M_w 5.0 or more as recorded by the USGS between 1900 to present day. Significant seismic activity for the period has been recorded surrounding the project site. However, relatively few earthquake epicenters have been recorded in the immediate area of the subject site. Notable historic earthquakes in Southern California of significance to the project are listed in Table 4. The most significant historic earthquake near the project site was the 1933 Long Beach earthquake.

Table 3
Nearby Known Active Faults

Referenced Site Latitude and Longitude: 33.883493°, -118.131557°

Fault Name	Approximate Fault Distance to Site¹ (miles)	Maximum Moment Magnitude² (Mmax)
Puente Hills Blind Thrust	1.4	7.1
Los Alamitos	2.8	6.2
Newport-Inglewood	5.3	7.1
Whittier	9.2	6.8
Compton Blind Thrust	10.0	7.3
THUMS-Huntington Beach	11.4	7
Palos Verdes	11.7	7.3
Charnock	14.3	6.5
Cabrillo	14.4	6.8
Redondo Canyon	16.2	6.5
Hollywood	16.2	6.4
Raymond	16.2	6.5
Verdugo	18.7	6.9
Sierra Madre	20.2	7.2
Santa Monica	20.6	6.6
Malibu Coast	31.9	6.7
Anacapa-Dume	37.5	7.5
San Andreas	41.6	7.8
Notes: ¹ per Jennings, 2010 ² per Cao, et al., 2003		

Table 4
Historic Earthquakes in Southern California
Referenced Site Latitude and Longitude: 33.883493°, -118.131557°

Earthquake Name	Year	Fault and Fault Type	Earthquake Magnitude*	Epicenter	
				Latitude	Longitude
La Habra	2014	Shallow previously unknown fault	5.1 M _w	33.933°N	117.916°W
Chino Hills	2008	Yorba Linda Trend	5.4 M _w	33.949°N	117.766°W
Northridge	1994	Northridge Thrust (Blind Thrust) (a.k.a. Pico Thrust)	6.7 M _w	34.21°N	118.54°W
Sierra Madre	1991	Clamshell-Sawpit Canyon Fault (Reverse)	5.8 M _L	34.20°N	118.14°W
Pasadena	1988	Raymond Fault (left lateral strike-slip)	5.0 M _w	34.14°N	118.13°W
Whittier Narrows	1987	Puente Hills Fault (Blind Thrust Fault)	5.9 M _L	34.06°N	118.08°W
San Fernando	1971	San Fernando Fault (thrust)	6.5-6.7 M _w	34.42°N	118.37°W
Torrance-Gardena	1941	Palos Verdes Fault (right-reverse)	4.8 M _L	33.82°N 33.78°N	118.22°W 118.25°W
Long Beach	1933	Newport-Inglewood Fault (right- lateral strike-slip)	6.4 M _w	33.63°N	118.00°W
San Jacinto	1923	San Jacinto Fault (right- lateral strike-slip)	6.3 M _L	34.00°N	117.24°W
San Jacinto	1918	San Jacinto Fault (right- lateral strike-slip)	6.7 M _w	33.65°N	117.43°W
Elsinore	1910	Elsinore Fault (right- lateral strike-slip)	6 M _L	33.75°N	117.45°W
Fort Tejon	1857	South Central Segment of the San Andreas Fault (right- lateral strike-slip)	7.9 M _w	35.43°N	120.19°W
*M _w refers to Moment Magnitude scale M _L refers to Local Magnitude scale					

Potential seismic sources of significance to the project include active faults previously described and faults that are not known to break the ground surface but are considered active. This latter group of faults includes “buried” or “blind” thrust faults. Current tectonic models for the Los Angeles Basin include buried thrust faults, several of which are considered partly responsible for the north-to-south compression of the basin. Although these faults are not currently zoned by the State of California for surface rupture hazards (Earthquake Fault Zones), many are considered capable of generating seismic shaking of significance to structures.

Of these buried active faults the closest to the site is the Puente Hills Trust Fault (PHTF). The PHTF is currently defined as 3 separate but juxtaposed, generally east-west trending and north-dipping, fault surfaces that combined extend from Downtown Los Angeles to Brea. From west to east these include the Los Angeles, Santa Fe Springs, and Coyote Hills segments. Based upon recent studies by several researchers, including: Shaw et al., (2002), Olsen and Cooke (2005), and Leon et al. (2007), the three fault surfaces are interpreted to extend from depths in excess of 9 miles on the north side of the Los Angeles basin to less than 1.2 miles at the southerly limits of the fault surfaces in the central portion of the basin. Fault surface geometries are interpreted from historical petroleum exploration data, geotechnical subsurface exploration data, and limited seismicity (e.g., the 1987 magnitude 5.9 Whittier Narrows earthquake).

Leon et al. (2007) estimates that upwards of 60 percent of the total Los Angeles basin compression may be attributed to strain along the PHTF. Although ground rupture has not been officially attributed to the fault, the presence of youthful hills (e.g., Coyote Hills) and shallow folding at depth in the upper portion of the interpreted thrust ramp suggests recent activity. The PHTF is considered capable of generating earthquake magnitudes up to about M_w 7.1.

7.2. Surface Fault Rupture

Official Maps of Earthquake Fault Zones were reviewed to evaluate the location of the project site relative to active fault zones. Earthquake Fault Zones (known as Special Studies Zones prior to 1994) have been established in accordance with the Alquist-Priolo Special Studies Zones Act enacted in 1972. The Act directs the State Geologist to delineate the regulatory zones that encompass surface traces of active faults that have a potential for future surface fault rupture. The purpose of the Alquist-Priolo Act is to regulate development near active faults in order to mitigate the hazard of surface fault rupture.

The site is not located within a designated Earthquake Fault Zone for fault surface rupture hazard. Based on a review of State of California Earthquake Fault Zone maps, the closest fault zone for surface rupture is the Newport Inglewood Fault Zone approximately 5.3 miles southwest of the site according to the CGS website application (<https://maps.conservation.ca.gov/cgs/EQZApp/app/>).

No surface traces of any active or potentially active faults are known to pass directly through or project towards the site. Neither our field exploration nor literature review disclosed an active fault trace projecting to the ground surface in the project area. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low.

7.3. Seismic Hazard Zones

Maps of seismic hazard zones are issued by the California Geological Survey (CGS, formerly California Department of Conservation, Division of Mines and Geology (CDMG)) in accordance with the Seismic Hazards Mapping Act enacted in April 1997. The intent of the Seismic Hazards Mapping Act is to provide for a statewide seismic hazard mapping and technical advisory program to assist cities and counties in developing compliance requirements to protect the public health and

safety from the effects of strong ground shaking, liquefaction, landslides, or other ground failure and other seismic hazards caused by earthquakes.

Based on the review of the South Gate Quadrangle Official Map of Seismic Hazard Zones issued March 25, 1998 (see Figure 6 – Seismic Hazard Zones Map), the proposed development is located within an area identified by the State of California as subject to the hazard of liquefaction. Because the site is located in a mapped area where the potential for liquefaction exists, liquefaction analyses were performed per the 2019 CBC to evaluate the site liquefaction potential.

7.4. Liquefaction Potential and Dynamic Settlement Assessment

The site, as previously discussed, is within an area identified by the State of California as subject to the hazard of liquefaction. Liquefaction of soils can be caused by ground shaking during earthquakes and is generally known to occur in saturated or nearly saturated cohesionless soils at depths shallower than about 50 feet. Research and historical data indicate that loose, relatively clean granular soils and low plasticity silts are susceptible to liquefaction whereas the stability of clayey silts, silty clays and clays are not typically adversely affected by ground shaking. Materials that are above the groundwater table are not considered susceptible to liquefaction.

Dynamic settlement can occur in both unsaturated and saturated sands when loose to medium-dense granular soils tend to undergo contractive volumetric changes during ground shaking. The following sections present our assessment of liquefaction potential and dynamic settlement at the site.

7.4.1. Groundwater Level for Liquefaction Analysis

Groundwater was encountered during the field exploration at depths ranging between 35 and 37 feet. However, the historic high groundwater at the site was mapped by CDMG (South Gate Quadrangle) at a depth of about 8 feet. Therefore, a groundwater depth of 8 feet was assumed for evaluation of liquefaction potential at the site.

7.4.2. Liquefaction Seismic Demand

Based on the Structural Engineers Association of California (SEAOC) and the Office of Statewide Health Planning and Development (OSHPD) website application (<https://www.seaoc.org/page/seismicdesignmaptool>), for a site with latitude and longitude 33.883493°, -118.131557°, respectively. The mapped Geometric Mean Peak Ground Acceleration (PGA_M) for a ground motion corresponding to the Maximum Considered Earthquake (MCE) for a Site Class D was determined to be 0.747g. From the USGS Seismic Hazard Interactive Deaggregation website (<https://earthquake.usgs.gov/hazards/interactive/>) this ground motion corresponds approximately to a predominant earthquake with a magnitude of M_w 7.3. The largest contributor to the seismic hazard at the site (about 18 percent) is the Compton Blind Thrust fault located about 6.5 miles from the site according to the website. These ground motion parameters were used in the liquefaction analyses. A summary of the seismic demand parameters is presented in Appendix D.

7.4.3. Evaluation of Liquefaction Potential and Sensitivity

The liquefaction potential of cohesionless (sandy) soils was evaluated utilizing the procedure published by Boulanger and Idriss and (2014) and as recommended in County of Los Angeles Administrative Manual GS 045.0 (2014), based on the SPT blowcounts from borings B-1 and B-2 from the previous exploration and from boring B-3 from the current exploration, and on laboratory test data.

The analyses based on standard penetration test (SPT) considered the energy ratio correction factor C_E of 1.48 for borings B-1 and B-2, and a C_E of 1.34 for boring B-3, as indicated in the hammer calibration records. A borehole diameter factor C_B of 1.0 was used per SP 117 based on the internal diameter of the hollow stem auger system used during drilling. The blowcounts recorded for soils driven with the 3-inch O.D. California-type Sampler with brass rings were converted to equivalent SPT blowcounts using a reduction factor of 0.67 as recommended by SP 117.

Seismic sensitivity of fine-grained soils was further evaluated per GS045.0 where the fine-grained soils are classified in the following 3 categories:

1. Soils with Plasticity Index < 12 and moisture content > 85 percent of the liquid limit are classified as fine-grained soils susceptible to liquefaction (typically includes silts);
2. Soils with Plasticity Index > 18 and a degree of sensitivity $S_t > 6$ are classified as fine-grained soils potentially susceptible to significant loss of strength during seismic shaking and require additional evaluation. The sensitivity of the on-site fine-grained soils is evaluated based on the water content, Atterberg limits, and effective vertical stresses using the procedures suggested by Holtz and Kovacs (1981) and Terzaghi, Peck and Mesri (1996).
3. Fine-grained soils falling outside the two categories described above are considered to behave like clays and are not considered susceptible to liquefaction or seismic sensitivity.

Evaluation of the liquefaction potential of granular soils and fine-grained soils with Plasticity Index less than 12 are presented in Appendix E and are summarized in Table 5 in the next section of the report. Analyses of the sensitivity of the saturated fine-grained soils to ascertain the potential for cyclic softening was not performed for the fine-grained materials based on the boring logs because the Plasticity Index of these soils fell below the threshold level of 18 indicating that the soils are not susceptible to cyclic softening. The analyses based on SPT data from the soil borings indicates that the on-site sandy silts and silty sands and poorly-graded sands found at the depth intervals 8 to 15 feet, 20 to 30 feet, and 35 to 50 feet are susceptible to liquefaction.

7.4.4. Dynamic Settlement

Dynamic settlement can occur in both unsaturated and saturated sands when loose to medium-dense granular soils undergo volumetric changes during ground shaking. Dynamic settlement can occur in saturated sands due to liquefaction or in dry sands due to densification of the soil matrix. The anticipated dynamic settlement of the saturated soils at the site was evaluated using SPT data

using procedures outlined by Yoshimine et al (2006). The estimated settlements by this procedure were further adjusted by a calibration factor of 0.9 as recommended by Cetin (2009).

The unsaturated dynamic settlement was calculated based on SPT data according to the procedure outlined by Pradel (1998a and 1998b). Table 5 presents the results of the liquefaction analyses and corresponding settlement evaluation as well as the dynamic settlement of unsaturated soils. Details of dynamic settlement analyses are presented in Appendix D.

As shown in Table 5, the total (combined) dynamic settlement was estimated to range between 3.4 and 6.5 inches at the ground surface. The differential seismic settlement is calculated to be about 3.3 inches over a horizontal distance of 30 feet. For the structures that are embedded or buried at least 20 feet the total seismic settlement is calculated to be 1.3 to 4.3 inches with an upper estimate of differential settlement of about 2.2 inches over 30 feet. Therefore, liquefaction is a hazard at the site and mitigation of the adverse effects of liquefaction induced settlement will be required for the development of the project.

Table 5
Results of Liquefaction and Dry Dynamic Settlement Analyses

Exploratio n No.	Assumed Groundwater Depth (feet)	Liquefiable Zone Depth Interval (feet)	FS _{liq}	Liquefaction Settlement (inches) ¹	Settlement of Dry Sands (inches) ¹	Combined Dynamic Settlement (inches) ¹
Preliminary Exploration						
B-1	8	8-15	0.5	6.5	-	6.5
		20-30	0.3			
		35-50	0.3			
B-2	8	8-15	0.3	3.4	-	3.4
		44-50	0.3			
Supplementary Exploration						
B-3	8	8-15	0.3	6.0	-	6.0
		25-30	0.3			
		40-50	0.2			

7.5. Earthquake-Induced Landslides

The site is not located in an Earthquake-induced Landslide Hazard Zone on the State of California Seismic Hazard Zones Map (see Figure 6). The site is relatively level and no evidence of landsliding was observed on or in the immediate vicinity of the site. Therefore, an earthquake-induced landslide is not considered to be hazard at the site.

7.6. Lateral Spreading

Since the overall site is relatively level and there are no open sloping faces, lateral spreading is not a hazard at the site.

7.7. Subsidence

Land subsidence is the settlement of the ground surface due to extraction or lowering of water levels or other fluids within the subsurface soil pores, or due to seismic activity. Groundwater withdrawal causes the alluvial sediments in the basin to compact. Fine-grained materials such as clays and silts that comprise the aquitard that separates the Upper and Lower aquifers in the east valley are more susceptible to compaction and subsidence than coarse-grained sediments, such as sands when groundwater is removed. Damage caused by subsidence can be visible cracks, fissures, or surface depression.

The site is mapped within an area of land subsidence in California (<https://ca.water.usgs.gov/land-subsidence/california-subsidence-areas.html>) due to groundwater pumping. Provided that groundwater management strategies that include subsidence mitigation through an existing groundwater recharge program continue to be employed in the area, ground subsidence beneath the site that could result in damage to future site improvements is unlikely to occur at the site.

7.8. Collapsible Soils

The phenomenon of hydro-consolidation is typically exhibited in geologically young, unconsolidated, low-density, loose, dry soils commonly present in arid to semi-arid regions. Collapsible soils are usually composed of granular particles that are supported by a clay or silt matrix that can be chemically cemented in place creating a porous structure. The bonds supporting this porous structure have enough shear strength to support loads at low moisture contents, however, once water is introduced the cemented bond structure breaks down and the granular particles are re-arranged causing significant volume loss.

Based on our field investigation, the soil materials below the artificial fill are alluvial deposits consisting of stiff to very stiff silts and medium dense sands. Such deposits are generally not susceptible to collapse; therefore, the potential of collapse is expected to be low. Based on the consolidation testing of 2 soil samples it was observed that none of the samples exhibited any collapse during wetting. Therefore, collapsible soils are not considered a hazard to the site.

7.9. Expansive Soils

Expansive soils are characterized by significant volume changes (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from precipitation, landscape irrigation, utility leakage, roof drainage, perched groundwater, drought, or other factors which can cause unacceptable settlement or heave that could negatively affect the performance of structures or slabs supported on grade.

The Expansion Index of two selected surficial samples taken as a composite of the soils within the upper 5 feet was 8 and 9, which indicates that the surficial soils at the site have a very low expansion potential. Consequently, expansive soils near the ground surface are not likely to be a hazard to the project.

7.10. Oil and Gas Fields

The site is not located within an oil field according to the State of California Department of Conservation's Division of Oil, Gas and Geothermal Resources Map (<https://maps.conservation.ca.gov/doggr/wellfinder/>).

7.11. Gaseous Hazards

Methane is a naturally occurring gas associated with the decomposition of organic materials. In high concentrations, methane is considered to be an explosion hazard. The site is not within 300 feet of an oil or gas well or 1,000 feet of a methane producing site. Therefore, a methane mitigation system may not be required according to the LA County website (<https://pw.lacounty.gov/epd/swims/OnlineServices/search-methane-hazards-esri.aspx>). The site is not located within an oil or gas field according to the State of California Department of Conservation's Division of Oil, Gas and Geothermal Resources Map (<https://maps.conservation.ca.gov/doggr/wellfinder/>).

7.12. Flood Hazard

The project site is not located within an area mapped by the California Department of Water Resources within the 100-year, 200-year, or 500-year projected floodplains (<https://gis.bam.water.ca.gov/bam/>).

7.13. Tsunamis and Seiches

A tsunami is a sea wave generated by large-scale displacements of the ocean floor that causes a sudden surge of water onto the land. Tsunamis are most commonly caused by movement along faults and underwater landslides activated by earthquakes. Seiches are earthquake-induced displacements of water within an enclosed body of water such as a lake. Strong ground motions from an earthquake cause the water to slosh back and forth onto land. The site is at elevation 65 feet or more above sea level, is not located near a lake, and is located outside of a designated Tsunami Inundation Area per https://maps.conservation.ca.gov/cgs/informationwarehouse/-ts_evacuation/. Therefore, hazard associated with a tsunami or seiches is not anticipated at the site.

8. DESIGN RECOMMENDATIONS

8.1. General

Based on the results of the field exploration and engineering analyses, it is Tetra Tech's opinion that the proposed construction is feasible from the geotechnical standpoint, provided that the recommendations contained in this report are incorporated into the design plans and implemented during construction. It is expected that conventional foundation and construction methods will be suitable for the proposed improvements.

The key geotechnical design consideration is the liquefaction hazard where settlements of about 6 inches due the design seismic event are calculated which will likely result in shifting, misalignment, or cracking distress that may result in a loss of function and require repairs. Consequently, recommendations are herein provided for augmentation of the foundation subgrade with a geogrid-reinforced raft. However, such a recommendation may not be commensurate for unoccupied infrastructure facilities, e.g., diversion structure, where the calculated liquefaction displacements can either be tolerated or will not immediately impact their functionality. Additional specific geotechnical design considerations include:

- Excavation and shoring design;
- Suitability of permeable pavements for the site.

The design recommendations presented below are based on Tetra Tech's current understanding of the project. Once the project configuration is finalized and the design is complete, Tetra Tech should review the plans and specifications to evaluate if the geotechnical design recommendations have been incorporated as intended.

8.2. Clearing and Grubbing

Prior to any site grading, surface vegetation, trash, and debris should be removed and disposed of offsite. Existing subsurface installations, such as abandoned foundations, pipes, cables, utility collectors, and/or tanks, if present, should be removed or abandoned per the Geotechnical Engineer's recommendations and in accordance with applicable regulations.

8.3. Site Preparation and Geogrid-Reinforced Raft Configuration

- All structures should be founded on a 3-foot-thick geogrid-reinforced select granular fill raft placed on processed native soils subgrade. To the extent practicable, the geogrid-reinforced raft should extend outside the structure footprint for a horizontal distance of at least 3 feet.

The geogrid-reinforced select fill raft system has been designed based on experience and performance records rather than based on rigorous engineering analyses. The concept of increasing bearing capacity and mitigating differential settlement by the geogrid v. select fill interaction is well understood, but standard engineering analyses have not yet been developed. Whereas advanced modeling of the response of the system to liquefaction settlements can be relatively easily analyzed, the selection of the differential settlement pattern can only be

discretionary. It is Tetra Tech's recommendation to accept the empirical experience for the geogrid-reinforced select fill raft design. Alternatively, this office can carry out engineering analyses of the response of the geogrid-reinforced select fill raft system to a settlement pattern deemed acceptable to the regulatory agency.

After completion of the removals to the geogrid-reinforced select fill raft bottom, the exposed subgrade should be uniform, firm and unyielding and be probed and accepted by the Geotechnical Engineer. The subgrade should be smoothened and compacted using appropriate compaction equipment. Fine-grained or granular soils should be moisture conditioned to a minimum of 125 percent or 110 percent of optimum moisture content, respectively, and compacted to at least 90 percent relative compaction per ASTM D1557.

The geogrid-reinforced select granular fill raft should be constructed in the following sequence:

1. Place 6 inches of select granular fill;
2. Install 1st geogrid;
3. Place 10 inches of select granular fill;
4. Install 2nd geogrid;
5. Place 10 inches of select granular fill;
6. Install 3rd geogrid;
7. Place 10 inches of select granular fill.

It is understood that for the building addition, the foundation system may need to match the foundation system of the existing building to minimize the possibility of differential settlement and therefore, no liquefaction mitigation would be implemented for that structure. However, if the geogrid-reinforced select fill raft is required for the building addition, an overexcavation of about 3 feet below the bottom of the existing foundations will be necessary. To construct the geogrid-reinforced select fill raft, excavations below the bottom of the existing foundations will be needed. These excavations will likely require underpinning of the existing foundations. The underpinning should be properly designed by the Contractor and will likely entail installing hydraulic jacks under the centerline of the existing footings.

- In areas where existing pavements will be replaced with permeable pavement the removals should extend to the depth of the new pavement subgrade elevation. To the extent practicable, the zone of overexcavation should extend a horizontal distance of at least 1 foot beyond the flatwork.
- Flatwork areas including exterior flatwork slabs should be overexcavated and recompacted to a depth of at least 1 foot below the proposed subgrade elevation, or to uniform acceptable soils, whichever is deeper. To the extent practicable, the zone of overexcavation should extend a horizontal distance of at least 1 foot beyond the flatwork.
- In non-structural/landscaped areas any existing fill may remain in place. However, depending on the future use of the area, existing fill may need to be excavated and replaced as compacted fill. This can be evaluated during grading.

- Disturbed soils at structural and non-structural areas will likely occur after demolition of existing site improvements. These soils should be overexcavated and recompacted to the total depth of the disturbed material.

The exposed overexcavation subgrade for all structures and slabs, should be probed and accepted by the Geotechnical Engineer. Localized zones of loose and/or unstable soils may be encountered during the grading operations at the overexcavation/subgrade level and should be overexcavated to a depth exposing firm and unyielding earth materials and then recompacted. If loose and/or unstable areas are encountered during grading operations that are not practical to be excavated and processed, Table 6 provides general options for stabilizing the subgrade. The specific type of remediation and associated area limits will need to be evaluated in the field by the Geotechnical Engineer.

Table 6
Options for Handling Unstable Materials at the Excavated Subgrade

Areas where the soils are soft and/or unstable at the excavation subgrade	• Overexcavate at least 2 feet • Stabilize the soft subgrade by gradually working single-grain thickness layers of open-graded aggregate material (typically 3/4" or 1.5" crushed rock, coarser for softer subgrade) at least 4 to 6 inches into the soil. • Place non-woven geotextile, Mirafi RS580i or approved equivalent, over the stabilized subgrade. • Place and compact well-graded fill (e.g., AB, CMB) or general approved backfill material to specified compaction over the geotextile.
<u>Larger</u> areas where the soils are <u>excessively</u> soft and/or unstable	• Overexcavate at least 2 feet • Improve the soft subgrade by gradually working single-grain thickness layers of open-graded aggregate material as much as possible/practical into the subgrade. • Place non-woven geotextile, Mirafi RS580i or approved equivalent, over the exposed soil. • Place at least 8 inches (12-18 inches preferred) of well graded aggregate material (e.g., AB, CMB); only reasonably achievable compaction is required. • Place non-woven geotextile, Mirafi RS580i or approved equivalent, over the aggregate layer. • Place and compact fill to specified compaction over the geotextile.

8.4. Grading Materials

8.4.1. General Fill

Excavated on-site soils may be re-used as compacted fill to build up the required grades or as utility trench backfill, provided they are free of organics, deleterious materials, debris, and particles over 3 inches in largest dimension. Locally, particles up to 6 inches in largest dimension may be incorporated in the fill soils during grading based on specific approval and placement recommendations provided by the Geotechnical Engineer. All fine-grained soils placed as fill to

achieve finish grade level or utility trench backfill, should be moisture conditioned to a minimum of 125 percent of optimum moisture content and compacted to at least 90 percent relative compaction per ASTM D1557; sandy soils should be moisture conditioned to a minimum of 110 percent of optimum moisture content and compacted to at least 90 percent relative compaction. Fill should be placed in horizontal lifts not more than 8 inches in loose, uncompacted thickness.

8.4.2. Select Granular Fill

The select granular fill required for the geogrid-reinforced select fill raft should generally be compliant with fine crushed miscellaneous base (CMB) specification per Greenbook 200-2.4 or similar, moisture-conditioned wet of optimum moisture content and compacted to at least 93 percent relative compaction per ASTM D1557. The manufacturer of the underground storage units may require a gravel layer (e.g., 3/4-inch gravel) underneath the units instead of select fill. In that case, the geogrid-reinforced select fill raft could be built entirely with the clean gravel, but a geotextile layer e.g., Mirafi 140NL or equivalent, should be placed for separation between the native soil subgrade after overexcavation takes place and the gravel raft. Alternatively, the geogrid-reinforced select fill raft could be built by replacing a portion of total thickness of the select fill with gravel as required by the manufacturer of the stormwater storage units, and in that case, there would not be a need to place a geotextile at the gravel and the select fill interface.

8.4.3. Geogrids

The geogrids for the geogrid-reinforced select fill or gravel raft should be high strength products, preferably Tensar InterAx850 or triaxial TX7 or some other approved equivalent.

8.4.4. Import Fill

In the event that any soil materials are imported to the site, such soils should be sampled, tested, and approved by the Geotechnical Engineer prior to arrival on-site. In general, any soils imported to the site for use as fill should be predominantly granular and have an Expansion Index less than 20. Additional recommendations for site grading are provided in the “General Site Grading Recommendations” section of this report.

8.5. Temporary Sloped Excavations

It is anticipated that all excavations to reach subgrade level for the underground stormwater storage facilities and for utility trenches can be performed as sloped excavations, i.e., without the need for shoring. Where space for sloped sides is not available, shoring will be required. Recommendations for shoring design are provided in the Temporary Shoring section of this report.

All excavations should be performed in accordance with CalOSHA regulations. The on-site soils to a depth of 25 feet may be considered Type C soils, as defined by the current Cal-OSHA soil classification.

Unsurcharged excavations: Sides of temporary, unsurcharged excavations less than 25 feet deep should be sloped back at an inclination of 1.5(H):1(V) or flatter.

Surcharge setback recommendations: Stockpiled (excavated) materials should be placed no closer to the edge of a trench excavation than a distance defined by a line drawn upward from the bottom of the trench at an inclination of 1(H):1(V), but no closer than 4 feet. A greater setback may be necessary when considering significant surcharge loads such as heavy vehicles, concrete trucks and cranes. Tetra Tech should be advised of such heavy surcharges so that specific setback requirements can be established. Alternatively, a shoring system may be designed to allow reduction in the setback distance.

The Geotechnical Engineer should observe the excavation progress so that appropriate modifications to the excavation design may be recommended, if necessary, due to conditions differing from the design assumptions.

8.6. Temporary Shored Excavations

Excavation depths for the construction of the proposed project are anticipated to be at a depth between 10 and 25 feet. If sloped excavations are not feasible because of lack of space, then shoring will be needed.

Cantilevered shoring systems are typically suitable and practicable for retained heights less than about 15 feet. Alternatively, restrained shoring system with internal bracing may be considered. Presented herein are design recommendations for both shoring systems. The shoring designer will need to take into account the presence of adjacent structures, utilities, conduits, and other underground structures and their impact on the installation and performance of the shoring system.

All components of the shoring system, including the penetration depth, should be designed by a specialist Civil Engineer registered in the State of California and should further satisfy requirements of Cal-OSHA. It is recommended that the final shoring design be reviewed by the Geotechnical Engineer.

8.6.1. Shoring Design

Either sheet pile or soldier pile and lagging systems may be utilized for the proposed construction. Table 7 below summarizes the governing geotechnical design parameters and loading diagrams for a shoring system for both cantilever and internal bracing conditions. These geotechnical parameters were developed based on the following assumptions:

- the shored soil grade is level;
- there are no hydrostatic pressures behind the wall;
- groundwater remains below the excavation bottom; and
- the shoring is temporary.

If any of these assumed conditions cannot be met due to field conditions or the contractor's method of construction, this office should be consulted regarding potential revisions to our recommendations.

Table 7
Geotechnical Design Parameters
Cantilevered and Restrained Temporary Shoring
(without hydrostatic pressure build-up behind the wall)

Excavation bottom depth (including overexcavation depth)	Up to 15 feet	
Subsurface materials	Alluvial Soils Mostly firm to stiff lean clays and silts and medium dense sands to depth of 40 feet Groundwater depth during construction assumed at a depth of at least 3 feet below the bottom of the excavation	
SHORING SYSTEM	Cantilevered shoring (excavation depth less than about 12 to 15 feet)	Restrained shoring with internal bracing
Soil unit weight, γ	125 pcf	
Design friction angle, ϕ	34°	
Design cohesion, c	0 psf	
K _a ... coefficient of active lateral pressure	0.28	
LOADING DIAGRAM ON SHORING		
Equivalent fluid density, EFD	43 pcf	n/a
Loading Diagram	Triangular distribution	Trapezoidal load distribution (see Diagram 2 below)
ALLOWABLE PASSIVE PRESSURE BELOW EXCAVATION BOTTOM		
Design friction angle, ϕ	34°	
K _p ... coefficient of passive lateral pressure	3.5	
Arching capability *	2.7 (for design of soldier pile systems)	
Equivalent fluid density (pcf EFD) ** (triangular distribution) – includes Safety Factor of 1.5 – ignore resistance within upper 12 inches	Sheet pile system: 290 / 145 (above / below groundwater) Soldier pile system (includes arching): 780 / 390 (above/below groundwater)	
* Per Caltrans Trenching and Shoring Manual (2011)		
** Valid without reduction for soldier pile spacing > 2.7 times the effective pile width		
*** Groundwater depth for temporary shoring can be assumed to be at 35 feet		

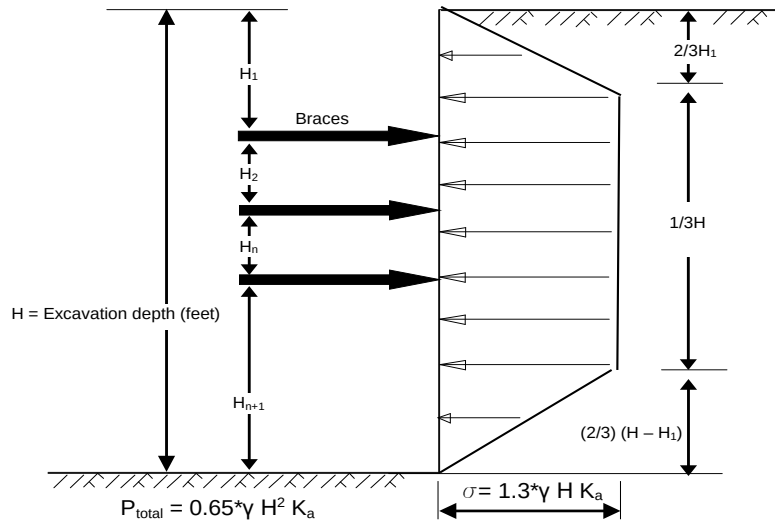


Diagram 2. Lateral pressures loading diagram for a braced excavation

Any surcharge (live or dead load) located within a 1(H):1(V) plane drawn up from the excavation bottom should be accounted for in the calculation of the lateral earth pressures. For cantilevered and restrained shoring systems, the lateral contribution of a uniform surcharge load may be calculated by multiplying the surcharge by a factor of 0.28 and 0.36, respectively. This constant lateral load, i.e., independent of depth, should be applied throughout the whole exposed height of the sheet pile or soldier pile wall. As a minimum, 3 feet of equivalent soil surcharge, i.e., 360 psf, is recommended to be included to account for nominal construction surcharge and traffic loads on the adjacent roads.

The required penetration depth of the soldier or sheet pile wall below the excavation bottom may be calculated based on the estimated passive soil resistances provided in Table 7. Passive soil resistance should be ignored for the upper 12 inches below the excavation bottom to account for potential near-surface soil disturbance.

8.6.2. Deflection

It should be realized that some shoring deflection will likely occur. It is difficult to accurately predict the magnitude of shoring deflection as it largely depends on the quality and sequence of construction. The shoring system should be designed so that deflection at the top of the shored excavation is kept below 1 inch. If greater deflection occurs during construction, additional bracing or restraint may be necessary to minimize settlement of the nearby improvements. If it is desired to reduce the deflection of the shoring, a greater lateral earth pressure could be used for the shoring design.

8.6.3. Internal Bracing

Internal bracing can be provided with struts or rakers designed for lateral earth pressures provided in Table 7 and shown schematically in Diagram 2 for the restrained shoring system.

If used, raker bracing could be supported on temporary concrete footings. For design of temporary footings poured with the bearing surface normal to the rakers inclined at 45 to 60 degrees from the vertical, an allowable bearing capacity value of 1,800 psf may be used, provided the shallowest point of the footing is at least 1 foot below the lowest adjacent grade. To reduce the movement of the shoring wall, the rakers should be tightly wedged against the footings and shoring wall.

8.6.4. Construction Staging

The shoring should be constructed utilizing a top-down method. Following the installation of the soldier or sheet piles, the soil is first partially excavated to allow for installation of the topmost row of internal bracing. For rakers, the staging will need to include installation of temporary rakers and their subsequent removal and replacement as the excavation advances. Following the installation of the topmost row of internal bracing, the excavation then proceeds, and each level of internal bracing should be installed as soon as practicable. The shoring designer should analyze each stage of internal bracing installation to ensure that the excavation has an adequate Factor of Safety.

For the soldier pile system, the lagging should be installed simultaneously as the excavation proceeds. To facilitate a tight connection between the lagging and the soils and to minimize settlement, any voids left behind the lagging should be filled with cement grout as the excavation advances. To continuously support the excavation, any unsupported height should not exceed 4 feet.

The annulus of the soldier pile beam borehole below the excavation bottom should be backfilled with concrete. Pea gravel may be used to backfill the hole from the excavation bottom to the top grade. If the soldier pile beams are to be retrieved after construction, they may be backfilled with a weaker cementitious slurry mix below the excavation bottom. If the contractor chooses to use a well-rounded uniform pea gravel material to fill the hole below the excavation bottom, a reduction of 33 percent should be applied to the passive soil resistance values provided in Table 7 to account for potential lateral yielding of the pea gravel backfill.

8.6.5. Shoring Performance Monitoring

Some means of monitoring the performance of the shoring system and nearby paved surfaces and structures is recommended. The monitoring should consist of periodic visual inspections and lateral and vertical surveying of the tops of the soldier / sheet piles and strategic survey points. It is recommended that a survey be performed before construction begins and then, as the excavation proceeds, the monitoring should be performed daily or whenever excavation activities are taking place. In addition, the Contractor should inspect the shoring daily and actively search for presence of cracks or excessive movements and report immediately to the shoring designer and the

Geotechnical Engineer. This office can provide further recommendations of the monitoring when the design of the shoring system is finalized.

8.7. Seismic Design Parameters per 2019 CBC

The project site is located in an area that is subject to the hazard of liquefaction. Therefore, per ASCE 7-16 Section 20.3.1, the site needs to be designated as Site Class F and seismic design parameters need to be obtained by a site response analysis performed in conformance with ASCE 7-16 Section 21.1. However, ACSE 7-16, Section 20.3.1 provides an exception for structures having fundamental periods of vibration equal to or less than 0.5s, where site response analysis is not required for liquefiable soils and the site class may be determined in accordance with Section 20.3, in which case this site may be classified as Site Class D. Although it is expected that this exception is applicable to the proposed construction, the Structural Engineer needs to verify that the fundamental period of the structures meets this condition. Alternative, this office can perform the site response analysis under a separate scope and budget.

The seismic design coefficients provided below in Table 8 are based on Chapter 16 Section 1613 of the 2019 CBC and obtained from the Structural Engineers Association of California (SEAOC) and the Office of Statewide Health Planning and Development (OSHPD) website application (<https://www.seaoc.org/page/seismicdesignmaptool>). According to ASCE 7-16 Section 11.4.8, a site-specific ground motion hazard analysis shall be performed if structures on Site Class D have an S_1 greater than or equal to 0.2 unless the seismic coefficient C_s determined by Equation (12.8-2) is used for values of $T \leq 1.5 T_s$ and taken as equal to 1.5 times the value computed in accordance with either Equation (12.8-3) for $T_L \geq T > 1.5 T_s$ or Equation (12.8-4) for $T > T_L$. Tetra Tech can provide site-specific ground motion hazard analysis under a separate scope and budget.

Table 8
2019 CBC and ASCE 7-16 Seismic Design Parameters

Referenced Site Latitude and Longitude: 33.883493°, -118.131557°

Site Class Table 20.3-1 ASCE 7-16		D
Coefficients for the Maximum Considered Earthquake, MCE_R , for Site Class B	Short Period (0.2 seconds), S_s	1.592*
	1 Second Period, S_1	0.57*
Coefficients for the Maximum Considered Earthquake, MCE_R (Site Modified)	Short Period (0.2 seconds), S_{MS}	1.592*
	1 Second Period, S_{M1}	1.026**
Coefficients for the Design Earthquake	Short Period (0.2 seconds), S_{DS}	1.061*
	1 Second Period, S_{D1}	0.684**
Design PGA (risk-targeted maximum rotated direction)		0.424g*
Site Modified Peak Ground Acceleration PGA_M (geometric mean)		0.747g*
* Values obtained from Structural Engineers Association of California (SEAOC) and the Office of Statewide Health Planning and Development (OSHPD) website application, https://www.seaoc.org/page/seismicdesignmaptool based on ASCE7-16 and 2018 International Building Code.		
** See requirements for site-specific ground motions in Section 11.4.8 of ASCE 7-16.		

8.8. Foundations

Underground stormwater facilities and other buried structures may be supported on footing pads or mat foundations, supported on a geogrid-reinforced select fill raft as recommended in this report. The addition to the existing building can be supported on shallow strip or spread foundations. Foundations should be designed and reinforced in accordance with the recommendations of the Structural Engineer and should conform to the 2019 California Building Code.

8.8.1. Foundations for the Building Addition and Structures less than 10 feet Deep

Table 9 presents the geotechnical design parameters for the at-grade building addition and any other structure founded shallower than 10 feet.

Table 9
Geotechnical Design Parameters

Continuous Strip Footings	
Dimensions (feet)	At least 18 inches wide but less than 4 feet wide
Depth of Embedment	Embedded at least 2 feet below the lowest adjacent grade
Allowable Bearing Pressure (without geogrid-reinforced select fill raft)	2,000 psf
Allowable Bearing Pressure (with geogrid-reinforced select fill raft)	3,000 psf
Spread Footings	
Dimensions (feet)	At least 24 inches wide but less than 5 feet wide
Depth of Embedment	At least 24 inches below the lowest adjacent grade
Allowable Bearing Pressure (without geogrid-reinforced select fill raft)	2,900 psf
Allowable Bearing Pressure (with geogrid-reinforced select fill raft)	4,800 psf

Table 9 (continuation)

All Footings	
Allowable Bearing Pressure for Transient Live Loads	The allowable bearing pressure value may be increased by one-third for transient live loads from wind or seismicity.
Estimated Settlement	- Approximate 1 inches of static settlement. - Approximate differential settlement of 0.5 inches between supports or over a distance of 30 feet
Allowable Coefficient of Friction (incorporates Factor of Safety of 1.5)	0.3 concrete on soil 0.40 for mass concrete on select fill raft 0.30 for formed concrete on select fill raft
Allowable Lateral Passive Resistance (incorporates Factor of Safety of 2)	220 pcf (EFD, equivalent fluid density, for compacted fill with $\phi=34^\circ$) - The passive resistance derived from the upper 12 inches should be neglected.
Allowable Combined Lateral Resistance	- Total allowable resistance to lateral loads can be calculated by combining lateral resistance due to friction at the base and lateral passive resistance. - Passive resistance values may be increased by one-third when considering transient wind or seismic loading

8.8.2. Foundations for Underground Structures/Vaults at a depth below 10 feet

For foundations for the underground storage units and other structures founded at depths greater than 10 feet the parameters presented in Table 10 should be used. Foundations should be designed and reinforced in accordance with the recommendations of the Structural Engineer and should conform to the 2019 California Building Code.

Table 10
Geotechnical Design Parameters

Spread Footings			
Dimensions (feet)	• Up to 5 feet x 5 feet	• Up to 10 feet x 10 feet	• Up to 15 feet x 15 feet
Allowable Bearing Pressure (Structures at least 10 feet deep)	• 5,500 psf	• 3,800 psf	• 3,000 psf
Allowable Bearing Pressure (Structures at least 17 feet deep)	• 6,700 psf	• 5,000 psf	• 4,200 psf
Modulus of Subgrade Reaction	• For design of large footings which behave like mat foundations supported on a geogrid-reinforced select fill raft, a reference modulus of subgrade reaction k_1 of 230 pci derived for a square bearing plate with 1-foot x 1-foot dimensions may be used. The modulus of subgrade reaction k (in pci) for the design of a concrete element of a given dimension can be calculated as: $k = k_1 \frac{(B + 1)^2}{4B^2} \frac{1 + 0.5 \frac{B}{L}}{1.5}$ where B and L are the governing width and the length of the element in feet, but no more than 14 times the thickness of the mat foundation.		
All Foundations			
Geogrid-Reinforced Select Fill Raft	• Structures founded on a 3-foot-thick geogrid-reinforced select fill raft		
Allowable Bearing Pressure	• The allowable bearing value may be increased by one-third for transient live loads from wind or seismicity.		
Estimated Static Settlement for Allowable Bearing Pressure	• Approximately 1-inch total settlement. • Approximately 0.5-inch differential settlement between supports or over a distance of 30 feet.		
Allowable Lateral Passive Resistance (incorporates Factor of Safety of 2)	• 220 pcf (EFD, equivalent fluid density, for compacted fill with $\phi=34^\circ$) • The passive resistance derived from the upper 12 inches should be neglected.		
Allowable Friction Factor at the base to compute Lateral Sliding Resistance (incorporates Factor of Safety of 1.5)	• 0.40 for mass concrete on geogrid-reinforced select fill raft • 0.30 for formed concrete on geogrid-reinforced select fill raft • 0.30 for select fill or gravel on geotextile		
Uplift Capacity	• The weight of the soil that contributes to the uplift capacity can be estimated as a zone defined by an angle of 20 degrees from the vertical projected from the top edge of the foundation upwards to the adjacent grade. • A total unit weight of 125 pcf may be used for the soil. • The lowest depth of foundation embedment from the adjacent grade shall be used in the calculations		

8.8.3. Foundations Adjacent to Trenches

The bottom of any trenches that are required for any buried utilities and piping should be kept outside a zone defined by a 1(H):1(V) plane projected from the outside bottom edge of any existing or proposed foundation. Backfill materials and procedures shall conform to the recommendations provided in the “Site Preparation and Geogrid-Reinforced Raft Configuration” and “General Site Grading” sections of this report. If any piping needs to be placed within the zone of influence, the pipes should be designed to account for the increased surcharge from the applied foundation pressures and to withstand potential differential settlement between the surcharged and unsurcharged segments of the pipe. Generally, the pipes within the impacted zone should be protected with concrete encasement, utilidors, or other suitable form of protection. The piping and encasement should be designed to withstand differential settlements of up to 1 inch over a distance equal to half of the depth of the pipe below the bottom of the footing. This office should be contacted to review any specific utility interaction configurations and their proposed mitigation.

8.9. Uplift of Buried Structures

Buried structures should be designed to resist uplift forces due to potential buoyancy exerted by a design groundwater depth of 8 feet. These buoyant forces need to be accounted to prevent buried structures including pipelines from floating or shifting upward. The designer must consider all the downward and upward forces on the structures and design for the worst-case scenario. The uplift resistance should be calculated as recommended in Table 10. If uplift becomes a governing design issue, Tetra Tech can perform a site-specific uplift analysis under a separate scope and budget.

8.10. Lateral Earth Pressures on Permanent Subterranean Walls

The static lateral pressures acting on the proposed on-site underground should be calculated based on the recommendations provided in Table 11. Lateral earth pressures presented in this table are for a level backfill.

Based on the 2019 CBC the design of retaining walls higher than 6 feet, as measured from the top of the footing, requires the inclusion of not only static lateral pressures but also of additional seismically induced lateral earth pressures. According to the 2019 CBC the dynamic seismic lateral earth pressures on foundation walls and retaining walls should be determined using the design earthquake ground motions. Based on the USGS U.S. Seismic Design Maps website application (<http://earthquake.usgs.gov/designmaps/us/application.php>), the PGA from the Design Response Spectrum at the site is approximately 0.424g as indicated in Table 8.

The seismic induced earth pressure increments based on the Design Earthquake Ground Motions were estimated using the method recommended by Agusti and Sitar (2013) and as required by the County of Los Angeles Administrative Manual S004 dated January 6, 2020 regarding Seismic Earth Pressures on Retaining Walls. Determination of whether the active or at-rest condition is appropriate for the design will depend on the flexibility of the walls. In walls with medium dense silty sand backfill that are free to rotate at least 0.002 radians (deflection at the top of the wall of at least 0.002 x H) may be designed for the active condition. Walls that are not capable of this movement, such as basement or vault walls, should be assumed rigid and designed for the at-rest

condition. The effect of any surcharge (dead or live load) located within a 1(H):1(V) plane drawn upward from the heel of the wall footing should be added to the lateral earth pressures.

The native soils are considered susceptible to liquefaction between the depths of 8 and 15 feet and between the depths of 20 and 25 feet. Therefore, the designer should consider the stability of the structure for the temporary liquefaction lateral pressure at these depth intervals. Earth pressures for the liquefaction condition are also included in Table 11. When considering the liquefiable condition the designer should not include any seismic pressure increments.

Recommendations for the combined static and seismic lateral passive earth pressure during the design seismic event computed as recommended by Taciroglu (2013) are also included in Table 11. Underground walls should be designed and constructed to withstand hydrostatic pressures behind the wall.

Table 11
Lateral Pressures due to Static and Seismic Loads

Active Pressure for Yielding Walls		
<u>Static active pressure</u> (psf)	above groundwater i.e., depth $z < z_w = 8$ feet	$31z + 0.28Q$
	below groundwater i.e., depth $z \geq z_w = 8$ feet	$31z_w + 80(z - z_w) + 0.28Q$
<u>Liquefaction active pressure</u> (psf)	below groundwater i.e., depth $z \geq z_w = 8$ feet	$125z_w + 125(z - z_w) + Q$ $- 2*0.09[125z_w + 63(z - z_w)]$
<u>Active seismic pressure increment</u> (psf)		$22z$
At rest Pressure for Non-yielding Walls		
<u>Static at-rest pressure</u> (psf)	above groundwater i.e., depth $z < z_w = 8$ feet	$55z + 0.44Q$
	below groundwater i.e., depth $z \geq z_w = 8$ feet	$55z_w + 90(z - z_w) + 0.44Q$
<u>Liquefaction at-rest pressure</u> (psf)	below groundwater i.e., depth $z \geq z_w = 8$ feet	$125z_w + 125(z - z_w) + Q$
<u>At-rest seismic pressure increment</u> (psf)		$36z$
Lateral Passive Resistance		
Allowable <u>static passive pressure</u> (psf) Includes Factor of Safety of 2	above groundwater i.e., depth $z < z_w = 8$ feet	$220z_1$
	below groundwater i.e., depth $z \geq z_w = 8$ feet	$220z_w + 110(z_1 - z_w)$
Ultimate seismic <u>total passive resistance</u> (psf) No Factor of Safety	above groundwater i.e., depth $z < z_w = 8$ feet	$310z_1$
	below groundwater i.e., depth $z \geq z_w = 8$ feet	$310z_w + 150(z_1 - z_w)$
<u>Liquefaction ultimate passive pressure</u> (psf) No Factor of Safety	below groundwater i.e., depth $z \geq z_w = 8$ feet	$125z_w + 63(z_1 - z_w)$ $+ 2*0.09[125z_w + 63(z - z_w)]$
Notes: - Lateral Pressures due to seismic loading are based on a $PGA=0.424g$ for a design response spectrum taken as 2/3 MCER response spectrum. The appropriate total seismic force (active plus seismic increment for yielding walls and at rest plus seismic increment for non-yielding walls) should be calculated by assuming a downward increasing triangular equivalent fluid pressure distribution. The resulting force should be assumed to act at 1/3 of the height of the wall above the bottom of the wall. - Pressure based on soil with $\phi = 34^\circ$, $c = 0$ psf, $\gamma_t = 125$ pcf (above groundwater), $\gamma_b = 125$ pcf (below groundwater) - The 2019 CBC requires that basement walls be designed for at rest earth pressures for static conditions. - Liquefaction residual strength (S_r), for $N = 10$ was assumed to be equal to 0.09 the effective normal stress, per Idriss and Boulanger (2015). Legend: z ... Depth (ft) below the grade behind the wall – depth measured from the ground surface to the depth where the soil lateral pressure is evaluated; z_1 ... Depth (ft) below the grade where passive conditions apply, i.e., usually in front of the wall – depth measured from the ground surface to the depth where the soil lateral pressure is evaluated; z_w ... Depth to groundwater (ft) – depth measured from the ground surface to the groundwater depth (constant); Q ... Uniform surcharge (psf) within a 1(H):1(V) plane drawn upward from the heel of the wall footing		

Suitable backfill materials within a zone immediately the behind the subterranean walls should have a Sand Equivalent of about 30, an Expansion Index of less than 20, and fines content (passing #200 sieve) of less than 15 percent. It is expected that due to the interspersed nature of the native materials comprising clayey and sandy soils, the on-site materials are not likely to be suitable as a backfill immediately behind. Consequently, a select on-site or import material with an Expansion Index less than 20 or approved non-expansive import material should be used for the backfill within at least 5 feet behind the back of the underground walls. It is expected that additional laboratory testing will be necessary to determine the suitability of the selected on-site or import materials. The select on-site or import materials that are approved as backfill materials should be moisture-conditioned to 110 percent of the optimum moisture content and placed in horizontal lifts not more than 8 inches of uncompacted thickness and compacted to at least 90 percent of the maximum dry density, as evaluated by the latest version of ASTM D1557.

8.11. Liquefaction-Induced Downdrag on Buried Structures

Structures founded below a depth of 8 feet will be subject to downdrag forces due to liquefaction potential identified at a depth between 8 and 15 feet (Table 5). These forces should be added to the structural design loads and verified against the allowable bearing pressures presented in Table 10.

The downdrag for this configuration is composed of the downdrag due to the non-liquefied material from the ground surface to a depth of 8 feet settling due to the settlement of the underlying liquefied zone and the downdrag of the liquefied interval itself. The downdrag force is calculated as the lateral pressure exerted by the non-liquefied and the liquefied zone multiplied by the appropriate coefficient of friction. The lateral pressure for the non-liquefied and liquefied interval should be selected using the respective condition in Table 11. For the non-liquefied and liquefied material, the coefficient of friction may be assumed as 0.4 and 0.1, respectively.

8.12. Concrete Slabs-On-Grade

The recommendations provided in the “Site Preparation and Geogrid-Reinforced Raft Configuration” section of this report and in this section are intended to provide a firm bearing subgrade to help reduce the occurrence of cracks in concrete and associated horizontal separation and vertical offset. However, it should be understood that concrete slabs may still crack due to structural design or detailing, curing, or construction execution even when these recommendations are implemented. Therefore, the concrete slab design is ultimately the responsibility of the Structural Engineer and/or Concrete Specialist who can account for all adverse factors and loading and provide reinforcement, concrete mix, and curing specifications to minimize their adverse effects.

8.12.1. Floor Slabs

For the design of concrete floor slabs, supported on a geogrid-reinforced select fill raft, a reference modulus of subgrade reaction k_1 of 230 pci derived for a square bearing plate with 1-foot x 1-foot dimensions may be used. The design modulus of subgrade reaction k in pci for a concrete rectangular element can be determined as:

$$k = k_1 \frac{(B + 1)^2}{4B^2} \frac{1 + 0.5 \frac{B}{L}}{1.5}$$

B and L are the governing width and length of the concrete element in feet, but no more than 14 times the thickness of the element.

Although the slab-on-grade should be designed by the Structural Engineer and/or Concrete Specialist the following recommendations are provided to assist with the preliminary design and initial cost estimate. Thus, the slab-on-grade should have a minimum thickness of 5 inches and be reinforced with a minimum of No.4 reinforcing bars spaced at 18 inches on-center, each way, placed in the middle one-third of the section. The slab should be doweled into the building footings to reduce the potential for differential movement. Reinforcement should be properly placed and supported on blocks or “chairs”. Welded wire mesh reinforcement is not recommended.

8.12.2. Exterior Slabs

Exterior concrete slabs should be placed on subgrade prepared in accordance with the recommendations provided in the “Site Preparation and Geogrid-Reinforced Raft Configuration” and section of this report. A Structural Engineer or an Engineer specialized in concrete design should be consulted if cracking of the exterior slabs is to be minimized. As a minimum for exterior walkways, it is recommended that narrow strip concrete slabs, such as sidewalks, be reinforced with at least No. 4 reinforcing bars placed longitudinally at 18 inches on center. Wide exterior slabs should be reinforced with at least No. 4 reinforcing bars placed 18 inches on center, each way. Welded wire mesh reinforcement is not recommended. Reinforcement should extend through the control joints to reduce the potential for differential movement.

8.12.3. Control Joints

For preliminary design considerations, control joints should be provided in all concrete slabs-on-grade as recommended by American Concrete Institute (ACI) guidelines and at a maximum spacing (in feet) of 2 to 3 times of the slab thickness (in inches), but generally no more than 10 feet. All joints should form approximately square patterns to reduce potential for randomly oriented shrinkage cracks. The control joints should be tooled at the time of the pour or sawcut to ¼ of slab depth within 6 to 8 hours of concrete placement. All joints in flatwork should be sealed to prevent moisture, vermin, or foreign material intrusion. Precautions should be taken to prevent curling of slabs in this semi-arid region (refer to ACI guidelines).

The recommendations provided in the “Site Preparation and Geogrid-Reinforced Raft Configuration” section of this report are intended to provide a firm bearing subgrade and the design recommendations provided in the “Mat Foundation” and “Concrete Exterior Slabs-On-Grade” sections are based on the subgrade soil conditions. These recommendations are intended to provide a gross stability and also to help reduce the occurrence of cracks in concrete and associated horizontal separation and vertical offset. However, it should be understood that regardless of the subgrade preparation, concrete slabs/mats may still crack due to structural design or detailing,

curing, or construction execution even when these subgrade and design recommendations are implemented. If cracking of the concrete is desired to be minimized, the reinforcement, concrete mix, and curing specifications should be designed by the Structural Engineer and Concrete Specialist.

8.13. Embedded Posts and Poles at Grade

Recommendations for design of posts and poles including fence poles and streetlamps are provided herein. The allowable static lateral soil bearing pressure can be assumed to be equal to 140 pcf equivalent fluid density (pcf EFD). The allowable passive earth pressure value incorporates a Factor of Safety of 2. For isolated poles not adversely affected by 0.5 inches of horizontal motion due to short-term lateral loads the lateral soil bearing pressure can be increased to 280 pcf EFD. Vertical compressive loading can be resisted utilizing an allowable end bearing pressure of 2,000 psf.

8.13.1. Non-Constrained Case

For the non-constrained case where the pole is not restricted to move at the ground level, the minimum depth of embedment required to resist lateral loads should be determined in accordance with the 2019 CBC Section 1807.3.2.1. Where bare ground (without concrete or asphalt cover) is present adjacent to the foundation, the lateral resistance should be ignored for the upper 12 inches below grade. Therefore, a trapezoidal pressure distribution should be used starting at 12 inches below grade.

8.13.2. Constrained Case

For the constrained case where the pole is restricted from movement at the ground level by encasement in surrounding concrete or similar, the minimum depth of embedment required to resist lateral loads should be determined in accordance with the 2019 CBC, Section 1807.3.2.2.

8.14. Utility Trenches

Utility trench excavations should follow the recommendations provided in the Temporary Sloped Excavations section of this report. Construction recommendations for the trench bottom preparation and trench backfill are provided below.

8.14.1. Utility Trench Bottom Preparation

Trench bottom preparation should produce a uniform, firm, and unyielding subgrade. The exposed trench bottom should be probed and accepted by the Geotechnical Engineer. Any particle size greater than 3 inches should be removed. Surface water should be controlled so that the trench subgrade is protected even during periods of heavy rainfall.

8.14.2. Utility Trench Backfill

Bedding and pipe zone backfill. The bedding is the material placed in the bottom of the trench on which the pipeline is laid. Bedding material should extend at least 6 inches below the bottom of pipe and up to the pipe springline level. The pipe zone is defined as the area above the bedding, around the pipe, and up to typically at least 12 inches over the pipe.

Bedding and pipe zone backfill material should consist of clean sand, i.e., sand with less than about 30 percent fines and no more than 20 percent of fine gravel. The pipe bedding material should be placed over the full width of the trench on firm and unyielding subgrade soils approved by the Geotechnical Engineer. After placement of the pipe, the pipe zone backfill should be brought up uniformly on both sides of the pipe to reduce the potential for unbalanced loads. No voids or uncompacted areas should be left beneath the pipe haunches. The silty sands and silty materials found within the upper 20 feet at the site are not expected to be suitable for pipe bedding and pipe zone backfill.

The bedding and pipe zone backfill should be moisture-conditioned to at least wet of optimum moisture content and compacted in horizontal lifts no greater than 6 inches to achieve relative compaction of at least 90 percent per ASTM D1557. The use of mechanized compaction equipment within the pipe zone should be carefully controlled to avoid overstressing or damaging the pipe.

General trench backfill. This zone extends from the top of the pipe zone to the finished grade. The backfill may consist of approved excavated soil devoid of particles greater than 3 inches in the largest dimension, moisture-conditioned wet of optimum moisture content and compacted to at least 90 percent of relative compaction per ASTM D1557. Lift thickness for backfill will be dependent on the type of compaction equipment utilized but should not generally exceed 8 inches in loose thickness. Care should be exercised to avoid damaging the pipe during compaction of the trench backfill. Within pavement areas the upper 12 inches of backfill should be compacted to at least 95 percent of maximum.

Compaction by ponding or jetting of the trench backfill materials may be permitted by the Geotechnical Engineer only where select sand backfill is used and adequate drainage is available in the sandier surrounding soil intervals. If ponding or jetting are used for compaction, it will likely be necessary to supplement these methods by the use of vibrators to achieve the required compaction.

8.15. Infiltration Suitability for Permeable Pavements

Based on the characteristics of the near surface soils i.e., sandy silts and silty sands, encountered within the parking lot area, shallow onsite infiltration is not considered feasible per County of Los Angeles Guidelines GS200.2 which require a minimum design infiltration rate of 0.3 inches/hour. The County of Los Angeles Low Impact Development (LID) Standards Manual (2014) also requires a design infiltration rate larger than 0.3 inches/hour for permeable pavements without underdrain. However, the County of Los Angeles Low Impact Development (LID) Standards Manual (2014) allows for the design and use of permeable pavements in areas with low infiltration

capacity if an underdrain is included in the design. Therefore, it is recommended that if permeable pavements are implemented in this project underdrains be provided.

8.15.1. Permeable Pavements

Permeable pavements are considered for replacement of the existing asphalt pavements in the parking lot stall areas. Permeable or porous pavements include: permeable interlocking concrete pavement (PICP), also known as interlocking pavers; pervious concrete pavement (PCP); and porous asphalt pavement (PAP). Pervious pavement is designed primarily to promote stormwater infiltration and improve the quality of stormwater runoff. Furthermore, the County of Los Angeles Low Impact Development (LID) Standards Manual (2014) allows for the design and use of permeable pavements in areas with low infiltration capacity if an underdrain reservoir layer is included in the design. The designer will need to incorporate all the considerations included in the LID Manual (County of Los Angeles, 2014).

Results of R-value testing indicate R-values of 56 and 58. For the design of the structural pavement section, the procedure recommended in the Design Manual for Pervious Pavement by Caltrans (2014) was followed assuming an R-value of 40 for the subgrade, an R-value of 75 for the reservoir layer (i.e., aggregate base layer). The permeable pavements for the applications considered in this project fall under Category B, i.e., parking area for passenger vehicles included trafficked area. The basic design section consists of a surface pavement layer with an underlying reservoir layer on top of subgrade. A geotextile liner is placed between the reservoir layer and the soil subgrade to provide separation. The geotextile liner will need to comply with the specifications provided in the County of Los Angeles Low Impact Development Standards Manual (2014) Table E-9. For PICP sections a bedding layer (also known as top filter layer) is placed under the pavers.

A collector perforated pipe or underdrain should be placed within the reservoir area to collect stored water. Underdrains should be sloped at a minimum of 0.5 percent and must drain freely to an acceptable discharge point. The underdrain should have a mainline diameter of eight inches using slotted PVC SDR 26 or PVC C9000. The slotted pipe should have two to four of slots cut perpendicular to the axis of the pipe or at right angles to the pitch of corrugations. Slots should be 0.04 to 0.1 inches wide with a length of 1 to 1.25 inches. Slots should be longitudinally-spaced such that the pipe has a minimum of one square inch opening per lineal foot and should face down. For further details regarding the underdrain please refer to the County of Los Angeles Low Impact Development (LID) Standards Manual (2014), Section T-5. Rigid non-perforated observation pipes with a diameter equal to the underdrain diameter should be connected to the underdrain to provide a clean-out port as well as an observation well to monitor drainage rates.

Table 12 presents the minimum pavement structural sections required for the 3 permeable pavement systems considered herein for the parking stalls area for mostly passenger vehicles assuming a Traffic Index of 5.5 which is adequate per Caltrans Highway Design Manual (2017) for parking lots and auto road areas with minimal traffic. If other Traffic Indexes are considered, Tetra Tech can provide the permeable pavement structural sections upon request.

Table 12
Parking Area - Minimum Structural Sections for Permeable Pavements, TI=5.5

Permeable Pavement Type	Layers	Thickness (inches)
PAP	Open Graded Friction Course (OGFC)	1.2
	Asphalt Treated Permeable Base (ATPB)	3.8
	Reservoir Layer Class 4 Aggregate Base AB/ ASTM No. 57 Aggregate	Min. 7.4
	Geotextile on subgrade	--
PCP	PCP	6
	Reservoir Layer Class 4 AB/ ASTM No. 57 Aggregate	Min. 8.4
	Geotextile on subgrade	--
PICP	Pavers	Min. 3.1
	Bedding Layer	1.8
	Reservoir Layer Class 3 Permeable	4.2
	Reservoir Layer Class 4 AB/ASTM No. 57 Aggregate	Min. 9.6
	Geotextile on subgrade	--

For an effective performance, any drainage from other landscaped areas into the pavement area should be prevented to minimize the potential for clogging. Permeable pavements should not be placed near structural foundations and the pavement subgrade should be at least 15 feet away and outside a 1:1 plane drawn up from the bottom of adjacent foundations.

A concrete and/or concrete curb and gutter should be provided for edge support. These barriers can also keep unwanted sediment off the pavement. For parking lot type applications, the subgrade slope of the facility should be relatively flat, no steeper than 2 percent slope, to enable even distribution and infiltration of stormwater.

8.16. Corrosion Potential Evaluation

The corrosion potential of the on-site soils to buried steel and concrete was evaluated based on laboratory testing of 2 near-surface soil samples. Table 13 below presents the results of the corrosivity testing.

Per 2019 CBC based on 2018 IBC, Section 1904.1, concrete subject to exposure to sulfates shall comply with the durability requirements set forth in ACI 318 Section 19.3. Based on the measured water-soluble sulfate results, the exposure of buried concrete to sulfate attack is not a concern, i.e., exposure class S0 per ACI 318-14, Table 19.3.1.1. Consequently, per Table 19.3.2.2, cement with a minimum 28-day compressive strength of 2,500 psi could be used.

Per 2019 CBC, Section 1904.1, concrete, including protection for corrosion and exposure to chlorides, shall conform to durability requirements established in ACI 318. Reinforcement should be protected from corrosion in accordance with ACI 318-14 Chapter 19.

Table 13
Corrosivity Test Results

Boring	Sample ID	Soil type	Depth (feet)	pH	Minimum Resistivity (ohm-cm)	Chlorides (ppm/%)	Soluble Sulfate Content in Soil (ppm/%)
B-3	SK-1	Sandy Silt	0-5	9.1	471	579/0.0579	871/0.0871
B-4	SK-1	Sandy Clay	0-5	9.0	2,619	47/0.0047	94/0.0094

The corrosion potential of buried metals was evaluated based on the minimum resistivity and our experience with similar soils. The on-site soils are anticipated to be “mildly corrosive to very corrosive” to buried metals as defined by the NACE (1984).

A Corrosion Specialist should be consulted regarding suitable types of piping and necessary protection for underground metal conduits. The corrosion potential of the on-site soils should be verified during construction for each encountered soil type. Imported fill materials should be tested prior to placement to confirm that their corrosion potential is not more severe than the one assumed for the project.

8.17. Drainage Control

The intent of this section is to provide general information regarding the control of surface water. The control of surface water is essential to the satisfactory performance of the building construction and site improvements. Surface water should be controlled so that conditions of uniform moisture are maintained beneath and adjacent to the structure, even during periods of heavy rainfall. The following recommendations should be considered as minimal.

- Ponding and areas of low flow gradients should be avoided.
- Paved surfaces within 10 feet from the building foundation should be provided with a gradient of at least 2 percent sloping away from improvements.
- Bare soil, e.g., planters, within 10 feet of the structure should be sloped away from the improvement at a gradient of 5 percent.
- Positive drainage devices, such as graded swales, paved ditches, and/or catch basins should be employed to accumulate and convey water to appropriate discharge points.
- Concrete walks and flatwork should not obstruct the free flow of surface water.
- Planters should not be located immediately adjacent to structures. If planters are to be located adjacent to a structure, they should be positively sealed, should incorporate a subdrain, and should be provided with free discharge capacity to a drainage device.
- Planting areas at grade should be provided with positive drainage. Wherever possible, the grade of exposed soil areas should be established above adjacent paved grades. Drainage

devices and curbing should be provided to prevent runoff from adjacent pavement or walks into planted areas.

- Gutter and downspout systems should be provided to capture discharge from roof areas. The accumulated roof water should be conveyed to an off-site disposal area by a pipe or concrete swale system.

9. GENERAL SITE GRADING RECOMMENDATIONS

The intent of this section is to provide general information regarding the grading of the site. Site grading operations should conform with applicable local building and safety codes and to the rules and regulations of those governmental agencies having jurisdiction over the subject construction.

The grading contractor is responsible for notifying governmental agencies, as required, and the Geotechnical Engineer at the start of site cleanup, at the initiation of grading, and any time that grading operations are resumed after an interruption. Each step of the grading should be accepted in a specific area by the Geotechnical Engineer, and where required, should be approved by the applicable governmental agencies prior to proceeding with subsequent work.

The following site grading recommendations should be regarded as minimal. The site grading recommendations should be incorporated into the project plans and specifications.

- Prior to grading, existing vegetation, trash, surface structures and debris should be removed and disposed off-site at a legal dumpsite. Any existing utility lines, or other subsurface structures which are not to be utilized, should be removed, destroyed, or abandoned in compliance with current governmental regulations.
- Subsequent to cleanup operations, and prior to initial grading, a reasonable search should be made for subsurface obstructions and/or possible loose fill or detrimental soil types. This search should be conducted by the contractor, with advice from and under the observation of the Geotechnical Engineer.
- Prior to the placement of fill or foundations, the site should be prepared in accordance with the recommendations presented in the “Site Preparation and Geogrid-Reinforced Raft Configuration” section of this report or the manufacturer’s specifications for the stormwater storage and infiltration, whichever is more stringent. Undocumented fill or disturbed soils within shallow foundation footprints should be removed and processed as recommended by the Geotechnical Engineer.
- The exposed subgrade and/or excavation bottom should be observed and approved by the Geotechnical Engineer for conformance with the intent of the recommendations presented in this report and prior to any further processing or fill placement. Proof rolling may be considered to provide a general guidance about the quality of the fill. It should be understood that the actual encountered conditions may warrant excavation and/or subgrade preparation beyond the extent recommended and/or anticipated in this report.
- On-site inorganic granular soils that are free of debris or contamination are considered suitable for placement as compacted fill. Any rock or other soil fragments greater than 3 inches in size should not be placed within 5 feet of a foundation subgrade.
- Any imported fill material required for backfill or grading should be tested and approved prior to delivery to the site.

- Visual observations and field tests should be performed during grading by the Geotechnical Engineer. This is necessary to assist the contractor in obtaining the proper moisture content and required degree of compaction.
- Wherever, in the opinion of the Geotechnical Engineer, an unsatisfactory condition is being created in any area, whether by cutting or filling, the work should not proceed in that area until the condition has been corrected.

10. DESIGN REVIEW CONSTRUCTION SERVICES

Geotechnical review of plans and specifications and participation during the construction are an integral part of the design practice. The following paragraphs present our recommendations relative to the review of construction documents and the monitoring of construction activities.

10.1. Plans and Specifications

Upon completion, the civil and structural design plans and specifications should be reviewed and approved by Tetra Tech prior to submittal for issuance of grading, building, and/or construction permits as the geotechnical recommendations may need to be re-evaluated based on the actual design configuration and loads. This review is intended to evaluate whether the recommendations contained in this report have been incorporated into the project plans and specifications as intended.

10.2. Construction Monitoring

The objective of the construction quality assurance (CQA) is to assist in the construction of the soils and soils-structure interaction components of the Project. Observation of site overexcavation, processing and assessment of fill materials, fill placement, and other site grading operations by the Geotechnical Engineer should be implemented during construction to allow for evaluation of the geotechnical-related conditions as they are encountered. This process provides the Geotechnical Engineer with the opportunity to recommend appropriate revisions as needed.

10.2.1. Grading Observations

Continuous observations by the Geotechnical Engineer should be provided to assess all encountered soil types, verify the extent of removals and overexcavation, suitability of import materials, lift thicknesses and densities during placement and compaction of fill materials. The Geotechnical Engineer should observe all temporary excavations and construction slopes, as well as the backfill operations so that appropriate modifications to the design criteria presented herein may be recommended, if necessary, due to encountered conditions differing from the design assumptions.

10.2.2. Foundation Construction Observations

The Geotechnical Engineer should observe and evaluate the presence of satisfactory materials at the foundation subgrade. Foundation excavations should be clean of loosened soil and debris before placing steel or concrete. If soft or loose soils or other unsatisfactory materials are encountered, such materials should be removed and replaced with compacted fill prior to pouring of concrete. The Geotechnical Engineer should also observe the placement of the geogrid-reinforced select fill or gravel raft including the placement of any high strength geogrids.

10.2.3. Permeable Pavement Construction Observations

The preparation of the pavement subgrade and the placement of permeable pavement sections should be observed by the Geotechnical Engineer. Careful observation is recommended to evaluate that the pavement subgrade is consistent with the design assumptions and that it is uniform and uniformly compacted where appropriate and that the recommended permeable pavement and reservoir thickness are achieved.

10.2.4. Construction Quality Assurance Reporting

The following list is intended to provide basic minimum guidelines for the reporting during the excavation and backfilling operations:

- A Daily Field Report should be generated each time a representative of the Geotechnical Engineer is performing QA work at the site.
- The Daily Field Reports should contain, at a minimum, a detailed description of the field activities, utilized equipment, areas of work, date, time, weather, and locations and results of all observations and performed tests.
- Provisions should be made for vertical and horizontal control for recording observations and test locations.
- A complete set of Daily Field Reports should be submitted as a part of formal final reporting.
- Visual observations and field tests should be performed during grading by the Geotechnical Engineer. This is necessary to assist the contractor in obtaining the proper moisture content and required degree of compaction. In general field compaction testing should be performed as recommended in Table 14.

Table 14
Field and Laboratory Testing

Test or Observation	Test Method	Minimum Testing Frequency
Field Density	Maximum dry density and optimum moisture content by ASTM D1557	As deemed necessary or the most frequent of: 1 per major material type 1 per 4 weeks of construction
Field Compaction	Field Density by Sand cone (ASTM D1556) or Nuclear gauge (ASTM D2922)	As deemed necessary or the most frequent of: 1 per 1,000 CYs, or 1 per each 2 compaction lifts, or 2 per full day of placement 1 per 250 LF of roadway

11. STATEMENT 111

Based on the data and evaluations presented in the report, it is the opinion of Tetra Tech that the subject project site for the proposed John Simms Park facilities will be safe against hazards from future landsliding, settlement or slippage and that the proposed grading construction as long as the recommendations provided herein are implemented will have no adverse impact on the geologic stability of property outside of the project site.

12. LIMITATIONS

The recommendations and opinions expressed in this report are based on Tetra Tech's geotechnical investigation data, laboratory testing, and engineering analyses. It should be noted that this study did not evaluate the presence of hazardous materials although the results of analytical testing for one soil sample are presented herein.

Due to the limited nature of the field explorations, conditions not observed and described in this report may be present on the site. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation and laboratory testing can be performed upon request. It should be understood that conditions different from those anticipated in this report may be encountered during grading operations, for example, the extent of unsuitable soil and the associated additional effort required to mitigate them.

Site conditions can change with time as a result of natural processes or the activities of man. Changes to the applicable laws, regulations, codes, and standards of practice may occur as a result of government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Tetra Tech has no control. Therefore, this report should be reviewed and recertified by Tetra Tech if it were to be used for a project design commencing more than one year after the date of issuance of this report.

Tetra Tech's recommendations for this site are dependent upon verification of the actual encountered field conditions, appropriate quality control of grading operations including shoring installation, overexcavation, processing and replacement of the on-site materials, and foundation construction. Accordingly, the recommendations are made contingent upon the opportunity for Tetra Tech to observe all aspects of subgrade preparation for the proposed construction. If parties other than Tetra Tech are engaged to provide such services, such parties are automatically and expressly assuming complete responsibility as the Geotechnical Engineer of Record for the project and are deemed concurring with the recommendations in this report and/or are obligated to provide alternative recommendations.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Tetra Tech should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document. Reliance by others on the data presented herein or for purposes other than those stated in the text is authorized only if so permitted in writing by Tetra Tech. It should be understood that such an authorization may incur additional expenses and charges.

Tetra Tech has endeavored to perform its evaluation using the degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical professionals with experience in this area in similar soil conditions. No other warranty, either expressed or implied, is made as to the conclusions and recommendations contained in this report.

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Figures

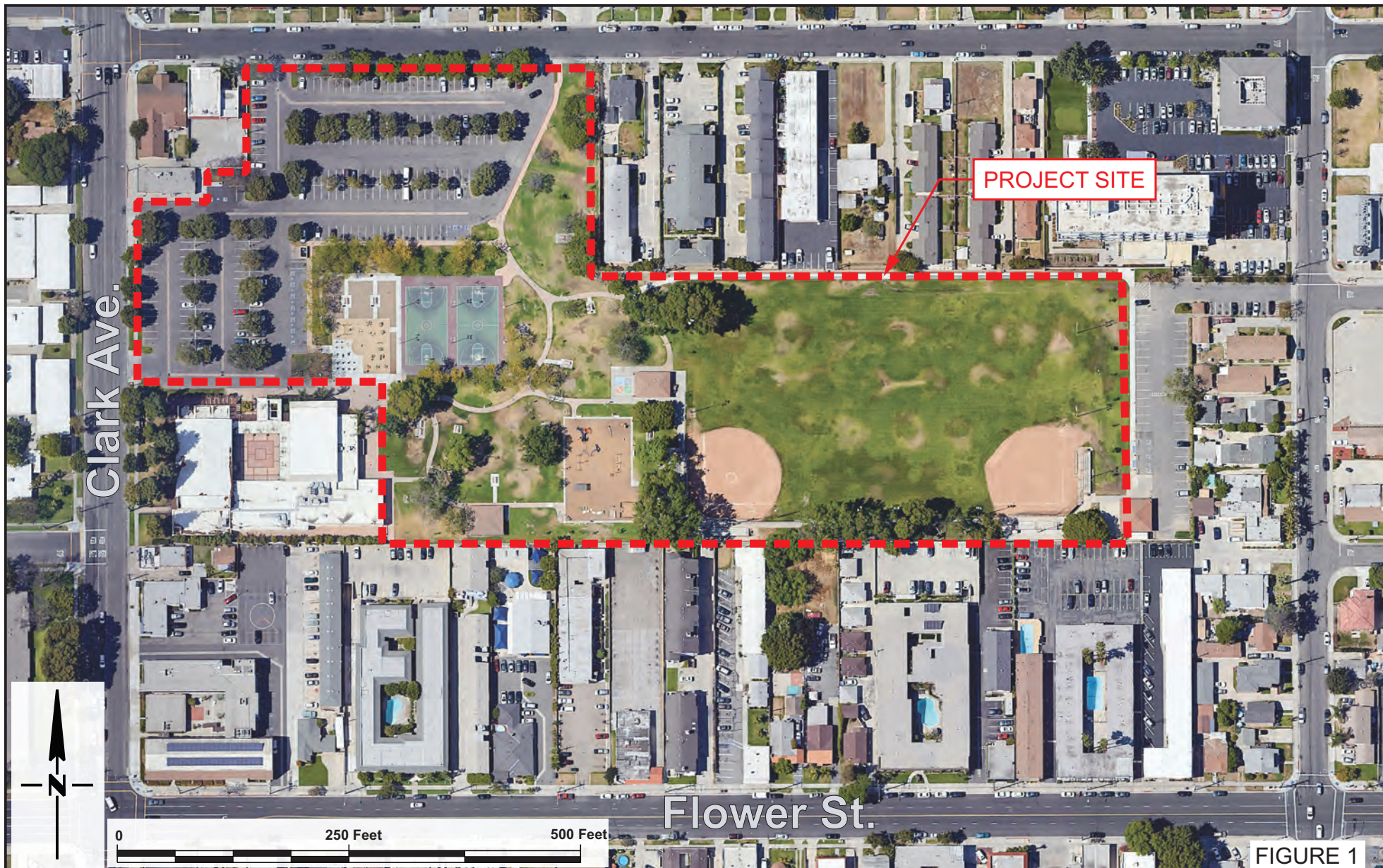


FIGURE 1



TETRA TECH

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JOHN SIMMS PARK - BELLFLOWER, CA

PROJECT LOCATION MAP

JOB NO.
CWT 22-05E

DATE
JUN 2022

DRAWN BY
TAC

CHECKED BY
FC



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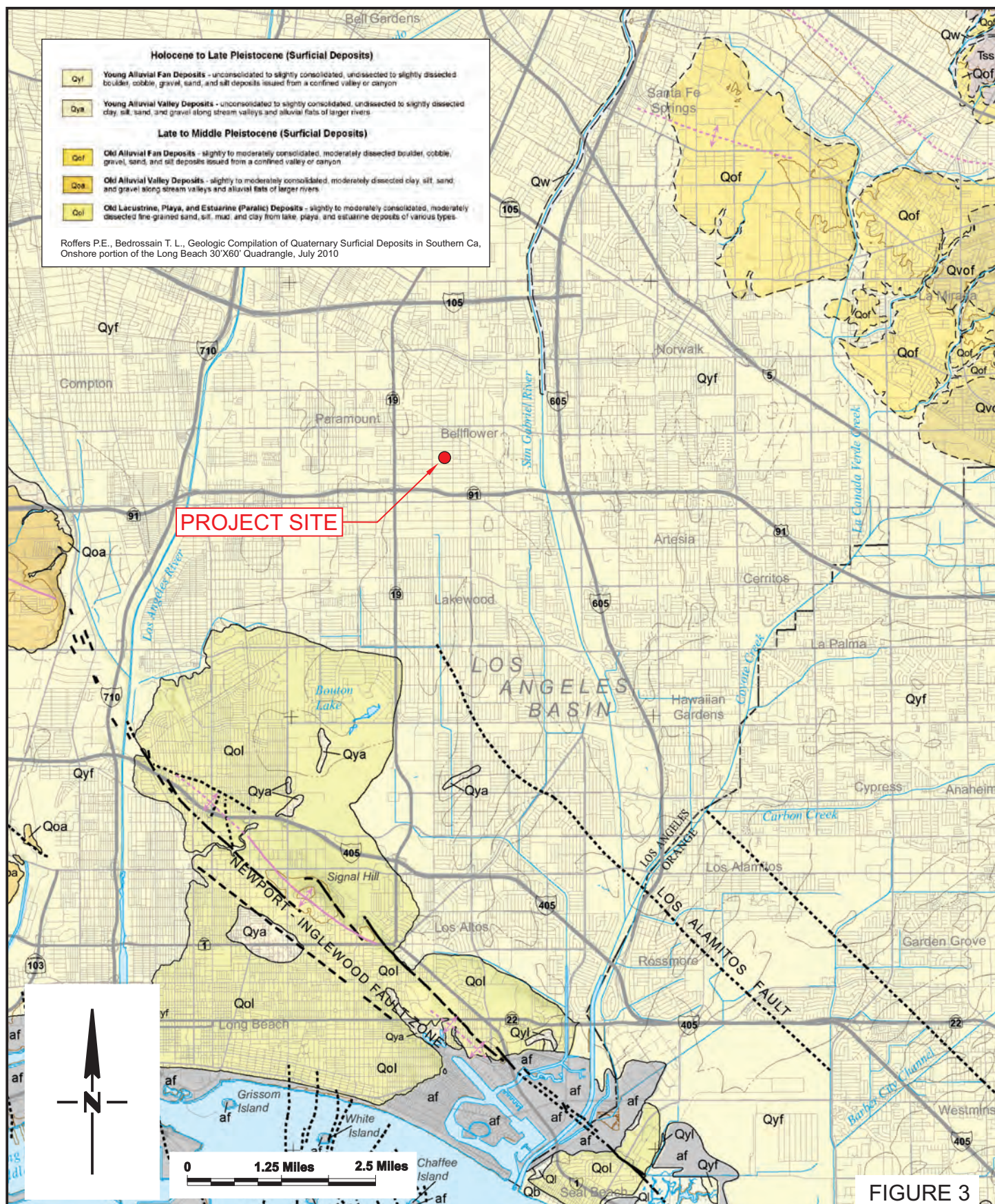
SITE LAYOUT AND BORING LOCATION MAP

JOB NO. CWT 22-05E

DATE JUN 2022

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JOHN SIMMS PARK - BELLFLOWER, CA

GEOLOGIC MAP

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FIGURE 4



TETRA TECH

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JOHN SIMMS PARK - BELLFLOWER, CA

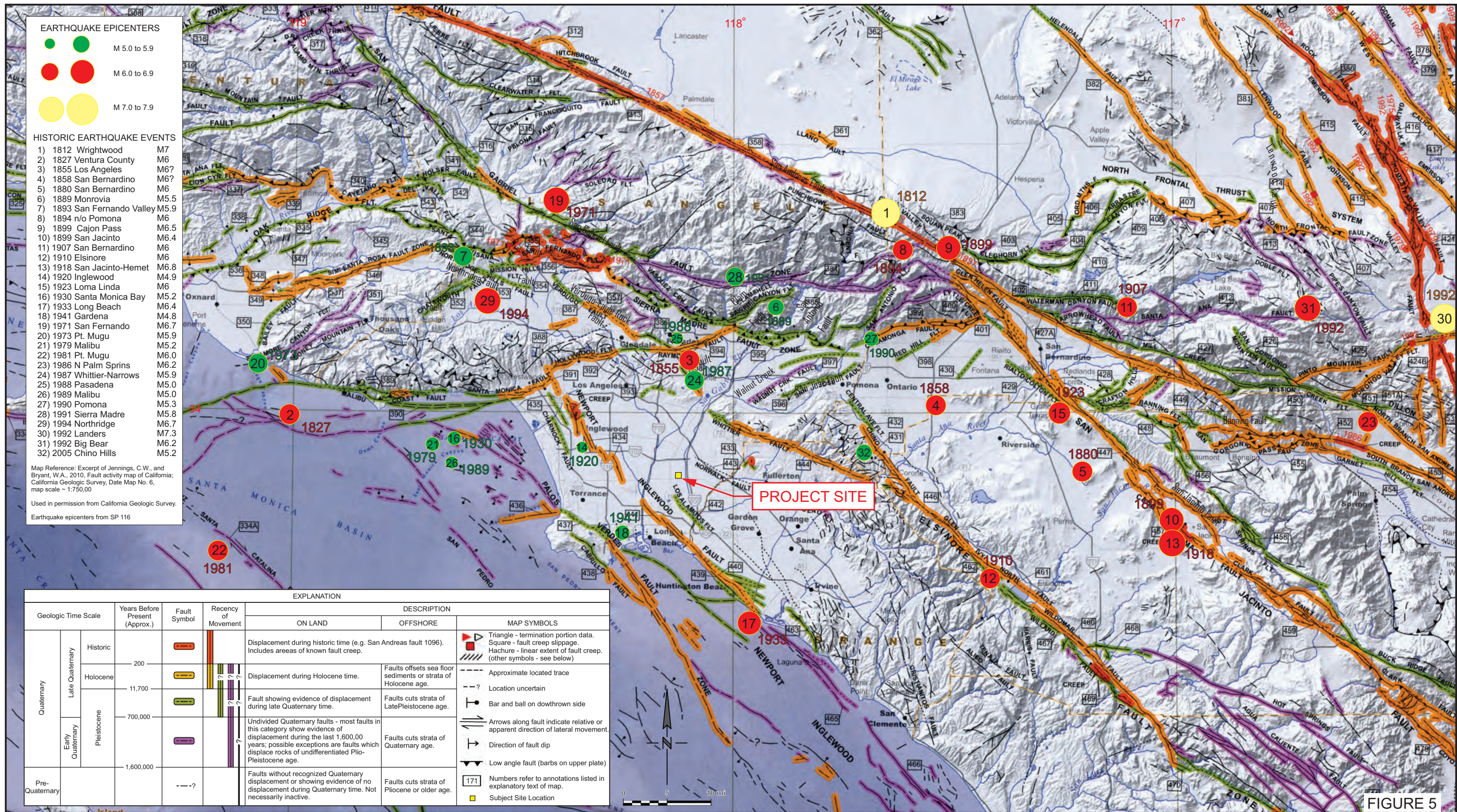
HISTORIC HIGH GROUNDWATER MAP

JOB NO.
CWT 22-05E

DATE
JUN 2022

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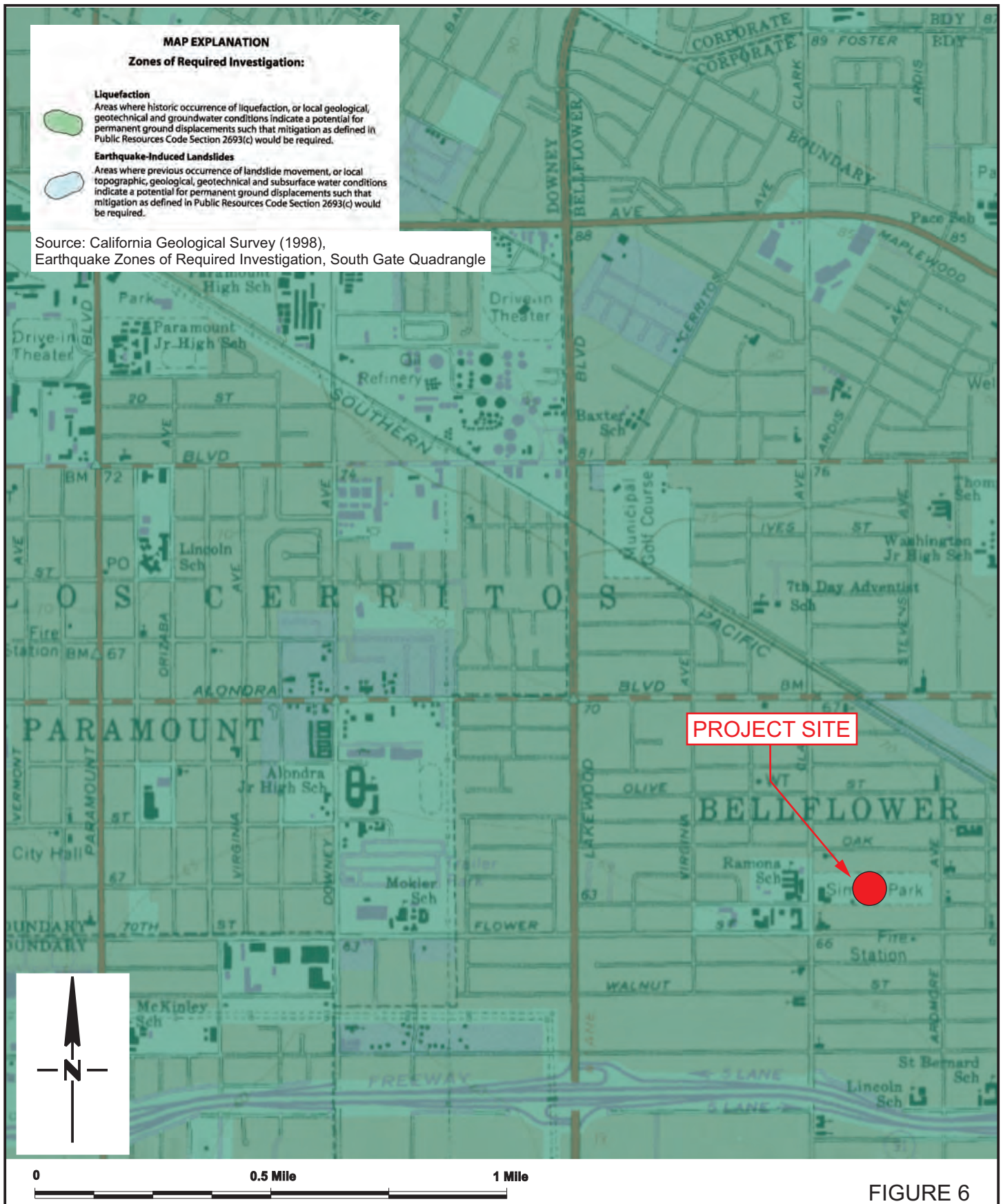


FIGURE 6

 TETRA TECH 21700 Copley Drive, Suite 200, Diamond Bar, CA 91765 TEL 909.860.7777 www.tetratech.com	JOHN SIMMS PARK - BELLFLOWER, CA		JOB NO. CWT 22-05E
	SEISMIC HAZARD ZONES MAP		DATE JUN 2022
			DRAWN BY TAC
			CHECKED BY FC

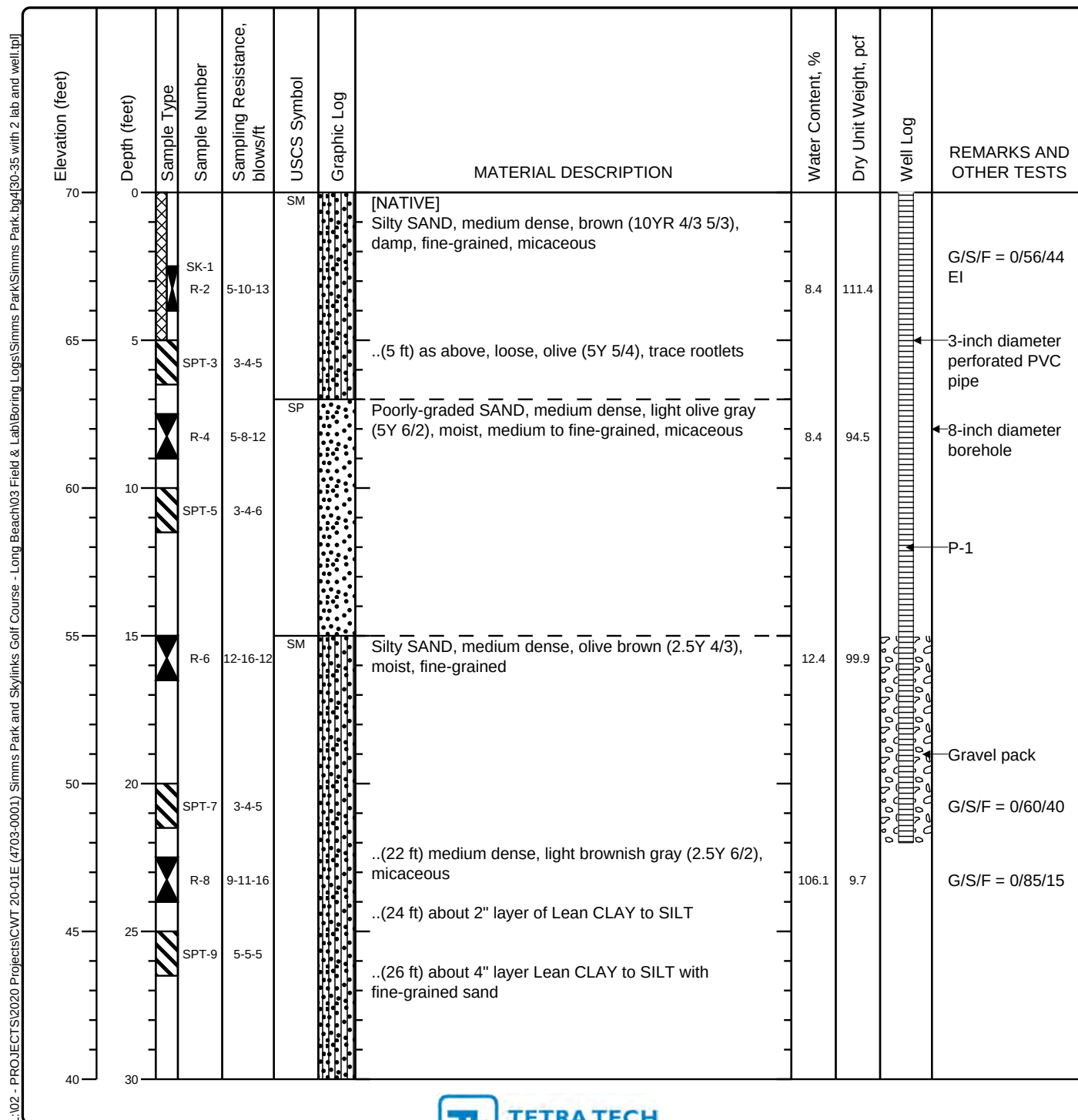
Appendix A1

Logs of Previous Exploratory Borings

Project: **John Simms Park**
 Project Location: **Bellflower, CA**
 Project Number: **CWT 20-01E (4703-0001)**

Log of Boring B-1/P-1
Sheet 1 of 2

Date(s) Drilled 4/16/2020	Logged By SCM	Checked By FC
Drilling Method Hollow-stem Auger	Drill Bit Size/Type 8-inch diameter	Total Depth of Borehole 51.5 feet
Drill Rig Type CME75 Truck Mounted Rig	Drilling Contractor 2R Drilling	Approximate Surface Elevation 70 feet
Groundwater Level and Date Measured 36.67 feet	Sampling Method(s) Bulk, Rings, SPT	Hammer Data 140lb auto-trip hammer, 30-inch drop
Borehole Backfill Grout 4-51.5', soil cuttings 0-4', sod plug at surface	Location Latitude: 33.883505° Longitude: -118.130304°	



Project: **John Simms Park**
 Project Location: **Bellflower, CA**
 Project Number: **CWT 20-01E (4703-0001)**

Log of Boring B-1/P-1
Sheet 2 of 2

Elevation (feet)	Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/ft	USCS Symbol	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Well Log	REMARKS AND OTHER TESTS
40	30		R-10	6-16-30	SP		Poorly-graded SAND, medium dense, greenish gray (5G 5/1), 2" to 4" layer of Lean CLAY to SILT, trace of organics, medium grained	14.7	102.2		NP
35	35		SPT-11	7-9-11							
					ML		Sandy SILT, stiff, light olive gray (5Y 6/2), wet, micaceous				
30	40		R-12	6-7-12	SM		Silty SAND, medium dense, greenish gray (5G 5/1), wet, fine-grained, micaceous	27.2	98.2		
25	45		SPT-13	6-7-9	ML		Sandy SILT, very stiff, light olive gray (5Y 6/2), wet, fine-grained sand, trace of clay, micaceous				
20	50		R-14	5-13-16	SM		Silty SAND, medium dense, greenish gray (5G 5/1), wet, fine grained	28.2	96.5		
15	55						Notes: -Total depth: 51.5 ft -Groundwater encountered at 36' 8" at 9:18 AM -Caving encountered at 22.5' after pulling out auger				
10	60										
5	65										

L:\02 - PROJECTS\2020 Projects\CWT 20-01E (4703-0001) Simms Park and Skylinks Golf Course - Long Beach\03 Field & Lab\Boring Logs\Simms Park\Bq4\30-35 with 2 lab and well.rpt]

Project: **John Simms Park**
 Project Location: **Bellflower, CA**
 Project Number: **CWT 20-01E (4703-0001)**

Log of Boring B-2/P-2
 Sheet 1 of 2

Date(s) Drilled 4/16/2020	Logged By TC	Checked By FC
Drilling Method Hollow-stem Auger	Drill Bit Size/Type 8-inch diameter	Total Depth of Borehole 51.5 feet
Drill Rig Type CME75 Truck Mounted Rig	Drilling Contractor 2R Drilling	Approximate Surface Elevation 69 feet
Groundwater Level and Date Measured 35 feet	Sampling Method(s) Bulk, Rings, SPT	Hammer Data 140lb auto-trip hammer, 30-inch drop
Borehole Backfill Grout 4-51.5', soil cuttings 0-4', sod plug at surface	Location Latitude: 33.883503° Longitude: -118.131257°	

Elevation (feet)	Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/ft	USCS Symbol	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Well Log	REMARKS AND OTHER TESTS
69	0		SK-1		SM		[NATIVE] Silty SAND, loose, dark brown (7.5YR 3/4), damp, fine-grained, trace of organics, micaceous				EI
			SPT-2	3-4-5							3-inch diameter perforated PVC pipe
64	5		R-3	6-10-13	ML		Sandy SILT, very stiff, brown (10YR 4/3 5/3), damp, micaceous	18.7	108.3		PL/LL/PI = 26/30/4
			SPT-4	4-5-6	SP		Poorly-graded SAND with silt, medium dense, light brownish gray (2.5Y 6/2), damp, fine-grained, micaceous				8-inch diameter borehole
59	10		R-5	6-10-13				6.1	96.2		P-2
			SPT-6	2-3-4	CL		Lean CLAY to SILT, firm, dark gray (N 4/1), moist, micaceous				Gravel pack
54	15										
49	20		R-7	6-10-12	SM		Silty SAND, medium dense, olive brown (2.5Y 4/3), moist, fine-grained, trace clay	13.6	115.4		G/S/F = 0/69/31
			SPT-8	7-9-11	SP		Poorly-graded SAND, medium dense, yellowish brown (10YR 5/6), moist, fine to medium grained, trace of silt				G/S/F = 0/73/27
44	25		R-9	14-13-21			...(25 ft) light olive gray (5Y 6/2)	4.6	99.5		
39	30										

L:\02 - PROJECTS\2020 Projects\CWT 20-01E (4703-0001) Simms Park and Skylinks Golf Course - Long Beach\03 Field & Lab\Boring Logs\Simms Park\Bq4\30-35 with 2 lab and well.jpg

Project: **John Simms Park**
 Project Location: **Bellflower, CA**
 Project Number: **CWT 20-01E (4703-0001)**

Log of Boring B-2/P-2
Sheet 2 of 2

Elevation (feet)	Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/ft	USCS Symbol	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Well Log	REMARKS AND OTHER TESTS
39	30		SPT-10	10-12-15			...(25 ft) light olive gray (5Y 6/2)				
34	35		R-11	17-27-33				4.6	92.8		
29	40		SPT-12	7-10-14							
					SM		Silty SAND, medium dense, bluish gray (5B 5/1), wet, fine-grained				
24	45		R-13	6-6-8				39.9	84.2		
					CL		Lean CLAY to SILT, stiff, dark gray (N 4/1), wet				
					SM		Silty SAND, medium dense, bluish gray (5B 5/1), wet, fine-grained				
19	50		SPT-14	5-5-7							
							Notes: -Total depth: 51.5 ft -Groundwater encountered at 35' at 1:16 PM -No caving encountered				
14	55										
9	60										
4	65										

L:\02 - PROJECTS\2020 Projects\CWT 20-01E (4703-0001) Simms Park and Skylinks Golf Course - Long Beach\03 Field & Lab\Boring Logs\Simms Park\Boring Log B-2/P-2 (4703-0001) (2) (1).pdf

Project: **John Simms Park**
 Project Location: **Bellflower, CA**
 Project Number: **CWT 20-01E (4703-0001)**

Key to Log of Boring Sheet 1 of 1

Elevation (feet)	Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/ft	USCS Symbol	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Well Log	REMARKS AND OTHER TESTS
1	2	3	4	5	6	7	8	9	10	11	12

COLUMN DESCRIPTIONS

- | | |
|---|--|
| <p>1 Elevation (feet): Elevation (MSL, feet).</p> <p>2 Depth (feet): Depth in feet below the ground surface.</p> <p>3 Sample Type: Type of soil sample collected at the depth interval shown.</p> <p>4 Sample Number: Sample identification number.</p> <p>5 Sampling Resistance, blows/ft: Number of blows to advance driven sampler one foot (or distance shown) beyond seating interval using the hammer identified on the boring log.</p> <p>6 USCS Symbol: USCS symbol of the subsurface material.</p> <p>7 Graphic Log: Graphic depiction of the subsurface material encountered.</p> <p>8 MATERIAL DESCRIPTION: Description of material encountered. May include consistency, moisture, color, and other descriptive text.</p> | <p>9 Water Content, %: Water content of the soil sample, expressed as percentage of dry weight of sample.</p> <p>10 Dry Unit Weight, pcf: Dry weight per unit volume of soil sample measured in laboratory, in pounds per cubic foot.</p> <p>11 Well Log: Graphical representation of well installed upon completion of drilling and sampling.</p> <p>12 REMARKS AND OTHER TESTS: Comments and observations regarding drilling or sampling made by driller or field personnel.</p> |
|---|--|

FIELD AND LABORATORY TEST ABBREVIATIONS

CHEM: Chemical tests to assess corrosivity COMP: Compaction test CONS: One-dimensional consolidation test LL: Liquid Limit, percent	PI: Plasticity Index, percent SA: Sieve analysis (percent passing No. 200 Sieve) UC: Unconfined compressive strength test, Qu, in ksf WA: Wash sieve (percent passing No. 200 Sieve)
---	--

MATERIAL GRAPHIC SYMBOLS

	Lean CLAY, CLAY w/SAND, SANDY CLAY (CL)		SILT, SILT w/SAND, SANDY SILT (ML)
	Gravel		Silty SAND (SM)
			Poorly graded SAND (SP)

TYPICAL SAMPLER GRAPHIC SYMBOLS

	Auger sampler		CME Sampler		Pitcher Sample
	Bulk Sample		Grab Sample		2-inch-OD unlined split spoon (SPT)
	3-inch-OD California w/ brass rings		2.5-inch-OD Modified California w/ brass liners		Shelby Tube (Thin-walled, fixed head)

OTHER GRAPHIC SYMBOLS

	Water level (at time of drilling, ATD)
	Water level (after waiting)
	Minor change in material properties within a stratum
	Inferred/gradational contact between strata
	Queried contact between strata

GENERAL NOTES

- Soil classifications are based on the Unified Soil Classification System. Descriptions and stratum lines are interpretive, and actual lithologic changes may be gradual. Field descriptions may have been modified to reflect results of lab tests.
- Descriptions on these logs apply only at the specific boring locations and at the time the borings were advanced. They are not warranted to be representative of subsurface conditions at other locations or times.

Appendix A2

Logs of Current Exploratory Borings



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BORING B-3

Sheet 1 of 2

PROJECT NAME Simms Park CLIENT Craftwater GROUND ELEVATION 64 ft
PROJECT NUMBER 197-4703-0005 PROJECT LOCATION Bellflower, CA GROUNDWATER DEPTH 35.4 ft
DATES DRILLED 5/3/2022 DRILLING CONTRACTOR MR Drilling BOREHOLE DEPTH 51.5 ft
LOGGED/CHECKED BY SMS/FC DRILLING METHOD CME 75 HSA BOREHOLE DIAMETER 8 in
COORDINATES 33.883502, -118.130882 HAMMER DATA 140 lbs Autohammer, 30" drop BACKFILL grout
LOCATION DESCRIPTION mid-field

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOW/COUNT blows/6" (bpf)	GRAPHIC LOG	USCS	Standard Penetration Test (SPT) California-Type Ring Sample No Recovery No Recovery Shelby Tube Grab/Bulk Sample	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0					MATERIAL DESCRIPTION				
0				ML	Surface is covered in grass. [NATIVE] Qyf Sandy SILT, dark brown 7.5YR 3/4, moist, rootlets, fine grained.				
5	SK-1 R-2	3-5-5 (10)				100.8	21.7	R Value=58, Corr.	60
5	SPT-3	4-6-7 (13)							
10	R-4	6-8-11 (19)				92.2	22.5	Shear	55
10	SPT-5	4-7-8 (15)		SP	Poorly-graded SAND, medium dense, olive 5Y 5/4, moist, oxidized, fine grained.				
15	R-6	4-5-9 (14)							50
15	SPT-7	3-3-4 (7)							
20				ML	SILT, firm, black 5Y 2.5/1 to dark bluish gray 5B 5/1, very moist to moist, some fine sand.				45
20	R-8	3-5-9 (14)		CL	CLAY, stiff, bluish gray 5B 5/1, moist, some fine sand.	104.8	21.3	Consol, Shear LL= 28 PL= 20 PI= 8	40
25	SPT-9	4-5-7 (12)		SP-SM	SAND to Silty SAND, medium dense, pale olive 5Y 6/4, moist, fine grained, layered, micaceous and silty in some layers.				
30	R-10	14-26-32 (58)			...@30ft becomes dense	107.3	4.0		35
35									30

Continued on next page

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BORING B-3

Sheet 2 of 2

PROJECT NAME Simms Park CLIENT Craftwater GROUND ELEVATION 64 ft
PROJECT NUMBER 197-4703-0005 PROJECT LOCATION Bellflower, CA GROUNDWATER DEPTH 35.4 ft

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	 Standard Penetration Test (SPT)  California-Type Ring Sample  No Recovery  No Recovery  Shelby Tube  Grab/Bulk Sample	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
35	 SPT-11	4-10-13 (23)		SM	Silty SAND, medium dense, dark gray N 4/1, wet, fine grained.					25
40	 R-12	8-10-13 (23)		ML	Sandy SILT, very stiff, greenish gray 5G 5/1, wet, fine sand.	95.3	28.1	LL= 33 PL= 29 PI= 4		20
45	 SPT-13	5-5-6 (11)		SP- SM	SAND to Silty SAND, medium dense, dark gray N 4/1, wet, layered, fine grained.					15
50	 R-14	11-16-27 (43)				110.4	20.9			

Total Depth: 51.5 ft
Groundwater encountered during drilling at 35.4 feet.
Backfill with cement bentonite grout, upper 5 feet soil replaced for landscaping.
Cuttings spread onsite.



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BORING B-4

Sheet 1 of 1

PROJECT NAME Simms Park CLIENT Craftwater GROUND ELEVATION 67 ft
PROJECT NUMBER 197-4703-0005 PROJECT LOCATION Bellflower, CA GROUNDWATER DEPTH Not encountered
DATES DRILLED 5/3/2022 DRILLING CONTRACTOR MR Drilling BOREHOLE DEPTH 31.5 ft
LOGGED/CHECKED BY SMS/FC DRILLING METHOD CME 75 HSA BOREHOLE DIAMETER 8 in
COORDINATES 33.884096, -118.132389 HAMMER DATA 140 lbs Autohammer, 30" drop BACKFILL grout
LOCATION DESCRIPTION east side of parking area

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOW/COUNT blows/6" (bpf)	GRAPHIC LOG	USCS	Standard Penetration Test (SPT) No Recovery Shelby Tube California-Type Ring Sample No Recovery Grab/Bulk Sample	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0						Surface is 4 inches of asphalt concrete over 6 inches of aggregate base.				
	SK-1			CL		[FILL] Sandy CLAY, firm, olive brown 2.5Y 4/3, moist, fine grained.				65
	SPT-2	2-3-3 (6)		ML		[NATIVE] Qyf Sandy SILT, firm, olive brown 2.5Y 4/3, moist, micaceous, layered.				
5							101.4	10.2	Corr. G/S/F/2μ=0/31/69/24	
	R-3	3-4-4 (8)								60
	SPT-4	3-3-5 (8)								
10										
	R-5	7-9-13 (22)		SM		Silty SAND, medium dense, light olive grey 5Y 6/2, moist, micaceous, fine grained, interbedded silt and sand, laminated to thinly bedded.	91.3	10.6	Shear	55
	SPT-6	3-5-6 (11)								
15						...@15 ft becomes dark brown 7.5Y 3/4, fine to medium grained.	120.3	12.5		50
	R-7	6-11-21 (32)								
20										
	SPT-8	5-7-8 (15)		ML		Sandy SILT, very stiff, dark brown 7.5Y 3/4, moist, micaceous, fine to medium sand.				45
				SM		Silty SAND, medium dense, dark brown, moist, fine grained.				
25						...(@25ft) becomes fine to medium grained.	101.6	16.7	Consol	40
	R-9	9-12-15 (27)								
30						...@30ft becomes light olive gray 5Y 6/2				
	SPT-10	10-13-14 (27)		SM						
Total Depth: 31.5 ft Groundwater not encountered during drilling. Backfill with cement bentonite grout, upper 5 feet tamped soil was replaced, asphalt cold patch to repair surface. Cuttings spread onsite.										

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BORING B-5

Sheet 1 of 1

PROJECT NAME Simms Park CLIENT Craftwater GROUND ELEVATION 66 ft
PROJECT NUMBER 197-4703-0005 PROJECT LOCATION Bellflower, CA GROUNDWATER DEPTH Not encountered
DATES DRILLED 5/3/2022 DRILLING CONTRACTOR MR Drilling BOREHOLE DEPTH 31.5 ft
LOGGED/CHECKED BY SMS/FC DRILLING METHOD CME 75 HSA BOREHOLE DIAMETER 8 in
COORDINATES 33.884034, -118.133736 HAMMER DATA 140 lbs Autohammer, 30" drop BACKFILL grout
LOCATION DESCRIPTION west side of parking area

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOW/COUNT blows/6" (bpf)	GRAPHIC LOG	USCS	Standard Penetration Test (SPT) California-Type Ring Sample No Recovery No Recovery Shelby Tube Grab/Bulk Sample	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0						Surface is 4 inches of asphalt concrete over 6 inches of aggregate base.				65
	SK-1	7-10-14 (24)		SM		[FILL] Silty SAND to Sandy SILT, medium dense, olive brown 2.5Y 4/3, moist, trace gravel.	99.2	11.4	Max=122.0@11.7%, R Value=56 G/S/F/2μ=0/36/64/17	
	R-2			ML		[NATIVE] Qyf SILT, very stiff, pale olive 5Y 6/4, slightly moist, rootlets.				
5										
	SPT-3	3-5-6 (11)		ML		SILT to Sandy SILT, stiff, light olive gray 5Y 6/2 to pale olive 5Y 6/4, slightly moist, very fine sand, rootlets.				60
	R-4	6-10-19 (29)				...@7.5ft becomes very stiff	101.7	13.5		
10										
	SPT-5	4-5-5 (10)				...@10ft becomes stiff				55
	R-6	8-10-15 (25)		SM		Silty SAND, medium dense, light olive gray 5Y 6/2, moist, very fine sand, micaceous, rootlets, some oxidation around roots.	94.6	9.2		
15										
	SPT-7	10-14-17 (31)				...@15ft becomes dense				50
20										
	R-8	3-4-5 (9)		ML		SILT, stiff, dark brown 7.5YR 3/4, very moist, some cohesion, trace gravel, some sand and clay, rootlets.	105.1	20.2		45
25										
	SPT-9	6-7-7 (14)		SM		Silty SAND, medium dense, light olive gray 5Y 6/2, moist, micaceous, fine sand.				40
30										
	R-10	10-15-22 (37)				...(@31 ft) becomes wet, no free water measured in boring.	104.5	22.1		35
Total Depth: 31.5 ft Groundwater not encountered during drilling. Backfill with cement bentonite grout, upper 5 feet tamped soil was replaced, asphalt cold patch to repair surface. Cuttings spread onsite.										

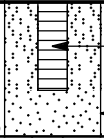


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WELL P-3

Sheet 1 of 1

PROJECT NAME Simms Park **CLIENT** Craftwater **GROUND ELEVATION** 67 ft
PROJECT NUMBER 197-4703-0005 **PROJECT LOCATION** Bellflower, CA **GROUNDWATER DEPTH** Not encountered
DATES DRILLED 5/3/2022 **DRILLING CONTRACTOR** MR Drilling **BOREHOLE DEPTH** 3.5 ft
LOGGED/CHECKED BY SMS/FC **DRILLING METHOD** CME 75 HSA **BOREHOLE DIAMETER** 8 in
COORDINATES 33.884476, -118.132289 **HAMMER DATA** Not Applicable **BACKFILL** cuttings
LOCATION DESCRIPTION northeast portion of parking area

DEPTH (ft)	SAMPLE TYPE NUMBER	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	WELL DIAGRAM
0					
		ML		Sandy SILT, pale olive 5Y 6/4, moist, fine sand.	 2" diameter schedule 40 PVC with 0.020 inch slotted screen #1 Rock backfill
				Bottom of borehole at 3.5 feet.	

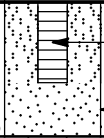


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WELL P-4

Sheet 1 of 1

PROJECT NAME Simms Park **CLIENT** Craftwater **GROUND ELEVATION** 66 ft
PROJECT NUMBER 197-4703-0005 **PROJECT LOCATION** Bellflower, CA **GROUNDWATER DEPTH** Not encountered
DATES DRILLED 5/3/2022 **DRILLING CONTRACTOR** MR Drilling **BOREHOLE DEPTH** 3.5 ft
LOGGED/CHECKED BY SMS/FC **DRILLING METHOD** CME 75 HSA **BOREHOLE DIAMETER** 8 in
COORDINATES 33.884128, -118.133090 **HAMMER DATA** Not Applicable **BACKFILL** cuttings
LOCATION DESCRIPTION central portion of parking area

DEPTH (ft)	SAMPLE TYPE NUMBER	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	WELL DIAGRAM
0					
		ML		Sandy SILT, pale olive 5Y 6/4, moist, fine sand.	 2" diameter schedule 40 PVC with 0.020 inch slotted screen #1 Rock backfill
			3.5	Bottom of borehole at 3.5 feet.	



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WELL P-5

Sheet 1 of 1

PROJECT NAME Simms Park **CLIENT** Craftwater **GROUND ELEVATION** 65 ft
PROJECT NUMBER 197-4703-0005 **PROJECT LOCATION** Bellflower, CA **GROUNDWATER DEPTH** Not encountered
DATES DRILLED 5/3/2022 **DRILLING CONTRACTOR** MR Drilling **BOREHOLE DEPTH** 3.5 ft
LOGGED/CHECKED BY SMS/FC **DRILLING METHOD** CME 75 HSA **BOREHOLE DIAMETER** 8 in
COORDINATES 33.883631, -118.133359 **HAMMER DATA** Not Applicable **BACKFILL** cuttings
LOCATION DESCRIPTION southwest portion of parking area

DEPTH (ft)	SAMPLE TYPE NUMBER	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	WELL DIAGRAM
0					
		ML		Sandy SILT, pale olive 5Y 6/4, moist, fine sand.	
				3.5	61.5
				Bottom of borehole at 3.5 feet.	2" diameter schedule 40 PVC with 0.020 inch slotted screen #1 Rock backfill

Appendix B1

Results of Previous Laboratory Testing

MOISTURE CONTENT AND DENSITY

ASTM D2937

Job Name: Simms Park Bellflower
Job Number: 197-4703-0001
Tested By: MG

Date Sampled: 4/27/2020
Date Completed: 5/2/2020
Note: Page 1 of 2

Boring / Test Pit / Trench		B-1	B-1	B-1	B-1	B-1	B-1	B-1	
Sample Number		R-1	R-4	R-6	R-8	R-10	R-12	R-14	
Sample Depth	feet	2.5ft	7.5ft	15ft	22.5ft	30ft	40ft	50ft	
USCS Soil Description		Olive Brown Silty SAND, (Qal)	Olive Poorly Graded SAND, (Qal)	Olive Silty SAND, (Qal)	Olive Silty SAND, (Qal)	Greenish Gray Poorly Graded SAND Native (Qal)	Greenish Gray Silty SAND Native (Qal)	Greenish Gray Silty SAND Native (Qal)	
Number of Rings		6	6	6	6	6	6	6	
Total Weight Rings + Soil	grams	1190.00	1006.50	1077.90	1107.70	1114.30	1169.50	1160.70	
* Volume of Rings	ft ³	0.0159	0.0159	0.0159	0.0159	0.0159	0.0159	0.0159	
* Weight of Rings	grams	265.80	265.80	265.80	265.80	265.80	265.80	265.80	
* Weight of Soil	grams	924.20	740.70	812.10	841.90	848.50	903.70	894.90	
* Wet Density	pcf	127.79	102.41	112.29	116.41	117.32	124.95	123.73	
Container ID		Z12	X4	X17	BB-1	X55	X27	X36	
Tare	grams	10.5	10.5	10.5	24.2	10.5	10.5	10.5	
Wet Soil + Tare	grams	295.1	248.4	300.7	415.6	296.1	276.5	274	
Dry Soil + Tare	grams	258.5	230	268.7	381.1	259.4	219.6	216.1	
* Weight of Water	grams	36.6	18.4	32	34.5	36.7	56.9	57.9	

* Dry Density	pcf	111.4	94.5	99.9	106.1	102.2	98.2	96.5	
* Moisture Content	%	14.8	8.4	12.4	9.7	14.7	27.2	28.2	

MOISTURE CONTENT AND DENSITY

ASTM D2937

Job Name: Simms Park Bellflower
Job Number: 197-4703-0001
Tested By: MG

Date Sampled: 4/27/2020
Date Completed: 5/2/2020
Note: Page 2 of 2

Boring / Test Pit / Trench		B-2	B-2	B-2	B-2	B-2	B-2		
Sample Number		R-3	R-5	R-7	R-9	R-11	R-13		
Sample Depth		5ft	10ft	20ft	25ft	35ft	45ft		
USCS Soil Description		Olive Brown Sandy SILT (Qal)	Greenish Gray Poorly Graded SAND (Qal)	Light Brownish Gray Silty SAND (Qal)	Olive Yellow Poorly Graded SAND (Qal)	Greenish Gray Poorly Graded SAND (Qal)	Greenish Gray SILT (Qal)		
Number of Rings		6	6	6	6	5	6		
Total Weight Rings + Soil		grams	1196.20	1004.20	1213.90	1018.30	806.70	1117.60	
* Volume of Rings		ft ³	0.0159	0.0159	0.0159	0.0159	0.0133	0.0159	
* Weight of Rings		grams	265.80	265.80	265.80	265.80	221.50	265.80	
* Weight of Soil		grams	930.40	738.40	948.10	752.50	585.20	851.80	
* Wet Density		pcf	128.64	102.10	131.09	104.05	97.10	117.78	
Container ID		Z54	X40	E5	X26	X52	X56		
Tare		grams	10.5	10.5	29	10.5	10.5	10.5	
Wet Soil + Tare		grams	291.2	265.3	487.4	238.1	274.9	288.3	
Dry Soil + Tare		grams	246.9	250.7	432.4	228.1	263.3	209	
* Weight of Water		grams	44.3	14.6	55	10	11.6	79.3	
* Dry Density		pcf	108.3	96.2	115.4	99.5	92.8	84.2	
* Moisture Content		%	18.7	6.1	13.6	4.6	4.6	39.9	

GRAIN SIZE DISTRIBUTION ANALYSIS

ASTM C136/C117/D7928

Job Name: Simms Park Bellflower

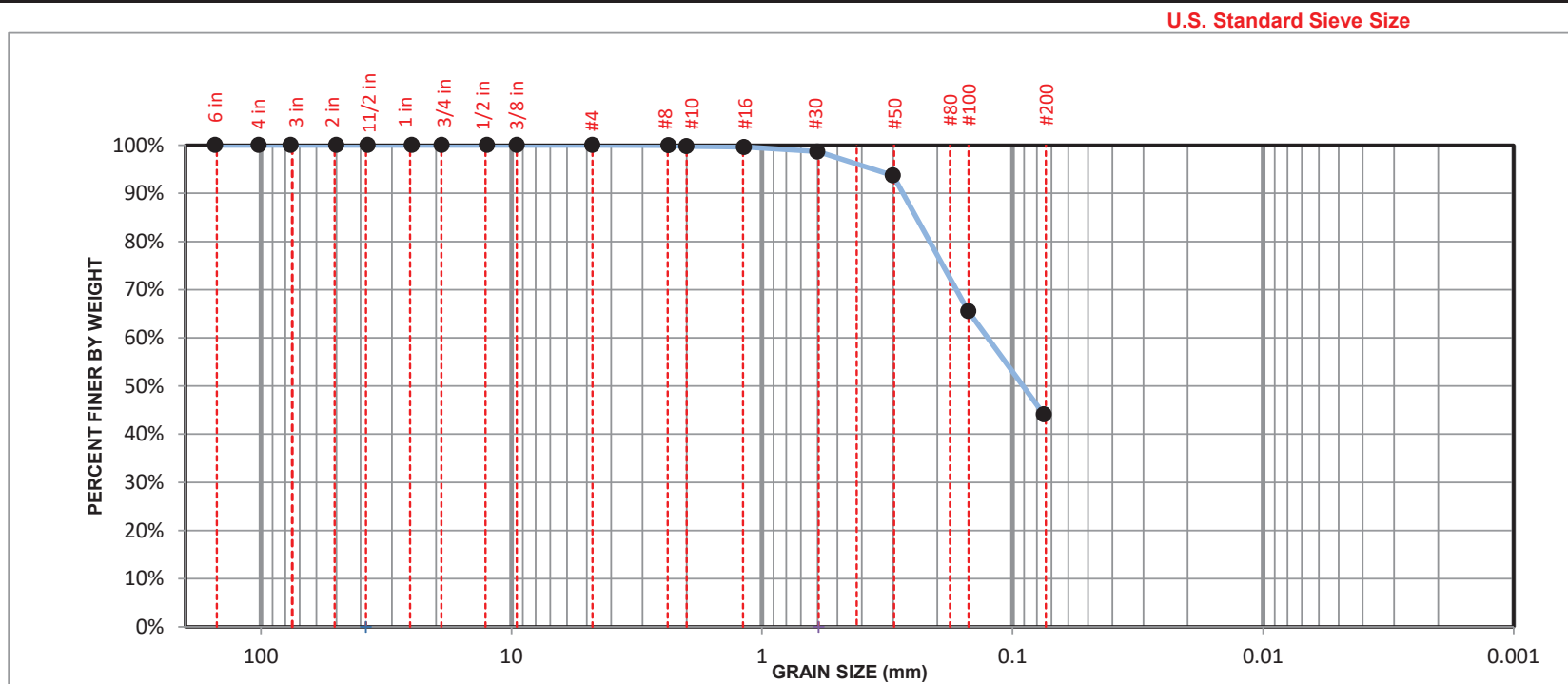
Tested By : MG

Job Number: 197-4703-0001

Date Completed: May 2, 2020

Address:

Date Sampled: April 27, 2020



Symbol	Boring No.	Sample #	Depth (feet)	LL	PI	USCS	Cobbles	Gravel	Sand	Fines	2 μ
●	B-1	SK-1	0-5ft	-	-	SM	0%	0%	56%	44%	

GRAIN SIZE DISTRIBUTION ANALYSIS

ASTM C136/C117/D7928

Job Name: Simms Park Bellflower

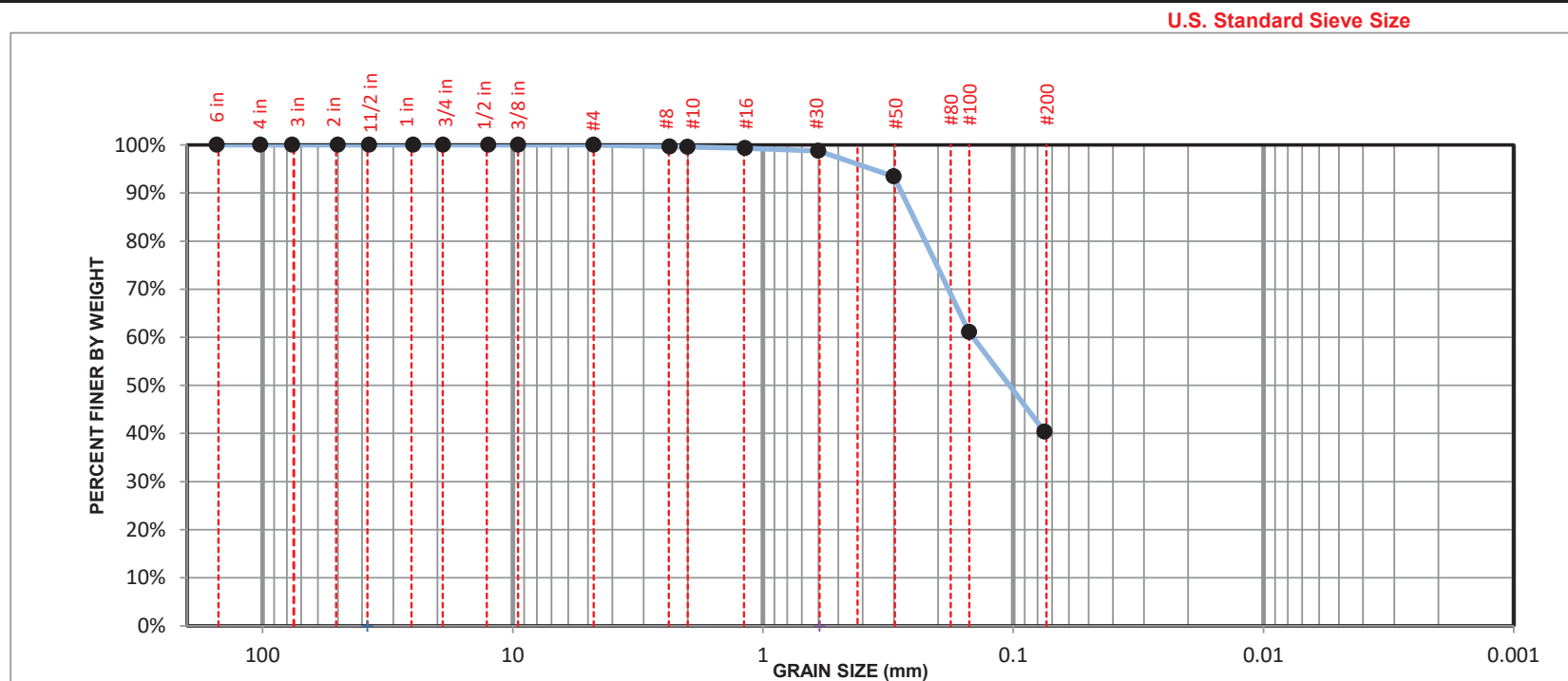
Tested By : MG

Job Number: 197-4703-0001

Date Completed: May 2, 2020

Address:

Date Sampled: April 27, 2020



Symbol	Boring No.	Sample #	Depth (feet)	LL	PI	USCS	Cobbles	Gravel	Sand	Fines	2 μ
●	B-1	SPT-7	20ft	-	-	SM	0%	0%	60%	40%	

GRAIN SIZE DISTRIBUTION ANALYSIS

ASTM C136/C117/D7928

Job Name: Simms Park Bellflower

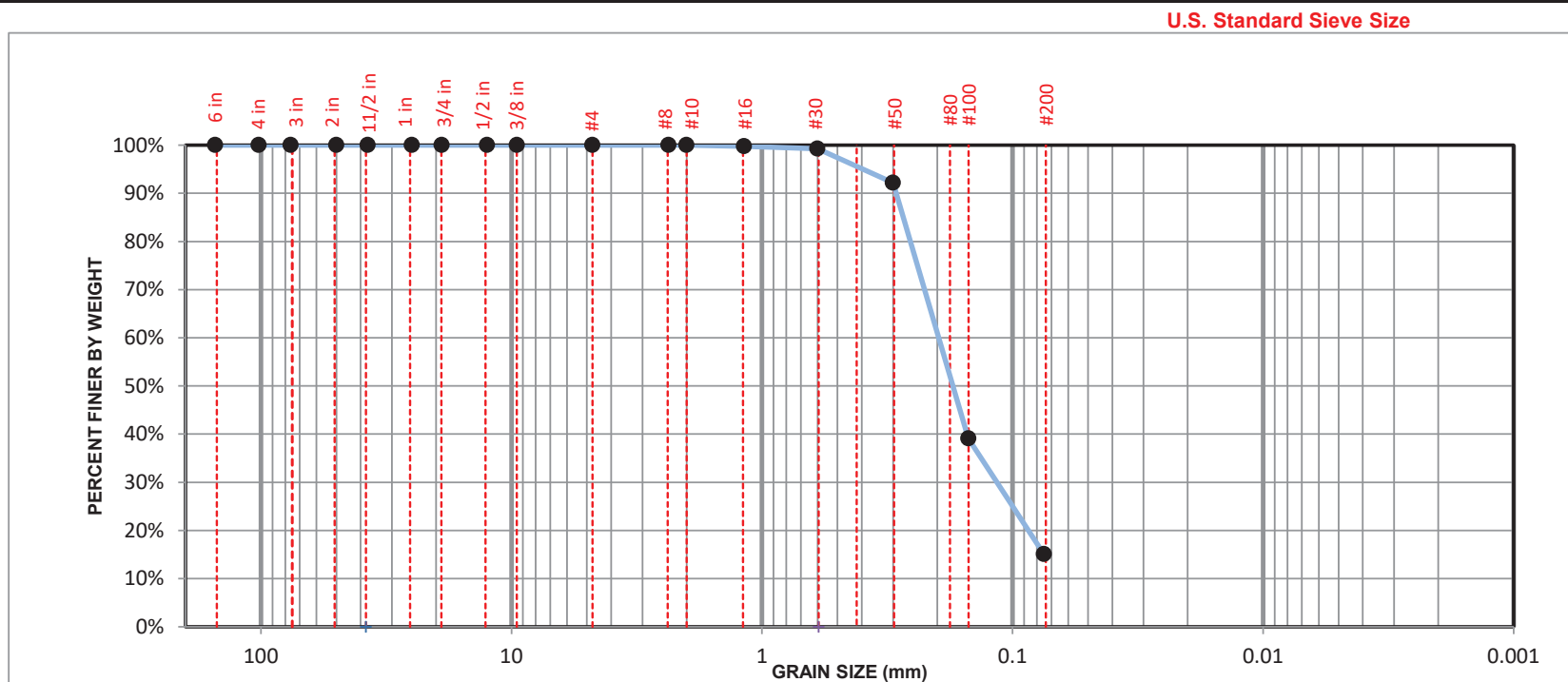
Tested By : MG

Job Number: 197-4703-0001

Date Completed: May 2, 2020

Address:

Date Sampled: April 27, 2020



Symbol	Boring No.	Sample #	Depth (feet)	LL	PI	USCS	Cobbles	Gravel	Sand	Fines	2 μ
●	B-1	R-8	22.5ft	-	-	SM	0%	0%	85%	15%	

GRAIN SIZE DISTRIBUTION ANALYSIS

ASTM C136/C117/D7928

Job Name: Simms Park Bellflower

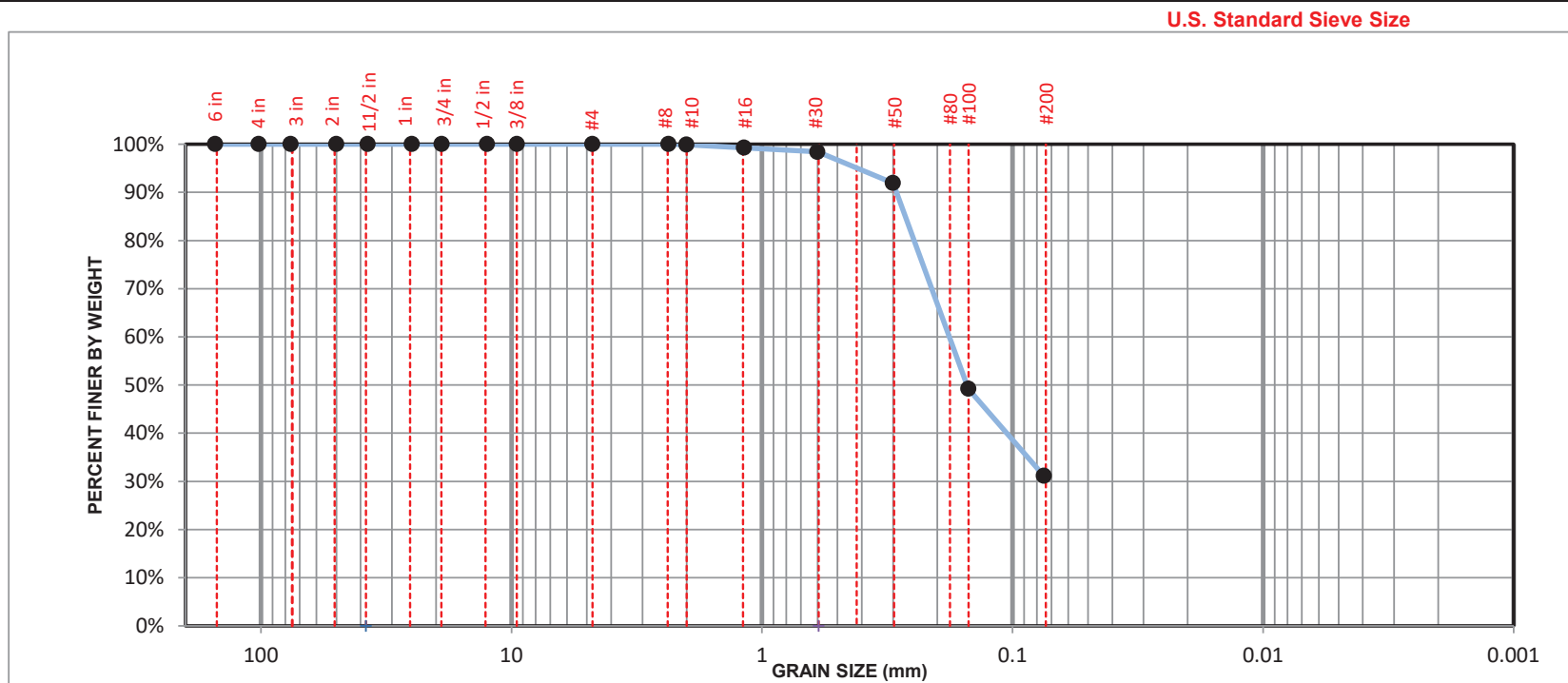
Tested By : MG

Job Number: 197-4703-0001

Date Completed: May 2, 2020

Address:

Date Sampled: April 27, 2020



Symbol	Boring No.	Sample #	Depth (feet)	LL	PI	USCS	Cobbles	Gravel	Sand	Fines	2 μ
●	B-2	R-7	20ft	-	-	SM	0%	0%	69%	31%	

GRAIN SIZE DISTRIBUTION ANALYSIS

ASTM C136/C117/D7928

Job Name: Simms Park Bellflower

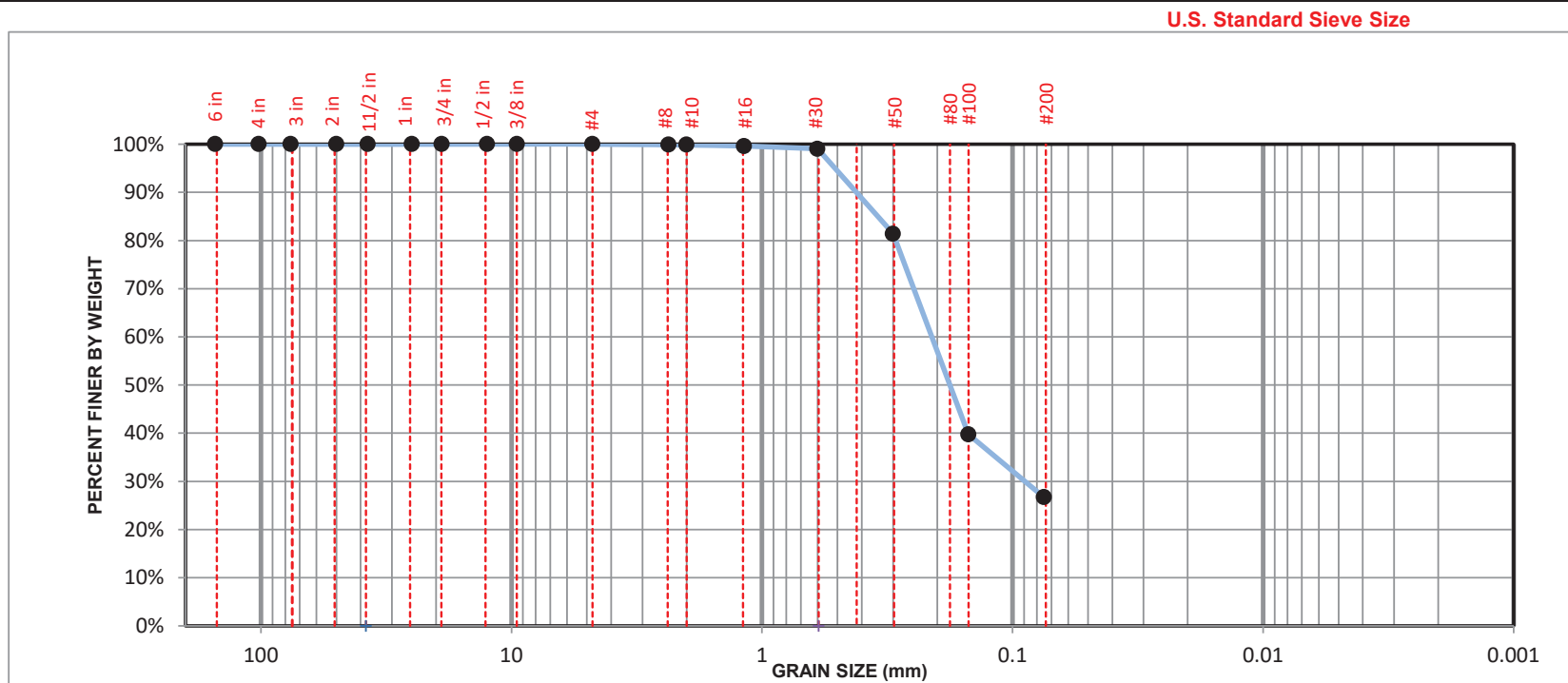
Tested By : MG

Job Number: 197-4703-0001

Date Completed: May 2, 2020

Address:

Date Sampled: April 27, 2020



Symbol	Boring No.	Sample #	Depth (feet)	LL	PI	USCS	Cobbles	Gravel	Sand	Fines	2 μ
●	B-2	SPT-8	22.5ft	-	-	SM	0%	0%	73%	27%	

ATTERBERG LIMITS

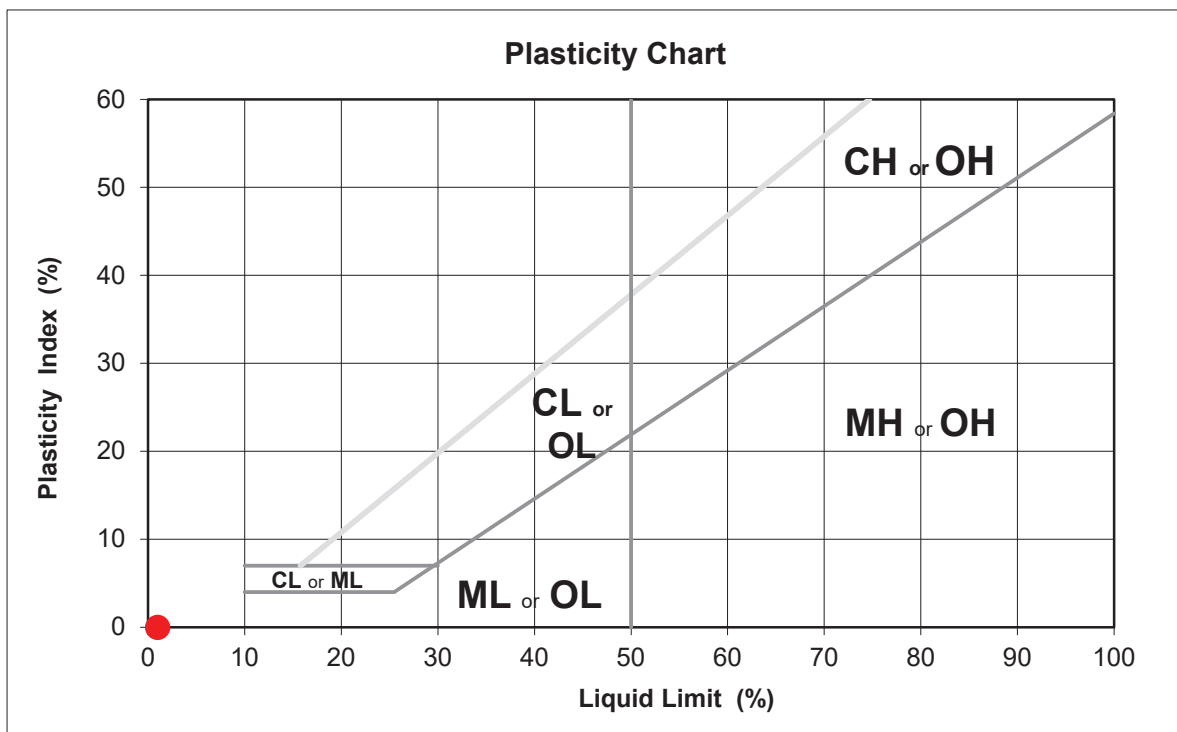
ASTM D4318

Job Name: Simms Park Bellflower
Job Number: 197-4703-0001
Tested By: MG
Note:
Sample Description: Greenish Gray Silty SAND, Qal

Date Sampled: 4/27/2020
Date Completed: 5/3/2020
Sample Identification: B-1 R-12
Sample Depth: 40ft

		PLASTIC LIMIT	
Test No.		1	2
Number of Blows			
Container ID			
Wet Weight of Soil + Cont.	grams	NP	NP
Dry Weight of Soil + Cont.	grams		
Weight of Container	grams		
* Moisture Weight	grams	0.00	0.00
* Weight of Dry Soil	grams	0.00	0.00
* Moisture Content	%	NP	NP

LIQUID LIMIT			
1	2	3	4
NP	NP	NP	
0.00	0.00	0.00	
0.00	0.00	0.00	
NP	NP	NP	



Plastic Limit NP
Liquid Limit NP
Plasticity Index NP

USCS Classification SM

ATTERBERG LIMITS

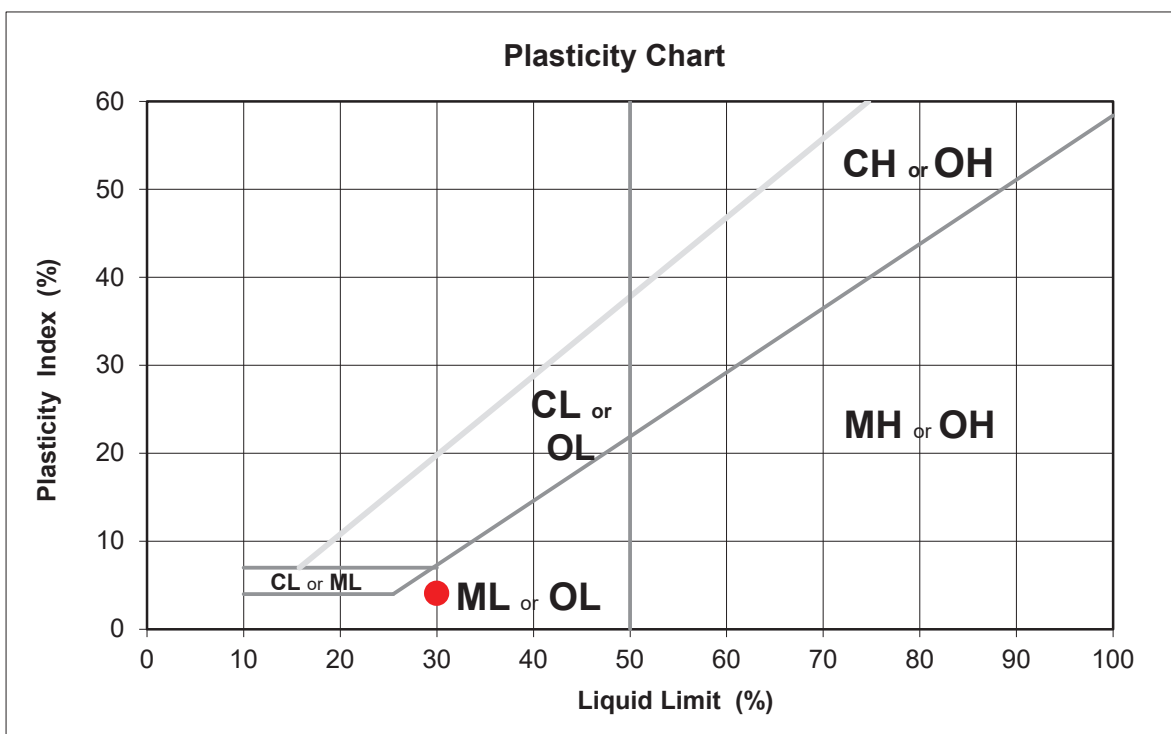
ASTM D4318

Job Name: Simms Park Bellflower
Job Number: 197-4703-0001
Tested By: MG
Note:
Sample Description: Olive Sandy SILT

Date Sampled: 4/27/2020
Date Completed: 5/3/2020
Sample Identification: B-2, R-3
Sample Depth: 5ft

		PLASTIC LIMIT	
Test No.		1	2
Number of Blows			
Container ID		P41	P9
Wet Weight of Soil + Cont.	grams	21.60	24.60
Dry Weight of Soil + Cont.	grams	19.70	22.10
Weight of Container	grams	12.40	12.40
* Moisture Weight	grams	1.90	2.50
* Weight of Dry Soil	grams	7.30	9.70
* Moisture Content	%	26.0	25.8

LIQUID LIMIT			
1	2	3	4
32	25	15	
P39	T23	T18	
61.20	58.10	44.50	
53.20	50.70	39.60	
25.30	26.00	25.60	
8.00	7.40	4.90	
27.90	24.70	14.00	
28.7	30.0	35.0	



Plastic Limit 26
Liquid Limit 30
Plasticity Index 4

USCS Classification ML

EXPANSION INDEX

ASTM D4829

Job Name: Simms Park Bellflower
Job Number: 197-4703-0001
Tested By: MG
Note:
Sample Description: Olive Brown Sandy Silt

Date Sampled: 4/27/2020
Date Completed: 5/3/2020
Sample Identification: B-1, SK-1
Sample Depth: 0-5ft

SAMPLE PROCESSING		
Percentage Passing #4 Sieve		
Total Air Dry Weight	2500.00	grams
Weight Retained on #4	1.00	grams
*% Retained	0.04	%
*% Passing # 4 Sieve	100.0	%

SAMPLE DIMENSIONS		
Sample Height	1.00	s
Sample Diameter	4.01	s

MOISTURE CALCS		INITIAL	FINAL
Tare ID or #		Z15	5A
Wet Weight of Soil + Tare		grams 173.50	617.30
Dry Weight of Soil + Tare		grams 156.30	546.00
Weight of Tare		grams 4.00	191.10
* Moisture Weight		grams 17.20	71.30
* Weight of Dry Soil		grams 152.30	354.90
* Moisture Content		% 11.3	20.1

AFTER REMOLDING		INITIAL	FINAL
Weight of Ring and Sample		grams 609.2	609.2
Weight of Ring		grams 202.2	202.2
*Remolded Wet Weight		grams 407.0	407.0
*Wet Density		122.7	122.1
*Dry Density		110.3	101.7
Assumed/Measured Specific Gravity		2.7	2.7
*Degree of Saturation		% 57.7	82.6

EXPANSION INDEX TEST			
DATE	TIME	DIAL	Δ H %
5/1/2020	11:50 AM	0.0345	0.0
		0.0345	0.0
		0.0348	0.0
		0.0386	0.4
		0.0387	0.4
5/2/2020	1:00 PM	0.0391	0.5
5/2/2020	1:30 PM	0.0391	0.5
*Total			0.5

UNCORRECTED EXPANSION INDEX 5
CORRECTED EXPANSION INDEX 8
 For degrees of Saturation ≠ 50%, >40% and <60%

very Low

EXPANSION INDEX

ASTM D4829

Job Name: Simms Park Bellflower
Job Number: 197-4703-0001
Tested By: MG
Note:
Sample Description: Olive Brown Sandy Silt

Date Sampled: 4/27/2020
Date Completed: 5/3/2020
Sample Identification: B-2, SK-1
Sample Depth: 0-5ft

SAMPLE PROCESSING		
Percentage Passing #4 Sieve		
Total Air Dry Weight	2500.00	grams
Weight Retained on #4	1.00	grams
*% Retained	0.04	%
*% Passing # 4 Sieve	100.0	%

SAMPLE DIMENSIONS		
Sample Height	1.00	s
Sample Diameter	4.01	s

MOISTURE CALCS	INITIAL	FINAL
Tare ID or #	Z17	P-1
Wet Weight of Soil + Tare	grams 200.80	564.90
Dry Weight of Soil + Tare	grams 179.60	488.70
Weight of Tare	grams 4.00	149.80
* Moisture Weight	grams 21.20	76.20
* Weight of Dry Soil	grams 175.60	338.90
* Moisture Content	% 12.1	22.5

AFTER REMOLDING	INITIAL	FINAL
Weight of Ring and Sample	grams 592.9	592.9
Weight of Ring	grams 198.7	198.7
*Remolded Wet Weight	grams 394.2	394.2
*Wet Density	118.8	118.1
*Dry Density	106.0	96.4
Assumed/Measured Specific Gravity	2.7	2.7
*Degree of Saturation	% 55.4	81.2

EXPANSION INDEX TEST			
DATE	TIME	DIAL	Δ H %
5/1/2020	1:00 PM	0.0434	0.0
	1:10 PM	0.0434	0.0
		0.0448	0.1
		0.0450	0.2
		0.0487	0.5
5/2/2020	1:00 PM	0.0505	0.7
5/2/2020	1:30 PM	0.0500	0.7
*Total			0.7

UNCORRECTED EXPANSION INDEX

7

CORRECTED EXPANSION INDEX
9

For degrees of Saturation ≠ 50%, >40% and <60%

very Low

Appendix B2

Results of Current Laboratory Testing



MOISTURE CONTENT AND DENSITY

ASTM D7263

Job Name:	Simms Park Belflower	Date Sampled:	5/9/2022
Job Number:	CWT 22-05	Date Completed:	5/16/2022
Tested By:	MG	Note:	Page 1 of 2

Boring / Test Pit / Trench		B-3	B-3	B-3	B-3	B-4	B-4	B-5	B-5	B-5
Sample Number		R-2	R-10	R-12	R-14	R-3	R-7	R-2	R-4	R-6
Sample Depth	ft	2.5	30	40	50	5	15	2.5	7.5	12.5
USCS Soil Description		Olive Gray Qal, SM	Light Olive Gray Qal, SM	Olive Gray Qal, SM	Olive Gray Qal, SM	Olive Brown Qal, SM	Olive Gray Qal, SM	Olive Gray Qal, SM	Olive Gray Qal, SM	Olive Gray Qal, SP
Number of Rings		6	6	6	6	6	6	6	6	6
Total Weight Rings + Soil	grams	1144.40	1063.20	1139.00	1221.60	1064.60	1235.70	1055.60	1091.80	1003.80
Volume of Rings	ft ³	0.0159	0.0159	0.0159	0.0159	0.0159	0.0159	0.0159	0.0159	0.0159
Weight of Rings	grams	256.46	256.46	256.46	256.46	256.46	256.46	256.46	256.46	256.46
Weight of Soil	grams	887.94	806.74	882.54	965.14	808.14	979.24	799.14	835.34	747.34
Wet Density	pcf	122.77	111.55	122.03	133.45	111.74	135.40	110.49	115.50	103.33
Container ID		X46	X31	X53	X38	X12	X21	E5	X24	X33
Tare	grams	10.5	10.5	10.5	10.5	10.5	10.5	29.2	10.5	10.5
Wet Soil + Tare	grams	327.9	243.1	294.9	355.2	260.7	342.4	529.2	282.2	234.9
Dry Soil + Tare	grams	271.2	234.2	232.5	295.7	237.5	305.4	478.2	249.8	216
Weight of Water	grams	56.7	8.9	62.4	59.5	23.2	37	51	32.4	18.9

Dry Density	pcf	100.8	107.3	95.3	110.4	101.4	120.3	99.2	101.7	94.6
Moisture Content	%	21.7	4.0	28.1	20.9	10.2	12.5	11.4	13.5	9.2



MOISTURE CONTENT AND DENSITY

ASTM D7263

Job Name:	Simms Park Belflower	Date Sampled:	5/9/2022
Job Number:	CWT 22-05	Date Completed:	5/16/2022
Tested By:	MG	Note:	Page 2 of 2

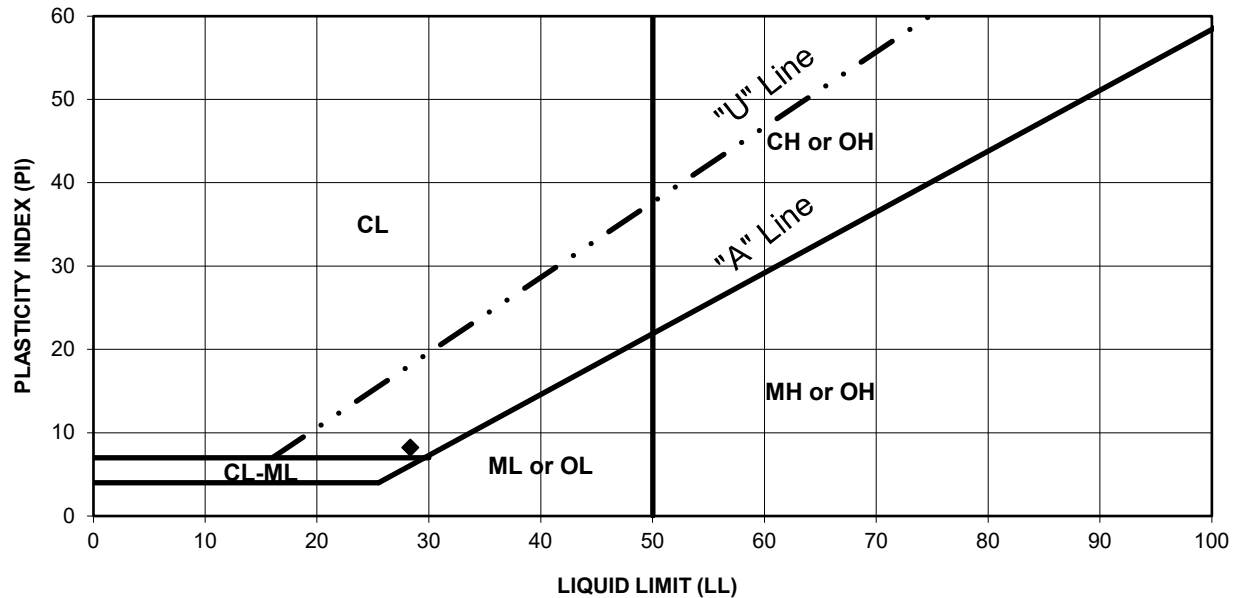
Boring / Test Pit / Trench		B-5	B-5							
Sample Number		R-8	R-10							
Sample Depth	ft	20	30							
USCS Soil Description		Olive Gray Qal, ML	Light Olive Gray Qal, SP							
Number of Rings		6	6							
Total Weight Rings + Soil	grams	1170.50	1178.90							
Volume of Rings	ft ³	0.0159	0.0159							
Weight of Rings	grams	256.46	256.46							
Weight of Soil	grams	914.04	922.44							
Wet Density	pcf	126.38	127.54							
Container ID		X30	X26							
Tare	grams	10.5	10.5							
Wet Soil + Tare	grams	309.2	298.2							
Dry Soil + Tare	grams	259	246.2							
Weight of Water	grams	50.2	52							

Dry Density	pcf	105.1	104.5							
Moisture Content	%	20.2	22.1							



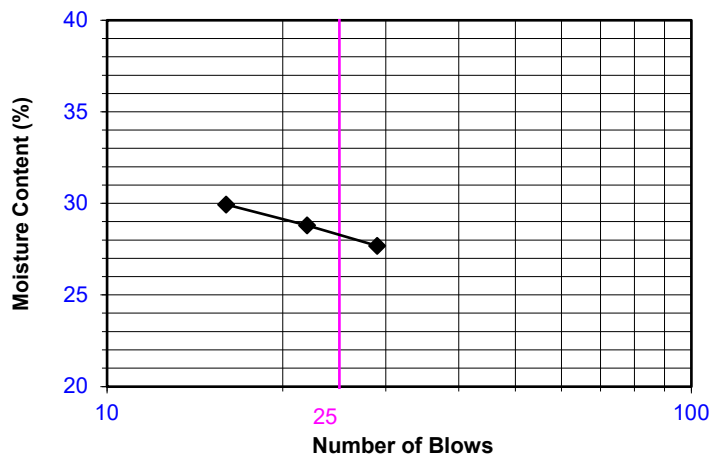
ATTERBERG LIMITS ASTM D 4318

Client Name: Tetra Tech Tested By: LS Date: 05/20/22
Project Name: Simms Park, Bellflower Computed By: NR Date: 05/23/22
Project No.: CWT22-05 Checked By: AP Date: 05/24/22



PROCEDURE USED

- ☐ Wet Preparation
- ☒ Dry Preparation
- ☒ Procedure A
Multipoint Test
- ☐ Procedure B
One-point Test



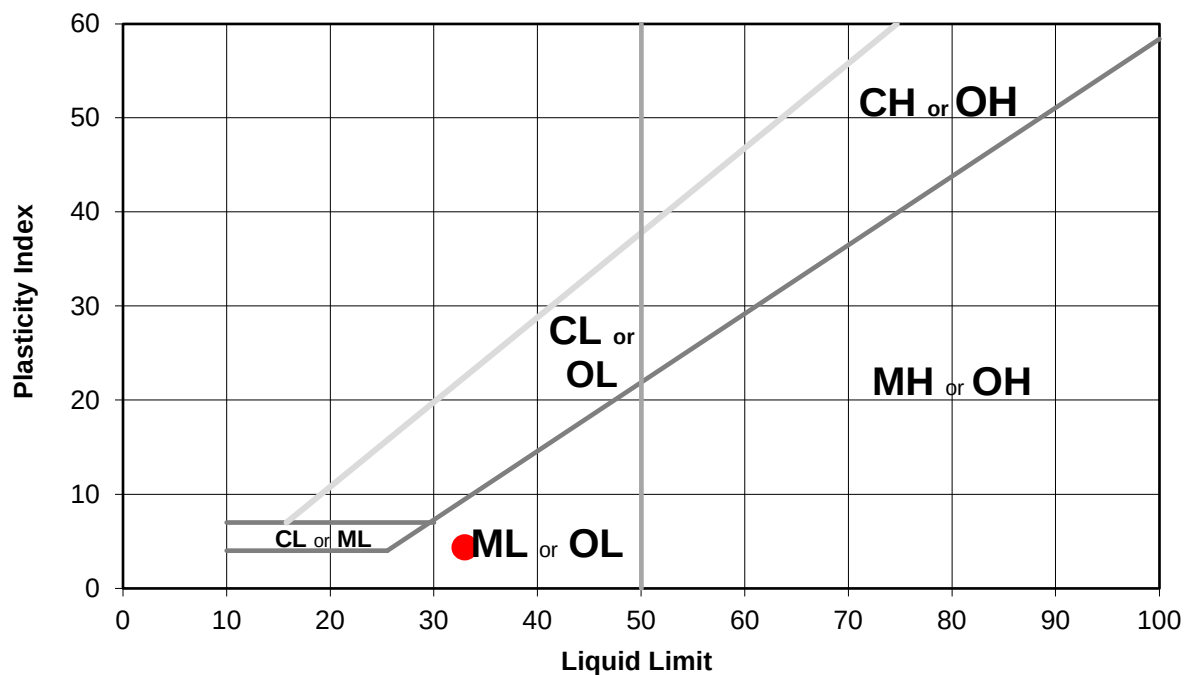
Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
◆	B-3	R-8	20	28	20	8	CL



Job Name:	Simms Park Belflower	Date Sampled:	5/9/2022
Job Number:	CWT22-05	Date Completed:	5/16/2022
Tested By:	MG	Sample Identification:	B-3, R-12
Note:		Sample Depth:	40ft
Sample Description:	Olive Gray ML Qal		

		Plastic Limit	
Test No.		1	2
Number of Blows			
Container ID		P1	P5
Wet Weight of Soil + Cont.	grams	22.40	22.60
Dry Weight of Soil + Cont.	grams	20.20	20.30
Weight of Container	grams	12.40	12.40
Moisture Weight	grams	2.20	2.30
Weight of Dry Soil	grams	7.80	7.90
Moisture Content	%	28.2	29.1

Liquid Limit			
1	2	3	4
16	23	32	
M14	P13	S12	
49.00	44.50	52.40	
42.60	39.70	46.30	
25.40	25.40	25.60	
6.40	4.80	6.10	
17.20	14.30	20.70	
37.2	33.6	29.5	

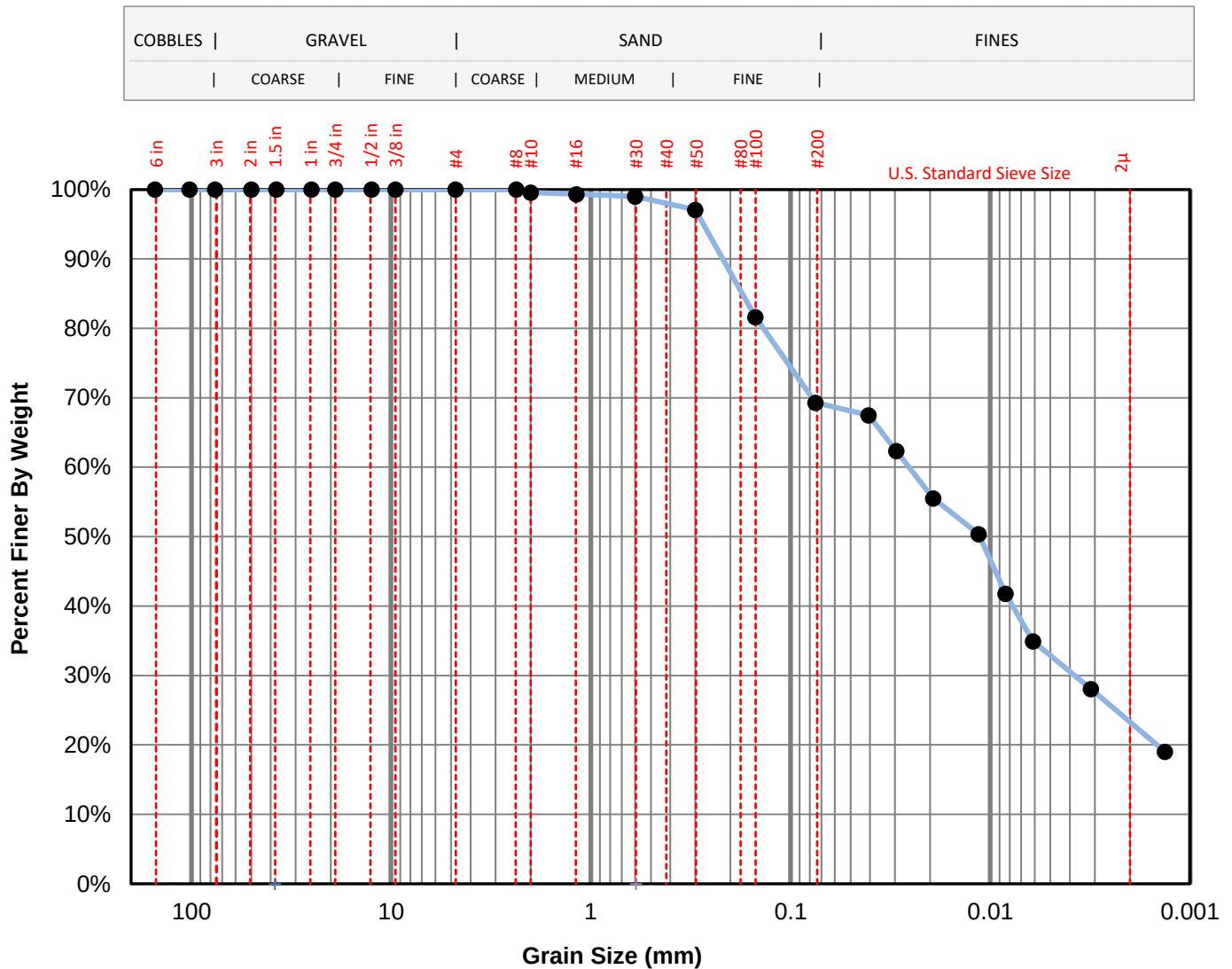


Plastic Limit **29**
 Liquid Limit **33**
 Plasticity Index **4**

USCS Classification **ML**
 Based on Atterberg Limits only



Job Name:	Simms Park Belflower	Date Sampled:	5/9/2022
Job Number:	CWT-22-05	Date Completed:	5/18/2022
Tested By:	MG	Sample Identification:	B-4 S-2
Note:		Sample Depth:	2.5ft
Sample Description:	Olive Gray Sandy SILT, ML		



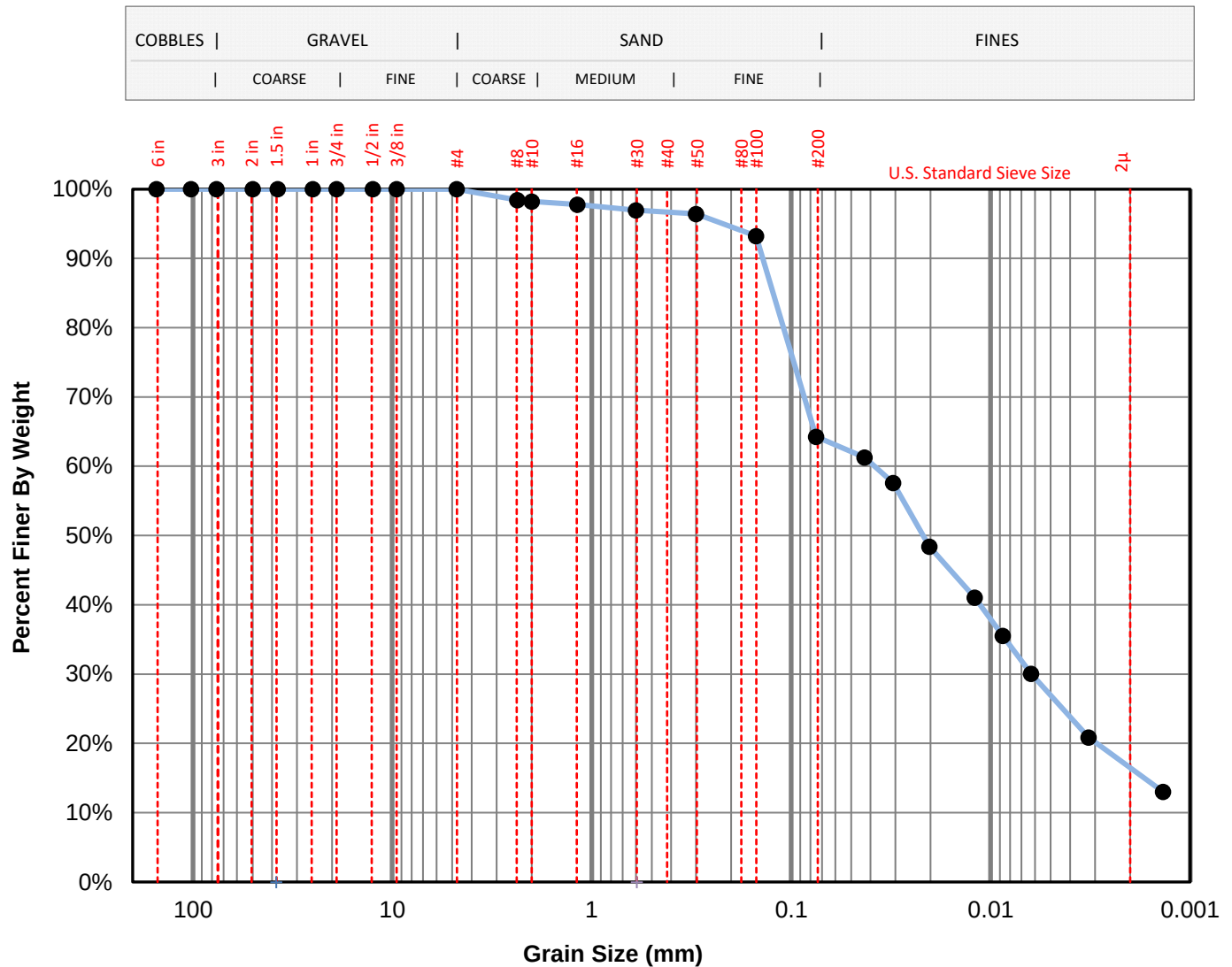
Symbol	Boring No.	Sample No.	Depth	LL	PI	USCS	Cobbles	Gravel	Sand	Fines	2 μ
●	B-4	S-2	2.5ft			ML	0%	0%	31%	69%	24%



GRAIN SIZE DISTRIBUTION ANALYSIS

ASTM C136/C117/D7928/D6913

Job Name:	Simms Park Belflower	Date Sampled:	5/9/2022
Job Number:	CWT-22-05	Date Completed:	5/18/2022
Tested By:	MG	Sample Identification:	B-5, R-2
Note:		Sample Depth:	2.5ft
Sample Description:	Olive Gray Sandy SILT, ML		



Symbol	Boring No.	Sample No.	Depth	LL	PI	USCS	Cobbles	Gravel	Sand	Fines	2 μ
●	B-5	R-2	2.5ft			ML	0%	0%	36%	64%	17%



COMPACTION TEST

Client: Tetra Tech
 Project Name: Simms Park, Bellflower
 Project No.: CWT22-05
 Boring No.: B-5
 Sample No.: SK-1
 Visual Sample Description: Sandy Silt

AP Number: 22-0526
 Tested By: TV Date: 05/19/22
 Calculated By: NR Date: 05/20/22
 Checked By: AP Date: 05/24/22
 Depth (ft.): 0-5

METHOD A
 MOLD VOLUME (CU.FT) 0.0333

Compaction Method ☒ ASTM D1557
☐ ASTM D698
 Preparation Method ☐ Moist
☒ Dry

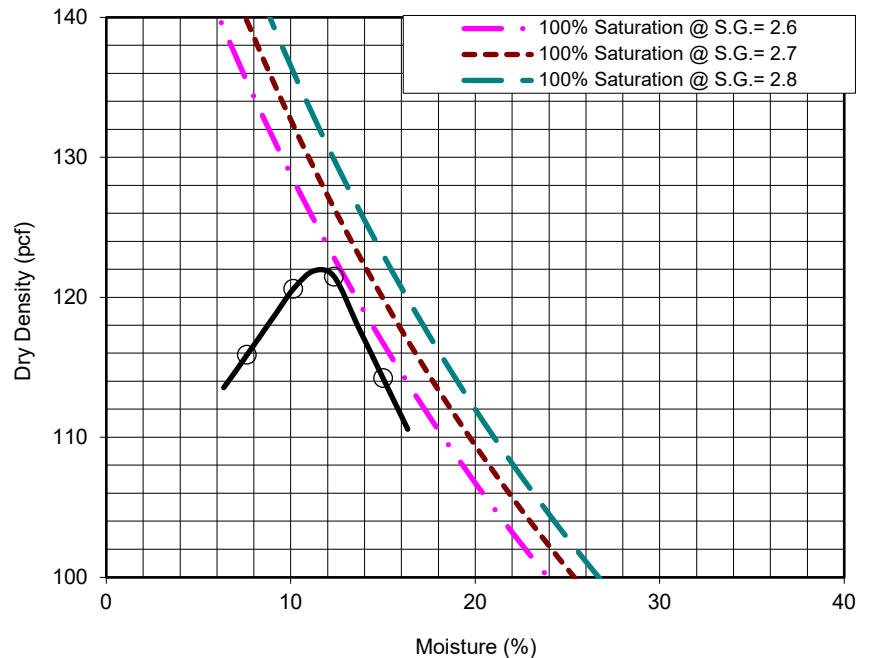
Wt. Comp. Soil + Mold (gm.)	3847	3902	3824	3724		
Wt. of Mold (gm.)	1838	1838	1838	1838		
Net Wt. of Soil (gm.)	2009	2064	1986	1886		
Container No.						
Wt. of Container (gm.)	144.63	181.50	135.88	145.25		
Wet Wt. of Soil + Cont. (gm.)	772.32	831.81	770.40	789.59		
Dry Wt. of Soil + Cont. (gm.)	714.54	760.38	687.64	743.95		
Moisture Content (%)	10.14	12.34	15.00	7.62		
Wet Density (pcf)	132.84	136.47	131.35	124.74		
Dry Density (pcf)	120.61	121.48	114.22	115.90		

Maximum Dry Density (pcf) 122.0
 Maximum Dry Density w/ Rock Correction (pcf) N/A

Optimum Moisture Content (%) 11.7
 Optimum Moisture Content w/ Rock Correction (%) N/A

PROCEDURE USED

- ☒ **METHOD A: Percent of Oversize:** 5.0%
 Soil Passing No. 4 (4.75 mm) Sieve
 Mold: 4 in. (101.6 mm) diameter
 Layers: 5 (Five)
 Blows per layer: 25 (twenty-five)
- ☐ **METHOD B: Percent of Oversize:** N/A
 Soil Passing 3/8 in. (9.5 mm) Sieve
 Mold: 4 in. (101.6 mm) diameter
 Layers: 5 (Five)
 Blows per layer: 25 (twenty-five)
- ☐ **METHOD C: Percent of Oversize:** N/A
 Soil Passing 3/4 in. (19.0 mm) Sieve
 Mold: 6 in. (152.4 mm) diameter
 Layers: 5 (Five)
 Blows per layer: 56 (fifty-six)



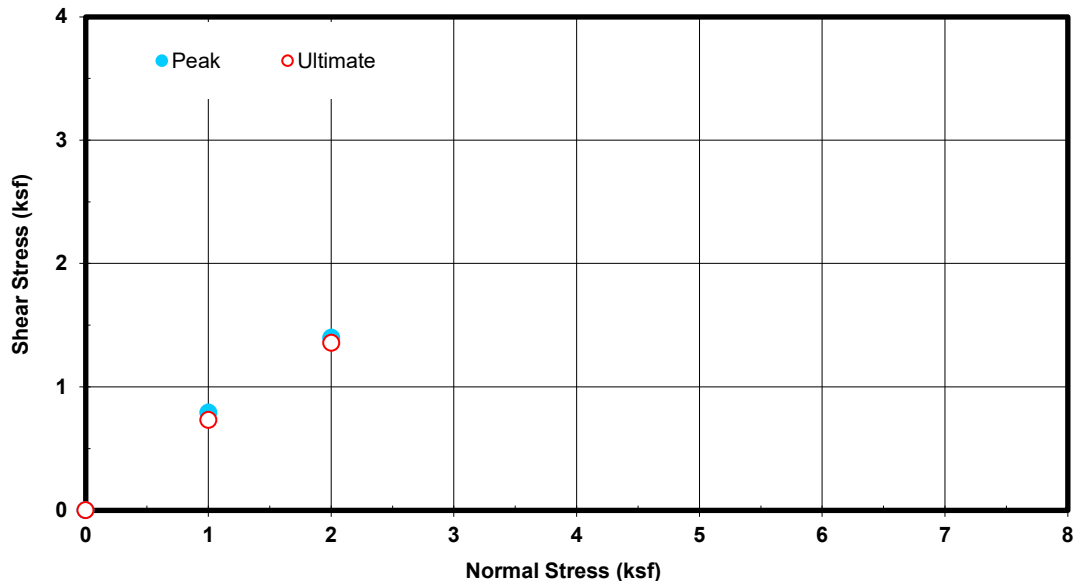
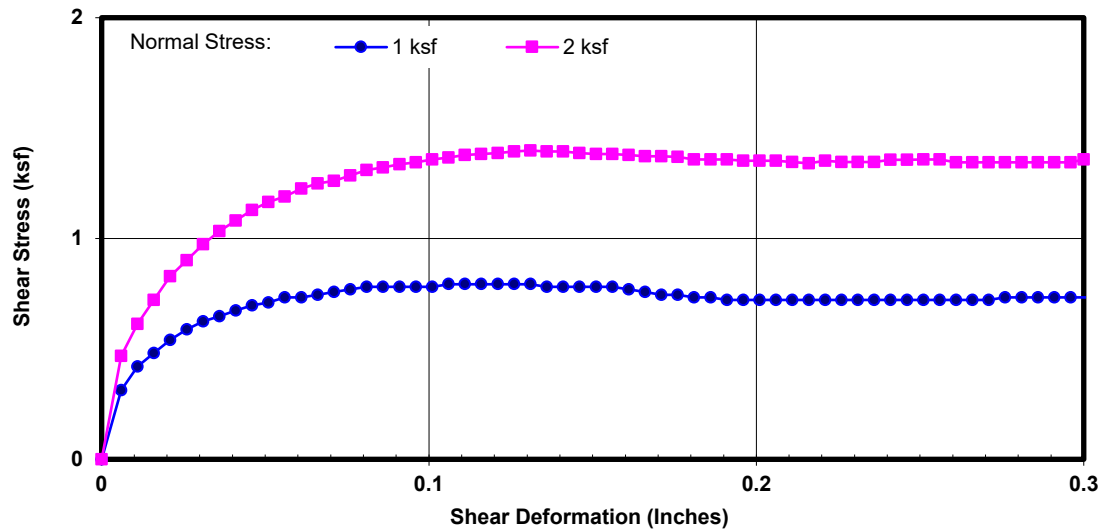


DIRECT SHEAR TEST RESULTS ASTM D 3080

Client: Tetra Tech
Project Name: Simms Park, Bellflower
Project No.: CWT22-05
Boring No.: B-3
Sample No.: R-4 Depth (ft): 7.5
Sample Type: Mod. Cal.
Soil Description: Sandy Silt
Test Condition: Inundated Shear Type: Regular

Tested By: LS Date: 05/17/22
Computed By: NR Date: 05/19/22
Checked by: AP Date: 05/24/22

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
112.8	92.2	22.5	30.4	73	99	1	0.793	0.733
						2	1.398	1.357



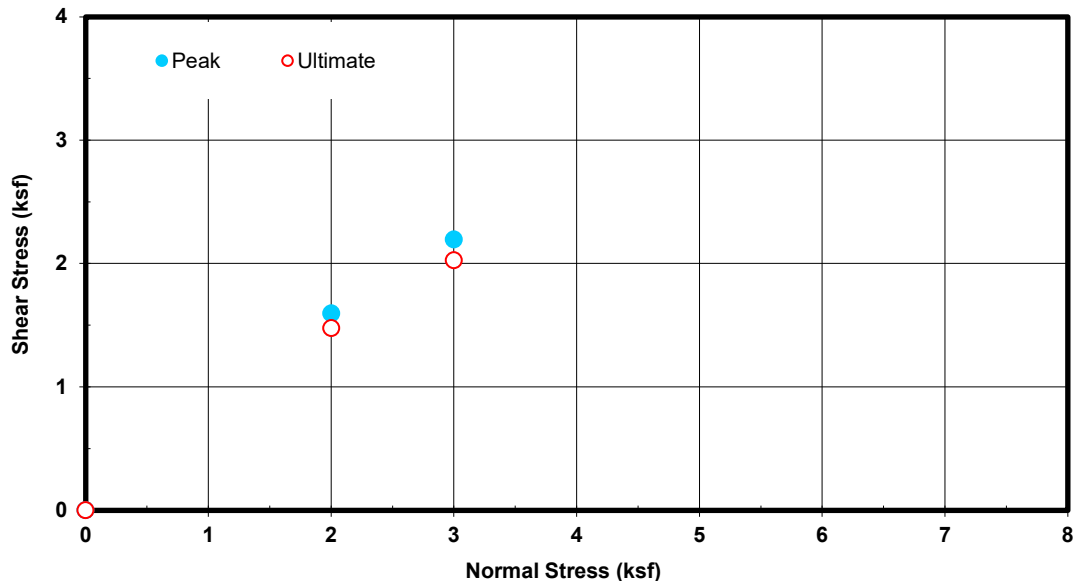
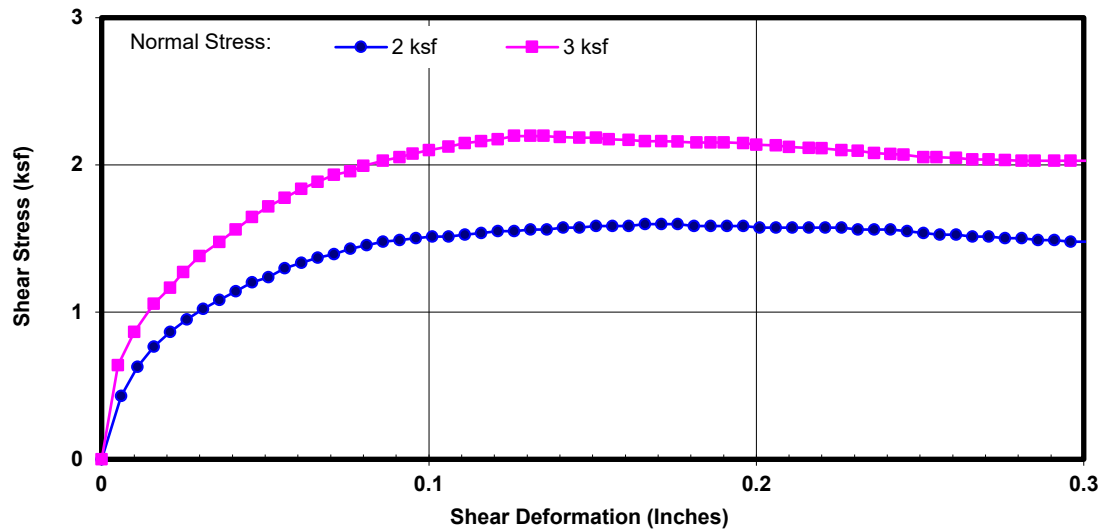


DIRECT SHEAR TEST RESULTS ASTM D 3080

Client: Tetra Tech
Project Name: Simms Park, Bellflower
Project No.: CWT22-05
Boring No.: B-3
Sample No.: R-8 Depth (ft): 20
Sample Type: Mod. Cal.
Soil Description: Sandy Lean Clay
Test Condition: Inundated Shear Type: Regular

Tested By: LS Date: 05/17/22
Computed By: NR Date: 05/19/22
Checked by: AP Date: 05/24/22

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
129.4	106.7	21.3	21.6	99	100	2	1.597	1.477
						3	2.196	2.028



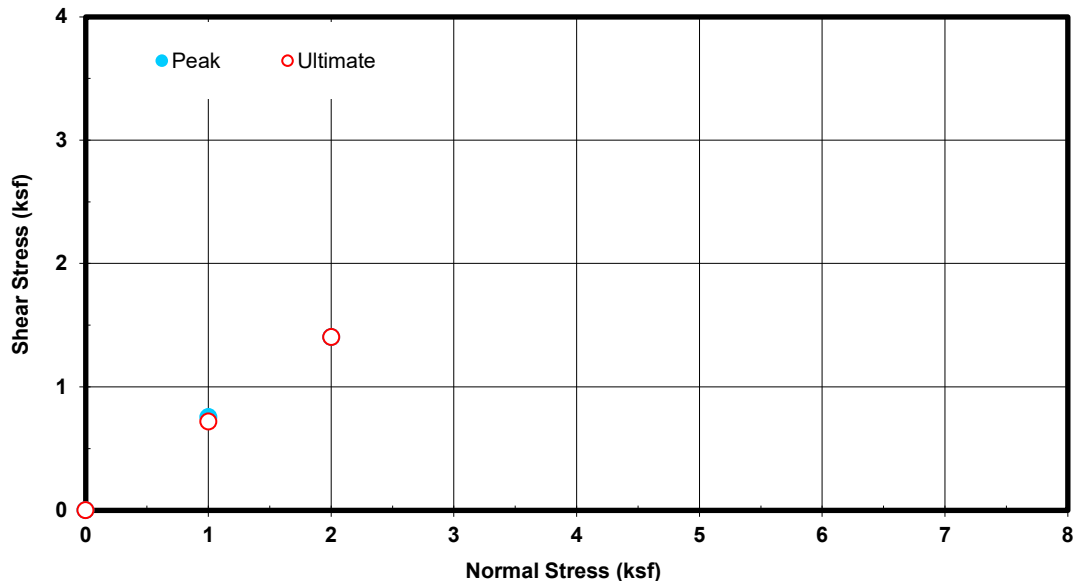
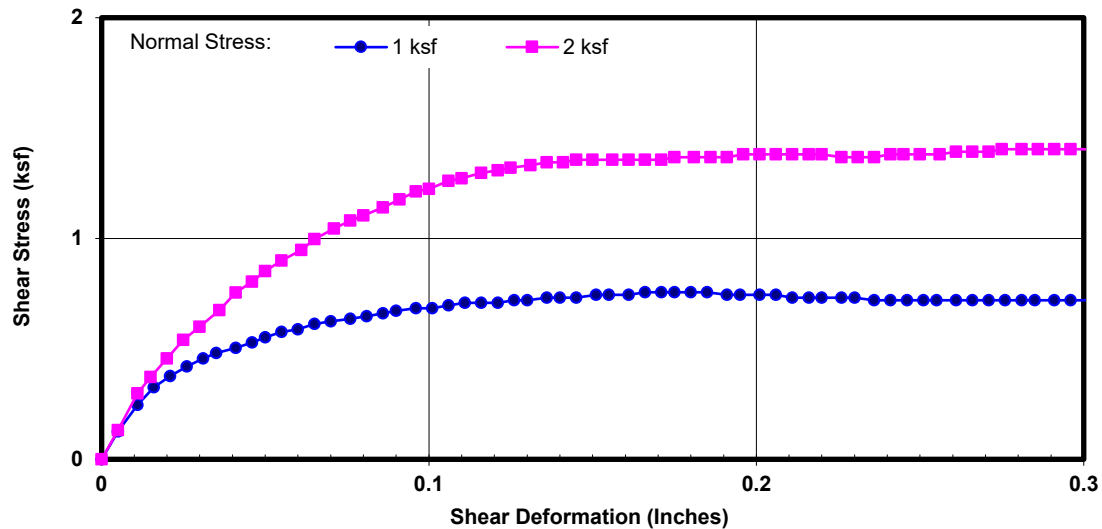


DIRECT SHEAR TEST RESULTS ASTM D 3080

Client: Tetra Tech
Project Name: Simms Park, Bellflower
Project No.: CWT22-05
Boring No.: B-4
Sample No.: R-5 Depth (ft): 10
Sample Type: Mod. Cal.
Soil Description: Silty Sand, fine-grained
Test Condition: Inundated Shear Type: Regular

Tested By: LS Date: 05/17/22
Computed By: NR Date: 05/19/22
Checked by: AP Date: 05/24/22

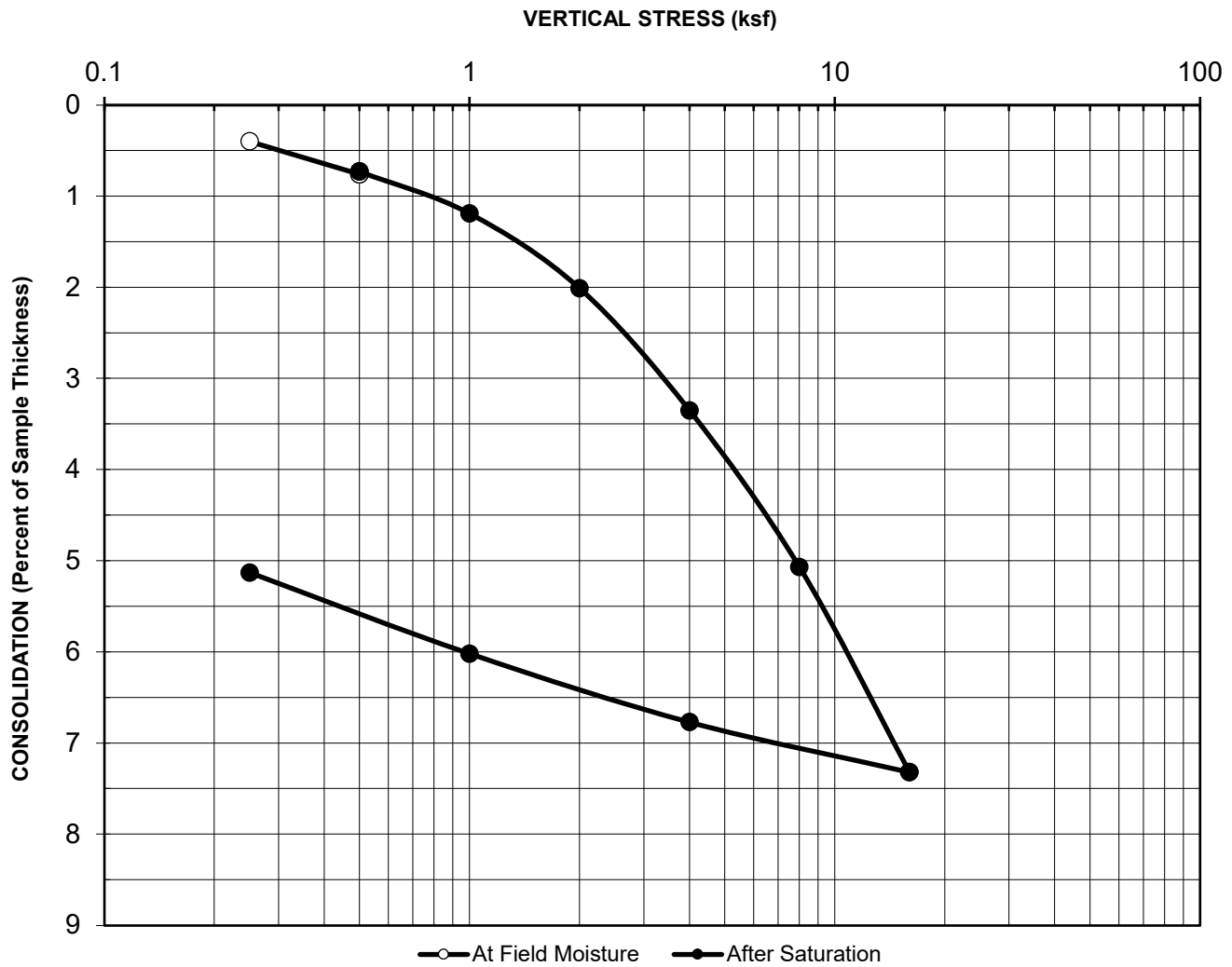
Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
101.0	91.3	10.6	31.3	34	100	1	0.756	0.720
						2	1.404	1.404



**AP Engineering and Testing, Inc.**

DBE|MBE|SBE

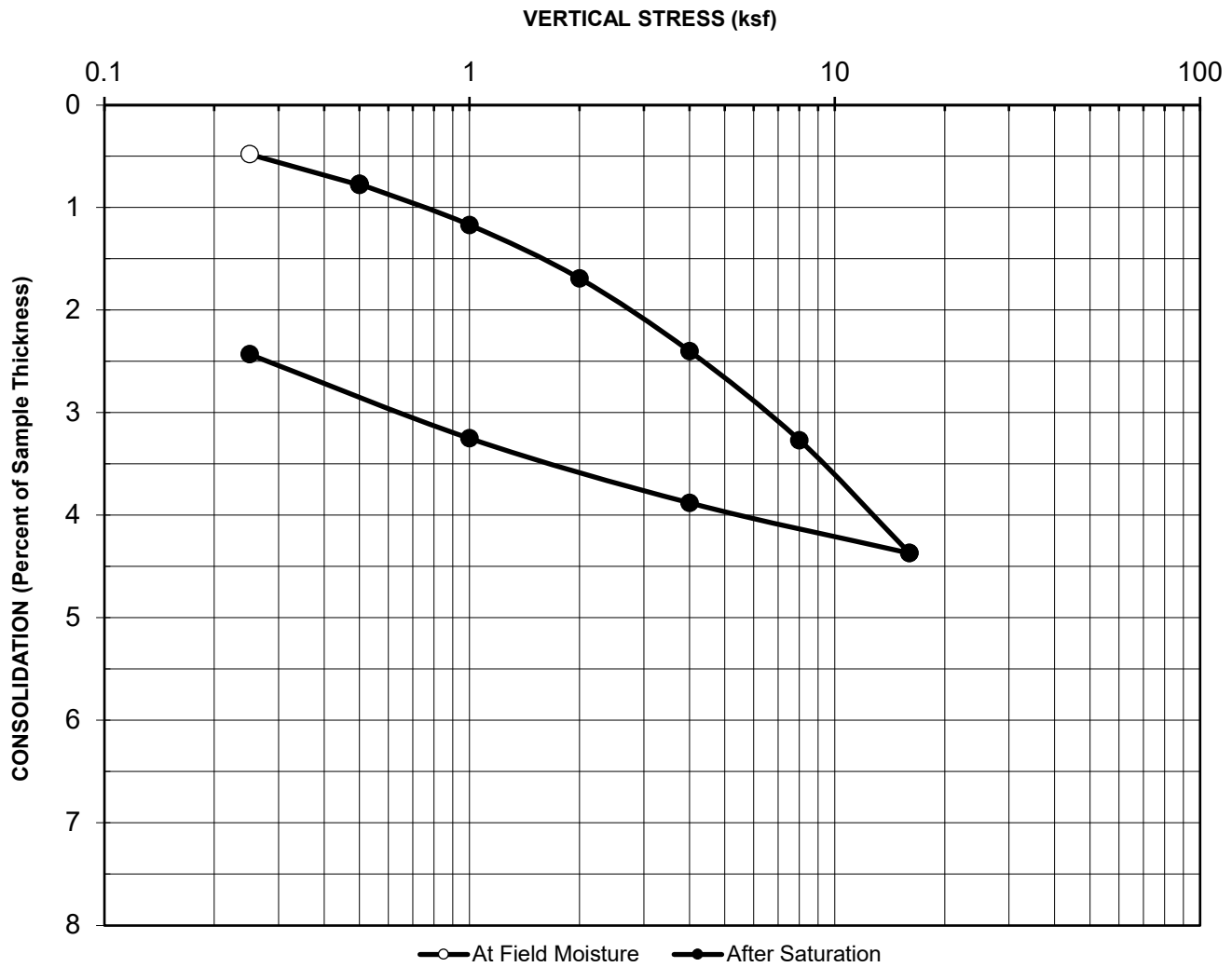
2607 Pomona Boulevard | Pomona, CA 91768

t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.comBoring No. : B-3Initial Dry Unit Weight (pcf): 104.8Sample No.: R-8Initial Moisture Content (%): 21.3Depth (feet): 20Final Moisture Content (%): 21.5Sample Type: Mod CalAssumed Specific Gravity: 2.7Soil Description: Sandy Lean ClayInitial Void Ratio: 0.61Remarks: Swell= 0.03% upon inundation**CONSOLIDATION CURVE
ASTM D 2435**Project Name: Simms Park, BellflowerProject No.: CWT22-05Date: 5/16/2022AP No: 22-0526 Sheet No: 1

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t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.comBoring No. : B-4Initial Dry Unit Weight (pcf): 101.6Sample No.: R-9Initial Moisture Content (%): 16.7Depth (feet): 25Final Moisture Content (%): 22.4Sample Type: Mod CalAssumed Specific Gravity: 2.7Soil Description: Sandy SiltInitial Void Ratio: 0.66Remarks: Swell= 0.01% upon inundation**CONSOLIDATION CURVE
ASTM D 2435**Project Name: Simms Park, BellflowerProject No.: CWT22-05Date: 5/16/2022AP No: 22-0526 Sheet No: 1

**AP Engineering and Testing, Inc.**

DBE | MBE | SBE

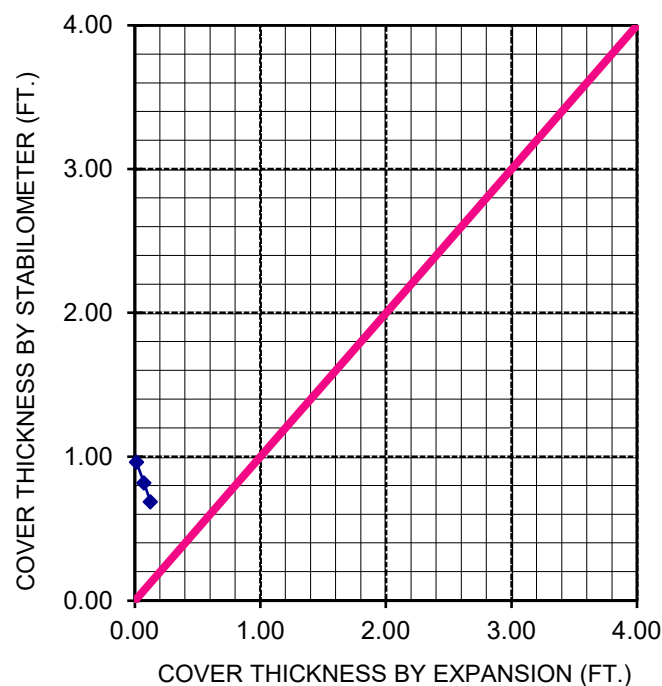
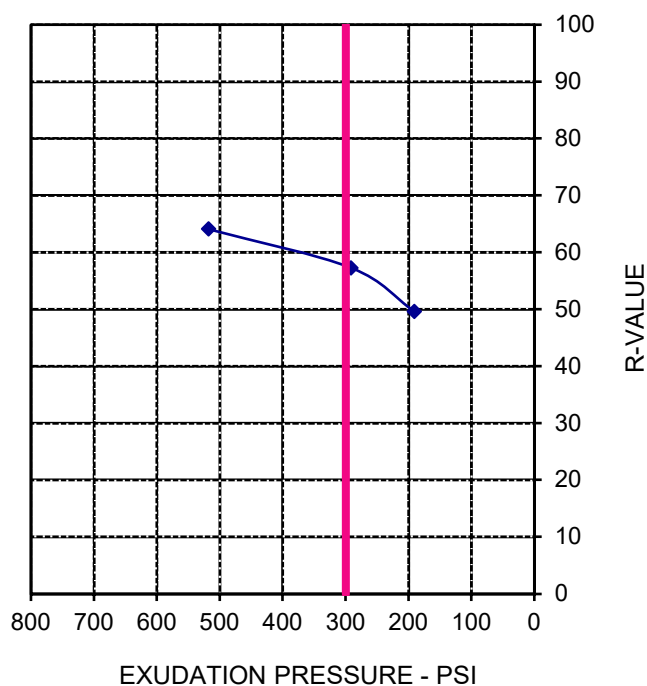
2607 Pomona Boulevard | Pomona, CA 91768

t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com**R-VALUE TEST DATA**

ASTM D2844

Project Name: Simms Park, BellflowerProject Number: CWT22-05Boring No.: B-3Sample No.: SK-1Location: N/ASoil Description: Sandy SiltTested By: ST Date: 05/16/22Computed By: KM Date: 05/18/22Checked By: AP Date: 05/24/22Depth (ft.): 0-5

Mold Number	R4	R5	R6		R-VALUE	By Exudation:	58
Water Added, g	46	56	66			By Expansion:	*N/A
Compact Moisture(%)	16.3	17.4	18.3			At Equilibrium:	58
Compaction Gage Pressure, psi	250	250	250			(by Exudation)	
Exudation Pressure, psi	518	291	191		Remarks	Gf = 1.34, and 0.3 % Retained on the 3/4" *Not Applicable	
Sample Height, Inches	2.6	2.5	2.5				
Gross Weight Mold, g	3102	3044	3019				
Tare Weight Mold, g	2017	2012	2012				
Net Sample Weight, g	1085	1032	1007				
Expansion, inchesx10 ⁻⁴	37	22	4				
Stability 2,000 (160 psi)	16/34	18/39	22/46				
Turns Displacement	5.57	5.79	6.29				
R-Value Uncorrected	62	57	50				
R-Value Corrected	64	57	50				
Dry Density, pcf	108.7	106.5	103.1				
Traffic Index	8.0	8.0	8.0				
G.E. by Stability	0.69	0.82	0.96				
G.E. by Expansion	0.12	0.07	0.01				



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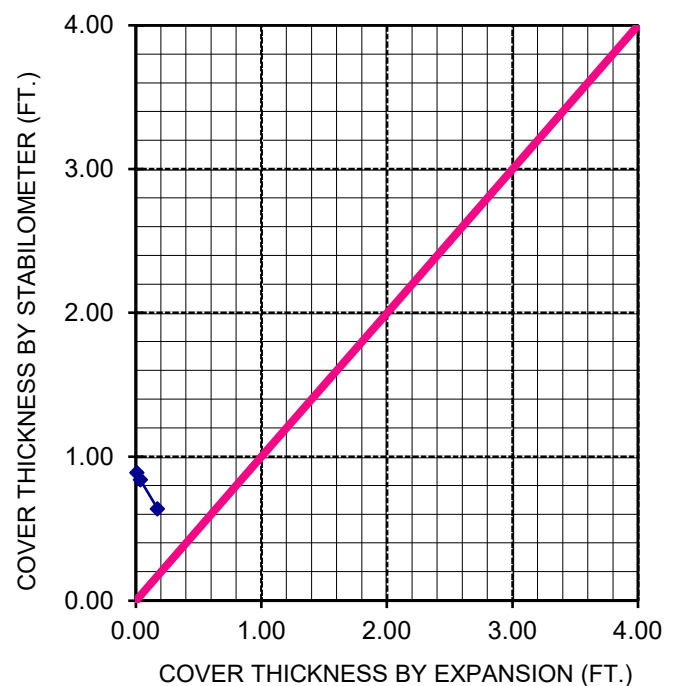
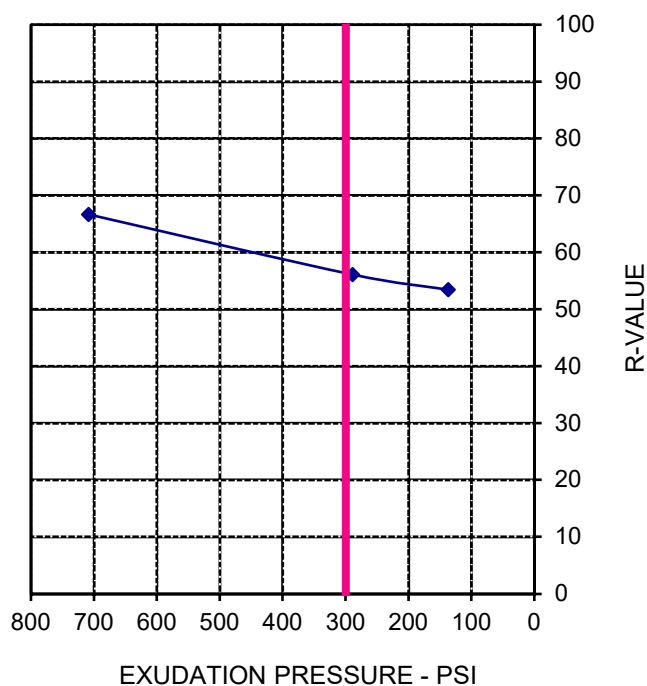
2607 Pomona Boulevard | Pomona, CA 91768

t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com**R-VALUE TEST DATA**

ASTM D2844

Project Name: Simms Park, BellflowerProject Number: CWT22-05Boring No.: B-5Sample No.: SK-1Location: N/ASoil Description: Sandy SiltTested By: ST Date: 05/16/22Computed By: KM Date: 05/18/22Checked By: AP Date: 05/24/22Depth (ft.): 0-5

Mold Number	D	F	E		R-VALUE	By Exudation:	56
Water Added, g	42	32	22			By Expansion:	*N/A
Compact Moisture(%)	15.1	14.1	13.1			At Equilibrium:	56
Compaction Gage Pressure, psi	250	250	250			(by Exudation)	
Exudation Pressure, psi	137	289	708		Remarks	Gf = 1.34, and 0.0 % Retained on the 3/4" *Not Applicable	
Sample Height, Inches	2.5	2.5	2.5				
Gross Weight Mold, g	3061	2928	3041				
Tare Weight Mold, g	1964	1869	1955				
Net Sample Weight, g	1096	1059	1086				
Expansion, inchesx10 ⁻⁴	3	11	52				
Stability 2,000 (160 psi)	20/44	19/42	14/30				
Turns Displacement	5.74	5.50	5.42				
R-Value Uncorrected	53	56	67				
R-Value Corrected	53	56	67				
Dry Density, pcf	115.5	112.5	116.4				
Traffic Index	8.0	8.0	8.0				
G.E. by Stability	0.89	0.84	0.64				
G.E. by Expansion	0.01	0.04	0.17				



**AP Engineering and Testing, Inc.**

DBE | MBE | SBE

2607 Pomona Boulevard | Pomona, CA 91768

t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com**CORROSION TEST RESULTS**Client Name: Tetra TechAP Job No.: 22-0526Project Name: Simms Park, BellflowerDate: 05/19/22Project No.: CWT22-05

Boring No.	Sample No.	Depth (feet)	Soil Description	Minimum Resistivity (ohm-cm)	pH	Sulfate Content (ppm)	Chloride Content (ppm)
B-3	SK-1	0-5	Sandy Silt	471	9.1	871	579
B-4	SK-1	0	Sandy Clay	2,619	9.0	94	47

NOTES: Resistivity Test and pH: California Test Method 643

Sulfate Content : California Test Method 417

Chloride Content : California Test Method 422

ND = Not Detectable

NA = Not Sufficient Sample

NR = Not Requested

Appendix C1

Results of Previous Infiltration Testing

PERCOLATION DATA SHEET

Project:	John Simms Park, Bellflower		Job No:	CWT 20-01E	
Test Hole No:	P-1 (EAST)		Date Excavated:	Thursday, April 16, 2020	
Test Hole Depth (feet):	22.0	feet	Soil Classification:	Silty Sand (SM)	
Stick up Length (feet):	0.00	foot			
Sandy Soil Criteria Tested By:	Shawn Morrish		Date:	Thursday, April 16, 2020	Presoak: 11:00 AM
Actual Percolation Tested By:	Shawn Morrish		Date:	Thursday, April 16, 2020	
Test Hole DiPMeter Inches):	8	inches	Latitude:	33.883505°	Longitude: -118.130304°
Casing DiPMeter:	3	inches	Elevation:	70 feet	

PERCOLATION TEST (after presoaking for more than 2 hours)

[illegible]

Reduction factor for boring percolation

$$RF_t = 2.00$$

Site variability, number of tests, and thoroughness of subsurface investigation

$$RF_v = 1.50$$

Long-term siltation, plugging, and maintenance

$$RF_s = 2.00$$

Percolation Test performed according to the Administrative Manual, County of Los Angeles, Department of Public Works, Guidelines for Geotechnical Investigation and Reporting Low Impact Development Stormwater Infiltration, GS 200.2 (6/30/17)

Corrected Percolation Rate (Infiltration Rate) =	0.05	in/hour
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PERCOLATION DATA SHEET

Project:	John Simms Park, Bellflower		Job No:	CWT 20-01E	
Test Hole No:	P-2 (WEST)		Date Excavated:	Thursday, April 16, 2020	
Test Hole Depth (feet):	20.0	feet	Soil Classification:	Silty Sand (SM)	
Stick up Length (feet):	0.00	foot			
Sandy Soil Criteria Tested By:	Shawn Morrish		Date:	Thursday, April 16, 2020	Presoak: 12:00 PM
Actual Percolation Tested By:	Shawn Morrish		Date:	Thursday, April 16, 2020	
Test Hole DiPMeter Inches):	8	inches	Latitude:	33.883503°	Longitude: -118.131257°
Casing DiPMeter:	3	inches	Elevation:	69 feet	

PERCOLATION TEST (after presoaking for more than 2 hours)

[illegible]

Reduction factor for boring percolation

$$RF_t = 2.00$$

Site variability, number of tests, and thoroughness of subsurface investigation

$$RF_v = 1.50$$

Long-term siltation, plugging, and maintenance

$$RF_s = 2.00$$

Percolation Test performed according to the Administrative Manual, County of Los Angeles, Department of Public Works, Guidelines for Geotechnical Investigation and Reporting Low Impact Development Stormwater Infiltration, GS 200.2 (6/30/17)

Corrected Percolation Rate (Infiltration Rate) =	0.04	in/hour
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Appendix C2

Results of Current Infiltration Testing

PERCOLATION DATA SHEET

Project:	Simms Park, Bellflower		Job No:	CWT22-05E	
Test Hole No:	P-3		Date Excavated:	Tuesday, May 3, 2022	
Test Hole Depth (feet):	3.5	feet	Soil Classification:	Sandy SILT	
Stick up Length (feet):	2.8	foot	Gravel Porosity:	0.35	
Sandy Soil Criteria Tested By:	SMS		Presoak Date:	Tuesday, May 3, 2022	Presoak: 4 hrs
Actual Percolation Tested By:	SMS/SM		Test Date:	Tuesday, May 3, 2022	
Test Hole Diameter (inches):	8	inches	Latitude:	33.8845	Longitude: -118.1323
Casing Diameter (inches):	2	inches	Elevation:	67	

PERCOLATION TEST (presoaking for 1 hour)

Time	Time Interval (min)	Initial Reading (feet below the top of the pipe)	Final Reading (feet below the top of the pipe)	Initial Water Level (feet above bottom of hole)	Final Water Level (feet above bottom of hole)	Δ in Water Level (inches)	Measured Raw Percolation Rate (inches/hour)
8:55 AM	39	3.05	3.10	3.25	3.20	0.60	0.018
9:34 AM							
9:34 AM	50	3.10	3.18	3.20	3.12	0.96	0.023
10:24 AM							
10:24 AM	65	3.18	3.25	3.12	3.05	0.84	0.016
11:29 AM							
11:29 AM	48	3.25	3.28	3.05	3.02	0.36	0.009
12:17 PM							
12:17 PM	30	3.28	3.34	3.02	2.96	0.72	0.030
12:47 PM							
12:47 PM	27	3.34	3.38	2.96	2.92	0.48	0.022
1:14 PM							
1:14 PM	31	3.38	3.48	2.92	2.82	1.20	0.050
1:45 PM							
1:45 PM	31	3.48	3.61	2.82	2.69	1.56	0.067
2:16 PM							
2:16 PM	21	3.61	3.70	2.69	2.60	1.08	0.071
2:37 PM							
2:37 PM	33	3.70	3.78	2.6	2.52	0.96	0.042
3:10 PM							
3:10 PM	30	3.78	3.85	2.5	2.45	0.84	0.041
3:40 PM							
3:40 PM	30	3.85	3.91	2.45	2.39	0.72	0.036
4:10 PM							

Reduction Factor Method (1 to 3)	2.00	Observed short-term infiltration rate (in/hr)=	0.036
Reduction Factor Site Variability (1 to 3)	1.50		
Reduction Factor Long Term Siltation (1 to 3)	3.00		

County of Los Angeles GS200.1 Guidelines for Geotechnical Investigation and Reporting. Low Impact Development Stormwater Infiltration 6/30/21

Corrected Percolation Rate = 0.006 in/hour

PERCOLATION DATA SHEET

Project: <u>Simms Park, Bellflower</u>		Job No: <u>CWT22-05E</u>	
Test Hole No: <u>P-4</u>		Date Excavated: <u>Tuesday, May 3, 2022</u>	
Test Hole Depth (feet): <u>3.5</u> feet		Soil Classification: <u>Sandy SILT</u>	
Stick up Length (feet): <u>3.0</u> foot		Gravel Porosity: <u>0.35</u>	
Sandy Soil Criteria Tested By: <u>SMS</u>		Presoak Date: <u>Tuesday, May 3, 2022</u>	Presoak: 4 hrs
Actual Percolation Tested By: <u>SMS/SM</u>		Test Date: <u>Tuesday, May 3, 2022</u>	
Test Hole Diameter (inches): <u>8</u> inches		Latitude: <u>33.8841</u>	Longitude: <u>-118.1331</u>
Casing Diameter (inches): <u>2</u> inches		Elevation: <u>66</u>	

PERCOLATION TEST (presoaking for 1 hour)

Time	Time Interval (min)	Initial Reading (feet below the top of the pipe)	Final Reading (feet below the top of the pipe)	Initial Water Level (feet above bottom of hole)	Final Water Level (feet above bottom of hole)	Δ in Water Level (inches)	Measured Raw Percolation Rate (inches/hour)
8:52 AM	34	2.73	2.76	3.77	3.74	0.36	0.011
9:26 AM							
9:26 AM	56	2.76	2.89	3.74	3.61	1.56	0.028
10:22 AM							
10:22 AM	64	2.89	3.33	3.61	3.17	5.28	0.091
11:26 AM							
11:26 AM	47	3.33	3.57	3.17	2.93	2.88	0.074
12:13 PM							
12:13 PM	30	3.57	3.67	2.93	2.83	1.20	0.051
12:43 PM							
12:43 PM	29	3.67	3.79	2.83	2.71	1.44	0.066
1:12 PM							
1:12 PM	31	3.79	3.92	2.71	2.58	1.56	0.070
1:43 PM							
1:43 PM	31	3.92	3.97	2.58	2.53	0.60	0.028
2:14 PM							
2:14 PM	20	3.97	4.05	2.53	2.45	0.96	0.071
2:34 PM							
2:34 PM	30	4.05	4.12	2.45	2.38	0.84	0.042
3:04 PM							
3:07 PM	30	4.00	4.10	2.50	2.40	1.20	0.060
3:37 PM							
3:37 PM	30	4.00	4.13	2.50	2.37	1.56	0.078
4:07 PM							

Reduction Factor Method (1 to 3)	2.00	Observed short-term infiltration rate (in/hr)=	0.078
Reduction Factor Site Variability (1 to 3)	1.50		
Reduction Factor Long Term Siltation (1 to 3)	3.00		

County of Los Angeles GS200.1 Guidelines for Geotechnical Investigation and Reporting. Low Impact Development Stormwater Infiltration 6/30/21

Corrected Percolation Rate = **0.012** in/hour

PERCOLATION DATA SHEET

Project:	Simms Park, Bellflower		Job No:	CWT22-05E	
Test Hole No:	P-5		Date Excavated:	Tuesday, May 3, 2022	
Test Hole Depth (feet):	2.8	feet	Soil Classification:	Sandy SILT	
Stick up Length (feet):	2.3	foot	Gravel Porosity:	0.35	
Sandy Soil Criteria Tested By:	SMS		Presoak Date:	Tuesday, May 3, 2022	Presoak: 1 hr
Actual Percolation Tested By:	SMS/SM		Test Date:	Tuesday, May 3, 2022	
Test Hole Diameter (inches):	8	inches	Latitude:	33.8836	Longitude: -118.1334
Casing Diameter (inches):	2	inches	Elevation:	65	

PERCOLATION TEST (presoaking for 1 hour)

Time	Time Interval (min)	Initial Reading (feet below the top of the pipe)	Final Reading (feet below the top of the pipe)	Initial Water Level (feet above bottom of hole)	Final Water Level (feet above bottom of hole)	Δ in Water Level (inches)	Measured Raw Percolation Rate (inches/hour)
8:48 AM	34	3.95	4.27	1.05	0.73	3.84	0.418
9:22 AM							
9:23 AM	55	4.00	4.32	1.00	0.68	3.84	0.271
10:18 AM							
10:19 AM	64	3.95	4.33	1.05	0.67	4.56	0.271
11:23 AM							
11:25 AM	45	4.00	4.27	1.00	0.73	3.24	0.273
12:10 PM							
12:11 PM	30	4.00	4.21	1.00	0.79	2.52	0.309
12:41 PM							
12:42 PM	27	4.00	4.21	1.00	0.79	2.52	0.343
1:09 PM							
1:10 PM	30	4.00	4.18	1.00	0.82	2.16	0.261
1:40 PM							
1:41 PM	30	4.00	4.20	1.00	0.80	2.40	0.293
2:11 PM							
2:12 PM	20	4.00	4.15	1.00	0.85	1.80	0.322
2:32 PM							
2:33 PM	30	4.00	4.19	1.00	0.81	2.28	0.277
3:03 PM							
3:04 PM	30	4.00	4.20	1.00	0.80	2.40	0.293
3:34 PM							
3:34 PM	30	4.00	4.19	1.00	0.81	2.28	0.277
4:04 PM							

Reduction Factor Method (1 to 3)	2.00	Observed short-term infiltration rate (in/hr)=	0.277
Reduction Factor Site Variability (1 to 3)	1.50		
Reduction Factor Long Term Siltation (1 to 3)	3.00		

County of Los Angeles GS200.1 Guidelines for Geotechnical Investigation and Reporting. Low Impact Development Stormwater Infiltration 6/30/21

Corrected Percolation Rate = **0.043** in/hour

Appendix D

Seismic Demand



Latitude, Longitude: 33.883493, -118.131557



Date	6/10/2022, 3:56:01 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.592	MCE_R ground motion. (for 0.2 second period)
S_1	0.57	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.592	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.061	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.679	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.747	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
S_{sRT}	1.592	Probabilistic risk-targeted ground motion. (0.2 second)
S_{sUH}	1.75	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
S_{sD}	2.451	Factored deterministic acceleration value. (0.2 second)
S_{1RT}	0.57	Probabilistic risk-targeted ground motion. (1.0 second)
S_{1UH}	0.632	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S_{1D}	0.825	Factored deterministic acceleration value. (1.0 second)
$PGAd$	0.99	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.91	Mapped value of the risk coefficient at short periods
C_{R1}	0.903	Mapped value of the risk coefficient at a period of 1 s

Unified Hazard Tool



Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the [U.S. Seismic Design Maps web tools](#) (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

^ Input

Edition

Dynamic: Continuous U.S. 2014 (update)

Spectral Period

Peak Ground Acceleration

Latitude

Decimal degrees

33.883493

Time Horizon

Return period in years

2475

Longitude

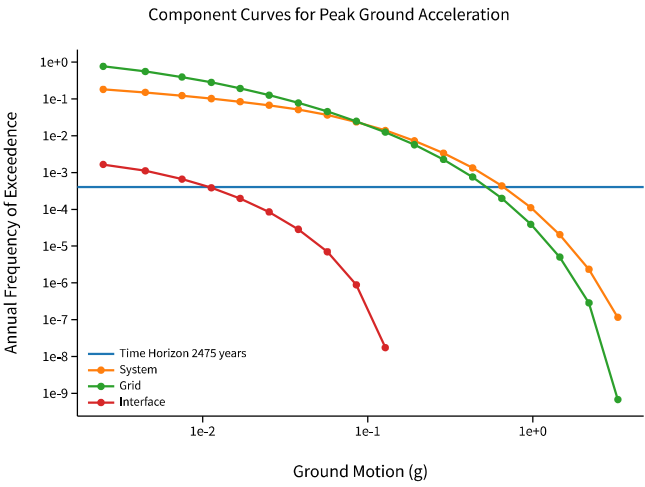
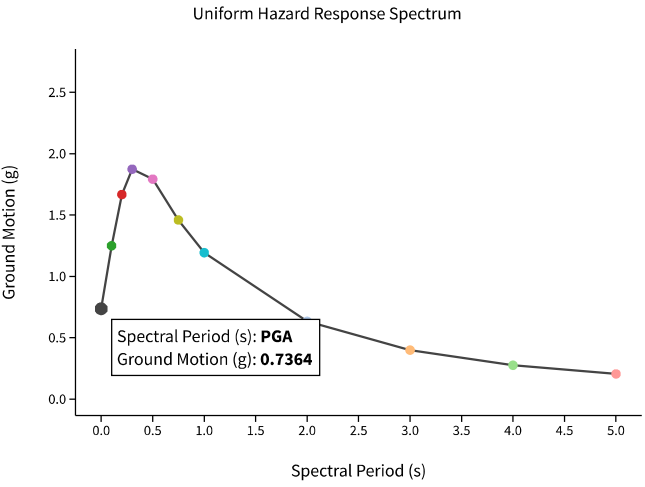
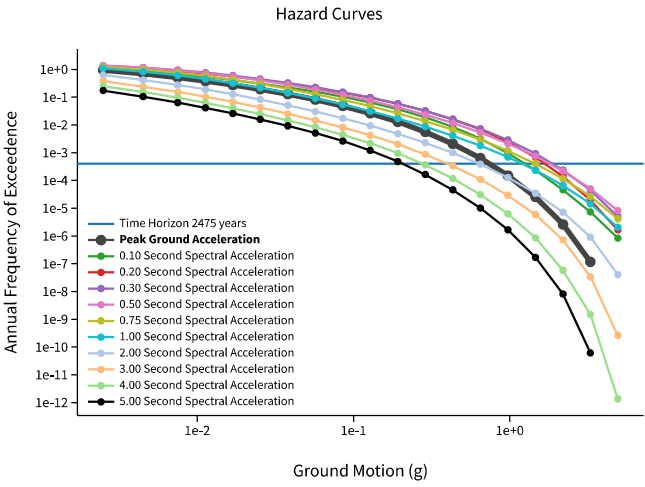
Decimal degrees, negative values for western longitudes

-118.131557

Site Class

259 m/s (Site class D)

^ Hazard Curve

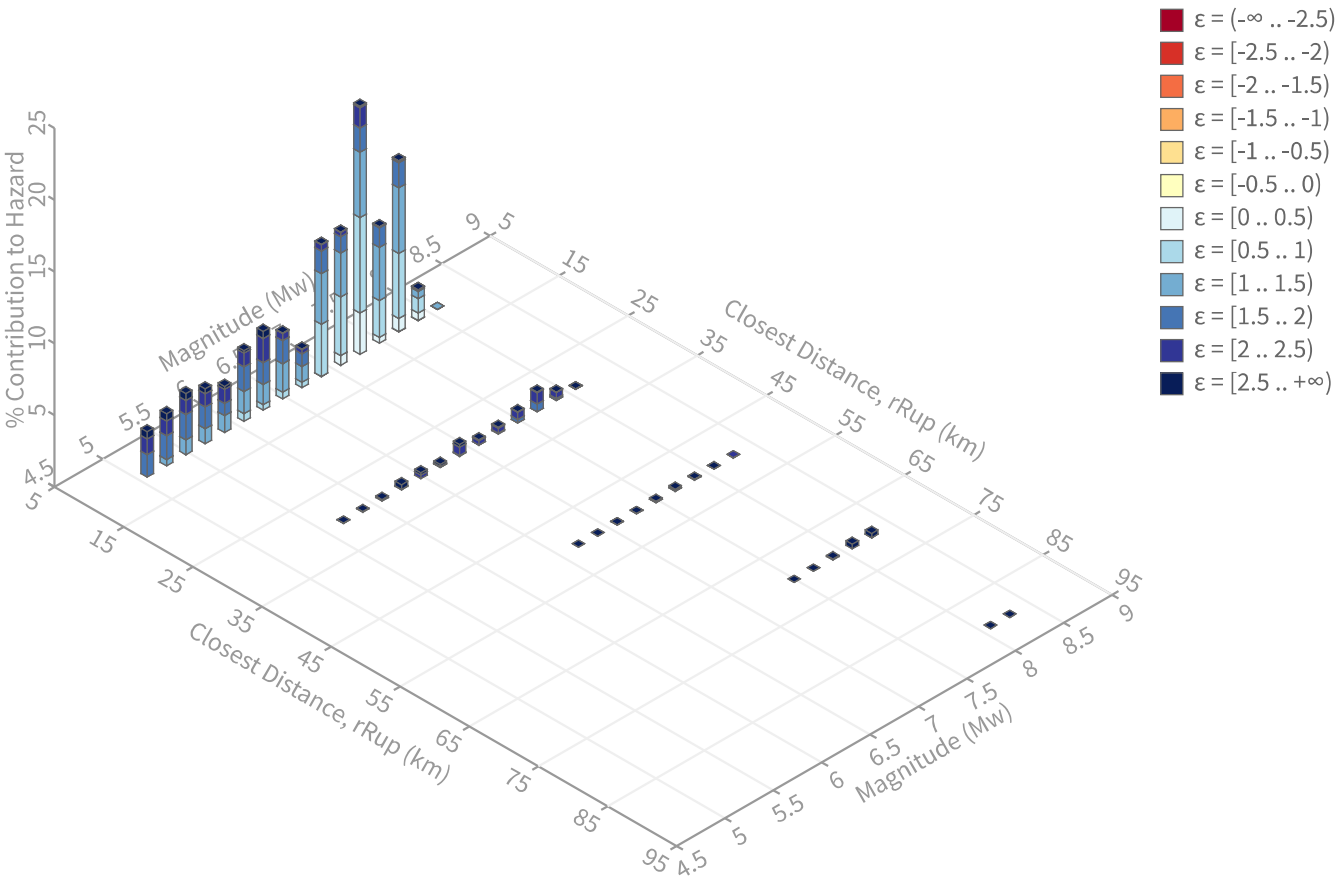


[View Raw Data](#)

^ Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr⁻¹
PGA ground motion: 0.73639848 g

Recovered targets

Return period: 2936.8909 yrs
Exceedance rate: 0.00034049614 yr⁻¹

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.05 %

Mean (over all sources)

m: 6.84
r: 11.51 km
ε₀: 1.38 σ

Mode (largest m-r bin)

m: 7.3
r: 11.19 km
ε₀: 0.98 σ
Contribution: 17.26 %

Mode (largest m-r-ε₀ bin)

m: 7.3
r: 9.46 km
ε₀: 0.66 σ
Contribution: 6.65 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↴ Source	Type	r	m	ϵ_0	lon	lat	az	%
UC33brAvg_FM32	System							38.33
Compton [1]		10.48	7.30	0.59	118.191°W	33.775°N	204.38	9.24
Puente Hills (Santa Fe Springs) [1]		6.34	7.05	1.04	118.144°W	33.926°N	346.61	7.58
Newport-Inglewood alt 2 [5]		8.72	7.43	1.12	118.214°W	33.848°N	242.53	5.03
Puente Hills (LA) [0]		11.95	7.18	1.48	118.148°W	33.986°N	352.24	2.37
Palos Verdes [10]		19.03	7.41	1.93	118.261°W	33.751°N	219.14	2.27
Puente Hills (Coyote Hills) [1]		9.62	7.36	1.28	118.044°W	33.915°N	66.75	2.15
Puente Hills (Santa Fe Springs) [0]		8.35	7.70	1.05	118.083°W	33.935°N	37.95	1.86
Anaheim [2]		8.01	7.17	0.89	118.063°W	33.881°N	92.93	1.74
UC33brAvg_FM31	System							32.23
Compton [1]		10.48	7.24	0.60	118.191°W	33.775°N	204.38	8.81
Newport-Inglewood alt 1 [4]		9.30	7.46	1.13	118.195°W	33.820°N	219.44	6.20
Puente Hills [2]		10.71	7.36	1.27	118.103°W	33.961°N	16.97	4.13
Palos Verdes [10]		19.03	7.26	2.02	118.261°W	33.751°N	219.14	2.17
Anaheim [2]		8.01	7.12	0.91	118.063°W	33.881°N	92.93	1.74
Whittier alt 1 [7]		15.10	7.13	1.87	118.046°W	33.999°N	31.60	1.47
Puente Hills [3]		11.51	6.86	1.55	118.143°W	33.972°N	354.05	1.18
UC33brAvg_FM32 (opt)	Grid							15.05
PointSourceFinite: -118.132, 33.915		6.16	5.67	1.43	118.132°W	33.915°N	0.00	2.88
PointSourceFinite: -118.132, 33.915		6.16	5.67	1.43	118.132°W	33.915°N	0.00	2.88
PointSourceFinite: -118.132, 33.942		7.92	5.75	1.67	118.132°W	33.942°N	0.00	1.61
PointSourceFinite: -118.132, 33.942		7.92	5.75	1.67	118.132°W	33.942°N	0.00	1.61
UC33brAvg_FM31 (opt)	Grid							14.40
PointSourceFinite: -118.132, 33.915		6.13	5.69	1.42	118.132°W	33.915°N	0.00	2.55
PointSourceFinite: -118.132, 33.915		6.13	5.69	1.42	118.132°W	33.915°N	0.00	2.55
PointSourceFinite: -118.132, 33.942		7.91	5.76	1.67	118.132°W	33.942°N	0.00	1.69
PointSourceFinite: -118.132, 33.942		7.91	5.76	1.67	118.132°W	33.942°N	0.00	1.69

Appendix E

Liquefaction Analyses

Summary of Liquefaction and Earthquake-Induced Settlement Analysis

Project:	Siims Park	Boring:	B-3	Engineer:	FC	Date:	6/13/2022
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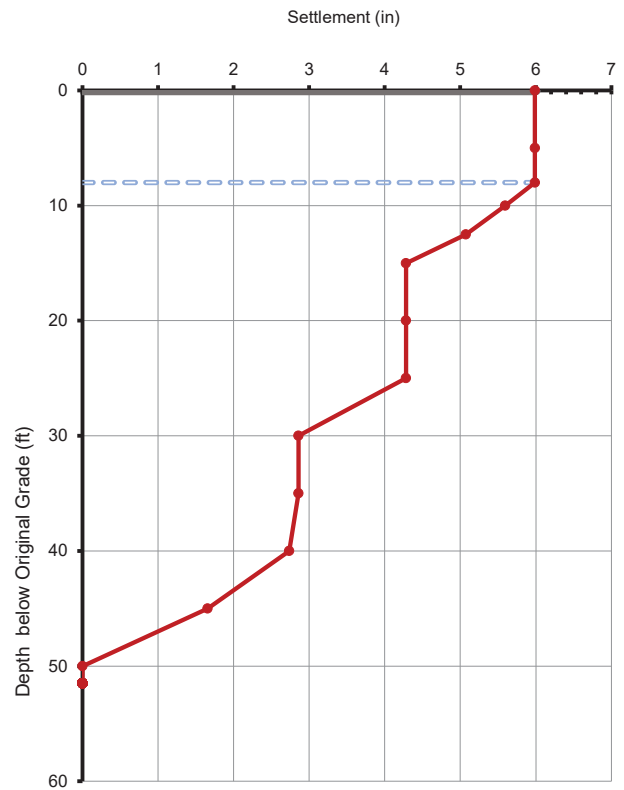
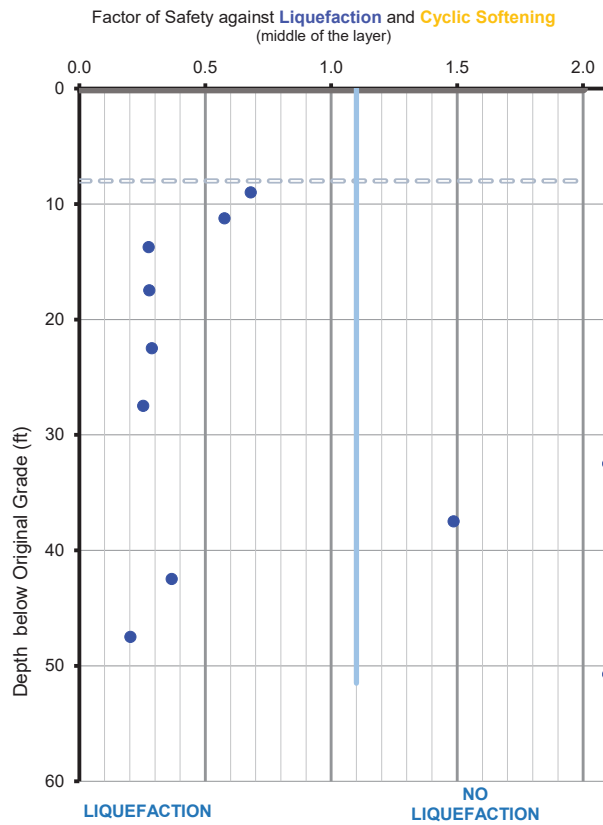
Liquefaction Evaluation Method			
Correction for fines content	Idriss & Boulang. 2008, 2014		
Correction for overburden C_N	Idriss & Boulang. 2014 (N1)60cs		
Cyclic resistance ratio of soil CRR_{CS}	Idriss & Boulang. 2004, 2014		
Correction for overburden K_σ	Idriss & Boulang. 2008, 2014		
Stress reduction factor r_D	Idriss 1999, I&B 2008,2014		
Magnitude scaling factor MSF	Idriss & Boulang. 2014		
Dry settlement	Pradel, 1998a,b		
Liquefaction settlement	Yoshimine et al., 2006 – w/ calibration		
Liquefaction behavior Plasticity Index threshold	less or equal to 12		
Saturated settlement threshold	less or equal to 70% fines		
Dry settlement threshold	less or equal to 25% fines		
Cyclic softening Plasticity Index threshold	greater or equal to 18		
Profile		Earthquake loading	
In-Situ Groundwater depth	35.00 feet	M	7.3
DESIGN Groundwater depth	8.00 feet	PGA	0.747
DESIGN Excavation depth	0.00 feet		
DESIGN Surcharge (fill)	0.00 feet		
Version v2 2022-03			

Liquefaction Analysis Statistics	
Total thickness of evaluated profile	51.5 feet
Profile thickness susceptible to liquefaction	32 feet
Number of evaluated intervals	13
Number of potentially liquefiable intervals	11
Average Factor of Safety of sandy intervals	0.88
Dry sand settlement	0.00 inches
Liquefaction settlement	5.99 inches
Total earthquake-induced settlement	5.99 inches
Checks	
Groundwater depth check	OK
Design groundwater/excavation depth check	OK
Fines correction method compatibility	OK
Idris & Boulanger, 2004 method for C_N	not used
Cetin 2009 settlement method	not used

[illegible]

B-3

Design excavation depth 0.00 feet
Design groundwater depth 8.00 feet



Summary of Liquefaction and Earthquake-Induced Settlement Analysis

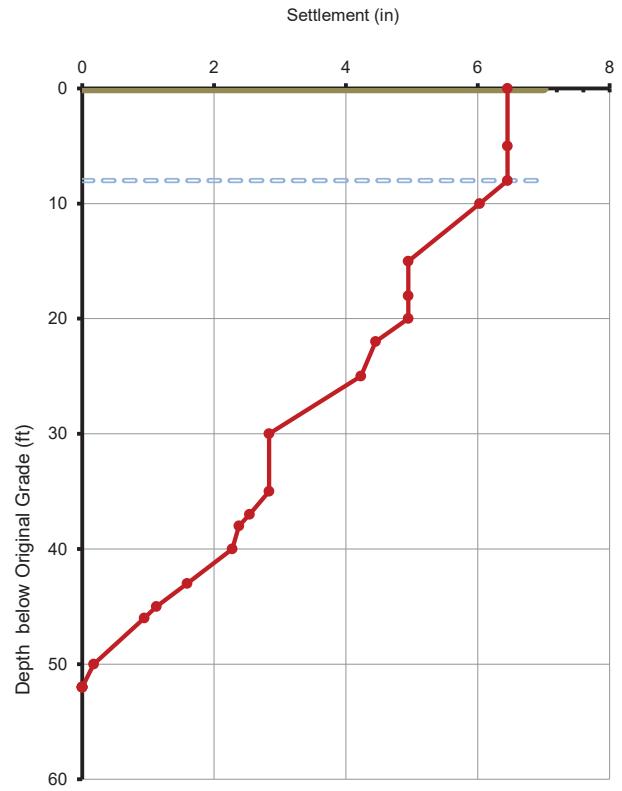
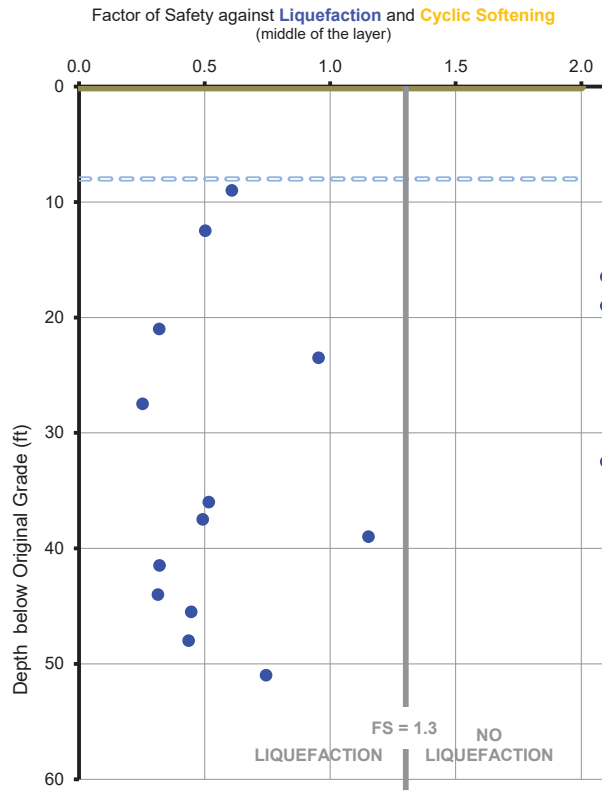
Project:	Simms Park	Boring:	B-1	Engineer:		Date:	5/13/2020
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Liquefaction Evaluation Method				Liquefaction Analysis Statistics			
Correction for fines content	Idriss & Boulang. 2008, 2014			Total thickness of evaluated profile	52 feet		
Correction for overburden C_N	Idriss & Boulang. 2014 (N1)60cs			Profile thickness susceptible to liquefaction	34 feet		
Cyclic resistance ratio of soil CRR_{CS}	Idriss & Boulang. 2004, 2014			Number of evaluated intervals	18		
Correction for overburden K_σ	Idriss & Boulang. 2008, 2014			Number of potentially liquefiable intervals	16		
Stress reduction factor r_D	Idriss 1999, I&B 2008,2014			Average Factor of Safety of sandy intervals	0.97		
Magnitude scaling factor MSF	Idriss & Boulang. 2014						
Dry settlement	Pradel, 1998a,b			Dry sand settlement	0.00 inches		
Liquefaction settlement	Yoshimine et al., 2006 – w/ calibration			Liquefaction settlement	6.45 inches		
				Total earthquake-induced settlement	6.45 inches		
Liquefaction behavior Plasticity Index threshold	less or equal to 12						
Saturated settlement threshold	less or equal to 70% fines						
Dry settlement threshold	less or equal to 25% fines						
Cyclic softening Plasticity Index threshold	greater or equal to 18						

Depth to Layer Top	Layer Thickness	Total Unit Weight		Fines %	Plasticity Index	Considered Blowcounts			Factor of Safety		Liquefaction potential rationale	Layer Settlement	Cumulative Settlement
feet	feet	In-situ pcf	Design pcf	%	–	SPT-N bpf	N1,60 bpf	N1,60,cs bpf	Liquefaction	Cyclic softening	–	in	in
0	5	119.88	119.9	44	n/plastic	15.4	34.9	40.5	–	–	- no groundwater	0.00	6.45
5	3	101.52	101.5	44	n/plastic	9.0	18.2	23.8	–	–	- no groundwater	0.00	6.45
8	2	101.52	101.5	3	n/plastic	13.4	24.0	24.0	0.61	–	- liquefiable - FS < 1.3	0.42	6.45
10	5	101.52	101.5	40	n/plastic	10.0	18.0	23.6	0.50	–	- liquefiable - FS < 1.3	1.08	6.03
15	3	112	112.0	40	n/plastic	18.8	29.8	35.4	2.18	–	- too dense – (N1)60,CS > 32	0.00	4.95
18	2	112	112.0	40	n/plastic	18.8	31.8	37.4	3.31	–	- too dense – (N1)60,CS > 32	0.00	4.95
20	2	116.6	116.6	40	n/plastic	9.0	14.6	20.2	0.32	–	- liquefiable - FS < 1.3	0.49	4.95
22	3	116.6	116.6	15	n/plastic	18.1	28.3	31.6	0.95	–	- liquefiable - FS < 1.3	0.23	4.45
25	5	116.6	116.6	15	n/plastic	10.0	14.2	17.4	0.25	–	- liquefiable - FS < 1.3	1.39	4.23
30	5	117.3	117.3	5	n/plastic	30.8	47.5	47.5	2.91	–	- too dense – (N1)60,CS > 32	0.00	2.84
35	2	117.3	117.3	5	n/plastic	20.0	27.6	27.6	0.52	–	- liquefiable - FS < 1.3	0.30	2.84
37	1	117.3	117.3	5	n/plastic	20.0	27.2	27.2	0.49	–	- liquefiable - FS < 1.3	0.16	2.54
38	2	124.46	124.5	60	n/plastic	20.0	27.8	33.4	1.15	–	- too dense – (N1)60,CS > 32	0.10	2.38
40	3	124.46	124.5	60	n/plastic	12.7	16.4	22.0	0.32	–	- liquefiable - FS < 1.3	0.69	2.28
43	2	124.46	124.5	40	n/plastic	12.7	16.2	21.7	0.31	–	- liquefiable - FS < 1.3	0.46	1.59
45	1	124.16	124.2	40	n/plastic	16.0	20.7	26.3	0.45	–	- liquefiable - FS < 1.3	0.18	1.13
46	4	124.16	124.2	60	n/plastic	16.0	20.4	26.0	0.44	–	- liquefiable - FS < 1.3	0.77	0.94
50	2	124.16	124.2	60	n/plastic	19.4	25.2	30.8	0.75	–	- liquefiable - FS < 1.3	0.17	0.17
487.00	52.00	2096.00	2096.00	616.00	0.00	290.13	443.02	516.62	15.47	0.00		6.45	65.60

Profile		Earthquake loading		Checks	
In-Situ Groundwater depth	37.00 feet	M	7.3	Groundwater depth check	OK
DESIGN Groundwater depth	8.00 feet	PGA	0.747	Design groundwater/excavation depth check	OK
DESIGN Excavation depth	0.00 feet			Fines correction method compatibility	OK
DESIGN Surcharge (fill)	0.00 feet			Idris & Boulanger, 2004 method for C_N	not used
				Cetin 2009 settlement method	not used

Version v2.201 -07



Summary of Liquefaction and Earthquake-Induced Settlement Analysis

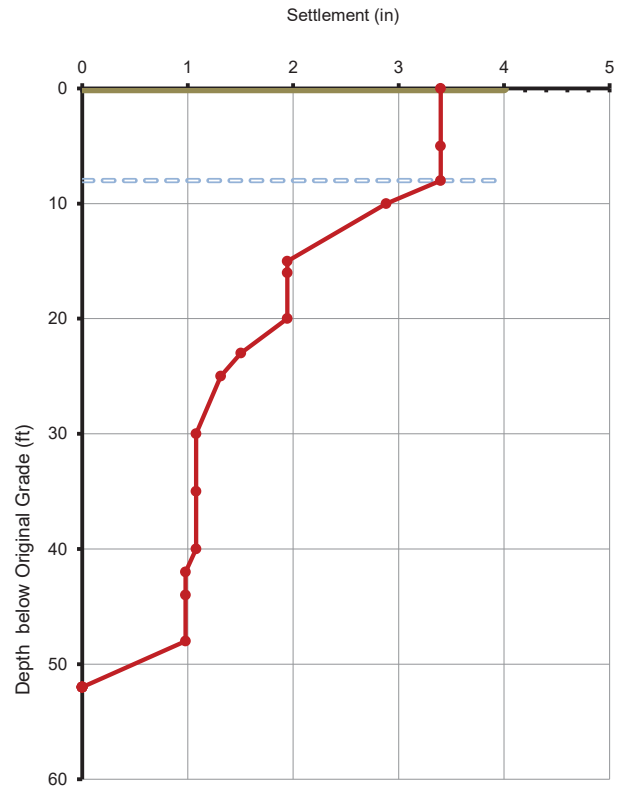
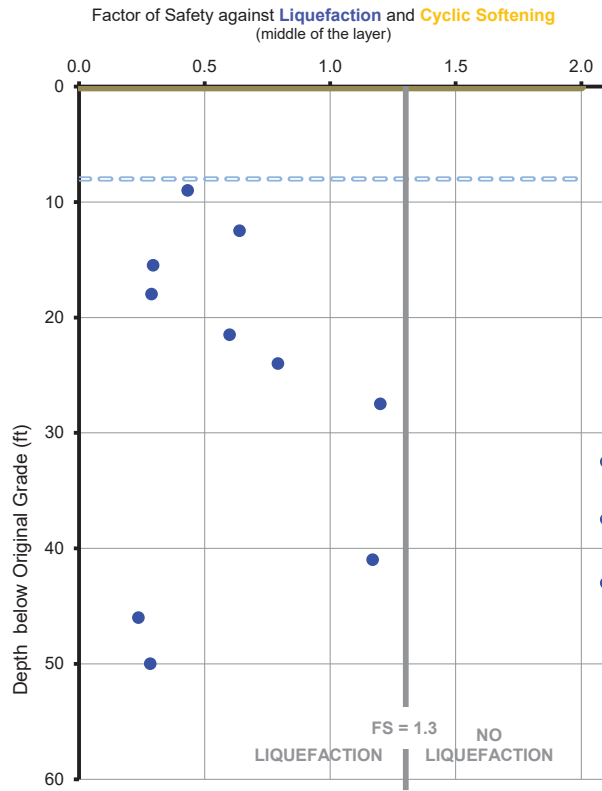
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Liquefaction Evaluation Method		
Correction for fines content	Idriss & Boulang. 2008, 2014	
Correction for overburden C_N	Idriss & Boulang. 2014 (N1)60cs	
Cyclic resistance ratio of soil CRR_{CS}	Idriss & Boulang. 2004, 2014	
Correction for overburden K_σ	Idriss & Boulang. 2008, 2014	
Stress reduction factor r_D	Idriss 1999, I&B 2008,2014	
Magnitude scaling factor MSF	Idriss & Boulang. 2014	
Dry settlement	Pradel, 1998a,b	
Liquefaction settlement	Yoshimine et al., 2006 – w/ calibration	
Liquefaction behavior Plasticity Index threshold	less or equal to 12	
Saturated settlement threshold	less or equal to 70% fines	
Dry settlement threshold	less or equal to 25% fines	
Cyclic softening Plasticity Index threshold	greater or equal to 18	

Liquefaction Analysis Statistics	
Total thickness of evaluated profile	52 feet
Profile thickness susceptible to liquefaction	32 feet
Number of evaluated intervals	15
Number of potentially liquefiable intervals	13
Average Factor of Safety of sandy intervals	1.11
Dry sand settlement	0.00 inches
Liquefaction settlement	3.40 inches
Total earthquake-induced settlement	3.40 inches

Depth to Layer Top	Layer Thickness	Total Unit Weight		Fines %	Plasticity Index	Considered Blowcounts			Factor of Safety		Liquefaction potential rationale	Layer Settlement	Cumulative Settlement
		In-situ	Design			SPT-N	N1,60	N1,60,cs	Liquefaction	Cyclic softening			
feet	feet	pcf	pcf	%	—	bpf	bpf	bpf			—	in	in
0	5	128.52	128.5	35	n/plastic	9.0	20.4	25.9	—	—	- no groundwater	0.00	3.40
5	3	128.52	128.5	60	12	15.4	28.0	33.6	—	—	- no groundwater	0.00	3.40
8	2	128.52	128.5	3	n/plastic	11.0	19.1	19.1	0.43	—	- liquefiable - FS < 1.3	0.52	3.40
10	5	101.76	101.8	3	n/plastic	15.4	26.2	26.2	0.64	—	- liquefiable - FS < 1.3	0.94	2.88
15	1	101.76	101.8	75	12	7.0	11.2	16.8	0.30	—	- liquefiable - FS < 1.3	0.00	1.95
16	4	101.76	101.8	75	12	7.0	11.8	17.3	0.29	—	- liquefiable - FS < 1.3	0.00	1.95
20	3	131.1	131.1	20	n/plastic	14.7	23.2	27.7	0.60	—	- liquefiable - FS < 1.3	0.44	1.95
23	2	131.1	131.1	3	n/plastic	20.0	30.2	30.2	0.79	—	- liquefiable - FS < 1.3	0.19	1.51
25	5	105	105.0	3	n/plastic	22.8	33.1	33.1	1.20	—	- too dense – (N1)60,CS > 32	0.24	1.31
30	5	105	105.0	3	n/plastic	27.0	40.3	40.3	2.93	—	- too dense – (N1)60,CS > 32	0.00	1.08
35	5	97.65	97.7	3	n/plastic	40.2	63.6	63.6	2.81	—	- too dense – (N1)60,CS > 32	0.00	1.08
40	2	97.65	97.7	3	n/plastic	24.0	33.6	33.6	1.17	—	- too dense – (N1)60,CS > 32	0.10	1.08
42	2	97.65	97.7	20	n/plastic	24.0	34.2	38.7	2.72	—	- too dense – (N1)60,CS > 32	0.00	0.98
44	4	117.6	117.6	75	n/plastic	9.4	11.7	17.3	0.24	—	- liquefiable - FS < 1.3	0.00	0.98
48	4	117.6	117.6	30	n/plastic	12.0	15.0	20.4	0.28	—	- liquefiable - FS < 1.3	0.98	0.98
361.00	52.00	1691.19	1691.19	411.00	36.00	258.92	401.74	443.84	14.41	0.00		3.40	27.91

Profile		Earthquake loading		Checks	
In-Situ Groundwater depth	35.00 feet	M	7.3	Groundwater depth check	OK
DESIGN Groundwater depth	8.00 feet	PGA	0.747	Design groundwater/excavation depth check	OK
DESIGN Excavation depth	0.00 feet			Fines correction method compatibility	OK
DESIGN Surcharge (fill)	0.00 feet			Idris & Boulanger, 2004 method for C _N	not used
				Cetin 2009 settlement method	not used
					Version v2 201 -07



Geotechnical Design Report Rev.1

BELLFLOWER YOUTH AND SENIOR CITIZEN CENTER 16614 South Clark Avenue Bellflower, California



Prepared for:

City of Bellflower
Public Works Manager
16600 Civic Center Drive
Bellflower, CA 90706

Prepared by:

Tetra Tech
21700 Copley Drive, Suite 200
Diamond Bar, California 91765

January 6, 2023
(supersedes report dated November 7, 2022)
Project No. ciBLF 22-01E

Mr. Bernardo Iniguez
Public Works Manager
City of Bellflower
16600 Civic Center Drive
Bellflower, CA 90706

Subject: **GEOTECHNICAL DESIGN REPORT Rev.1**
BELLFLOWER YOUTH AND SENIOR CITIZEN CENTER
16614 South Clark Avenue
Bellflower, California

Dear Mr. Iniguez:

Presented herein is Tetra Tech's geotechnical design report for the proposed Bellflower Youth and Senior Citizen Center located near the northwest corner of the John Simms Park at 16614 South Clark Avenue in the City of Bellflower, California. This report summarizes the results of our geotechnical investigation to characterize the soils at the site and provides geotechnical recommendations for the design and construction of the proposed facility. The appendices of the report include logs of borings from current and previous explorations, results of laboratory testing from current and previous investigations, and a summary of the seismic demand and liquefaction analyses. This revised report supersedes our previous report dated November 7, 2022 as it includes recommendations based on the 2022 California Building Code and provides additional recommendations for moisture vapor intrusion mitigation and for ancillary structures foundations.

We appreciate the opportunity to provide our professional services on this project. If you have any questions regarding this report or if we can be of further service, please do not hesitate to contact the undersigned.

Respectfully submitted,
Tetra Tech BAS, Inc.


Fernando Cuenca, Ph.D., G.E.
Senior Engineer




Shannon Siegel, P.G.
Project Geologist




Peter Skopek, Ph.D., G.E.
Principal



Distribution: Addressee (pdf by email biniguez@bellflower.org)

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- Figure 2 – Site Layout and Boring Location Map
- Figure 3 – Geologic Map
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Appendices

- Appendix A1 – Logs of Current Exploratory Boring
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- Appendix B1 – Results of Current Laboratory Testing
- Appendix B2 – Results of Previous Laboratory Testing
- Appendix C – Seismic Demand
- Appendix D – Liquefaction Analyses

1. INTRODUCTION AND PROJECT DESCRIPTION

This report presents the results of Tetra Tech's geotechnical engineering evaluation for the proposed Bellflower Youth and Senior Citizen Center located in the vicinity of the northwest corner of the John Simms Park at 16614 South Clark Avenue in the City of Bellflower, California (see Figure 1 – Project Location Map). It is noted that this project is a part of the redevelopment of the John Simms Park that includes construction of stormwater infiltration facilities and associated at-grade pavements and improvements.

The Youth and Senior Citizen Center project entails the construction of a new single story 11,834-square-foot building. The building is anticipated to be supported on shallow foundations. It is understood that other ancillary at-grade structures are also planned (e.g., trash enclosures, etc.). It is noted that two geotechnical investigations conducted by Tetra Tech (June 2022 and May 2019) in the area immediately adjacent to the Youth and Senior Citizen Center project site identified liquefaction as a significant geologic hazard at the site and this finding was confirmed by the investigation performed for the Youth and Senior Citizen Center project. Therefore, remedial measures are herein provided to mitigate the liquefaction hazard for the design of the foundations for the proposed building. The construction of the proposed building will entail excavations to install geogrid-reinforced raft for liquefaction mitigation measures. It is anticipated that all excavations can be performed as sloped excavations, i.e., without the need for shoring, however, depending on the final location of the building, shoring or slot cutting could be needed to protect off-site improvements.

The appendices to this report include logs of borings and results of laboratory testing from the current and previous investigations, and a summary of the seismic demand and liquefaction analyses.

2. SCOPE OF WORK

Tetra Tech's scope of services for this project consisted of the following tasks:

- Review readily available background data.
- Perform a reconnaissance site visit to observe ground conditions and mark boring locations.
- Obtain drilling permits from the Los Angeles County Department of Public Health (LACDPH).
- Coordinate with City of Bellflower's engineering staff, park personnel, and Underground Service Alert (USA) for clearance of buried on-site utilities prior to drilling.
- Conduct a subsurface investigation within the proposed building footprint area, consisting of excavating, logging, and geotechnical sampling of 1 soil exploratory boring to a maximum depth of 56.5 feet.
- Perform laboratory testing of selected soil samples recovered from the soil boring to evaluate geotechnical engineering properties of the on-site soils.
- Conduct an evaluation of the collected data to develop geotechnical recommendations for the design and construction of the proposed structures including the following items:
 - ♦ An evaluation of subsurface conditions and description of types, distribution, and engineering characteristics of subsurface materials.
 - ♦ An evaluation of the liquefaction potential and dynamic settlement of the on-site granular materials.
 - ♦ An evaluation of the suitability of on-site soils for the support of structures.
 - ♦ Recommendations for shoring and slot cuts.
 - ♦ Recommendations for design of foundation systems for proposed building including allowable bearing capacity, lateral resistance, and settlement estimates.
 - ♦ Determination of seismic design parameters in accordance with the 2022 California Building Code (CBC).
 - ♦ Recommendations for flatwork.
 - ♦ Recommendations for grading.
 - ♦ An evaluation of the corrosion potential of on-site soils to buried concrete.
- Prepare this written report documenting the work performed, physical data acquired, and provide geotechnical recommendations.

3. SUBSURFACE EXPLORATION

Prior to starting the current field exploration program, a field reconnaissance was conducted to observe surface conditions and to mark the location of the planned exploratory boring in agreement with the City of Bellflower staff. A drilling permit was obtained from the LACDPH for the subsurface exploration. Underground Service Alert, the LACDPH inspector, and City of Bellflower staff were also notified of the subsurface exploration schedule at least 48 hours prior to drilling.

Two previous subsurface investigations were carried out by Tetra Tech within the John Simms Park area on April 16, 2020 and May 3, 2022 and included the drilling, logging, and sampling of 5 hollow stem auger exploratory borings, B-1 through B-5, and 5 percolation test borings denoted P-1 through P-5. The boring logs from these previous investigations are presented in Appendix A2. The current subsurface investigation was carried out on October 11, 2022 and included the drilling, logging, and sampling of 1 boring B-6 located within the footprint area of the proposed building as shown on Figure 2 – Site Layout and Boring Location Map. The boring was excavated using a G-16 truck-mounted drill rig equipped with 8-inch diameter hollow stem augers to a depth of 56.5 feet. The boring log is presented in Appendix A1. The approximate latitude and longitude of the current and previous soil explorations, the approximate elevations, and depths of are included in Table 1.

Table 1
Boring Information

Boring Number	General Location	Approximate			
		Latitude	Longitude	Ground Elevation (feet)*	Depth (feet)
April 16, 2020 Investigation – John Simms Park					
B-1/P-1	East end of open field area	33.883505	-118.130304	64	51.5 / 22
B-2/P-2	West end of open field area	33.883503	-118.131257	64	51.5 / 20
May 3, 2022 Investigation – John Simms Park					
B-3	Center of the field area	33.883502	-118.130882	64	51.5
B-4	East side of parking area	33.884096	-118.132389	67	31.5
B-5	West side of parking area	33.884034	-118.133736	66	31.5
P-3	Northeast portion of parking area	33.884476	-118.132289	67	3.5
P-4	Central portion of parking area	33.884128	-118.133090	66	3.5
P-5	Southwest portion of parking area	33.883631	-118.133359	65	3.5
Current Investigation – October 11, 2022 – Youth and Senior Citizen Center					
B-6	Proposed Youth and Senior Citizen Center area	33.8843799	-118.1334905	67	56.5
* Estimated from Google Earth Cells highlighted in light blue correspond percolation borings from the supplementary investigation					

Bulk, driven ring samples and small bag samples were retrieved at selected depths during drilling of the exploratory boring. Standard Penetration Testing (SPT) was performed using an SPT sampler driven by an automatic 140-pound hammer with a drop of 30 inches in general accordance with ASTM D1586. The hammer calibration record indicated an energy transfer ratio of 90 percent. Ring samples were collected utilizing a California-type sampler driven by the same equipment used for the SPTs. Sampling was carried out at 2.5-foot vertical intervals within the upper 15 feet and at 5-foot vertical intervals at deeper depths.

The soil boring was surface-logged by a California Professional Geologist in general accordance with the visual-manual procedure for description and identification of soils, ASTM D2488. The Geologist prepared the recovered samples for subsequent reference and laboratory testing.

At the completion of drilling, the boring was backfilled with a bentonite cement grout in accordance with LACDPH requirements. The excavated soils from the borehole were disposed at a location specified by City of Bellflower personnel.

4. LABORATORY TESTING

The current investigation included laboratory tests on selected samples recovered from the soil boring to aid in the classification of soils and to evaluate their pertinent engineering properties. The following tests were performed:

- Moisture Content of Soil, ASTM D2216;
- Density of Soil Specimens, ASTM D7263;
- Particle Size Analysis of Soils, ASTM D6913 and ASTM D7928;
- Atterberg Limits, ASTM D4318;
- Expansion Index, ASTM D4829;
- Direct Shear Testing, ASTM D3080;
- Consolidation Testing, ASTM D2435;
- Corrosion Testing: Minimum electrical resistivity and pH, CTM 643;
Sulphate Content, CTM 417;
Chloride Content, CTM 422.

The following tests were performed during the course of the 2 previous investigations:

- Moisture Content of Soil, ASTM D2216;
- Density of Soil Specimens, ASTM D7263;
- Particle Size Analysis of Soils, ASTM D6913 and ASTM D7928;
- Atterberg Limits, ASTM D4318;
- Compaction Testing, ASTM D1557;
- Direct Shear Testing, ASTM D3080;
- Consolidation Testing, ASTM D2435;
- Expansion Index, ASTM D4829;
- R-value, ASTM D2844;
- Corrosion Testing: Minimum electrical resistivity and pH, CTM 643;
Sulphate Content, CTM 417;
Chloride Content, CTM 422.

Results of all laboratory tests are presented in Appendices B1 and B2 for the current and previous investigations, respectively. For ease of referral to the soil profile, most of the laboratory results are also included on the boring logs in the respective Appendices A1 and A2.

5. SUBSURFACE CONDITIONS

5.1. Regional Geology

The project site is located in the central portion of the greater Los Angeles. The Los Angeles Basin is located within Peninsular Ranges geomorphic province which is characterized by generally northwest trending elevated ground, hills and mountain ranges with intervening valleys. Topographic relief across the basin includes a low-lying plain that rises gently from the Pacific Ocean inland to the surrounding Santa Monica and San Gabriel Mountains to the north, Puente Hills to the northeast, the Santa Ana Mountains to the southeast, and the San Joaquin hills and Palos Verdes Peninsula to the south. Structurally, the Peninsular Ranges province includes northwest-southeast trending structural blocks separated by northwest-southeast trending strike-slip faults. The coastal portion of the basin is filled with several thousand feet of recent to later Tertiary age sediments.

5.2. Site Geology

Regional geologic mapping published by Roffers and Bedrossian (2010) (Figure 3 – Geologic Map), shows the site to be underlain by young alluvial fan deposits (Qyf) of Holocene to late Pleistocene age consisting of gravel, sand, and silt. Generalized descriptions of subsurface materials reported in this investigation includes mostly native alluvial soils. Minor amounts of artificial fill soil were encountered in the exploratory boring conducted during this study and these findings are in good agreement with the findings from previous explorations. Reported subsurface conditions are described in the following sections and the boring logs are included in Appendices A1 and A2.

5.3. Artificial Fill

The project site is underlain by about 2.5 feet of artificial fill material associated with the previous development of the site, generally consisting of fine silty sand, medium dense, moist, and containing varying amounts of fine gravel and some concrete fragments. The material is similar to the underlying native alluvium, suggesting that the native soil was amended for use as fill.

5.4. Native Alluvium (Qyf)

Native young alluvial soils were encountered below the artificial fill to the maximum explored depth of 56.5 feet and included both fine- and coarse-grained soils. Most of the encountered materials were coarse-grained and consisted of light olive gray to greenish gray, brown to olive brown silty sands with a few interspersed layers of poorly-graded sands. The fine-grained materials consisted of light olive gray to dark gray, and brown silts and sandy silts.

Uncorrected SPT blowcounts in the native alluvium for the fine-grained (clay, silt) soils ranged from 6 to 17 indicating firm to very stiff consistencies. SPT blowcounts in the native alluvial coarse-grained (sandy) soils ranged from 9 to 39 indicating loose to dense materials. The characteristics and properties of the native alluvium encountered in the current investigation

matches very closely the properties and characteristics of the native alluvium encountered during the 2 previous investigations.

5.5. Groundwater

Groundwater after drilling was encountered during the current investigation in boring B-6 at a depth of 32 feet (elevation 35 feet). During previous investigations groundwater was encountered at similar depths and elevations.

According to the State of California Seismic Hazard Zone Report for the South Gate 7.5-minute Quadrangle (CDMG, 1998), the historic high groundwater level near the site has been mapped at a depth of about 8 feet, i.e., about elevation 56 feet (Figure 4 – Historic High Groundwater Map), i.e., about 20 feet higher than the groundwater level encountered during the field investigation. It is noted that the CDMG groundwater contours are based on early last century water well logs (Mendenhall, 1905, Conkling, 1927) and also include water measurements from wells from the Central Basin Investigation (State Water Resources Board, 1952), Department of Water Resources (circa 1940's) which reflect conditions previous to massive infrastructure and urban development that has taken place in the last 50 years and therefore they do not likely correspond or apply to current conditions and circumstances.

Fluctuations of the groundwater level, localized zones of perched water, and increased soil moisture content should be anticipated during and following the rainy season. Irrigation of landscaped areas on or adjacent to the site can also cause a fluctuation of local groundwater levels. Evaluation of such factors is beyond the scope of our services.

Based on the research and observed conditions, groundwater is not expected to impact the construction of the proposed development, however the historic high groundwater depth of about 8 feet should be considered for the assessment of liquefaction and for structural design. Therefore, any proposed structures embedded or buried deeper than 8 feet from the existing grade should consider hydrostatic lateral and uplift forces. During construction the Contractor should expect groundwater depths below about 30 feet from existing grade.

6. ENGINEERING SEISMOLOGY AND GEOLOGIC HAZARDS

6.1. General Seismic Setting

The Southern California region is known to be seismically active. Earthquakes occurring within approximately 60 miles of the site are considered capable of generating ground shaking of engineering significance to the proposed construction. The project is located in the general proximity of several active and potentially active faults, as shown on Figure 5 – Regional Faults and Seismicity Map. Active faults are defined as those that demonstrated evidence of surface displacement within the Holocene period (approximately the last 11,000 years).

Active faults within approximately 10 miles of the subject site include the Puente Hills Blind Thrust fault located 1.4 miles north of the site, the Los Alamitos fault located approximately 2.8 miles south of the site, the Newport-Inglewood fault located approximately 5.3 miles southwest of the site, the Whittier fault located approximately 9.2 miles northeast of the site, and the Compton Blind Thrust fault located about 10 miles southwest of the site. The San Andreas Fault is located about 41.6 miles to the northeast of the site.

Table 2 lists selected known active faults that may affect the subject site and their associated potential maximum moment magnitude (M_{max}) as published by Cao et al. (2003) for the California Geological Survey (CGS). The approximate distances to the site were calculated from Jennings (2010).

Superimposed on the area map in Figure 5 are epicenters of earthquake magnitudes greater than M_w 5.0 as recorded by the USGS between 1900 to present day. Significant seismic activity for the period has been recorded surrounding the project site. However, relatively few earthquake epicenters have been recorded in the immediate area of the subject site. Notable historic earthquakes in Southern California of significance to the project are listed in Table 3. The most significant historic earthquake near the project site was the 1933 Long Beach earthquake.

Table 2
Nearby Known Active Faults

Referenced Site Latitude and Longitude: 33.884412°, -118.133623°

Fault Name	Approximate Fault Distance to Site¹ (miles)	Maximum Moment Magnitude² (Mmax)
Puente Hills Blind Thrust	1.4	7.1
Los Alamitos	2.8	6.2
Newport-Inglewood	5.3	7.1
Whittier	9.2	6.8
Compton Blind Thrust	10.0	7.3
THUMS-Huntington Beach	11.4	7
Palos Verdes	11.7	7.3
Charnock	14.3	6.5
Cabrillo	14.4	6.8
Redondo Canyon	16.2	6.5
Hollywood	16.2	6.4
Raymond	16.2	6.5
Verdugo	18.7	6.9
Sierra Madre	20.2	7.2
Santa Monica	20.6	6.6
Malibu Coast	31.9	6.7
Anacapa-Dume	37.5	7.5
San Andreas	41.6	7.8
Notes: ¹ per Jennings, 2010 ² per Cao, et al., 2003		

Table 3
Historic Earthquakes in Southern California
Referenced Site Latitude and Longitude: 33.884412°, -118.133623°

Earthquake Name	Year	Fault and Fault Type	Earthquake Magnitude*	Epicenter	
				Latitude	Longitude
La Habra	2014	Shallow previously unknown fault	5.1 M _w	33.933°N	117.916°W
Chino Hills	2008	Yorba Linda Trend	5.4 M _w	33.949°N	117.766°W
Northridge	1994	Northridge Thrust (Blind Thrust) (a.k.a. Pico Thrust)	6.7 M _w	34.21°N	118.54°W
Sierra Madre	1991	Clamshell-Sawpit Canyon Fault (Reverse)	5.8 M _L	34.20°N	118.14°W
Pasadena	1988	Raymond Fault (left lateral strike-slip)	5.0 M _w	34.14°N	118.13°W
Whittier Narrows	1987	Puente Hills Fault (Blind Thrust Fault)	5.9 M _L	34.06°N	118.08°W
San Fernando	1971	San Fernando Fault (thrust)	6.5-6.7 M _w	34.42°N	118.37°W
Torrance-Gardena	1941	Palos Verdes Fault (right-reverse)	4.8 M _L	33.82°N 33.78°N	118.22°W 118.25°W
Long Beach	1933	Newport-Inglewood Fault (right- lateral strike-slip)	6.4 M _w	33.63°N	118.00°W
San Jacinto	1923	San Jacinto Fault (right- lateral strike-slip)	6.3 M _L	34.00°N	117.24°W
San Jacinto	1918	San Jacinto Fault (right- lateral strike-slip)	6.7 M _w	33.65°N	117.43°W
Elsinore	1910	Elsinore Fault (right- lateral strike-slip)	6 M _L	33.75°N	117.45°W
Fort Tejon	1857	South Central Segment of the San Andreas Fault (right- lateral strike-slip)	7.9 M _w	35.43°N	120.19°W
Notes: *M _w refers to Moment Magnitude scale M _L refers to Local Magnitude scale					

Potential seismic sources of significance to the project include previously described active faults and faults that are not known to break the ground surface but are considered active. This latter group of faults includes “buried” or “blind” thrust faults. Current tectonic models for the Los Angeles Basin include buried thrust faults, several of which are considered partly responsible for the north-to-south compression of the basin. Although these faults are not currently zoned by the State of California for surface rupture hazards (Earthquake Fault Zones), many are considered capable of generating seismic shaking of significance to structures.

Of these buried active faults, the closest to the site is the Puente Hills Trust Fault (PHTF). The PHTF is currently defined as 3 separate but juxtaposed, generally east-west trending and north-dipping, fault surfaces that combined extend from Downtown Los Angeles to Brea. From west to east these include the Los Angeles, Santa Fe Springs, and Coyote Hills segments. Based upon recent studies by several researchers, including Shaw et al., (2002), Olsen and Cooke (2005), and Leon et al. (2007), the three fault surfaces are interpreted to extend from depths in excess of 9 miles on the north side of the Los Angeles basin to less than 1.2 miles at the southerly limits of the fault surfaces in the central portion of the basin. Fault surface geometries are interpreted from historical petroleum exploration data, geotechnical subsurface exploration data, and limited seismicity (e.g., the 1987 magnitude 5.9 Whittier Narrows earthquake).

Leon et al. (2007) estimates that upwards of 60 percent of the total Los Angeles basin compression may be attributed to strain along the PHTF. Although ground rupture has not been officially attributed to the fault, the presence of youthful hills (e.g., Coyote Hills) and shallow folding at depth in the upper portion of the interpreted thrust ramp suggests recent activity. The PHTF is considered capable of generating earthquake magnitudes up to about M_w 7.1.

6.2. Surface Fault Rupture

Official Maps of Earthquake Fault Zones were reviewed to evaluate the location of the project site relative to active fault zones. Earthquake Fault Zones (known as Special Studies Zones prior to 1994) have been established in accordance with the Alquist-Priolo Special Studies Zones Act enacted in 1972. The Act directs the State Geologist to delineate the regulatory zones that encompass surface traces of active faults that have a potential for future surface fault rupture. The purpose of the Alquist-Priolo Act is to regulate development near active faults in order to mitigate the hazard of surface fault rupture.

The site is not located within a designated Earthquake Fault Zone for fault surface rupture hazard. Based on a review of State of California Earthquake Fault Zone maps, the closest fault zone for surface rupture is the Newport Inglewood Fault Zone approximately 5.3 miles southwest of the site according to the CGS website application (<https://maps.conservation.ca.gov/cgs/EQZApp/app/>).

No surface traces of any active or potentially active faults are known to pass directly through or project towards the site. Neither our field exploration nor literature review disclosed an active fault trace projecting to the ground surface in the project area. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low.

6.3. Seismic Hazard Zones

Maps of seismic hazard zones are issued by the California Geological Survey (CGS, formerly California Department of Conservation, Division of Mines and Geology (CDMG)) in accordance with the Seismic Hazards Mapping Act enacted in April 1997. The intent of the Seismic Hazards Mapping Act is to provide for a statewide seismic hazard mapping and technical advisory program to assist cities and counties in developing compliance requirements to protect the public health and

safety from the effects of strong ground shaking, liquefaction, landslides, or other ground failure and other seismic hazards caused by earthquakes.

Based on the review of the South Gate Quadrangle Official Map of Seismic Hazard Zones issued March 25, 1998 (see Figure 6 – Seismic Hazard Zones Map), the proposed development is located within an area identified by the State of California as subject to the hazard of liquefaction. Because of the encountered subsurface conditions and because the site is located in a mapped area where the potential for liquefaction exists, analyses were performed per the 2022 CBC to evaluate the site liquefaction potential.

The site is relatively level and no evidence of landsliding was observed on or in the immediate vicinity of the site. Also, the site is not located in an earthquake-induced landslide hazard zone on the State of California Seismic Hazard Zones Map Earthquake-Induced Landslides. Therefore, an earthquake-induced landsliding is not considered to be hazard at the site.

6.4. Liquefaction Potential and Seismic Settlement Assessment

The site, as previously noted, is within an area identified by the State of California as potentially subject to the hazard of liquefaction. Liquefaction of soils can be caused by ground shaking during earthquakes and is generally known to occur in saturated or nearly saturated soils at depths shallower than about 50 feet. Research and historical data indicate that loose, relatively clean granular soils and low plasticity silts are susceptible to liquefaction whereas the stability of clayey silts, silty clays and clays are not typically adversely affected by ground shaking. Materials that are above the groundwater table are not considered susceptible to liquefaction.

Seismic settlement can occur in both unsaturated and saturated soils when loose to medium-dense granular soils undergo contractive volumetric changes during ground shaking. The following sections present our assessment of liquefaction potential and seismic settlement at the site.

6.4.1. Groundwater Level for Liquefaction Analysis

Groundwater was encountered during the field exploration at a depth of 32 feet. However, the historic high groundwater at the site was mapped by CDMG (South Gate Quadrangle) at a depth of about 8 feet. Therefore, consistent with SP 117, a groundwater depth of 8 feet was assumed for evaluation of liquefaction potential at the site.

6.4.2. Liquefaction Seismic Demand

Based on the Structural Engineers Association of California (SEAOC) website application (<https://www.seaoc.org/page/seismicdesignmaptool>), for a project site with latitude and longitude 33.8834412°, -118.133623° the mapped geometric mean peak ground acceleration (PGAM) for a ground motion corresponding to the Maximum Considered Earthquake (MCE) for a Site Class D was determined to be 0.749g. From the USGS Seismic Hazard Interactive Deaggregation website (<https://earthquake.usgs.gov/hazards/interactive/>) this ground motion corresponds approximately to a predominant earthquake with a magnitude of M_w 7.3. The largest contributor to the seismic hazard at the site (about 18 percent) is the Compton Blind Thrust fault located about 6.5 miles

from the site according to the website. These ground motion parameters were used in the liquefaction analyses. A summary of the seismic demand parameters is presented in Appendix C.

6.4.3. Evaluation of Liquefaction Potential and Sensitivity

The liquefaction potential was evaluated utilizing the procedure published by Boulanger and Idriss and (2014) and as recommended in County of Los Angeles Administrative Manual GS 045.0 (2014), based on the SPT blowcounts from borings B-1, B-2, and B-3 from the previous explorations and from boring B-6 from the current exploration, and on laboratory test data.

The analyses based on standard penetration test (SPT) considered the energy ratio correction factor C_E of 1.48 for borings B-1 and B-2, and a C_E of 1.34 for boring B-3, consistent with the respective hammer calibration records. A borehole diameter factor C_B of 1.0 was used per SP 117 based on the internal diameter of the hollow stem auger used during drilling. The blowcounts recorded for soils driven with the 3-inch O.D. California-type sampler with brass rings were converted to equivalent SPT blowcounts using a reduction factor of 0.67 as recommended by SP 117.

Seismic sensitivity of fine-grained soils was further evaluated per GS045.0 where the fine-grained soils are classified in the following 3 categories:

1. Soils with Plasticity Index < 12 and moisture content > 85 percent of the liquid limit are classified as fine-grained soils susceptible to liquefaction (typically includes silts);
2. Soils with Plasticity Index > 18 and a degree of sensitivity $S_t > 6$ are classified as fine-grained soils potentially susceptible to significant loss of strength during seismic shaking and require additional evaluation. The sensitivity of the on-site fine-grained soils is evaluated based on the water content, Atterberg limits, and effective vertical stresses using the procedures suggested by Holtz and Kovacs (1981) and Terzaghi, Peck and Mesri (1996).
3. Fine-grained soils falling outside the two categories described above are considered to behave like clays and are not considered susceptible to liquefaction or seismic sensitivity.

Evaluation of the liquefaction potential of granular soils and fine-grained soils with Plasticity Index less than 12 are presented in Appendix D and the results are summarized in Table 4 in the next section of the report. Analyses of the seismic sensitivity of saturated fine-grained soils were not performed because the Plasticity Index of the on-site fine-grained soils was less than 18 indicating that the soils are not susceptible to cyclic softening. The analyses based on SPT data from the soil borings indicate that most of the on-site sandy silts and silty sands and poorly-graded sands to a depth of about 50 feet are susceptible to liquefaction.

6.4.4. Seismic Settlement

Seismic settlement can occur in saturated soils due to liquefaction or in unsaturated soils due to densification of the soil matrix. The anticipated seismic settlement of the saturated soils was evaluated using SPT blowcounts using procedures outlined by Yoshimine et al (2006). The settlements calculated by this procedure were further adjusted by a calibration factor of 0.9 as

recommended by Cetin (2009). The seismic settlement of unsaturated sands was calculated based on SPT blowcounts according to the procedure outlined by Pradel (1998a and 1998b).

Table 4 presents the results of the liquefaction analyses and the corresponding settlement as well as the seismic settlement of unsaturated soils. Details of seismic settlement analyses are presented in Appendix D.

As shown in Table 4, the total (combined) seismic settlement was estimated to range between 3.4 and 6.5 inches. The differential seismic settlement is estimated to be on average about 2.7 inches over a horizontal distance of 30 feet. Because these settlement magnitudes exceed the generally accepted thresholds for structural mitigation of seismic total and differential settlement of 4 inches and 1 inch in 30 feet, respectively, mitigation of the adverse effects of liquefaction induced settlement will be required for the development of the project.

Table 4
Results of Seismic Settlement Analyses

Exploration No.	Liquefiable Zone Depth Interval (feet)	FS _{liq}	Settlement (inches)		
			Liquefaction	Unsaturated	Combined Seismic
April 16, 2020 Investigation – John Simms Park					
B-1	8-15	0.5	6.5	-	6.5
	20-30	0.3			
	35-50	0.3			
B-2	8-15	0.3	3.4	-	3.4
	44-50	0.3			
May 3, 2022 Investigation – John Simms Park					
B-3	8-15	0.3	6.0	-	6.0
	25-30	0.3			
	40-50	0.2			
Current Investigation – October 11, 2022 – Youth and Senior Citizen Center					
B-6	10-15	0.3	5.9	-	5.9
	20-25	0.3			
	35-50	0.3			

6.5. Lateral Spreading

Since the overall site is leveled and there are no open sloping faces, lateral spreading is not a hazard at the site.

6.6. Subsidence

Land subsidence is the settlement of the ground surface due to extraction or lowering of water levels or other fluids within the subsurface soil pores, or due to seismic activity. Groundwater

withdrawal causes the alluvial sediments in the basin to compact. Fine-grained materials such as clays and silts that comprise the aquitard that separates the Upper and Lower aquifers in the east valley are more susceptible to compaction and subsidence than coarse-grained sediments, such as sands when groundwater is removed. Damage caused by subsidence can be visible cracks, fissures, or surface depression.

The site is mapped within an area of land subsidence in California (https://ca.water.usgs.gov/land_subsidence/california-subsidence-areas.html) due to groundwater pumping. Provided that groundwater management strategies that include subsidence mitigation through an existing groundwater recharge program continue to be employed in the area, ground subsidence beneath the site that could result in damage to future site improvements is unlikely to occur at the site.

6.7. Collapsible Soils

The phenomenon of hydro-consolidation is typically exhibited in geologically young, unconsolidated, low-density, loose, dry soils commonly present in arid to semi-arid regions. Collapsible soils are usually composed of granular particles that are supported by a clay or silt matrix that can be chemically cemented in place creating a porous structure. The bonds supporting this porous structure have enough shear strength to support loads at low moisture contents, however, once water is introduced the cemented bond structure breaks down and the granular particles are re-arranged causing significant volume loss and associated settlement.

Based on our field investigation, the soil materials below the artificial fill are alluvial deposits consisting of stiff to very stiff silts and medium dense sands. Such deposits are generally not susceptible to collapse; therefore, the potential of collapse is expected to be low. Based on the consolidation testing of 4 soil samples (2 samples tested during this investigation) it was observed that none of the samples exhibited any collapse during wetting. Therefore, collapsible soils are not considered a hazard to the site.

6.8. Expansive Soils

Expansive soils are characterized by volume changes (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from precipitation, landscape irrigation, utility leakage, roof drainage, perched groundwater, drought, or other factors which can cause unacceptable settlement or heave that could negatively affect the performance of structures or slabs supported on grade.

The Expansion Index of 3 selected surficial samples taken as a composite of the soils within the upper 5 feet was 0 (sample tested during this investigation), 8 and 9, which indicates that the surficial soils at the site are considered non-expansive. Consequently, expansive soils are not considered a geologic hazard to the project.

6.9. Oil and Gas Fields

The site is not located within an oil field according to the State of California Department of Conservation's Division of Oil, Gas and Geothermal Resources Map (<https://maps.conservation.ca.gov/doggr/wellfinder/>).

6.10. Gaseous Hazards

Methane is a naturally occurring gas associated with the decomposition of organic materials. In high concentrations, methane is considered to be an explosion hazard. The site is not within 300 feet of an oil or gas well or 1,000 feet of a methane producing site. Therefore, a methane mitigation system may not be required according to the LA County website (<https://pw.lacounty.gov/epd/swims/OnlineServices/search-methane-hazards-esri.aspx>). The site is not located within an oil or gas field according to the State of California Department of Conservation's Division of Oil, Gas and Geothermal Resources Map (<https://maps.conservation.ca.gov/doggr/wellfinder/>).

6.11. Flood Hazard

The project site is not located within an area mapped by the California Department of Water Resources within the 100-year, 200-year, or 500-year projected floodplains (<https://gis.bam.water.ca.gov/bam/>).

6.12. Tsunamis and Seiches

A tsunami is a sea wave generated by large-scale displacements of the ocean floor that causes a sudden surge of water onto the land. Tsunamis are most commonly caused by movement along faults and underwater landslides activated by earthquakes. Seiches are earthquake-induced displacements of water within an enclosed body of water such as a lake. Strong ground motions from an earthquake cause the water to slosh back and forth onto land. The site is at an elevation of about 67 feet above sea level, is not located near a lake, and is located outside of a designated Tsunami Inundation Area per https://maps.conservation.ca.gov/cgs/informationwarehouse/-ts_evacuation/. Therefore, hazard associated with a tsunami or seiches is not anticipated at the site.

7. DESIGN RECOMMENDATIONS

7.1. General

Based on the results of the field exploration and engineering analyses, it is Tetra Tech's opinion that the proposed construction is feasible from the geotechnical standpoint, provided that the recommendations contained in this report are incorporated into the design plans and implemented during construction. It is expected that following implementation of seismic settlement mitigation measures conventional foundation and construction methods will be suitable for the proposed improvements.

The key geotechnical design consideration is the seismic settlement in excess of 4 inches. It is recommended that the seismic settlement be mitigated by installation of a mat foundation founded on subgrade augmented with a geogrid-reinforced aggregate raft.

The design recommendations presented below are based on Tetra Tech's current understanding of the project. Once the project configuration is finalized and the design is complete, Tetra Tech should review the plans and specifications to evaluate if the geotechnical design recommendations have been incorporated as intended.

7.2. Clearing and Grubbing

Prior to any site grading, surface vegetation, trash, and debris should be removed and disposed of offsite. Existing subsurface installations, such as abandoned foundations, pipes, cables, utility collectors, and/or tanks, if present, should be removed or abandoned per the Geotechnical Engineer's recommendations and in accordance with applicable regulations.

7.3. Site Preparation

- The proposed building mat foundation should be supported on a 52-inch-thick geogrid-reinforced aggregate fill raft (geogrid raft) placed on processed native soils subgrade. The geogrid-reinforced raft should extend outside the structure footprint for a horizontal distance of at least 5 feet. After completion of the removals to the geogrid raft bottom to a depth of about 5 feet and 10 inches as discussed in Section 7.4 below, the exposed subgrade should be scarified and recompact at 110 percent of optimum moisture content to at least 90 percent relative compaction per ASTM D1557.
- Lightly loaded ancillary structure areas (e.g., trash enclosures, fence walls) should be overexcavated to a depth of at least 1.5 feet below the foundation subgrade or to competent native soils, whichever is deeper. The excavation should extend, whenever feasible, for a horizontal distance of at least 2 feet beyond the outside perimeter of the structure. The exposed overexcavation subgrade should be scarified and recompact at 110 percent of optimum moisture content to at least 90 percent relative compaction per ASTM D1557.
- Flatwork areas including exterior flatwork slabs should be overexcavated and recompact to a depth of at least 1 foot below the proposed subgrade, or to uniform acceptable soils,

whichever is deeper. To the extent practicable, the zone of overexcavation should extend a horizontal distance of at least 1 foot beyond the flatwork.

- In non-structural/landscaped areas any existing fill may remain in place. However, depending on the future use of the area, existing fill may need to be excavated and replaced as compacted fill. This can be evaluated during grading.
- Disturbed soils at structural and non-structural areas will likely occur after demolition of existing site improvements. These soils should be overexcavated and recompacted to the total depth of the disturbed material.

The exposed overexcavation subgrade for all structures, slabs, pavements and flatwork should be probed and accepted by the Geotechnical Engineer. Localized zones of loose and/or unstable soils that may be encountered during overexcavation efforts should be overexcavated to a depth exposing firm and unyielding earth materials and then recompacted. If loose and/or unstable areas are encountered during grading operations that are not practical to be excavated and processed, Table 5 provides general options for stabilizing the subgrade. The specific type of remediation and associated area limits will need to be evaluated in the field by the Geotechnical Engineer.

Table 5
Options for Handling Unstable Materials at the Excavated Subgrade

Areas where the soils are soft and/or unstable at the excavation subgrade	• Overexcavate at least 2 feet • Stabilize the soft subgrade by gradually working single-grain thickness layers of open-graded aggregate material (typically 3/4" or 1.5" crushed rock, coarser for softer subgrade) at least 4 to 6 inches into the soil. • Place non-woven geotextile, Mirafi RS580i or approved equivalent, over the stabilized subgrade. • Place and compact well-graded fill (e.g., AB, CMB) or general approved backfill material to specified compaction over the geotextile.
<u>Larger</u> areas where the soils are <u>excessively</u> soft and/or unstable	• Overexcavate at least 2 feet • Improve the soft subgrade by gradually working single-grain thickness layers of open-graded aggregate material as much as possible/practical into the subgrade. • Place non-woven geotextile, Mirafi RS580i or approved equivalent, over the exposed soil. • Place at least 8 inches (12-18 inches preferred) of well graded aggregate material (e.g., AB, CMB); only reasonably achievable compaction is required. • Place non-woven geotextile, Mirafi RS580i or approved equivalent, over the aggregate layer. • Place and compact fill to specified compaction over the geotextile.

7.4. Geogrid-Reinforced Raft Configuration

The geogrid raft system has been designed utilizing a method developed by Tensar (2021) using their proprietary design software (<https://www.tensarplus.com/software/>) for foundations on liquefied soils, based on the method developed by Lees (2018 and 2019). The design provides a Factor of Safety of 3 against a bearing capacity failure of liquefied soils present in proximity of the geogrid raft bottom. It is shown (Bray and Macedo, 2017) that the bearing failure Factor of Safety of 3 reduces the seismic shear-induced building settlement to 1 inch. This design methodology has been empirically verified and broadly implemented in New Zealand following the 2010 and 2012 Christ Church earthquakes (MBIE, 2015 and 2017).

The geogrid raft should consist of 4 layers of high strength geogrid spaced vertically 10 inches embedded in coarse aggregate fill. The total thickness of the geogrid raft is 52 inches (4 feet 4 inches). It is recommended that the building foundation consists of a thickened edge mat. Considering that an embedment of at least 1.5 feet is recommended, the total depth to the geogrid raft bottom is 70 inches (5 feet and 10 inches).

Following the recompaction of the geogrid raft subgrade, the geogrid raft should be constructed in the following sequence:

1. Place 8 inches of coarse aggregate fill;
2. Install 1st geogrid;
3. Place 10 inches of coarse aggregate fill;
4. Install 2nd geogrid;
5. Place 10 inches coarse aggregate fill;
6. Install 3rd geogrid;
7. Place 10 inches of coarse aggregate fill;
8. Install 4th geogrid;
9. Place 14 inches coarse aggregate fill;
10. Install thickened edge mat foundation

7.5. Grading Materials

7.5.1. General Fill

Excavated on-site soils may be re-used as compacted fill outside the geogrid raft to build up the required grades or as utility trench backfill, provided they are free of organics, deleterious materials, debris, and particles over 3 inches in largest dimension. Locally, particles up to 6 inches in largest dimension may be incorporated in the fill soils during grading based on specific approval and placement recommendations provided by the Geotechnical Engineer. All fine-grained soils placed as fill to achieve finish grade level or utility trench backfill, should be moisture conditioned to a minimum of 125 percent of optimum moisture content and compacted to at least 90 percent relative compaction per ASTM D1557; sandy soils should be moisture conditioned to a minimum of 110 percent of optimum moisture content and compacted to at least 90 percent relative compaction. Fill should be placed in horizontal lifts not more than 8 inches in loose, uncompacted thickness.

7.5.2. Coarse Aggregate Fill for Geogrid Raft

The coarse aggregate fill required for the 52-inch geogrid raft should be generally compliant with fine crushed miscellaneous base (CMB) specification per Greenbook 200-2.4 or similar, moisture-conditioned wet of optimum moisture content and compacted to at least 93 percent relative compaction per ASTM D1557.

7.5.3. Geogrids

The geogrids for the geogrid raft should be high strength products, preferably Tensar InterAx NX850 or other high strength biaxial or triaxial approved equivalent.

7.5.4. Import Fill

In the event that any soil materials are imported to the site, such soils should be sampled, tested, and approved by the Geotechnical Engineer prior to arrival on-site. In general, any soils imported to the site for use as fill should be predominantly granular and have an Expansion Index less than 20. Additional recommendations for site grading are provided in the “General Site Grading Recommendations” section of this report.

7.6. Temporary Sloped Excavations

It is anticipated that all excavations to reach subgrade level for the proposed building and for utility trenches can be performed as sloped excavations, i.e., without the need for shoring. Where space for sloped sides is not available, shoring will be required. Recommendations for shoring design are provided in the Temporary Shoring section of this report.

All excavations should be performed in accordance with Cal/OSHA regulations. The on-site soils to a depth of 20 feet may be considered Type B soils, as defined by the current Cal/OSHA soil classification.

Unsurcharged excavations: Sides of temporary unsurcharged excavations less than 20 feet deep may be cut vertically up to 4 feet and then sloped back at an inclination of 1 (H):1(V) or flatter.

Surcharge setback recommendations: Stockpiled (excavated) materials should be placed no closer than 4 feet, or 50 percent of the excavation height, whichever is greater, from the top of the excavation. A greater setback may be necessary when considering surcharge loads such as heavy vehicles, concrete trucks and cranes. The Geotechnical Engineer should be advised of such heavy vehicle loadings so that specific setback requirements can be established for the used equipment. Alternatively, a shoring system may be designed to allow reduction in the setback distance.

The bottom of any trenches that are required for any underground utilities and piping should be kept outside a zone defined by a 1(H): 1(V) plane projected from the outside bottom edge of the footing or mat foundations. Backfill materials and procedures shall conform to the recommendations provided in the “Site Preparation” and “General Site Grading” sections of this report. If any piping needs to be placed within the zone of influence, the pipes should be designed

to account for the increased surcharge which results from the applied footing pressures, and the pipes within that zone should be protected with concrete encasement or other suitable form of protection. The piping and encasement should be designed to withstand differential settlements of up to 1 inch in relation to the area outside of the zone of influence.

7.7. Temporary Shored Excavations

Excavation depths for the construction of the proposed project are anticipated to be at a depth between 6 to 7 feet. If sloped excavations are not feasible because of lack of space, particularly near existing sidewalks or pavements, then shoring will be needed.

Cantilevered shoring systems are typically suitable and practicable for retained heights less than about 15 feet. The shoring designer will need to take into account the presence of adjacent structures, utilities, conduits, and other underground structures and their impact on the installation and performance of the shoring system.

All components of the shoring system, including the penetration depth, should be designed by a specialist Civil Engineer registered in the State of California and should further satisfy requirements of Cal-OSHA. It is recommended that the final shoring design be reviewed by the Geotechnical Engineer.

7.7.1. Shoring Design

Either sheet pile or soldier pile and lagging systems may be utilized for the proposed construction. Table 6 below summarizes the governing geotechnical design parameters and loading diagrams for a shoring system for both cantilever and internal bracing conditions. These geotechnical parameters were developed based on the following assumptions:

- the shored soil grade is level;
- there are no hydrostatic pressures behind the wall; and
- the shoring is temporary.

If any of these assumed conditions cannot be met due to field conditions or the contractor's method of construction, this office should be consulted regarding potential revisions to our recommendations.

Any surcharge (live or dead load) located within a 1(H):1(V) plane drawn up from the excavation bottom should be accounted for in the calculation of the lateral earth pressures. For cantilevered shoring systems the lateral contribution of a uniform surcharge load may be calculated by multiplying the surcharge by a factor of 0.28. This constant lateral load, i.e., independent of depth, should be applied throughout the whole exposed height of the sheet pile or soldier pile wall. As a minimum, 3 feet of equivalent soil surcharge, i.e., 360 psf, is recommended to be included to account for nominal construction surcharge and traffic loads on the adjacent roads.

The required penetration depth of the soldier or sheet pile wall below the excavation bottom may be calculated based on the estimated passive soil resistances provided in Table 6. Passive soil

resistance should be ignored for the upper 12 inches below the excavation bottom to account for potential near-surface soil disturbance.

Table 6
Geotechnical Design Parameters
Cantilevered Temporary Shoring
(without hydrostatic pressure build-up behind the wall)

Excavation bottom depth (including overexcavation depth)	Up to 15 feet
Subsurface materials	Alluvial Soils Mostly firm to stiff silts and medium dense sands
SHORING SYSTEM	Cantilevered shoring (excavation depth less than about 12 to 15 feet)
Soil unit weight, γ	125 pcf
Design friction angle, ϕ	34°
Design cohesion, c	0 psf
K_a ... coefficient of active lateral pressure	0.28
LOADING DIAGRAM ON SHORING	
Equivalent fluid density, EFD	43 pcf
Loading Diagram	Triangular distribution
ALLOWABLE PASSIVE PRESSURE BELOW EXCAVATION BOTTOM	
Design friction angle, ϕ	34°
K_p ... coefficient of passive lateral pressure	3.5
Arching capability *	2.7 (for design of soldier pile systems)
Equivalent fluid density (pcf EFD) ** (triangular distribution) – includes Safety Factor of 1.5 – ignore resistance within upper 12 inches	Sheet pile system: 290 Soldier pile system (includes arching): 780
* Per Caltrans Trenching and Shoring Manual (2011) ** Valid without reduction for soldier pile spacing > 2.7 times the effective pile width *** Groundwater depth for temporary shoring can be assumed to be at 32 feet	

7.7.2. Deflection

It should be realized that some shoring deflection will likely occur. It is difficult to accurately predict the magnitude of shoring deflection as it largely depends on the quality and sequence of construction. The shoring system should be designed so that deflection at the top of the shored excavation is kept below 1 inch. If greater deflection occurs during construction, additional bracing or restraint may be necessary to minimize settlement of the nearby improvements. If it is

desired to reduce the deflection of the shoring, a greater lateral earth pressure could be used for the shoring design (i.e., use $K_o = 0.44$ instead of K_a , or $EFD = 68$ pcf).

7.7.3. Construction Staging

The shoring should be constructed utilizing a top-down method. Following the installation of the soldier or sheet piles, the soil is first partially excavated to allow for installation of the topmost row of internal bracing. For rakers, the staging will need to include installation of temporary rakers and their subsequent removal and replacement as the excavation advances. Following the installation of the topmost row of internal bracing, the excavation then proceeds, and each level of internal bracing should be installed as soon as practicable. The shoring designer should analyze each stage of internal bracing installation to ensure that the excavation has an adequate Factor of Safety.

For the soldier pile system, the lagging should be installed simultaneously as the excavation proceeds. To facilitate a tight connection between the lagging and the soils and to minimize settlement, any voids left behind the lagging should be filled with cement grout as the excavation advances. To continuously support the excavation, any unsupported height should not exceed 4 feet.

The annulus of the soldier pile beam borehole below the excavation bottom should be backfilled with concrete. Pea gravel may be used to backfill the hole from the excavation bottom to the top grade. If the soldier pile beams are to be retrieved after construction, they may be backfilled with a weaker cementitious slurry mix below the excavation bottom. If the contractor chooses to use a well-rounded uniform pea gravel material to fill the hole below the excavation bottom, a reduction of 33 percent should be applied to the passive soil resistance values provided in Table 6 to account for potential lateral yielding of the pea gravel backfill.

7.7.4. Shoring Performance Monitoring

Some means of monitoring the performance of the shoring system and nearby flatwork, paved surfaces and structures is recommended. The monitoring should consist of periodic visual inspections and lateral and vertical surveying of the tops of the soldier / sheet piles and strategic survey points. It is recommended that a survey be performed before construction begins and then, as the excavation proceeds, the monitoring should be performed daily or whenever excavation activities are taking place. In addition, the Contractor should inspect the shoring daily and actively search for presence of cracks or excessive movements and report immediately to the shoring designer and the Geotechnical Engineer. This office can provide further recommendations of the monitoring when the design of the shoring system is finalized.

7.8. Slot Cuts

As an alternative to shoring along the property lines, slot cuts could also be performed to a depth of 7 feet. Partial grade lowering followed by slot cut excavation to facilitate the overexcavation and perimeter foundation subgrade preparation is recommended, i.e., no shoring would be required. The construction sequence should be as follows:

1. Lower the grade within the construction footprint to form a vertical cut up to 4 feet high along the property line.
2. At a horizontal distance of at least 7 feet from the property line continue with the excavation as needed to the full overexcavation depth. It is expected that up to about 3 feet of additional excavation will be needed. This step will leave a bench at least 7 feet wide and up to about 3 feet high in front of the property line. The leading edge of the bench (away from the property line) may be cut vertically or sloped. This bench forms the buttress to stabilize the vertical cut along the property line for the subsequent slot excavation.
3. Perform slot cut excavations through the bench to reach the final overexcavation grade in front of the property line and promptly backfill by placing the reinforced-geogrid raft materials to the top of the bench. The slot width may not exceed 20 feet. Simultaneously opened slots may not be closer than 20 feet.

7.9. Seismic Design Parameters per 2022 CBC

The project site is located in an area that is subject to the hazard of liquefaction. Therefore, per ASCE 7-16 Section 20.3.1, the site needs to be designated as Site Class F and seismic design parameters need to be obtained by a site response analysis performed in conformance with ASCE 7-16 Section 21.1. However, ACSE 7-16, Section 20.3.1 provides an exception for structures having fundamental periods of vibration equal to or less than 0.5s, where site response analysis is not required for liquefiable soils and the site class may be determined in accordance with Section 20.3, in which case this site may be classified as Site Class D. Although it is expected that this exception is applicable to the proposed construction, the Structural Engineer needs to verify that the fundamental period of the proposed building meets this condition. Alternatively, this office can perform a site response analysis under a separate scope and budget.

The seismic design coefficients provided below in Table 7 are based on Chapter 16 Section 1613 of the 2022 CBC and obtained from the Structural Engineers Association of California (SEAOC) website application (<https://www.seaoc.org/page/seismicdesignmaptool>). According to ASCE 7-16 Section 11.4.8, a site-specific ground motion hazard analysis shall be performed if structures on Site Class D have an S_1 greater than or equal to 0.2 unless the seismic coefficient C_s determined by Equation (12.8-2) is used for values of $T \leq 1.5 T_s$ and taken as equal to 1.5 times the value computed in accordance with either Equation (12.8-3) for $T_L \geq T > 1.5 T_s$ or Equation (12.8-4) for $T > T_L$. Tetra Tech can provide site-specific ground motion hazard analysis under a separate scope and budget.

Table 7
2022 CBC and ASCE 7-16 Seismic Design Parameters
Referenced Site Latitude and Longitude: 33.884412°, -118.133623°

Site Class Table 20.3-1 ASCE 7-16		D
Coefficients for the Maximum Considered Earthquake, MCE_R, for Site Class B	Short Period (0.2 seconds), S_s	1.594*
	1 Second Period, S_1	0.571*
Coefficients for the Maximum Considered Earthquake, MCE_R (Site Modified)	Short Period (0.2 seconds), S_{MS}	1.594*
	1 Second Period, S_{M1}	0.999**
Coefficients for the Design Earthquake	Short Period (0.2 seconds), S_{DS}	1.063*
	1 Second Period, S_{D1}	0.666**
Design PGA (risk-targeted maximum rotated direction)		0.425g*
Site Modified Peak Ground Acceleration PGA_M (geometric mean)		0.749g*
* Values obtained from Structural Engineers Association of California (SEAOC) website application, https://www.seaoc.org/page/seismicdesignmapprool based on ASCE7-16.		
** See requirements for site-specific ground motions in Section 11.4.8 of ASCE 7-16.		

7.10. Mat Foundations

The proposed building may be supported on a structural mat foundation with subgrade augmented by geogrid-reinforced coarse aggregate fill raft as recommended in this report. Foundations should be designed and reinforced in accordance with the recommendations of the Structural Engineer and should conform to the 2022 California Building Code. The flexural demands caused by liquefaction need not be considered if the mat is detailed in accordance with the requirements of Section 18.6.3.1 of ACI 318R-19. Mat foundations should have longitudinal reinforcement in both directions at the top and bottom. The geotechnical design parameters presented in Table 8 should be used. For the liquefaction condition the foundation mat should be designed for the limiting differential threshold per ASCE 7-16 Table 12.13.3 for single story structures with concrete or masonry wall systems for Risk Categories I and II for a maximum allowable differential settlement of 2.7 inches over 30 feet (0.0075L) or for other single-story structures for the same Risk Categories for a differential settlement of 5.4 inches over 30 feet (0.015L), as applicable.

Table 8
Geotechnical Design Parameters
Mat Foundation on Geogrid Raft

Embedment depth	At least 1.5 feet below the existing grade
Allowable Bearing Pressure	Average allowable bearing pressure 2,000 psf and accounts for the presence of the geogrid-reinforced select fill raft.
Allowable Bearing Pressure for Transient Loads	The allowable bearing value may be increased by one-third for transient live loads from wind and seismic loading.
Estimated Settlement	- Approximate 1 inches of total static settlement and 0.5 inches of differential settlement over a distance of 30 feet. - Approximate seismic differential settlement of less than 2.7 inches over 30 feet.
Modulus of Subgrade Reaction	For static design of the mat foundations, a reference modulus of subgrade reaction k_1 of 200 pci derived for a square bearing plate with 1-foot x 1-foot dimensions may be used. For the geogrid mat, the modulus of subgrade reaction k (in pci) for the design of a concrete element (mat foundation) of given dimensions can be calculated as: $k = k_1 \frac{1 + 0.5 * \frac{B}{L}}{1.5 * B}$ Where B and L are the governing width and the length of the element in feet, but no more than 14 times the thickness of the element.
Allowable Friction Coefficient (incorporates Factor of Safety of 1.5)	0.40 for mass concrete on geogrid-reinforced coarse aggregate raft
Allowable Lateral Passive Resistance (incorporates Factor of Safety of 2)	220 pcf (EFD, equivalent fluid density for compacted fill with $\phi=34^\circ$) - The passive resistance derived from the upper 12 inches should be neglected.
Allowable Combined Lateral Resistance	- Total allowable resistance to lateral loads can be calculated by combining lateral resistance due to friction at the base and lateral passive resistance. - Passive resistance values may be increased by one-third when considering transient wind or seismic loading

The bottom of any trenches that are required for any buried utilities and piping should be kept outside a zone defined by a 2(H):1(V) plane projected from the outside bottom edge plus 9 inches above of any existing or proposed concrete foundation and no closer than 18 inches from the face of the foundation. Backfill materials and procedures shall conform to the recommendations provided in the “Site Preparation and Geogrid Reinforced Raft Configuration” and “General Site Grading” sections of this report. If any piping needs to be placed within the zone of influence, the pipes should be designed to account for the increased surcharge from the applied foundation pressures and to withstand potential differential settlement between the surcharged and unsurcharged segments of the pipe. Generally, the pipes within the impacted zone should be protected with concrete encasement, utilidors, or other suitable form of protection. The piping and encasement should be designed to withstand differential settlements of up to 1 inch over a distance equal to half of the depth of the pipe below the bottom of the footing. This office should be contacted to review any specific utility interaction configurations and their proposed mitigation.

7.11. Foundations For Ancillary At-Grade Structures

Table 9 presents the geotechnical design parameters for ancillary at-grade structures. Foundations should be designed and reinforced in accordance with the recommendations of the Structural Engineer and should conform to the 2022 California Building Code.

Table 9
Geotechnical Design Parameters
At-Grade Structures

Continuous Strip Footings	
Dimensions (feet)	At least 1.5 feet wide but less than 5 feet wide
Depth of Embedment	Embedded at least 1.5 feet below the lowest adjacent grade
Allowable Bearing Pressure	2,000 psf
Allowable Bearing Pressure for Transient Loads	The allowable bearing pressure value may be increased by one-third for transient live loads from wind or seismicity.
Estimated Settlement	- Approximately 1-inch static settlement - Approximately 0.5-inch static differential settlement between supports or over a distance of 30 feet - Approximate seismic differential settlement of less than 2.7 inches over 30 feet
Allowable Coefficient of Friction (incorporates Factor of Safety of 1.5)	0.3 mass concrete on soil 0.25 formed concrete on soil
Allowable Lateral Passive Resistance (incorporates Factor of Safety of 2)	220 pcf (EFD, equivalent fluid density for compacted fill with $\phi=34^\circ$) - The passive resistance derived from the upper 12 inches should be neglected.
Allowable Combined Lateral Resistance	- Total allowable resistance to lateral loads can be calculated by combining lateral resistance due to friction at the base and lateral passive resistance. - Passive resistance values may be increased by one-third when considering transient wind or seismic loading

7.12. Interior Floor Slabs or Mats

Interior floor slabs, including the foundation mats, supported on the subgrade augmented by geogrid-reinforced coarse aggregate fill raft should be designed based on recommendations provided in the Mat Foundations section of this report and reinforced in accordance with 2022 California Building Code per the Structural Engineer's recommendations.

Although the concrete slab/mat should be designed by the Structural Engineer and/or Concrete Specialist the following recommendations are provided to assist with the preliminary design and initial cost estimate. The concrete slab/mat should be reinforced with at least No.4 reinforcing bars spaced at 18 inches on-center, each way, placed at the top and at the bottom of the section. The

slab should be doweled into the building foundations to reduce the potential for differential movement. Reinforcement should be properly placed and supported on blocks or “chairs”. Welded wire mesh reinforcement is not recommended.

Control joints should be included in the design of the concrete slab as recommended by American Concrete Institute (ACI) guidelines and at a maximum spacing (in feet) of 2 to 3 times of the slab thickness (in inches), but generally no more than 10 feet. All joints should form approximately square patterns to reduce potential for randomly oriented shrinkage cracks. The control joints should be tooled at the time of the pour or sawcut to $\frac{1}{4}$ of slab depth within 6 to 8 hours of concrete placement. All joints in the concrete slabs should be sealed to prevent moisture, vermin, or foreign material intrusion. Reinforcement should extend through the control joints to reduce the potential for differential movement. Precautions should be taken to prevent curling of slabs (refer to ACI guidelines).

Per 2022 California Building Code, Section 1907.1, a minimum 6-mil thick vapor retarder is required for moisture intrusion control. Tetra Tech does not practice in the field of moisture vapor transmission evaluation/mitigation. However, to assist with the selection of the slab underlayment system, Table 10 provides alternatives for control of vapor transmission through concrete slab-on-grade that go beyond the minimum code requirements. The provided alternatives are based on local experience and may be considered appropriate for standard applications. If moisture vapor transmission is considered a risk to the use and operation of the proposed structure, we recommend that a qualified specialist be engaged to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction and to provide recommendations for mitigation of potential adverse impact of moisture vapor transmission on various components of the structure as deemed appropriate.

All underslab materials should be nominally moisture-conditioned and compacted prior to the placement of concrete. Care should be taken during placement of the concrete to prevent displacement of the underslab materials. Concrete slabs should be allowed to cure properly prior to placing vinyl or other moisture-sensitive floor covering. Floor covering placement should be performed in accordance with the manufacturer’s recommendations.

Table 10
Alternatives for Control of Vapor Migration through Concrete Slab

Objective	Recommendation (Top to bottom layering)
“Best” protection against vapor intrusion	• Concrete floor slab-on-grade placed directly on moisture vapor barrier membrane at least 15 mils in thickness, e.g., Stego Wrap • 2 inches of protective course if naturally moist silty sand^{2,5} • Separation non-woven geotextile, Mirafi 140N or equivalent. Alternatively, the voids at the top of the underlying aggregate course may be filled with fine sand to minimize potential for tearing and damage to the membrane during construction. • Capillary aggregate course of at least 4 inches of ¾-inch crushed rock³ or clean gravel⁴ • Prepared finished grade
“Better” protection against vapor intrusion	• Concrete floor slab-on-grade placed directly on plastic membrane at least 10 mils in thickness¹ • 2 inches of protective course if naturally moist silty sand^{2,5} • Prepared finished grade.
Standard protection against vapor intrusion	• Concrete floor slab-on-grade placed on 2 inches of naturally moist silty sand² • Vapor retarder barrier membrane at least 10 mils in thickness • 2 inches of naturally moist silty sand^{2,5} • Prepared finished grade
¹ If additional protection is desired, the plastic membrane may be replaced with a 15-mil thick moisture vapor barrier that meets the requirements of ASTM E 1745 Class C (for example, Stego Wrap or similar). ² The silty sand should have a gradation between approximately 15 and 35 percent passing the No. 200 sieve and a plasticity index (PI) of less than 4. ³ The ¾-inch crushed rock should conform to Section 200-1.2 of the latest edition of the Standard Specifications for Public Works Construction (Greenbook). ⁴ The gravel should contain less than 10 percent of material passing the No. 4 sieve and less than 3 percent passing the No. 200 sieve. ⁵ The silty sand layer is intended as a levelling course and to protect the membrane from any gravel grains that may be at the finished subgrade. It is not strictly required for vapor migration protection. It is recognized that the silty sand course may be difficult to protect during construction activities, consequently if a thicker/stronger membrane is utilized, e.g., 15-mil Stego Wrap, and an adequate caution is taken during the subgrade preparation and slab construction, then the silty sand course may be eliminated.	

7.13. Exterior Slabs

Exterior concrete slabs should be placed on subgrade prepared in accordance with the recommendations provided in the “Site Preparation and Geogrid-Reinforced Raft Configuration” and section of this report. A Structural Engineer or an Engineer specialized in concrete design should be consulted if cracking of the exterior slabs is to be minimized. As a minimum for exterior walkways, it is recommended that narrow strip concrete slabs, such as sidewalks, be reinforced with at least No. 4 reinforcing bars placed longitudinally at 18 inches on center. Wide exterior slabs should be reinforced with at least No. 4 reinforcing bars placed 18 inches on center, each way. Welded wire mesh reinforcement is not recommended. Reinforcement should extend through the control joints to reduce the potential for differential movement.

Placement of control joints within exterior flabs should follow the recommendations presented in section “Interior Floor Slabs or Mats” of this report.

7.14. Embedded Posts and Poles at Grade

Recommendations for design of posts and poles including fence poles and streetlamps are provided herein. The allowable static lateral soil bearing pressure can be assumed to be equal to 220 pcf equivalent fluid density (pcf EFD). The allowable passive earth pressure value incorporates a Factor of Safety of 2. For isolated poles not adversely affected by 0.5 inches of horizontal motion due to short-term lateral loads the lateral soil bearing pressure can be increased to 300 pcf EFD. Vertical compressive loading can be resisted utilizing an allowable end bearing pressure of 2,000 psf.

For the non-constrained case where the pole is not restricted to move at the ground level, the minimum depth of embedment required to resist lateral loads should be determined in accordance with the 2022 CBC Section 1807.3.2.1. For the constrained case where the pole is restricted from movement at the ground level by encasement in surrounding concrete or similar, the minimum depth of embedment required to resist lateral loads should be determined in accordance with the 2022 CBC, Section 1807.3.2.2. Where bare ground (without concrete or asphalt cover) is present adjacent to the foundation, the lateral resistance provided by the soil within the upper 12 inches below grade should be ignored.

7.15. Utility Trenches

Utility trench excavations should follow the recommendations provided in the Temporary Sloped Excavations section of this report. Construction recommendations for the trench bottom preparation and trench backfill are provided below.

7.15.1. Utility Trench Bottom Preparation

Trench bottom preparation should produce a uniform, firm, and unyielding subgrade. The exposed trench bottom should be probed and accepted by the Geotechnical Engineer. Any particle size greater than 3 inches should be removed. Surface water should be controlled so that the trench subgrade is protected even during periods of heavy rainfall.

7.15.2. Utility Trench Backfill

Bedding and pipe zone backfill. The bedding is the material placed in the bottom of the trench on which the pipeline is laid. Bedding material should extend at least 6 inches below the bottom of pipe and up to the pipe springline level. The pipe zone is defined as the area above the bedding, around the pipe, and up to typically at least 12 inches over the pipe.

Bedding and pipe zone backfill material should consist of clean sand, i.e., sand with less than about 30 percent fines and no more than 20 percent of fine gravel. The pipe bedding material should be placed over the full width of the trench on firm and unyielding subgrade soils approved by the Geotechnical Engineer. After placement of the pipe, the pipe zone backfill should be brought up

uniformly on both sides of the pipe to reduce the potential for unbalanced loads. No voids or uncompacted areas should be left beneath the pipe haunches. The silty sands and silty materials found within the upper 20 feet at the site are not expected to be suitable for pipe bedding and pipe zone backfill.

The bedding and pipe zone backfill should be moisture-conditioned to at least wet of optimum moisture content and compacted in horizontal lifts no greater than 6 inches to achieve relative compaction of at least 90 percent per ASTM D1557. The use of mechanized compaction equipment within the pipe zone should be carefully controlled to avoid overstressing or damaging the pipe. Due to the presence of near surface fine grained soils, jetting of the bedding and pipe zone backfill is not recommended.

General trench backfill. This zone extends from the top of the pipe zone to the finished grade. The backfill may consist of approved excavated soil devoid of particles greater than 3 inches in the largest dimension, moisture-conditioned wet of optimum moisture content and compacted to at least 90 percent of relative compaction per ASTM D1557. Lift thickness for backfill will be dependent on the type of compaction equipment utilized but should not generally exceed 8 inches in loose thickness. Care should be exercised to avoid damaging the pipe during compaction of the trench backfill. Within pavement areas the upper 12 inches of backfill should be compacted to at least 95 percent of maximum. Jetting of the trench backfill is not recommended.

7.16. Corrosion Potential Evaluation

The corrosion potential of the on-site soils to buried steel and concrete was evaluated based on laboratory testing of 2 near-surface soil samples from previous investigations and 1 near-surface soil sample from the current investigation. Table 11 below presents the results of the corrosivity testing.

Per 2022 CBC based on 2021 IBC, Section 1904.1, concrete subject to exposure to sulfates shall comply with the durability requirements set forth in ACI 318 Section 19.3. Based on the measured water-soluble sulfate results, the exposure of buried concrete to sulfate attack is not a concern, i.e., exposure class S0 per ACI 318-14, Table 19.3.1.1. Consequently, per Table 19.3.2.2, cement with a minimum 28-day compressive strength of 2,500 psi could be used.

Per 2022 CBC, Section 1904.1, concrete, including protection for corrosion and exposure to chlorides, shall conform to durability requirements established in ACI 318. Reinforcement should be protected from corrosion in accordance with ACI 318-14 Chapter 19.

The corrosion potential of buried metals was evaluated based on the minimum resistivity and our experience with similar soils. The on-site soils are anticipated to be “mildly corrosive to very corrosive” to buried metals as defined by the NACE (1984).

A Corrosion Specialist should be consulted regarding suitable types of piping and necessary protection for underground metal conduits. The corrosion potential of the on-site soils should be verified during construction for each encountered soil type. Imported fill materials should be

tested prior to placement to confirm that their corrosion potential is not more severe than the one assumed for the project.

Table 11
Corrosivity Test Results

Boring	Sample ID	Soil type	Depth (feet)	pH	Minimum Resistivity (ohm-cm)	Chlorides (ppm/%)	Soluble Sulfate Content in Soil (ppm/%)
May 3, 2022 Investigation – John Simms Park							
B-3	SK-1	Sandy Silt	0-5	9.1	471	579/0.0579	871/0.0871
B-4	SK-1	Sandy Clay	0-5	9.0	2,619	47/0.0047	94/0.0094
Current Investigation – October 11, 2022							
B-6	SK-1	Sandy Silt	0-5	8.1	3,320	17/0.0017	85/0.0085

7.17. Drainage Control

The intent of this section is to provide general information regarding the control of surface water. The control of surface water is essential to the satisfactory performance of the building construction and site improvements. Surface water should be controlled so that conditions of uniform moisture are maintained beneath and adjacent to the structure, even during periods of heavy rainfall. The following recommendations should be considered as minimal.

- Ponding and areas of low flow gradients should be avoided.
- Paved surfaces within 10 feet from the building foundation should be provided with a gradient of at least 2 percent sloping away from improvements.
- Bare soil, e.g., planters, within 10 feet of the structure should be sloped away from the improvement at a gradient of 5 percent.
- Positive drainage devices, such as graded swales, paved ditches, and/or catch basins should be employed to accumulate and convey water to appropriate discharge points.
- Concrete walks and flatwork should not obstruct the free flow of surface water.
- Planters should not be located immediately adjacent to structures. If planters are to be located adjacent to a structure, they should be positively sealed, should incorporate a subdrain, and should be provided with free discharge capacity to a drainage device.
- Planting areas at grade should be provided with positive drainage. Wherever possible, the grade of exposed soil areas should be established above adjacent paved grades. Drainage devices and curbing should be provided to prevent runoff from adjacent pavement or walks into planted areas.
- Gutter and downspout systems should be provided to capture discharge from roof areas. The accumulated roof water should be conveyed to an off-site disposal area by a pipe or concrete swale system.

- Landscape watering should be performed judiciously to preclude either soaking or desiccation of soils. The watering should be such that it just sustains plant growth without excessive infiltration. Sprinkler systems should be checked periodically to detect leakage and irrigation efforts should be reduced or halted during the rainy season.

8. GENERAL SITE GRADING RECOMMENDATIONS

The intent of this section is to provide general information regarding the grading of the site. Site grading operations should conform with applicable local building and safety codes and to the rules and regulations of those governmental agencies having jurisdiction over the subject construction.

The grading contractor is responsible for notifying governmental agencies, as required, and the Geotechnical Engineer at the start of site cleanup, at the initiation of grading, and any time that grading operations are resumed after an interruption. Each step of the grading should be accepted in a specific area by the Geotechnical Engineer, and where required, should be approved by the applicable governmental agencies prior to proceeding with subsequent work.

The following site grading recommendations should be regarded as minimal. The site grading recommendations should be incorporated into the project plans and specifications.

- Prior to grading, existing vegetation, trash, surface structures and debris should be removed and disposed off-site at a legal dumpsite. Any existing utility lines, or other subsurface structures which are not to be utilized, should be removed, destroyed, or abandoned in compliance with current governmental regulations.
- Subsequent to cleanup operations, and prior to initial grading, a reasonable search should be made for subsurface obstructions and/or possible loose fill or detrimental soil types. This search should be conducted by the contractor, with advice from and under the observation of the Geotechnical Engineer.
- Prior to the placement of fill or foundations, the site should be prepared in accordance with the recommendations presented in the “Site Preparation and Geogrid-Reinforced Raft Configuration” section of this report. Undocumented fill or disturbed soils within shallow foundation footprints should be removed and processed as recommended by the Geotechnical Engineer.
- The exposed subgrade and/or excavation bottom should be observed and approved by the Geotechnical Engineer for conformance with the intent of the recommendations presented in this report and prior to any further processing or fill placement. Proof rolling may be considered to provide a general guidance about the quality of the fill. It should be understood that the actual encountered conditions may warrant excavation and/or subgrade preparation beyond the extent recommended and/or anticipated in this report.
- On-site inorganic granular soils that are free of debris or contamination are considered suitable for placement as compacted fill. Any rock or other soil fragments greater than 3 inches in size should not be placed within 5 feet of a foundation subgrade.
- Any imported fill material required for backfill or grading should be tested and approved prior to delivery to the site.

- Visual observations and field tests should be performed during grading by the Geotechnical Engineer. This is necessary to assist the contractor in obtaining the proper moisture content and required degree of compaction.
- Wherever, in the opinion of the Geotechnical Engineer, an unsatisfactory condition is being created in any area, whether by cutting or filling, the work should not proceed in that area until the condition has been corrected.

9. DESIGN REVIEW CONSTRUCTION SERVICES

Geotechnical review of plans and specifications and participation during the construction are an integral part of the design practice. The following paragraphs present our recommendations relative to the review of construction documents and the monitoring of construction activities.

9.1. Plans and Specifications

Upon completion, the civil and structural design plans and specifications should be reviewed and approved by Tetra Tech prior to submittal for issuance of grading, building, and/or construction permits as the geotechnical recommendations may need to be re-evaluated based on the actual design configuration and loads. This review is intended to evaluate whether the recommendations contained in this report have been incorporated into the project plans and specifications as intended.

9.2. Construction Monitoring

The objective of the construction quality assurance (CQA) is to assist in the construction of the soils and soils-structure interaction components of the Project. Observation of site overexcavation, processing and assessment of fill materials, fill placement, and other site grading operations by the Geotechnical Engineer should be implemented during construction to allow for evaluation of the geotechnical-related conditions as they are encountered. This process provides the Geotechnical Engineer with the opportunity to recommend appropriate revisions as needed.

9.2.1. Grading Observations

Continuous observations by the Geotechnical Engineer should be provided to assess all encountered soil types, verify the extent of removals and overexcavation, suitability of import materials, lift thicknesses and densities during placement and compaction of fill materials. The Geotechnical Engineer should observe all temporary excavations and construction slopes, as well as the backfill operations so that appropriate modifications to the design criteria presented herein may be recommended, if necessary, due to encountered conditions differing from the design assumptions.

9.2.2. Shoring and Excavation Observations

The Geotechnical Engineer should observe the shoring installation and excavation progress so that appropriate modifications to the excavation design may be recommended, if necessary, due to conditions differing from the design assumptions. The observations should focus on verification of the on-site materials, soldier pile installation depths and backfill, excavation and lagging installation sequencing, lagging backfill, and review and interpretation of the shoring movement survey.

9.2.3. Foundation Subgrade Observations and Construction of Geogrid- Reinforced Raft

The Geotechnical Engineer should observe and evaluate the presence of satisfactory materials at the foundation subgrade. Foundation excavations should be clean of loosened soil and debris before placing steel or concrete. If soft or loose soils or other unsatisfactory materials are encountered, such materials should be removed and replaced with compacted fill prior to pouring of concrete. The Geotechnical Engineer should also observe the placement of the geogrid-reinforced raft including the placement of high strength geogrids.

9.2.4. Construction Quality Assurance Reporting

The following list is intended to provide basic minimum guidelines for the reporting during the excavation and backfilling operations:

- A Daily Field Report should be generated each time a representative of the Geotechnical Engineer is performing QA work at the site.
- The Daily Field Reports should contain, at a minimum, a detailed description of the field activities, utilized equipment, areas of work, date, time, weather, and locations and results of all observations and performed tests.
- Provisions should be made for vertical and horizontal control for recording observations and test locations.
- A complete set of Daily Field Reports should be submitted as a part of formal final reporting.
- Visual observations and field tests should be performed during grading by the Geotechnical Engineer. This is necessary to assist the contractor in obtaining the proper moisture content and required degree of compaction. In general field compaction testing should be performed as recommended in Table 12.

Table 12
Field and Laboratory Testing

Test or Observation	Test Method	Minimum Testing Frequency
Field Density	Maximum dry density and optimum moisture content by ASTM D1557	As deemed necessary or the most frequent of: 1 per major material type 1 per 4 weeks of construction
Field Compaction	Field Density by Sand cone (ASTM D1556) or Nuclear gauge (ASTM D2922)	As deemed necessary or the most frequent of: 1 per 1,000 CYs, or 1 per each 2 compaction lifts, or 2 per full day of placement 1 per 250 LF of roadway

10. SECTION 111 STATEMENT

Based on the data and evaluations presented in the report, it is the opinion of Tetra Tech that the subject project site for the proposed Bellflower Youth and Senior Citizen Center facilities will be safe against hazards from future landsliding, settlement or slippage and that the proposed grading construction as long as the recommendations provided herein are implemented will have no adverse impact on the geologic stability of property outside of the project site.

11. LIMITATIONS

The recommendations and opinions expressed in this report are based on Tetra Tech's review of background documents, geotechnical investigation data obtained for this project, laboratory testing, and engineering analyses. It should be noted that this study did not evaluate the presence of hazardous materials on any portion of the site.

Due to the limited nature of the field explorations, conditions not observed and described in this report may be present on the site. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation and laboratory testing can be performed upon request. It should be understood that conditions different from those anticipated in this report may be encountered during grading operations, for example, the extent of unsuitable soil and the associated additional effort required to mitigate them.

Site conditions can change with time as a result of natural processes or the activities of man. Changes to the applicable laws, regulations, codes, and standards of practice may occur as a result of government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Tetra Tech has no control. Therefore, this report should be reviewed and recertified by Tetra Tech if it were to be used for a project design commencing more than one year after the date of issuance of this report.

Tetra Tech's recommendations for this site are dependent upon verification of the actual encountered field conditions, appropriate quality control of grading operations including shoring installation, overexcavation, processing and replacement of the on-site materials, and foundation construction. Accordingly, the recommendations are made contingent upon the opportunity for Tetra Tech to observe all aspects of subgrade preparation for the proposed construction. If parties other than Tetra Tech are engaged to provide such services, such parties are automatically and expressly assuming complete responsibility as the Geotechnical Engineer of Record for the project and are deemed concurring with the recommendations in this report and/or are obligated to provide alternative recommendations.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Tetra Tech should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document. Reliance by others on the data presented herein or for purposes other than those stated in the text is authorized only if so permitted in writing by Tetra Tech. It should be understood that such an authorization may incur additional expenses and charges.

Tetra Tech has endeavored to perform its evaluation using the degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical professionals with experience in this area in similar soil conditions. No other warranty, either expressed or implied, is made as to the conclusions and recommendations contained in this report.

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Figures

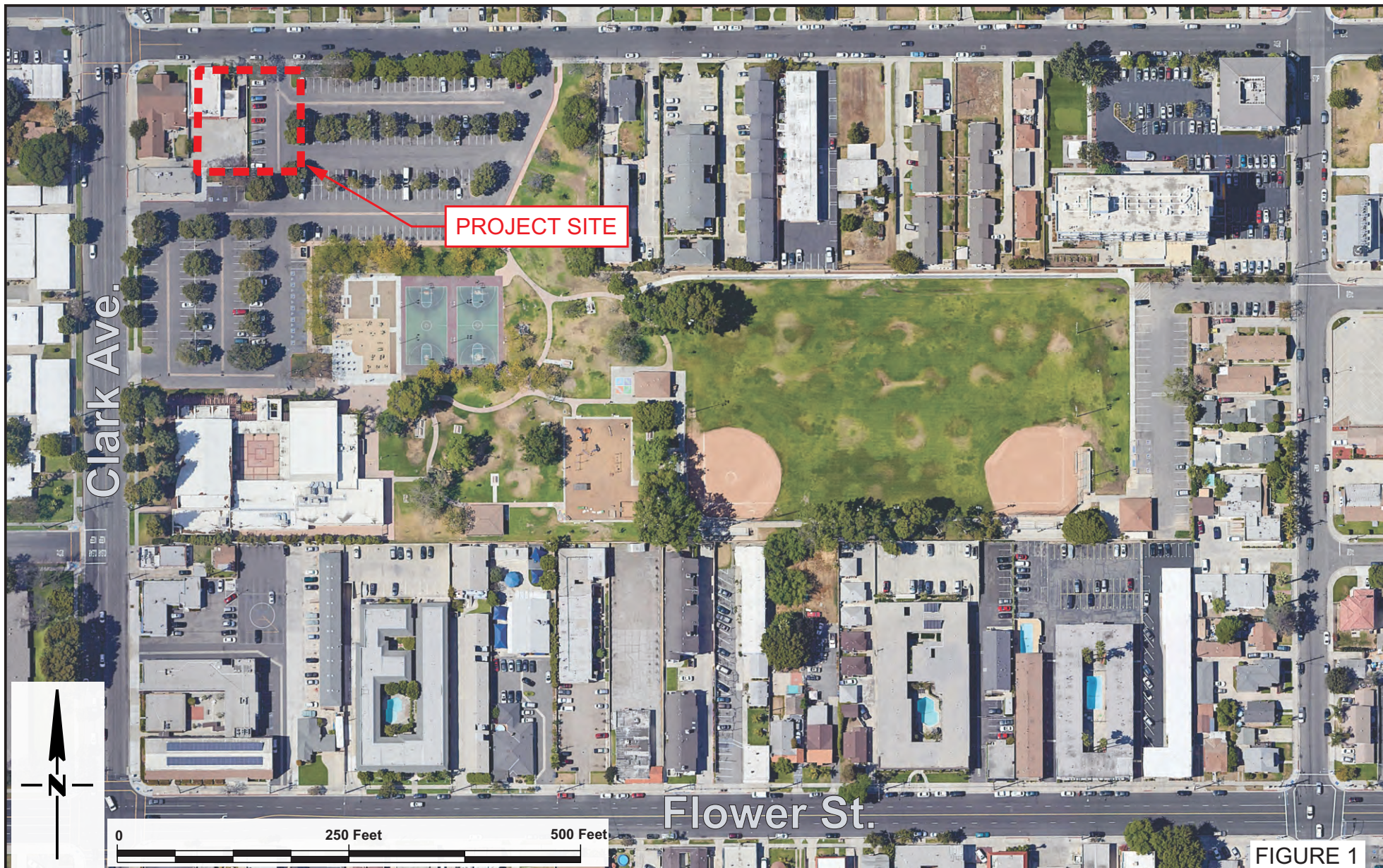


FIGURE 1



TETRA TECH

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TEL 909.860.7777 www.tetrattech.com

BELLFLOWER YOUTH AND SENIOR CENTER - BELLFLOWER, CA

PROJECT LOCATION MAP

JOB NO.
ciBLF 22-01E

DATE
NOV 2022

DRAWN BY
DL

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FIGURE 2



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BELLFLOWER YOUTH AND SENIOR CENTER - BELLFLOWER, CA

SITE LAYOUT AND BORING LOCATION MAP

JOB NO.
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BELLFLOWER YOUTH AND SENIOR CENTER - BELLFLOWER, CA

GEOLOGIC MAP

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FIGURE 4



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BELLFLOWER YOUTH AND SENIOR CENTER - BELLFLOWER, CA

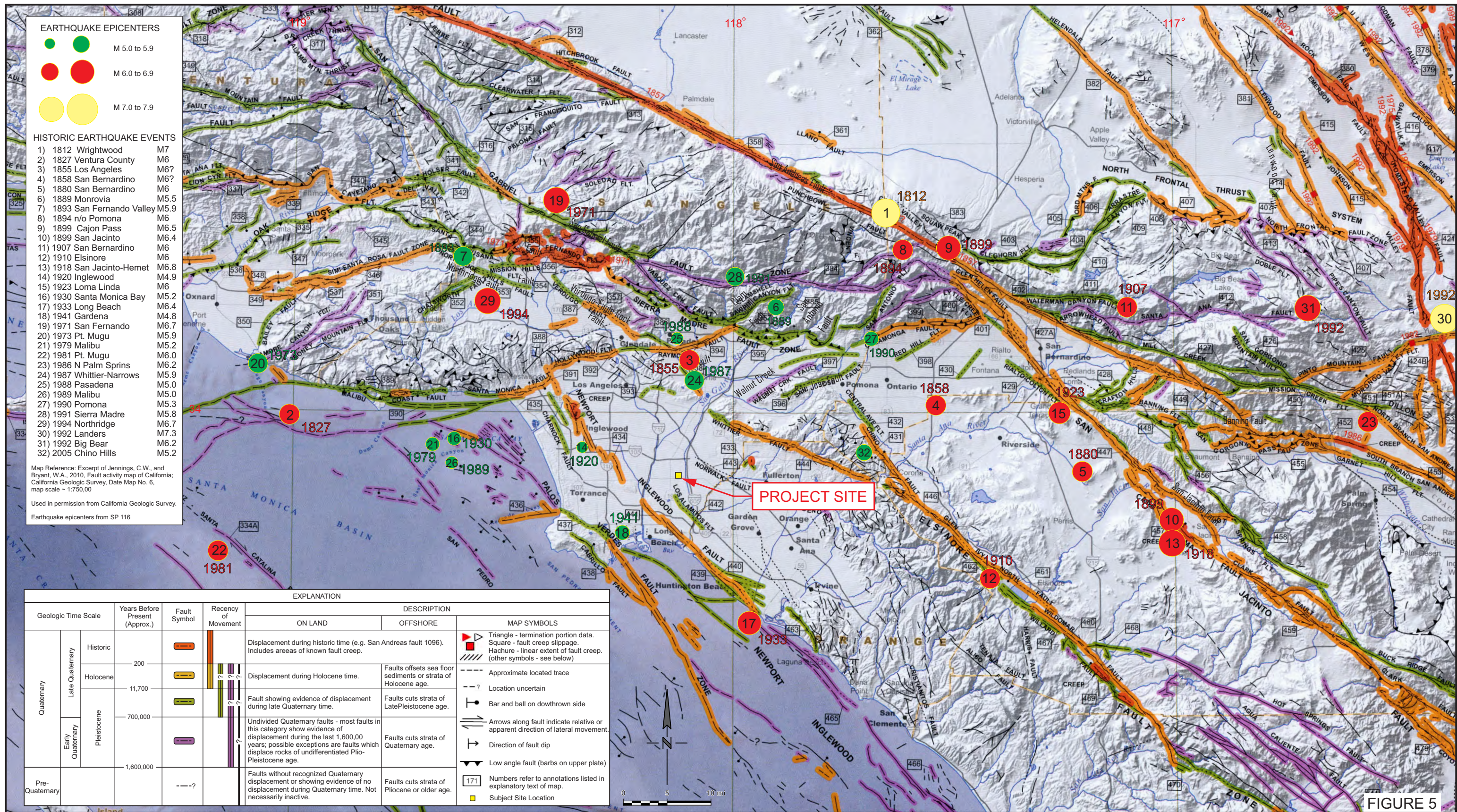
HISTORIC HIGH GROUNDWATER MAP

JOB NO.
ciBLF 22-01E

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NOV 2022

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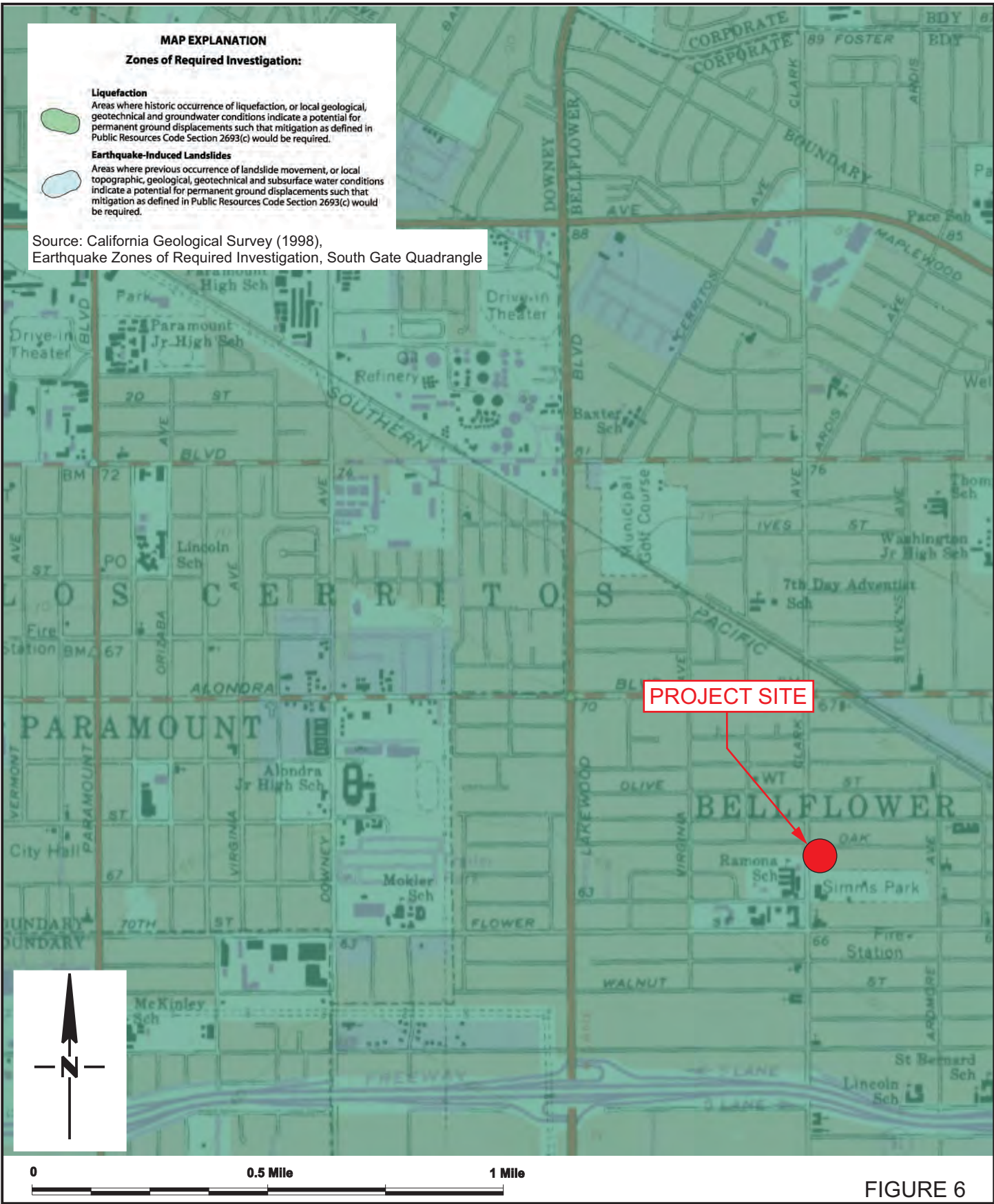


FIGURE 6

 TETRA TECH 21700 Copley Drive, Suite 200, Diamond Bar, CA 91765 TEL 909.860.7777 www.tetratech.com	BELLFLOWER YOUTH AND SENIOR CENTER - BELLFLOWER, CA		JOB NO. cIBLF 22-01E
	SEISMIC HAZARD ZONES MAP		DATE NOV 2022
			DRAWN BY DL
			CHECKED BY FC

Appendix A1

Logs of Current Exploratory Boring



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BORING B-6

Sheet 1 of 1

PROJECT NAME Bellflower Youth and Senior Center CLIENT City of Bellflower GROUND ELEVATION 67 ft
PROJECT NUMBER ciBLF22-01 PROJECT LOCATION 16614 Clark Ave GROUNDWATER DEPTH 32 ft
DATES DRILLED 10/11/2022 DRILLING CONTRACTOR 2R Drilling Company BOREHOLE DEPTH 56.5 ft
LOGGED/CHECKED BY SMS/FC DRILLING METHOD GT-16 Hollow Stem Auger BOREHOLE DIAMETER 8" in
COORDINATES 33.8843799, -118.1334905 HAMMER DATA 140 lbs Autohammer, 30" drop BACKFILL Grout
LOCATION DESCRIPTION Southeast corner of Oak and Clark.

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	☒ Standard Penetration Test (SPT) ☐ No Recovery ☒ Shelby Tube ☒ California-Type Ring Sample ☐ No Recovery ☒ Grab/Bulk Sample	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0										
	SK-1			SM		Surface is covered with 4-inches of wood chips.				
	R-2	5-7-10 (17)		SM		[FILL] Silty SAND with gravel, medium dense, dark brown (7.5YR 3/4), moist, fine sand, concrete rubble.	106.4	10.1	Corr EI=0	
	SPT-3	3-3-3 (6)				[NATIVE] Qyf				
	R-4	4-8-11 (19)		ML		Silty SAND, medium dense, yellow brown (10YR 5/6), moist, fine grained, some pinhole porosity.	99.8	20.7	Consol, DS, #200=82.4%, LL/PL/PI=NP	60
10	SPT-5	3-4-5 (9)				SILT, firm, light olive gray (5Y 6/2), moist. ...(7.5 ft) becomes sandy, stiff, dark brown (7.5 YR 3/4).				
	R-6	6-5-9 (14)		SP		Poorly graded SAND, loose, pale yellow (2.5Y 7/3) to light olive gray (5Y 6/2), mottled, moist, fine grained.	96.1	22.8	#200=86%	
	SPT-7	3-6-11 (17)		ML		SILT, stiff, olive (5Y 5/4), moist, laminated. ...(15 ft) sandy SILT, very stiff, brown (10YR 4/3), fine grained, red stringers.			LL/PL/PI=NP	50
20	R-8	3-9-11 (20)					109.8	17.1	Consol, DS	
	SPT-9	3-4-6 (10)		SM		...(25 ft) stiff, light yellow brown (10YR 6/4), finely laminated, some clay.			#200=69%	40
30	R-10	12-19-21 (40)				Silty SAND, medium dense, olive brown (2.5Y 4/3), moist, fine grained.	114.4	14.5	LL/PL/PI=NP	
	SPT-11	4-5-5 (10)		ML		...(35 ft) becomes wet.				30
40	R-12	7-13-15 (28)		SM		Silty SAND, medium dense, dark gray (N 4/1), wet, fine grained.	100.6	20.9		
	SPT-13	3-4-6 (10)							#200=49	20
50	R-14	17-27-31 (58)		SP-SM		Poorly graded SAND with silt to Silty SAND, dense, dark gray (N 4/1), wet, fine to medium grained.	101.7	22.0		
	SPT-15	4-9-10 (19)				...(55 ft) medium dense.				
Total Depth: 56.5 feet below existing grade. Groundwater encountered at 32 feet after drilling. Boring backfilled with cement-bentonite grout upon completion. Surface repaired with 2 feet of native soil and mulch replaced.										

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Appendix A2

Logs of Previous Exploratory Borings



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BORING B-3

Sheet 1 of 2

PROJECT NAME Simms Park CLIENT Craftwater GROUND ELEVATION 64 ft
PROJECT NUMBER 197-4703-0005 PROJECT LOCATION Bellflower, CA GROUNDWATER DEPTH 35.4 ft
DATES DRILLED 5/3/2022 DRILLING CONTRACTOR MR Drilling BOREHOLE DEPTH 51.5 ft
LOGGED/CHECKED BY SMS/FC DRILLING METHOD CME 75 HSA BOREHOLE DIAMETER 8 in
COORDINATES 33.883502, -118.130882 HAMMER DATA 140 lbs Autohammer, 30" drop BACKFILL grout
LOCATION DESCRIPTION mid-field

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOW/COUNT blows/6" (bpf)	GRAPHIC LOG	USCS	Standard Penetration Test (SPT) California-Type Ring Sample No Recovery No Recovery Shelby Tube Grab/Bulk Sample	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0					MATERIAL DESCRIPTION				
0				ML	Surface is covered in grass. [NATIVE] Qyf Sandy SILT, dark brown 7.5YR 3/4, moist, rootlets, fine grained.				
5	SK-1 R-2	3-5-5 (10)				100.8	21.7	R Value=58, Corr.	60
5	SPT-3	4-6-7 (13)							
10	R-4	6-8-11 (19)				92.2	22.5	Shear	55
10	SPT-5	4-7-8 (15)		SP	Poorly-graded SAND, medium dense, olive 5Y 5/4, moist, oxidized, fine grained.				
15	R-6	4-5-9 (14)							50
15	SPT-7	3-3-4 (7)							
20				ML	SILT, firm, black 5Y 2.5/1 to dark bluish gray 5B 5/1, very moist to moist, some fine sand.				45
20	R-8	3-5-9 (14)		CL	CLAY, stiff, bluish gray 5B 5/1, moist, some fine sand.	104.8	21.3	Consol, Shear LL= 28 PL= 20 PI= 8	40
25	SPT-9	4-5-7 (12)		SP-SM	SAND to Silty SAND, medium dense, pale olive 5Y 6/4, moist, fine grained, layered, micaceous and silty in some layers.				
30	R-10	14-26-32 (58)			...@30ft becomes dense	107.3	4.0		35
35									30

Continued on next page

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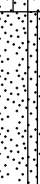


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BORING B-3

Sheet 2 of 2

PROJECT NAME Simms Park CLIENT Craftwater GROUND ELEVATION 64 ft
PROJECT NUMBER 197-4703-0005 PROJECT LOCATION Bellflower, CA GROUNDWATER DEPTH 35.4 ft

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	 Standard Penetration Test (SPT)  California-Type Ring Sample  No Recovery  No Recovery  Shelby Tube  Grab/Bulk Sample	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
35	 SPT-11	4-10-13 (23)		SM	Silty SAND, medium dense, dark gray N 4/1, wet, fine grained.					25
40										
45	 R-12	8-10-13 (23)		ML	Sandy SILT, very stiff, greenish gray 5G 5/1, wet, fine sand.	95.3	28.1	LL= 33 PL= 29 PI= 4		20
50	 SPT-13	5-5-6 (11)		SP-SM	SAND to Silty SAND, medium dense, dark gray N 4/1, wet, layered, fine grained.					15
	 R-14	11-16-27 (43)				110.4	20.9			

Total Depth: 51.5 ft
Groundwater encountered during drilling at 35.4 feet.
Backfill with cement bentonite grout, upper 5 feet soil replaced for landscaping.
Cuttings spread onsite.



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BORING B-4

Sheet 1 of 1

PROJECT NAME Simms Park **CLIENT** Craftwater **GROUND ELEVATION** 67 ft
PROJECT NUMBER 197-4703-0005 **PROJECT LOCATION** Bellflower, CA **GROUNDWATER DEPTH** Not encountered
DATES DRILLED 5/3/2022 **DRILLING CONTRACTOR** MR Drilling **BOREHOLE DEPTH** 31.5 ft
LOGGED/CHECKED BY SMS/FC **DRILLING METHOD** CME 75 HSA **BOREHOLE DIAMETER** 8 in
COORDINATES 33.884096, -118.132389 **HAMMER DATA** 140 lbs Autohammer, 30" drop **BACKFILL** grout
LOCATION DESCRIPTION east side of parking area

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOW/COUNT blows/6" (bpf)	GRAPHIC LOG	USCS	Standard Penetration Test (SPT) California-Type Ring Sample No Recovery No Recovery Shelby Tube Grab/Bulk Sample	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0					MATERIAL DESCRIPTION				
				CL	Surface is 4 inches of asphalt concrete over 6 inches of aggregate base.				65
	SK-1			ML	[FILL] Sandy CLAY, firm, olive brown 2.5Y 4/3, moist, fine grained.				
	SPT-2	2-3-3 (6)			[NATIVE] Qyf Sandy SILT, firm, olive brown 2.5Y 4/3, moist, micaceous, layered.				
5						101.4	10.2	Corr. G/S/F/2μ=0/31/69/24	
	R-3	3-4-4 (8)							60
	SPT-4	3-3-5 (8)							
10									
	R-5	7-9-13 (22)		SM	Silty SAND, medium dense, light olive grey 5Y 6/2, moist, micaceous, fine grained, interbedded silt and sand, laminated to thinly bedded.	91.3	10.6	Shear	55
	SPT-6	3-5-6 (11)							
15					...@15 ft becomes dark brown 7.5Y 3/4, fine to medium grained.	120.3	12.5		50
	R-7	6-11-21 (32)							
20									
	SPT-8	5-7-8 (15)		ML	Sandy SILT, very stiff, dark brown 7.5Y 3/4, moist, micaceous, fine to medium sand.				45
				SM	Silty SAND, medium dense, dark brown, moist, fine grained.				
25					...(@25ft) becomes fine to medium grained.	101.6	16.7	Consol	40
	R-9	9-12-15 (27)							
30					...@30ft becomes light olive gray 5Y 6/2				
	SPT-10	10-13-14 (27)		SM					
Total Depth: 31.5 ft Groundwater not encountered during drilling. Backfill with cement bentonite grout, upper 5 feet tamped soil was replaced, asphalt cold patch to repair surface. Cuttings spread onsite.									

GEOTECH BH COLUMNS - GINT STD US LAB GDT - L103 - TECHNICAL RESOURCES02 SOFTWARE LIBRARY\GINT\PROJECTS\197-4703-0005 SIMMS PARK BELLFLOWER.GPJ



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BORING B-5

Sheet 1 of 1

PROJECT NAME Simms Park CLIENT Craftwater GROUND ELEVATION 66 ft
PROJECT NUMBER 197-4703-0005 PROJECT LOCATION Bellflower, CA GROUNDWATER DEPTH Not encountered
DATES DRILLED 5/3/2022 DRILLING CONTRACTOR MR Drilling BOREHOLE DEPTH 31.5 ft
LOGGED/CHECKED BY SMS/FC DRILLING METHOD CME 75 HSA BOREHOLE DIAMETER 8 in
COORDINATES 33.884034, -118.133736 HAMMER DATA 140 lbs Autohammer, 30" drop BACKFILL grout
LOCATION DESCRIPTION west side of parking area

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOW/COUNT blows/6" (bpf)	GRAPHIC LOG	USCS	Standard Penetration Test (SPT) California-Type Ring Sample No Recovery No Recovery Shelby Tube Grab/Bulk Sample	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0										
	SK-1	7-10-14 (24)		SM		Surface is 4 inches of asphalt concrete over 6 inches of aggregate base.				65
	R-2			ML		[FILL] Silty SAND to Sandy SILT, medium dense, olive brown 2.5Y 4/3, moist, trace gravel. [NATIVE] Qyf SILT, very stiff, pale olive 5Y 6/4, slightly moist, rootlets.	99.2	11.4	Max=122.0@11.7%, R Value=56 G/S/F/2μ=0/36/64/17	
5										
	SPT-3	3-5-6 (11)		ML		SILT to Sandy SILT, stiff, light olive gray 5Y 6/2 to pale olive 5Y 6/4, slightly moist, very fine sand, rootlets.				60
	R-4	6-10-19 (29)				...@7.5ft becomes very stiff	101.7	13.5		
10										
	SPT-5	4-5-5 (10)				...@10ft becomes stiff				55
	R-6	8-10-15 (25)		SM		Silty SAND, medium dense, light olive gray 5Y 6/2, moist, very fine sand, micaceous, rootlets, some oxidation around roots.	94.6	9.2		
15										
	SPT-7	10-14-17 (31)				...@15ft becomes dense				50
20										
	R-8	3-4-5 (9)		ML		SILT, stiff, dark brown 7.5YR 3/4, very moist, some cohesion, trace gravel, some sand and clay, rootlets.	105.1	20.2		45
25										
	SPT-9	6-7-7 (14)		SM		Silty SAND, medium dense, light olive gray 5Y 6/2, moist, micaceous, fine sand.				40
30										
	R-10	10-15-22 (37)				...(@31 ft) becomes wet, no free water measured in boring.	104.5	22.1		35
Total Depth: 31.5 ft Groundwater not encountered during drilling. Backfill with cement bentonite grout, upper 5 feet tamped soil was replaced, asphalt cold patch to repair surface. Cuttings spread onsite.										

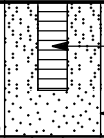


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WELL P-3

Sheet 1 of 1

PROJECT NAME Simms Park **CLIENT** Craftwater **GROUND ELEVATION** 67 ft
PROJECT NUMBER 197-4703-0005 **PROJECT LOCATION** Bellflower, CA **GROUNDWATER DEPTH** Not encountered
DATES DRILLED 5/3/2022 **DRILLING CONTRACTOR** MR Drilling **BOREHOLE DEPTH** 3.5 ft
LOGGED/CHECKED BY SMS/FC **DRILLING METHOD** CME 75 HSA **BOREHOLE DIAMETER** 8 in
COORDINATES 33.884476, -118.132289 **HAMMER DATA** Not Applicable **BACKFILL** cuttings
LOCATION DESCRIPTION northeast portion of parking area

DEPTH (ft)	SAMPLE TYPE NUMBER	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	WELL DIAGRAM
0					
		ML		Sandy SILT, pale olive 5Y 6/4, moist, fine sand.	
				Bottom of borehole at 3.5 feet.	

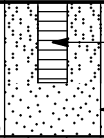


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WELL P-4

Sheet 1 of 1

PROJECT NAME Simms Park **CLIENT** Craftwater **GROUND ELEVATION** 66 ft
PROJECT NUMBER 197-4703-0005 **PROJECT LOCATION** Bellflower, CA **GROUNDWATER DEPTH** Not encountered
DATES DRILLED 5/3/2022 **DRILLING CONTRACTOR** MR Drilling **BOREHOLE DEPTH** 3.5 ft
LOGGED/CHECKED BY SMS/FC **DRILLING METHOD** CME 75 HSA **BOREHOLE DIAMETER** 8 in
COORDINATES 33.884128, -118.133090 **HAMMER DATA** Not Applicable **BACKFILL** cuttings
LOCATION DESCRIPTION central portion of parking area

DEPTH (ft)	SAMPLE TYPE NUMBER	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	WELL DIAGRAM
0					
		ML		Sandy SILT, pale olive 5Y 6/4, moist, fine sand.	
				Bottom of borehole at 3.5 feet.	



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WELL P-5

Sheet 1 of 1

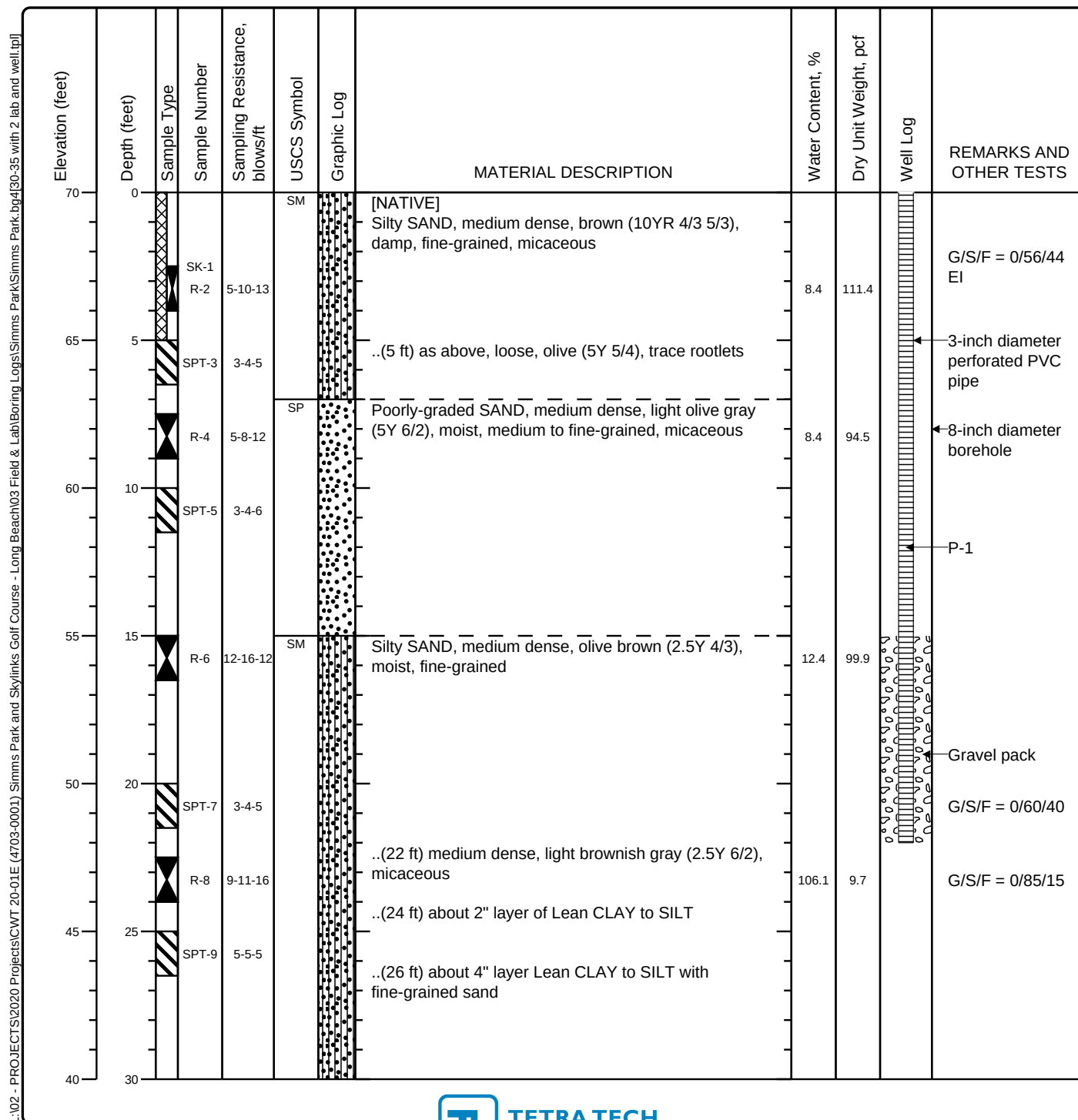
PROJECT NAME Simms Park **CLIENT** Craftwater **GROUND ELEVATION** 65 ft
PROJECT NUMBER 197-4703-0005 **PROJECT LOCATION** Bellflower, CA **GROUNDWATER DEPTH** Not encountered
DATES DRILLED 5/3/2022 **DRILLING CONTRACTOR** MR Drilling **BOREHOLE DEPTH** 3.5 ft
LOGGED/CHECKED BY SMS/FC **DRILLING METHOD** CME 75 HSA **BOREHOLE DIAMETER** 8 in
COORDINATES 33.883631, -118.133359 **HAMMER DATA** Not Applicable **BACKFILL** cuttings
LOCATION DESCRIPTION southwest portion of parking area

DEPTH (ft)	SAMPLE TYPE NUMBER	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	WELL DIAGRAM
0					
		ML		Sandy SILT, pale olive 5Y 6/4, moist, fine sand.	
				3.5	61.5
				Bottom of borehole at 3.5 feet.	2" diameter schedule 40 PVC with 0.020 inch slotted screen #1 Rock backfill

Project: **John Simms Park**
 Project Location: **Bellflower, CA**
 Project Number: **CWT 20-01E (4703-0001)**

Log of Boring B-1/P-1
Sheet 1 of 2

Date(s) Drilled 4/16/2020	Logged By SCM	Checked By FC
Drilling Method Hollow-stem Auger	Drill Bit Size/Type 8-inch diameter	Total Depth of Borehole 51.5 feet
Drill Rig Type CME75 Truck Mounted Rig	Drilling Contractor 2R Drilling	Approximate Surface Elevation 70 feet
Groundwater Level and Date Measured 36.67 feet	Sampling Method(s) Bulk, Rings, SPT	Hammer Data 140lb auto-trip hammer, 30-inch drop
Borehole Backfill Grout 4-51.5', soil cuttings 0-4', sod plug at surface	Location Latitude: 33.883505° Longitude: -118.130304°	



Project: **John Simms Park**
 Project Location: **Bellflower, CA**
 Project Number: **CWT 20-01E (4703-0001)**

Log of Boring B-1/P-1
Sheet 2 of 2

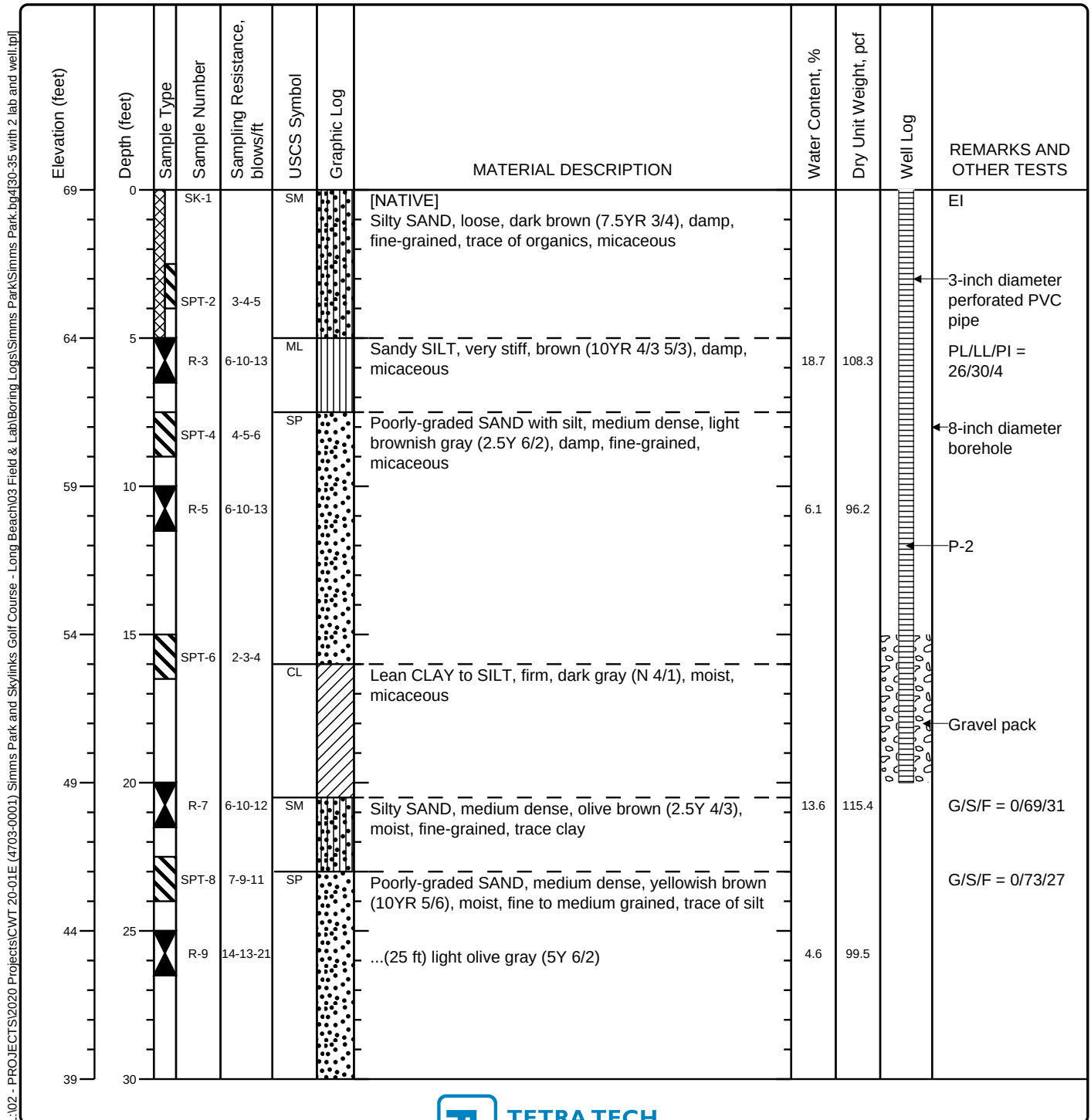
Elevation (feet)	Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/ft	USCS Symbol	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Well Log	REMARKS AND OTHER TESTS
40	30		R-10	6-16-30	SP		Poorly-graded SAND, medium dense, greenish gray (5G 5/1), 2" to 4" layer of Lean CLAY to SILT, trace of organics, medium grained	14.7	102.2		NP
35	35		SPT-11	7-9-11							
30	40		R-12	6-7-12	SM		Silty SAND, medium dense, greenish gray (5G 5/1), wet, fine-grained, micaceous	27.2	98.2		
25	45		SPT-13	6-7-9	ML		Sandy SILT, very stiff, light olive gray (5Y 6/2), wet, fine-grained sand, trace of clay, micaceous				
20	50		R-14	5-13-16	SM		Silty SAND, medium dense, greenish gray (5G 5/1), wet, fine grained	28.2	96.5		
15	55						Notes: -Total depth: 51.5 ft -Groundwater encountered at 36' 8" at 9:18 AM -Caving encountered at 22.5' after pulling out auger				
10	60										
5	65										

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Project: **John Simms Park**
 Project Location: **Bellflower, CA**
 Project Number: **CWT 20-01E (4703-0001)**

Log of Boring B-2/P-2
Sheet 1 of 2

Date(s) Drilled 4/16/2020	Logged By TC	Checked By FC
Drilling Method Hollow-stem Auger	Drill Bit Size/Type 8-inch diameter	Total Depth of Borehole 51.5 feet
Drill Rig Type CME75 Truck Mounted Rig	Drilling Contractor 2R Drilling	Approximate Surface Elevation 69 feet
Groundwater Level and Date Measured 35 feet	Sampling Method(s) Bulk, Rings, SPT	Hammer Data 140lb auto-trip hammer, 30-inch drop
Borehole Backfill Grout 4-51.5', soil cuttings 0-4', sod plug at surface	Location Latitude: 33.883503° Longitude: -118.131257°	



Project: **John Simms Park**
 Project Location: **Bellflower, CA**
 Project Number: **CWT 20-01E (4703-0001)**

Log of Boring B-2/P-2
Sheet 2 of 2

Elevation (feet)	Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/ft	USCS Symbol	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Well Log	REMARKS AND OTHER TESTS
39	30		SPT-10	10-12-15			...(25 ft) light olive gray (5Y 6/2)				
34	35		R-11	17-27-33				4.6	92.8		
29	40		SPT-12	7-10-14							
					SM		Silty SAND, medium dense, bluish gray (5B 5/1), wet, fine-grained				
24	45		R-13	6-6-8	CL		Lean CLAY to SILT, stiff, dark gray (N 4/1), wet	39.9	84.2		
					SM		Silty SAND, medium dense, bluish gray (5B 5/1), wet, fine-grained				
19	50		SPT-14	5-5-7							
14	55						Notes: -Total depth: 51.5 ft -Groundwater encountered at 35' at 1:16 PM -No caving encountered				
9	60										
4	65										

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Project: **John Simms Park**
 Project Location: **Bellflower, CA**
 Project Number: **CWT 20-01E (4703-0001)**

Key to Log of Boring Sheet 1 of 1

Elevation (feet)	Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/ft	USCS Symbol	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Well Log	REMARKS AND OTHER TESTS
1	2	3	4	5	6	7	8	9	10	11	12

COLUMN DESCRIPTIONS

- | | |
|---|--|
| <p>1 Elevation (feet): Elevation (MSL, feet).</p> <p>2 Depth (feet): Depth in feet below the ground surface.</p> <p>3 Sample Type: Type of soil sample collected at the depth interval shown.</p> <p>4 Sample Number: Sample identification number.</p> <p>5 Sampling Resistance, blows/ft: Number of blows to advance driven sampler one foot (or distance shown) beyond seating interval using the hammer identified on the boring log.</p> <p>6 USCS Symbol: USCS symbol of the subsurface material.</p> <p>7 Graphic Log: Graphic depiction of the subsurface material encountered.</p> <p>8 MATERIAL DESCRIPTION: Description of material encountered. May include consistency, moisture, color, and other descriptive text.</p> | <p>9 Water Content, %: Water content of the soil sample, expressed as percentage of dry weight of sample.</p> <p>10 Dry Unit Weight, pcf: Dry weight per unit volume of soil sample measured in laboratory, in pounds per cubic foot.</p> <p>11 Well Log: Graphical representation of well installed upon completion of drilling and sampling.</p> <p>12 REMARKS AND OTHER TESTS: Comments and observations regarding drilling or sampling made by driller or field personnel.</p> |
|---|--|

FIELD AND LABORATORY TEST ABBREVIATIONS

CHEM: Chemical tests to assess corrosivity COMP: Compaction test CONS: One-dimensional consolidation test LL: Liquid Limit, percent	PI: Plasticity Index, percent SA: Sieve analysis (percent passing No. 200 Sieve) UC: Unconfined compressive strength test, Qu, in ksf WA: Wash sieve (percent passing No. 200 Sieve)
---	--

MATERIAL GRAPHIC SYMBOLS

	Lean CLAY, CLAY w/SAND, SANDY CLAY (CL)		SILT, SILT w/SAND, SANDY SILT (ML)
	Gravel		Silty SAND (SM)
			Poorly graded SAND (SP)

TYPICAL SAMPLER GRAPHIC SYMBOLS

	Auger sampler		CME Sampler		Pitcher Sample
	Bulk Sample		Grab Sample		2-inch-OD unlined split spoon (SPT)
	3-inch-OD California w/ brass rings		2.5-inch-OD Modified California w/ brass liners		Shelby Tube (Thin-walled, fixed head)

OTHER GRAPHIC SYMBOLS

	Water level (at time of drilling, ATD)
	Water level (after waiting)
	Minor change in material properties within a stratum
	Inferred/gradational contact between strata
	Queried contact between strata

GENERAL NOTES

- Soil classifications are based on the Unified Soil Classification System. Descriptions and stratum lines are interpretive, and actual lithologic changes may be gradual. Field descriptions may have been modified to reflect results of lab tests.
- Descriptions on these logs apply only at the specific boring locations and at the time the borings were advanced. They are not warranted to be representative of subsurface conditions at other locations or times.

Appendix B1

Results of Current Laboratory Testing



MOISTURE CONTENT AND DENSITY

ASTM D7263

Job Name:	Youth and Senior Citizen Center	Date Sampled:	10/14/2022
Job Number:	ciBLF 22-01E	Date Completed:	10/22/2022
Tested By:	MG	Note:	

Boring / Test Pit / Trench		B-6	B-6	B-6	B-6	B-6				
Sample Number		R-2	R-6	R-10	R-12	R-14				
Sample Depth	ft	3	12.5	30	40	50				
USCS Soil Description		Olive Gray SM, Qal	Olive Gray ML,Qal	Olive Gray SM,Qal	Dark Bluish Gray SM, Qal	Olive Gray SM, Qal				
Number of Rings		6	6	6	6	6				
Total Weight Rings + Soil	grams	1096.50	1102.60	1196.60	1128.80	1146.70				
Volume of Rings	ft ³	0.0159	0.0159	0.0159	0.0159	0.0159				
Weight of Rings	grams	249.00	249.00	249.00	249.00	249.00				
Weight of Soil	grams	847.50	853.60	947.60	879.80	897.70				
Wet Density	pcf	117.18	118.02	131.02	121.65	124.12				
Container ID		X41	BB-3	X14	X54	X30				
Tare	grams	10.6	24.4	10.5	10.5	10.5				
Wet Soil + Tare	grams	143.5	393	302.7	305.2	312.2				
Dry Soil + Tare	grams	131.3	324.5	265.7	254.3	257.8				
Weight of Water	grams	12.2	68.5	37	50.9	54.4				

Dry Density	pcf	106.4	96.1	114.4	100.6	101.7				
Moisture Content	%	10.1	22.8	14.5	20.9	22.0				



PERCENT PASSING # 200 SIEVE

ASTM D1140

Job Name:	Youth and Senior Citizen Center	Date Completed:	10/14/2022
Job Number:	ciBLF 22-01E	Date Sampled:	October 22, 2022
Note:			
Tested By :	MG		

Boring Number	Sample Number	Depth (ft)	Weight Before Wash - Dry (grams)	Weight After Wash - Dry (grams)	Percent Passing # 200 Sieve	USCS Classification
B-6	R-6	12.5	299.9	42.5	86%	CL or ML
B-6	SPT-9	25.0	438.6	133.8	69%	ML
B-6	SPT-13	45.0	443.9	224.2	49%	SM

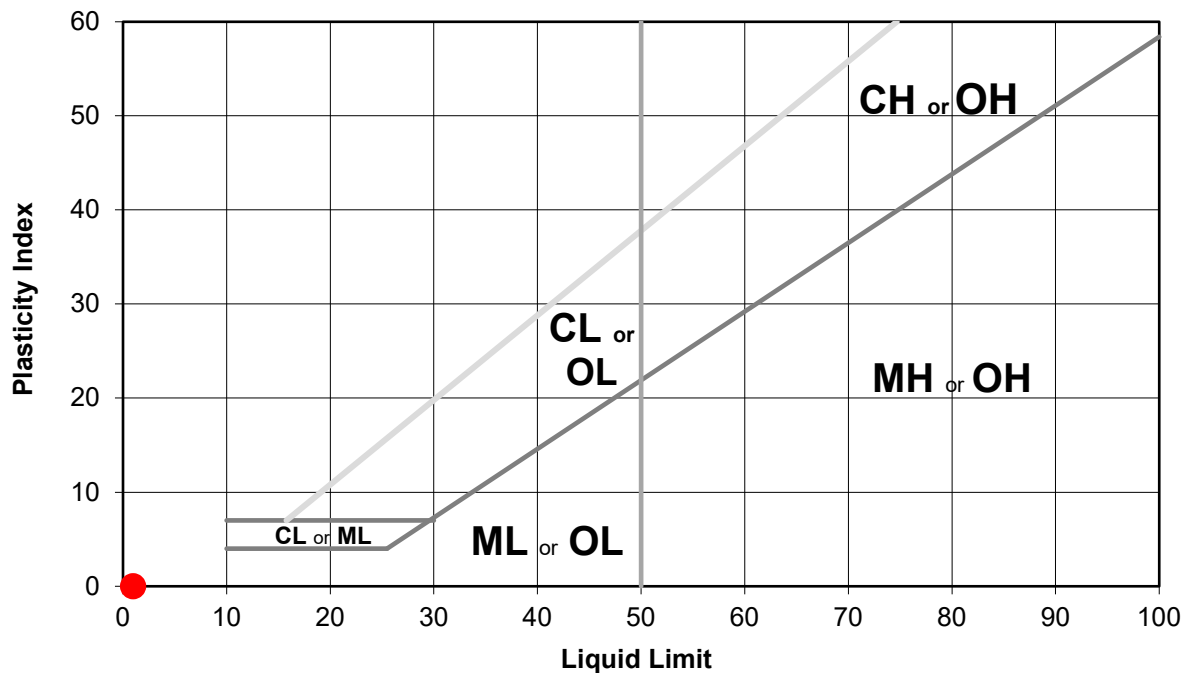
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Job Name:	Youth and Senior Citizen Center	Date Sampled:	10/4/2022
Job Number:	ciBLF 22-01E	Date Completed:	10/20/2022
Tested By:	MG	Sample Identification:	B-6, SPT-7
Note:		Sample Depth:	15ft
Sample Description:	Olive Gray Silty SAND, SM		

		Plastic Limit	
Test No.		1	2
Number of Blows			
Container ID			
Wet Weight of Soil + Cont.	grams	NP	NP
Dry Weight of Soil + Cont.	grams		
Weight of Container	grams		
Moisture Weight	grams	0.00	0.00
Weight of Dry Soil	grams	0.00	0.00
Moisture Content	%	NP	NP

Liquid Limit			
1	2	3	4
NP	NP	NP	NP
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
NP	NP	NP	NP



Plastic Limit NP
 Liquid Limit NP
 Plasticity Index NP

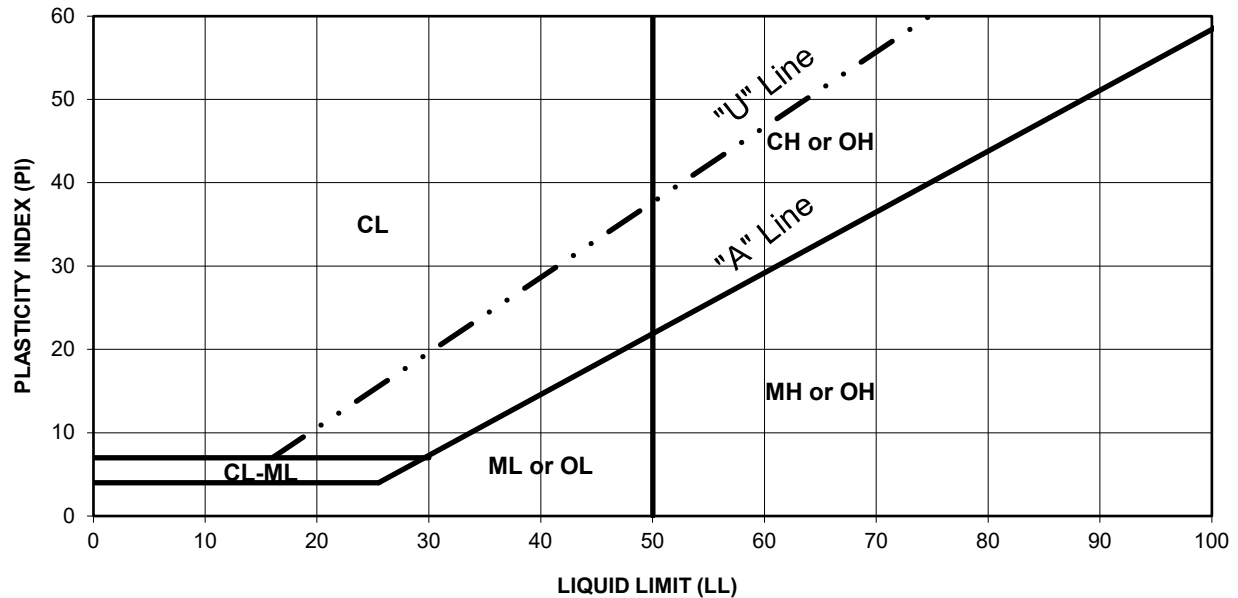
USCS Classification

Based on Atterberg Limits only



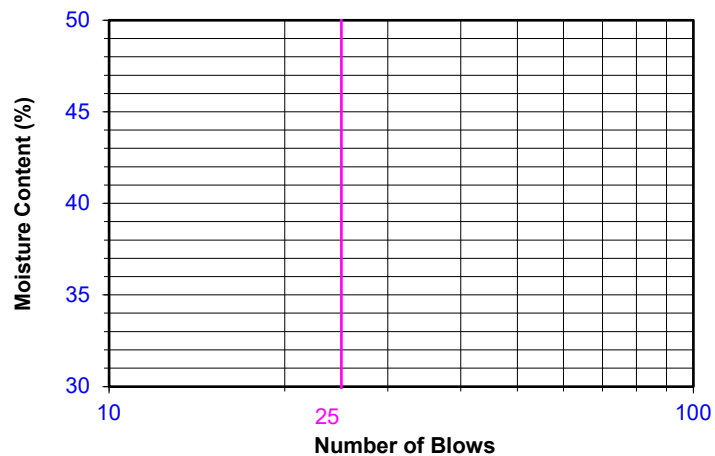
ATTERBERG LIMITS ASTM D 4318

Client Name: Tetra Tech Tested By: LS Date: 10/18/22
Project Name: City of Bellflower Youth and Senior Center Computed By: NR Date: 10/19/22
Project No.: ciBLF 22-01E Checked By: AP Date: 10/24/22



PROCEDURE USED

- ☐ Wet Preparation
☒ Dry Preparation
☒ Procedure A
Multipoint Test
☐ Procedure B
One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
	B-6	R-4	7.5	NP	NP	NP	

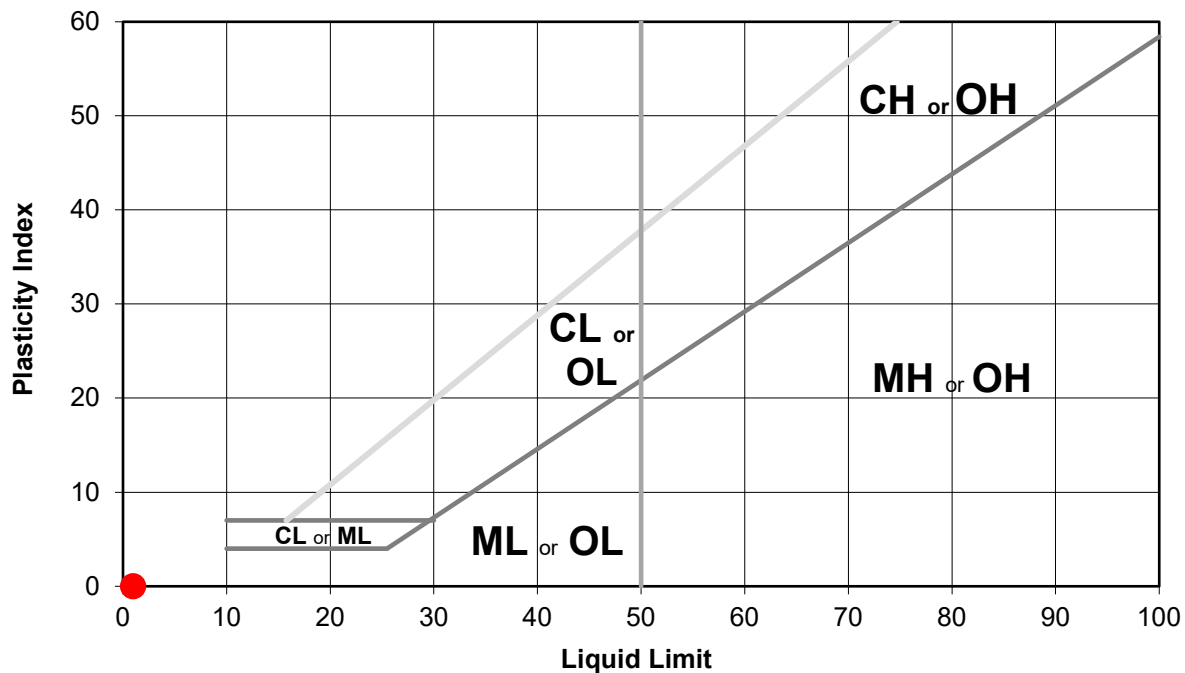
* NP denotes "non-plastic"



Job Name:	Youth and Senior Center	Date Sampled:	10/4/2022
Job Number:	ciBLF-22-01E	Date Completed:	10/20/2022
Tested By:	MG	Sample Identification:	B-6, R-10
Note:		Sample Depth:	30ft
Sample Description:	Olive Gray Silty SAND, SM		

		Plastic Limit	
Test No.		1	2
Number of Blows			
Container ID			
Wet Weight of Soil + Cont.	grams	NP	NP
Dry Weight of Soil + Cont.	grams		
Weight of Container	grams		
Moisture Weight	grams	0.00	0.00
Weight of Dry Soil	grams	0.00	0.00
Moisture Content	%	NP	NP

Liquid Limit			
1	2	3	4
NP	NP	NP	NP
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
NP	NP	NP	NP



Plastic Limit	NP
Liquid Limit	NP
Plasticity Index	NP

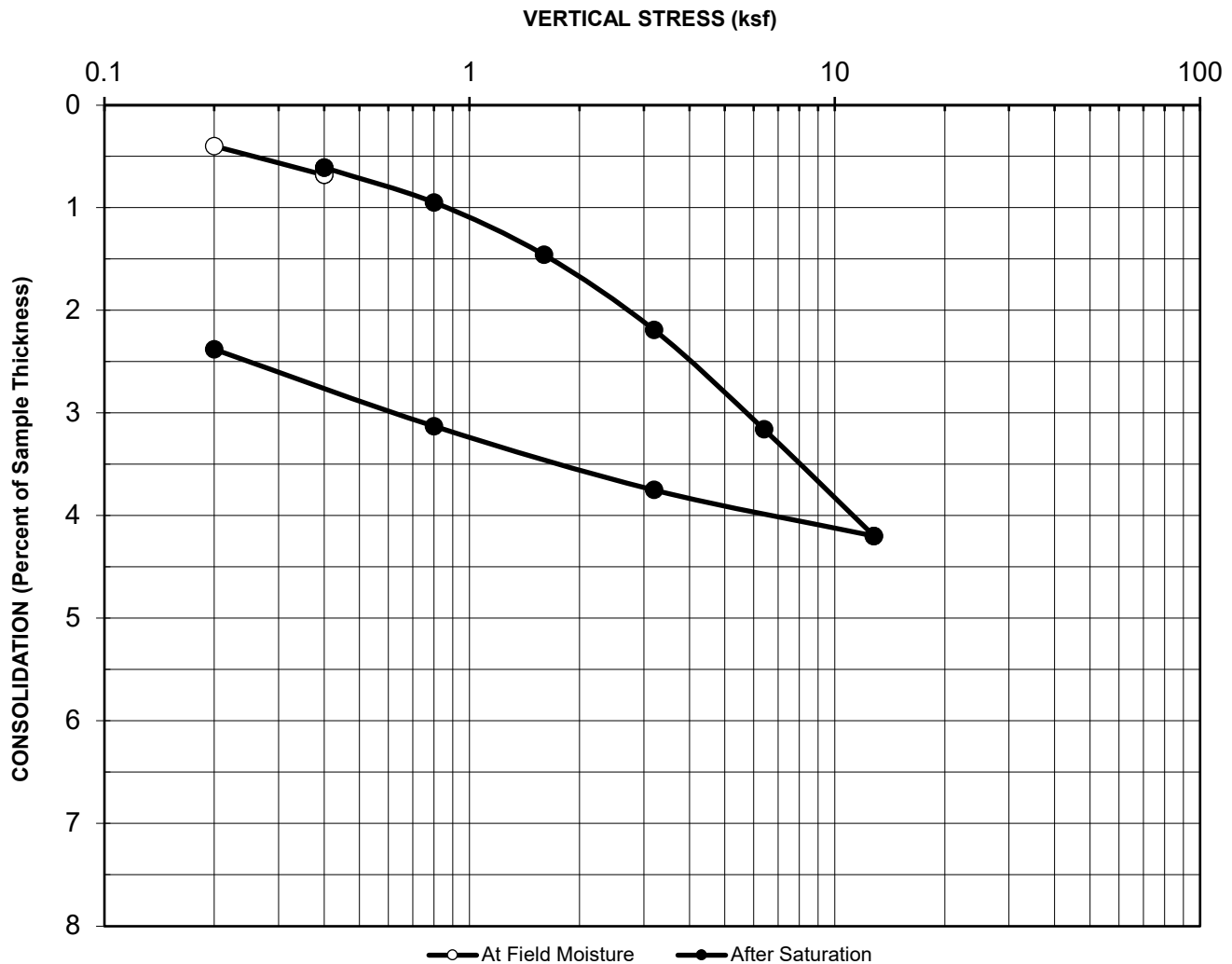
USCS Classification

Based on Atterberg Limits only

**AP Engineering and Testing, Inc.**

DBE | MBE | SBE

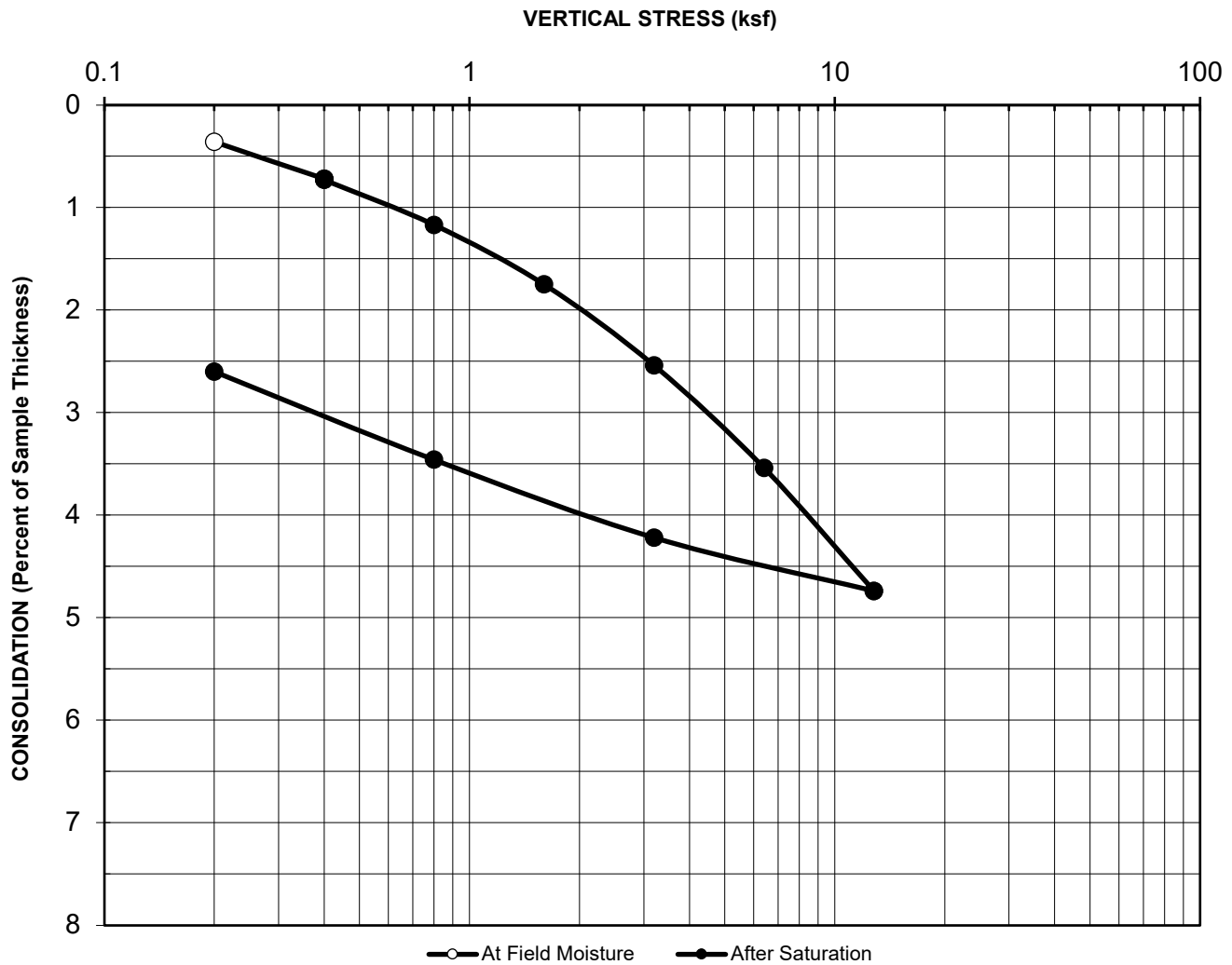
2607 Pomona Boulevard | Pomona, CA 91768

t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.comBoring No. : B-6Initial Dry Unit Weight (pcf): 109.1Sample No.: R-8Initial Moisture Content (%): 17.1Depth (feet): 20Final Moisture Content (%): 19.7Sample Type: Mod CalAssumed Specific Gravity: 2.7Soil Description: Sandy Silt Initial Void Ratio: 0.54Remarks: Swell= 0.07% upon inundation**CONSOLIDATION CURVE
ASTM D 2435**Project Name: City of Bellflower Youth and Senior CenterProject No.: ciBLF 22-01EDate: 10/18/2022AP No: 22-1032 Sheet No: 1

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t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.comBoring No. : B-6Initial Dry Unit Weight (pcf): 95.9Sample No.: R-4Initial Moisture Content (%): 20.7Depth (feet): 7.5Final Moisture Content (%): 27.5Sample Type: Mod CalAssumed Specific Gravity: 2.7Soil Description: Silt w/sandInitial Void Ratio: 0.76Remarks: Collapse= 0.01% upon inundation**CONSOLIDATION CURVE
ASTM D 2435**Project Name: City of Bellflower Youth and Senior CenterProject No.: ciBLF 22-01EDate: 10/18/2022AP No: 22-1032 Sheet No: 1

**AP Engineering and Testing, Inc.**

DBE|MBE|SBE

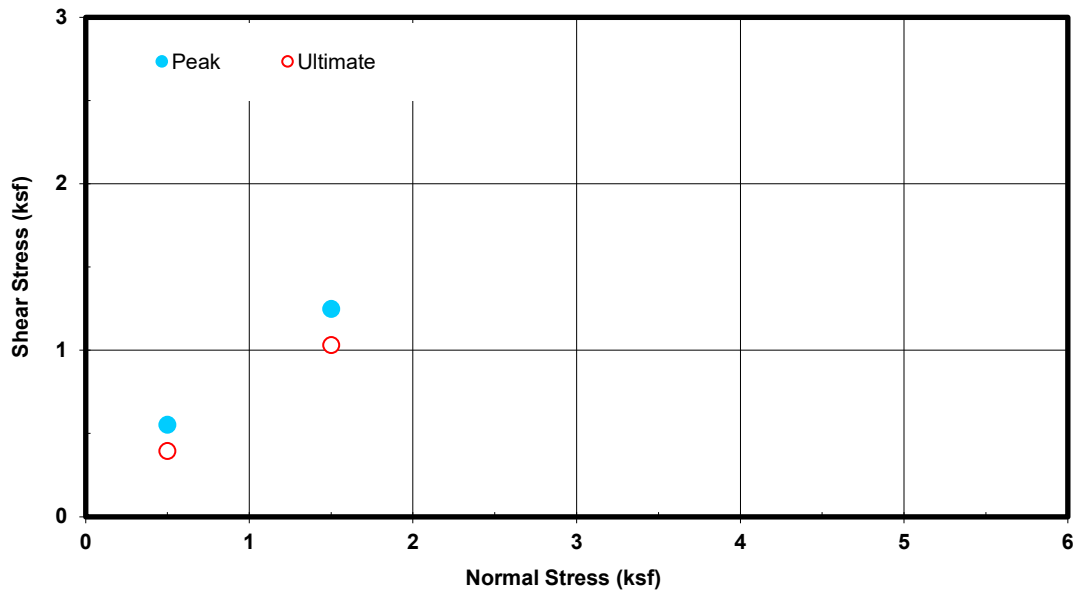
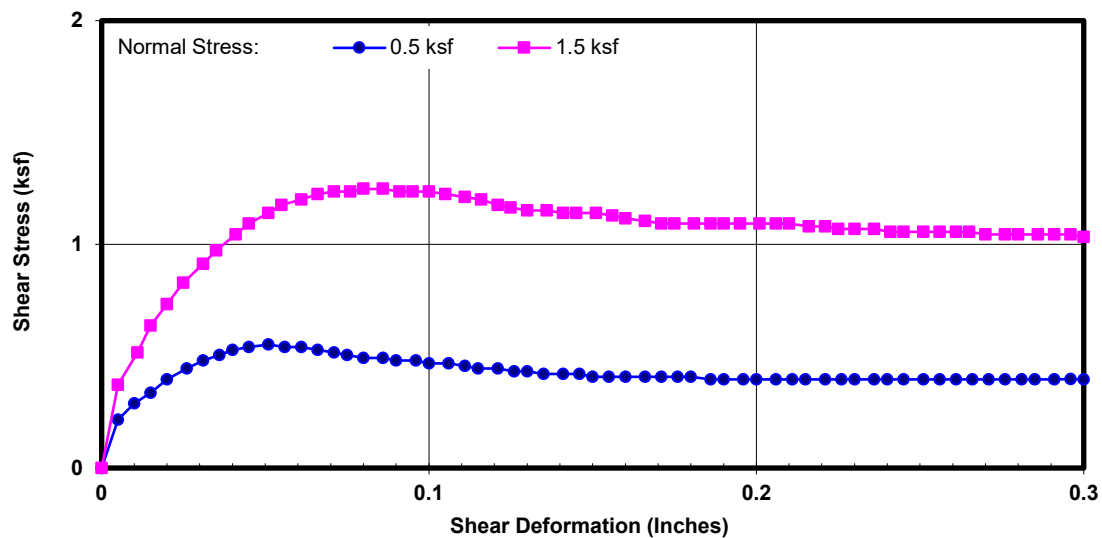
2607 Pomona Boulevard | Pomona, CA 91768

t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com**DIRECT SHEAR TEST RESULTS**
ASTM D 3080

Project Name: City of Bellflower Youth and Senior Center
Project No.: ciBLF 22-01E
Boring No.: B-6
Sample No.: R-4 **Depth (ft):** 7.5
Sample Type: Mod. Cal.
Soil Description: Silt w/sand
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 10/19/22
Computed By: NR **Date:** 10/20/22
Checked by: AP **Date:** 10/24/22

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
120.4	99.8	20.7	25.6	81	100	0.5	0.552	0.396
						1.5	1.248	1.032



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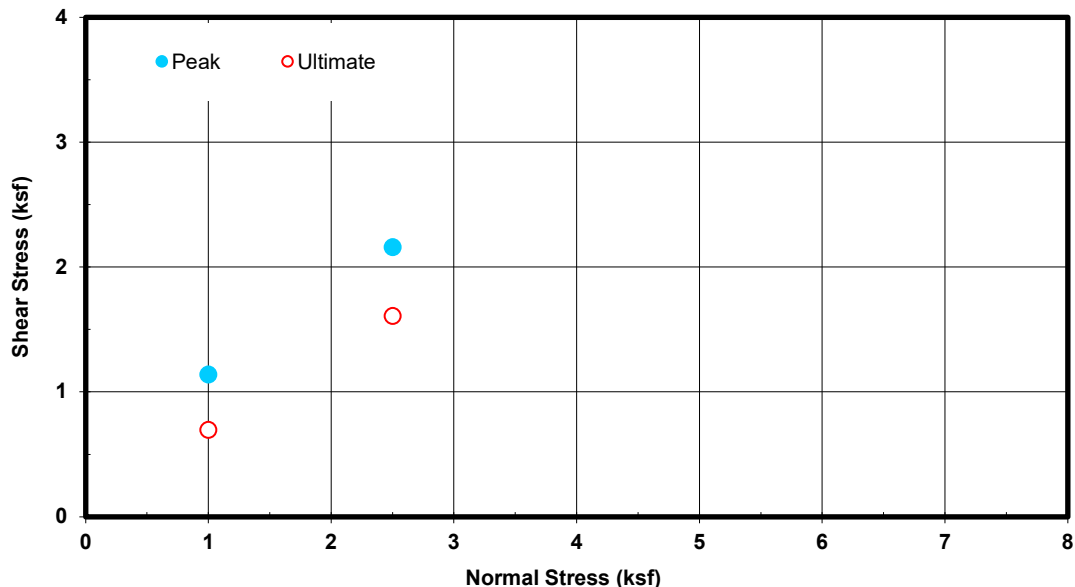
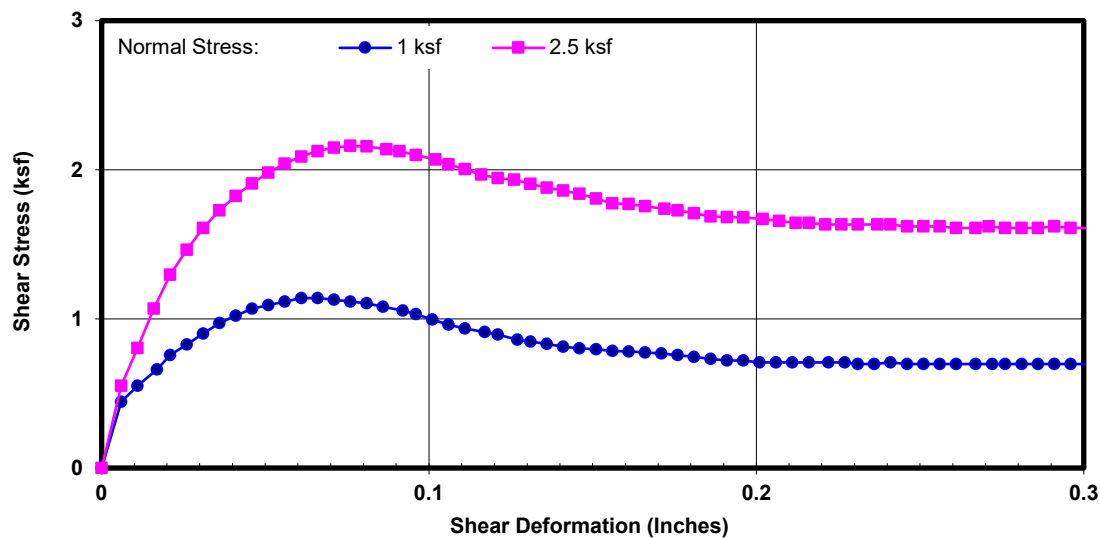
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t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com**DIRECT SHEAR TEST RESULTS**
ASTM D 3080

Project Name: City of Bellflower Youth and Senior Center
Project No.: ciBLF 22-01E
Boring No.: B-6
Sample No.: R-8 **Depth (ft):** 20
Sample Type: Mod. Cal.
Soil Description: Sandy Silt
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 10/19/22
Computed By: NR **Date:** 10/20/22
Checked by: AP **Date:** 10/24/22

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
128.7	109.8	17.1	19.0	87	96	1	1.140	0.696
						2.5	2.160	1.608



EXPANSION INDEX

ASTM D4829

Job Name:	Youth and Senior Center	Date Sampled:	10/14/2022
Job Number:	ciBLF 22-01E	Date Completed:	10/20/2022
Tested By:	MG	Sample Identification:	B-6, R-2
Note:		Sample Depth:	3ft
Sample Description:	Olive Gray Silty SAND, SM		

SAMPLE PROCESSING		
Percentage Passing #4 Sieve		
Total Air Dry Weight	grams	998.00
Weight Retained on #4	grams	0.07
*% Retained	%	0.01
*% Passing # 4 Sieve	%	100.0

SAMPLE DIMENSIONS		
Sample Height	inches	1.00
Sample Diameter	inches	4.01

MOISTURE CALCS		INITIAL	FINAL
Tare ID or #		X27	X10
Wet Weight of Soil + Tare	grams	310.6	425.4
Dry Weight of Soil + Tare	grams	284.5	355.4
Weight of Tare	grams	10.5	10.5
* Moisture Weight	grams	26.1	70.0
* Weight of Dry Soil	grams	274.0	344.9
* Moisture Content	%	9.5	20.3

AFTER REMOLDING		INITIAL	FINAL
Weight of Ring and Sample	grams	575.2	575.2
Weight of Ring	grams	196.8	196.8
*Remolded Wet Weight	grams	378.4	378.4
*Wet Density	pcf	114.1	113.7
*Dry Density	pcf	104.2	94.5
Assumed/Measured Specific Gravity		2.7	2.7
*Degree of Saturation	%	41.7	70.1

EXPANSION INDEX TEST			
DATE	TIME	DIAL	Δ H %
10/16/2022	10:40 AM	0.0346	0.0
10/16/2022	10:50 AM	0.0346	0.0
		0.0349	0.0
		0.0353	0.1
		0.0359	0.1
		0.0378	0.3
10/17/2022	10:45 AM	0.0378	0.3
*Total		0.0378	0.3

EI	Expansion Potential
0 to 20	Very Low
21 to 50	Low
51 to 90	Medium
90 to 130	High
>130	Very High

UNCORRECTED EXPANSION INDEX	3
CORRECTED EXPANSION INDEX	0
For degrees of Saturation ≠ 50%, >40% and <60%	

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t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com**CORROSION TEST RESULTS**Client Name: Tetra TechAP Job No.: 22-1032Project Name: City of Bellflower Youth and Senior CenterDate: 10/18/22Project No.: ciBLF 22-01E

Boring No.	Sample No.	Depth (feet)	Soil Description	Minimum Resistivity (ohm-cm)	pH	Sulfate Content (ppm)	Chloride Content (ppm)
B-6	SK-1	0-5	Sandy Silt	3,320	8.1	85	17

NOTES: Resistivity Test and pH: California Test Method 643
Sulfate Content : California Test Method 417
Chloride Content : California Test Method 422
ND = Not Detectable
NA = Not Sufficient Sample
NR = Not Requested

Appendix B2

Results of Previous Laboratory Testing

MOISTURE CONTENT AND DENSITY

ASTM D2937

Job Name: Simms Park Bellflower
Job Number: 197-4703-0001
Tested By: MG

Date Sampled: 4/27/2020
Date Completed: 5/2/2020
Note: Page 1 of 2

Boring / Test Pit / Trench		B-1	B-1	B-1	B-1	B-1	B-1	B-1	
Sample Number		R-1	R-4	R-6	R-8	R-10	R-12	R-14	
Sample Depth	feet	2.5ft	7.5ft	15ft	22.5ft	30ft	40ft	50ft	
USCS Soil Description		Olive Brown Silty SAND, (Qal)	Olive Poorly Graded SAND, (Qal)	Olive Silty SAND, (Qal)	Olive Silty SAND, (Qal)	Greenish Gray Poorly Graded SAND Native (Qal)	Greenish Gray Silty SAND Native (Qal)	Greenish Gray Silty SAND Native (Qal)	
Number of Rings		6	6	6	6	6	6	6	
Total Weight Rings + Soil	grams	1190.00	1006.50	1077.90	1107.70	1114.30	1169.50	1160.70	
* Volume of Rings	ft ³	0.0159	0.0159	0.0159	0.0159	0.0159	0.0159	0.0159	
* Weight of Rings	grams	265.80	265.80	265.80	265.80	265.80	265.80	265.80	
* Weight of Soil	grams	924.20	740.70	812.10	841.90	848.50	903.70	894.90	
* Wet Density	pcf	127.79	102.41	112.29	116.41	117.32	124.95	123.73	
Container ID		Z12	X4	X17	BB-1	X55	X27	X36	
Tare	grams	10.5	10.5	10.5	24.2	10.5	10.5	10.5	
Wet Soil + Tare	grams	295.1	248.4	300.7	415.6	296.1	276.5	274	
Dry Soil + Tare	grams	258.5	230	268.7	381.1	259.4	219.6	216.1	
* Weight of Water	grams	36.6	18.4	32	34.5	36.7	56.9	57.9	

* Dry Density	pcf	111.4	94.5	99.9	106.1	102.2	98.2	96.5	
* Moisture Content	%	14.8	8.4	12.4	9.7	14.7	27.2	28.2	

MOISTURE CONTENT AND DENSITY

ASTM D2937

Job Name: Simms Park Bellflower
Job Number: 197-4703-0001
Tested By: MG

Date Sampled: 4/27/2020
Date Completed: 5/2/2020
Note: Page 2 of 2

Boring / Test Pit / Trench		B-2	B-2	B-2	B-2	B-2	B-2		
Sample Number		R-3	R-5	R-7	R-9	R-11	R-13		
Sample Depth		5ft	10ft	20ft	25ft	35ft	45ft		
USCS Soil Description		Olive Brown Sandy SILT (Qal)	Greenish Gray Poorly Graded SAND (Qal)	Light Brownish Gray Silty SAND (Qal)	Olive Yellow Poorly Graded SAND (Qal)	Greenish Gray Poorly Graded SAND (Qal)	Greenish Gray SILT (Qal)		
Number of Rings		6	6	6	6	5	6		
Total Weight Rings + Soil		grams	1196.20	1004.20	1213.90	1018.30	806.70	1117.60	
* Volume of Rings		ft ³	0.0159	0.0159	0.0159	0.0159	0.0133	0.0159	
* Weight of Rings		grams	265.80	265.80	265.80	265.80	221.50	265.80	
* Weight of Soil		grams	930.40	738.40	948.10	752.50	585.20	851.80	
* Wet Density		pcf	128.64	102.10	131.09	104.05	97.10	117.78	
Container ID		Z54	X40	E5	X26	X52	X56		
Tare		grams	10.5	10.5	29	10.5	10.5	10.5	
Wet Soil + Tare		grams	291.2	265.3	487.4	238.1	274.9	288.3	
Dry Soil + Tare		grams	246.9	250.7	432.4	228.1	263.3	209	
* Weight of Water		grams	44.3	14.6	55	10	11.6	79.3	
* Dry Density		pcf	108.3	96.2	115.4	99.5	92.8	84.2	
* Moisture Content		%	18.7	6.1	13.6	4.6	4.6	39.9	

GRAIN SIZE DISTRIBUTION ANALYSIS

ASTM C136/C117/D7928

Job Name: Simms Park Bellflower

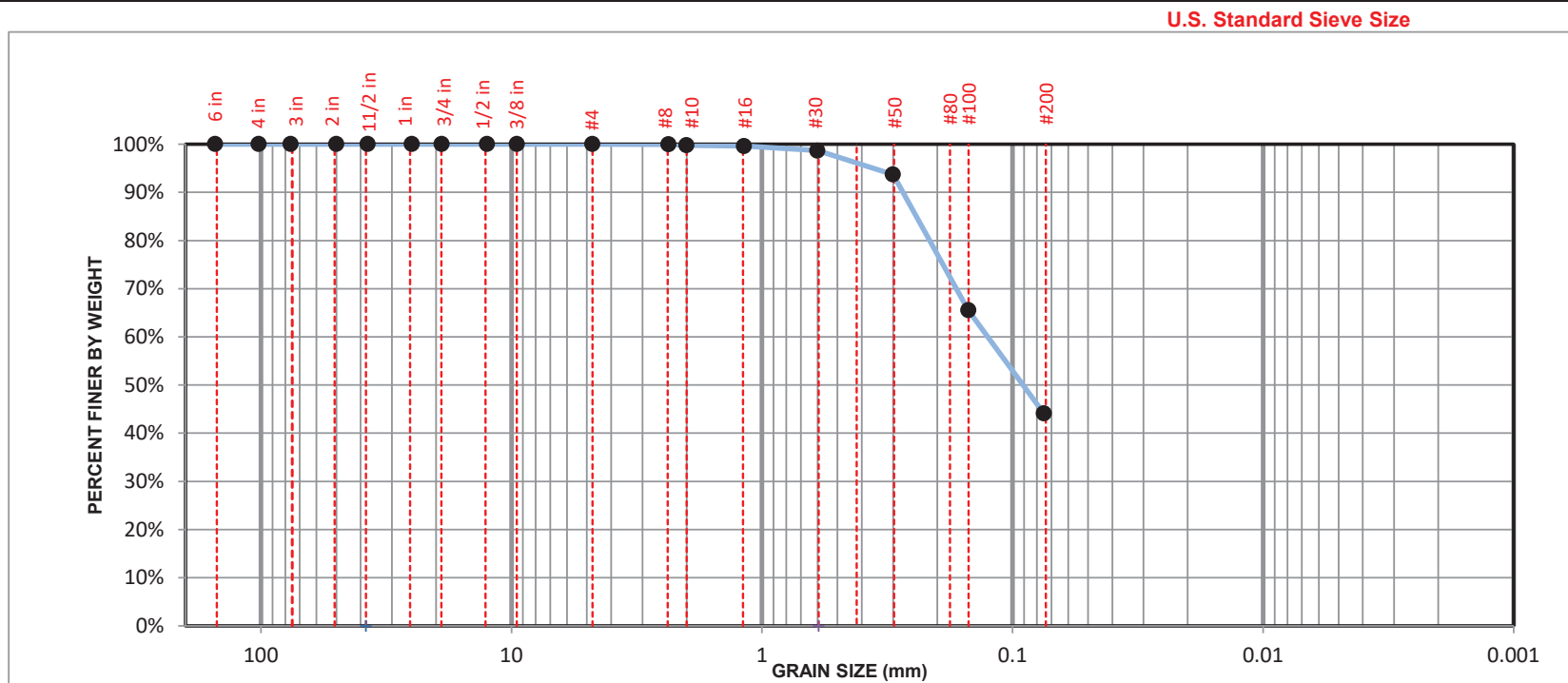
Tested By : MG

Job Number: 197-4703-0001

Date Completed: May 2, 2020

Address:

Date Sampled: April 27, 2020



Symbol	Boring No.	Sample #	Depth (feet)	LL	PI	USCS	Cobbles	Gravel	Sand	Fines	2 μ
●	B-1	SK-1	0-5ft	-	-	SM	0%	0%	56%	44%	

GRAIN SIZE DISTRIBUTION ANALYSIS

ASTM C136/C117/D7928

Job Name: Simms Park Bellflower

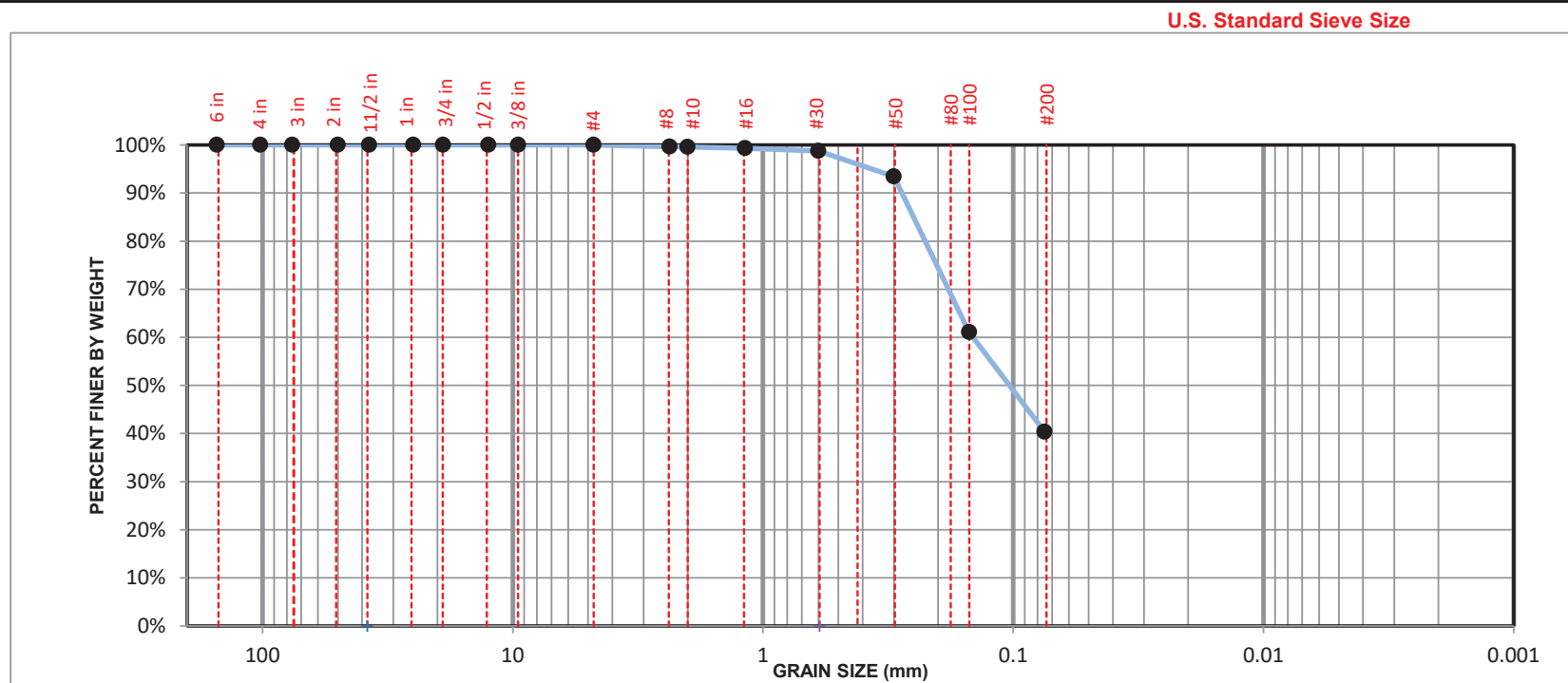
Tested By : MG

Job Number: 197-4703-0001

Date Completed: May 2, 2020

Address:

Date Sampled: April 27, 2020



Symbol	Boring No.	Sample #	Depth (feet)	LL	PI	USCS	Cobbles	Gravel	Sand	Fines	2 μ
●	B-1	SPT-7	20ft	-	-	SM	0%	0%	60%	40%	

GRAIN SIZE DISTRIBUTION ANALYSIS

ASTM C136/C117/D7928

Job Name: Simms Park Bellflower

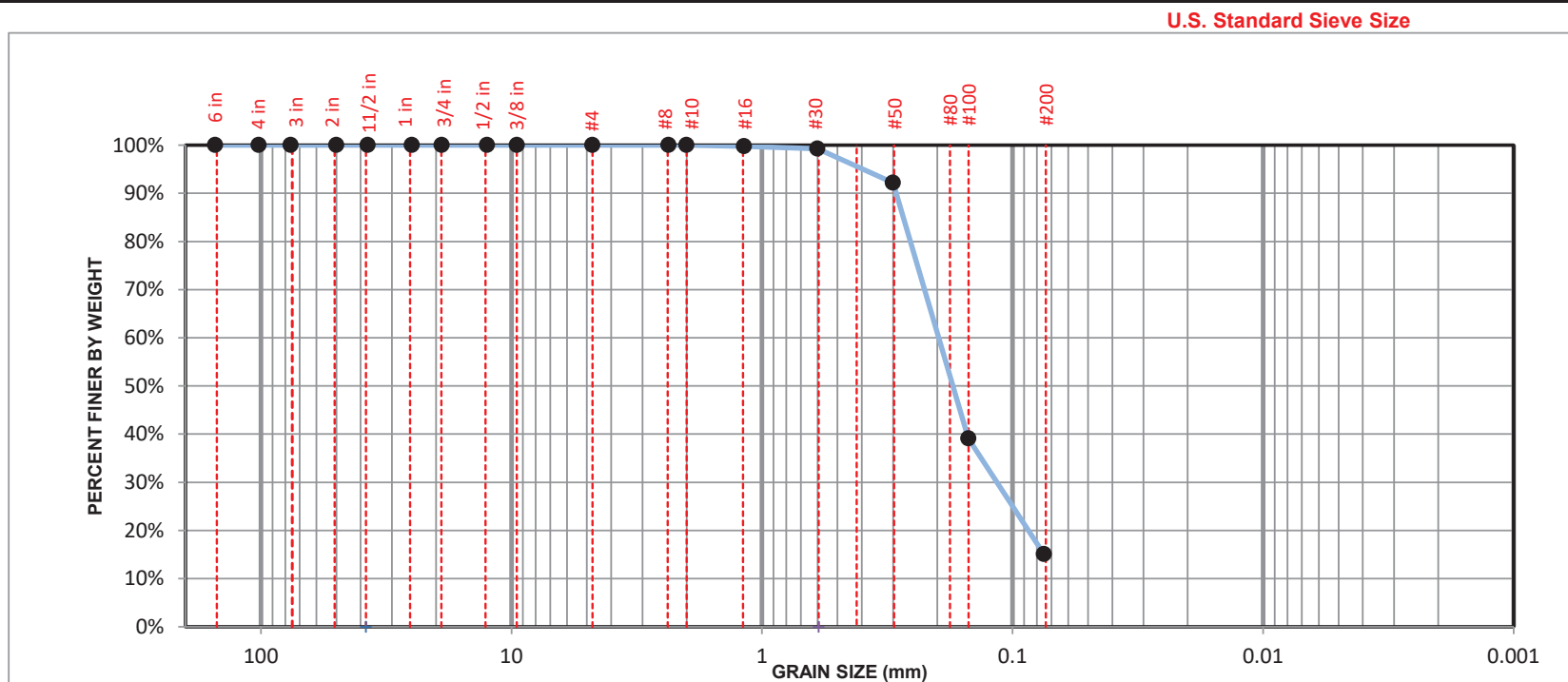
Tested By : MG

Job Number: 197-4703-0001

Date Completed: May 2, 2020

Address:

Date Sampled: April 27, 2020



Symbol	Boring No.	Sample #	Depth (feet)	LL	PI	USCS	Cobbles	Gravel	Sand	Fines	2 μ
●	B-1	R-8	22.5ft	-	-	SM	0%	0%	85%	15%	

GRAIN SIZE DISTRIBUTION ANALYSIS

ASTM C136/C117/D7928

Job Name: Simms Park Bellflower

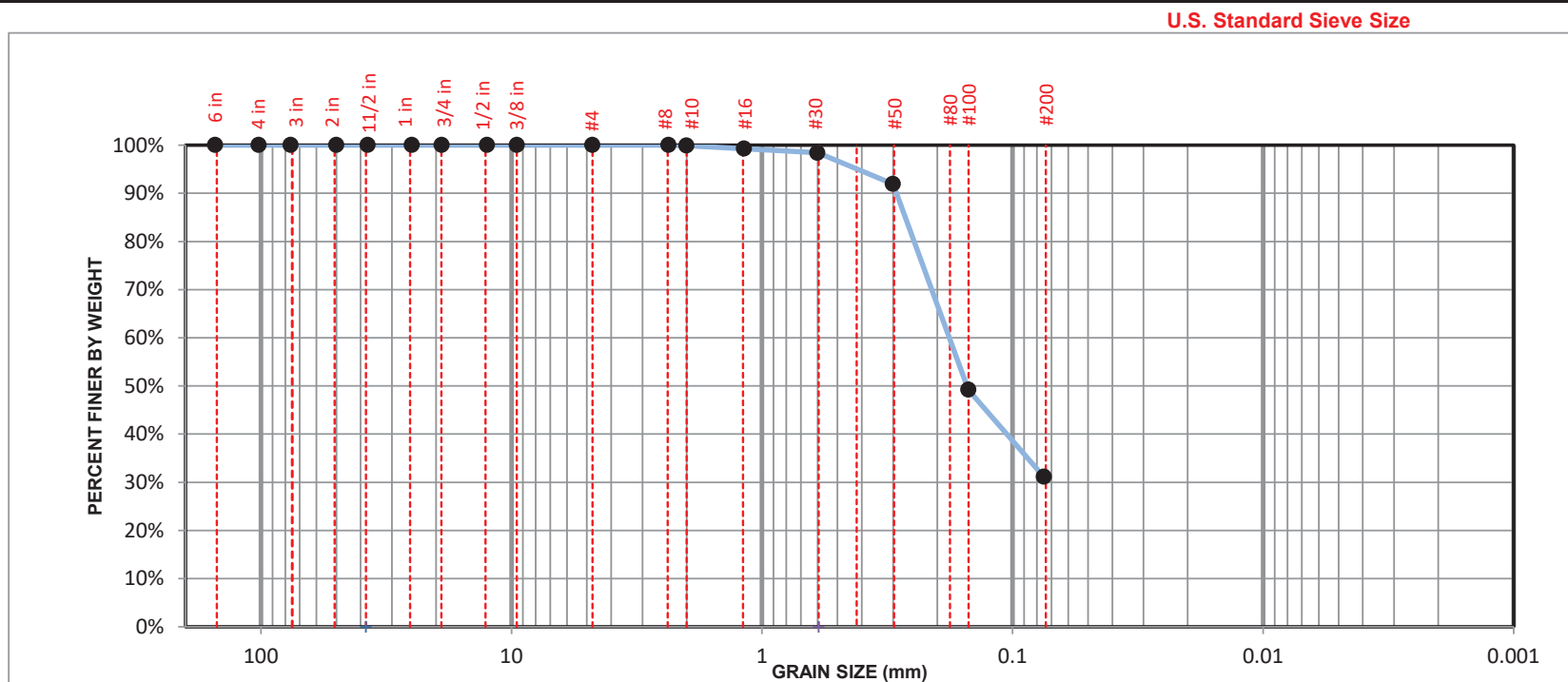
Tested By : MG

Job Number: 197-4703-0001

Date Completed: May 2, 2020

Address:

Date Sampled: April 27, 2020



Symbol	Boring No.	Sample #	Depth (feet)	LL	PI	USCS	Cobbles	Gravel	Sand	Fines	2 μ
●	B-2	R-7	20ft	-	-	SM	0%	0%	69%	31%	

GRAIN SIZE DISTRIBUTION ANALYSIS

ASTM C136/C117/D7928

Job Name: Simms Park Bellflower

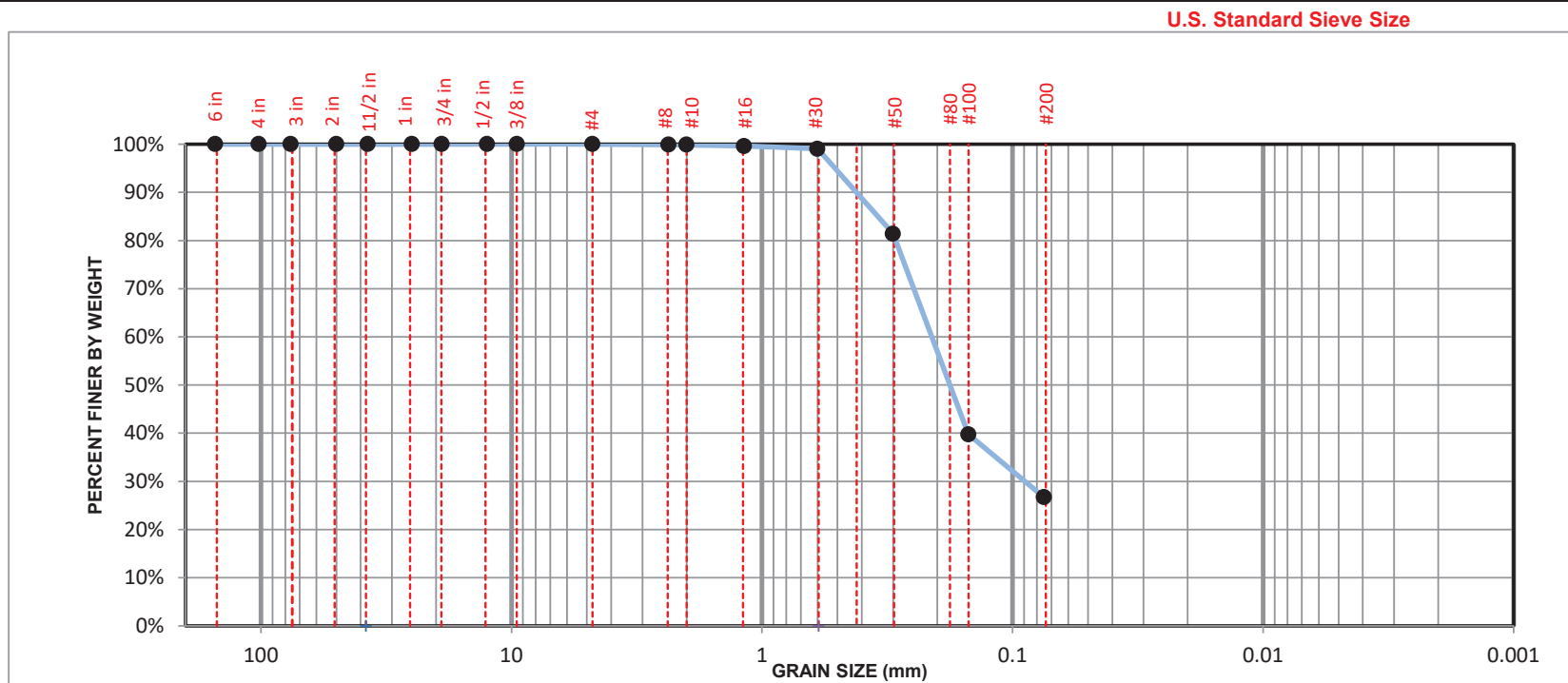
Tested By : MG

Job Number: 197-4703-0001

Date Completed: May 2, 2020

Address:

Date Sampled: April 27, 2020



Symbol	Boring No.	Sample #	Depth (feet)	LL	PI	USCS	Cobbles	Gravel	Sand	Fines	2 μ
●	B-2	SPT-8	22.5ft	-	-	SM	0%	0%	73%	27%	

ATTERBERG LIMITS

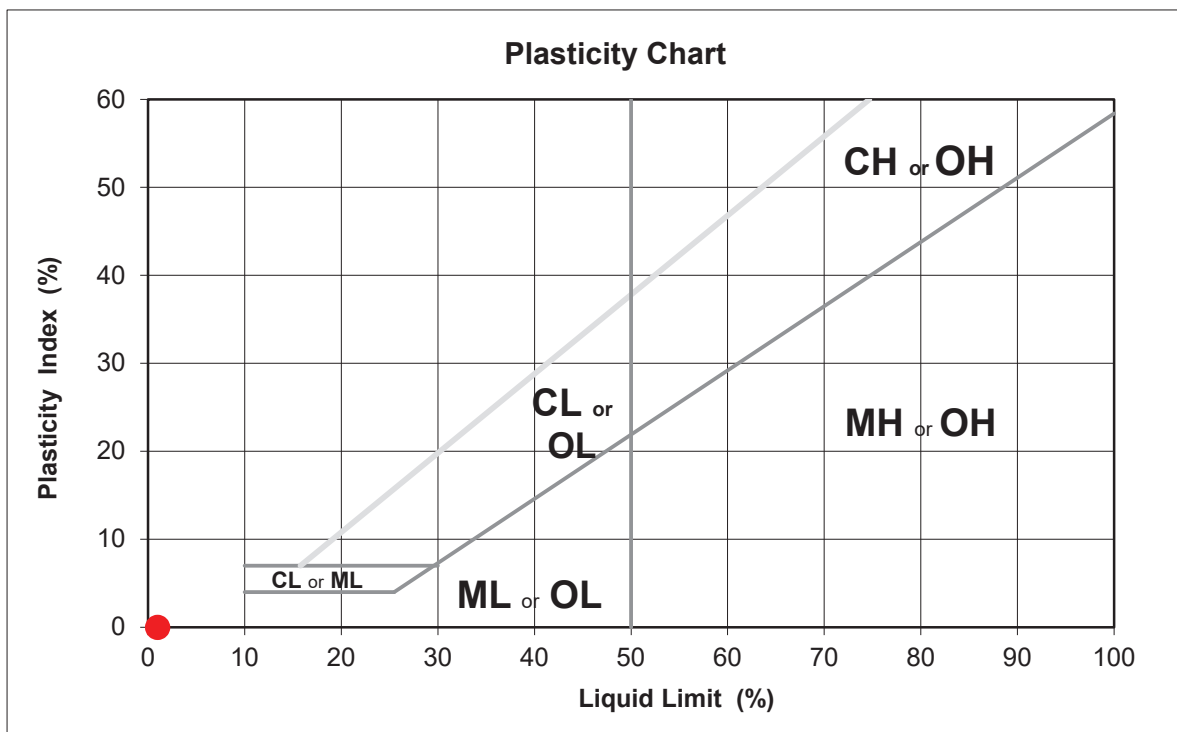
ASTM D4318

Job Name: Simms Park Bellflower
Job Number: 197-4703-0001
Tested By: MG
Note:
Sample Description: Greenish Gray Silty SAND, Qal

Date Sampled: 4/27/2020
Date Completed: 5/3/2020
Sample Identification: B-1 R-12
Sample Depth: 40ft

		PLASTIC LIMIT	
Test No.		1	2
Number of Blows			
Container ID			
Wet Weight of Soil + Cont.	grams	NP	NP
Dry Weight of Soil + Cont.	grams		
Weight of Container	grams		
* Moisture Weight	grams	0.00	0.00
* Weight of Dry Soil	grams	0.00	0.00
* Moisture Content	%	NP	NP

LIQUID LIMIT			
1	2	3	4
NP	NP	NP	
0.00	0.00	0.00	
0.00	0.00	0.00	
NP	NP	NP	



Plastic Limit NP
Liquid Limit NP
Plasticity Index NP

USCS Classification SM

ATTERBERG LIMITS

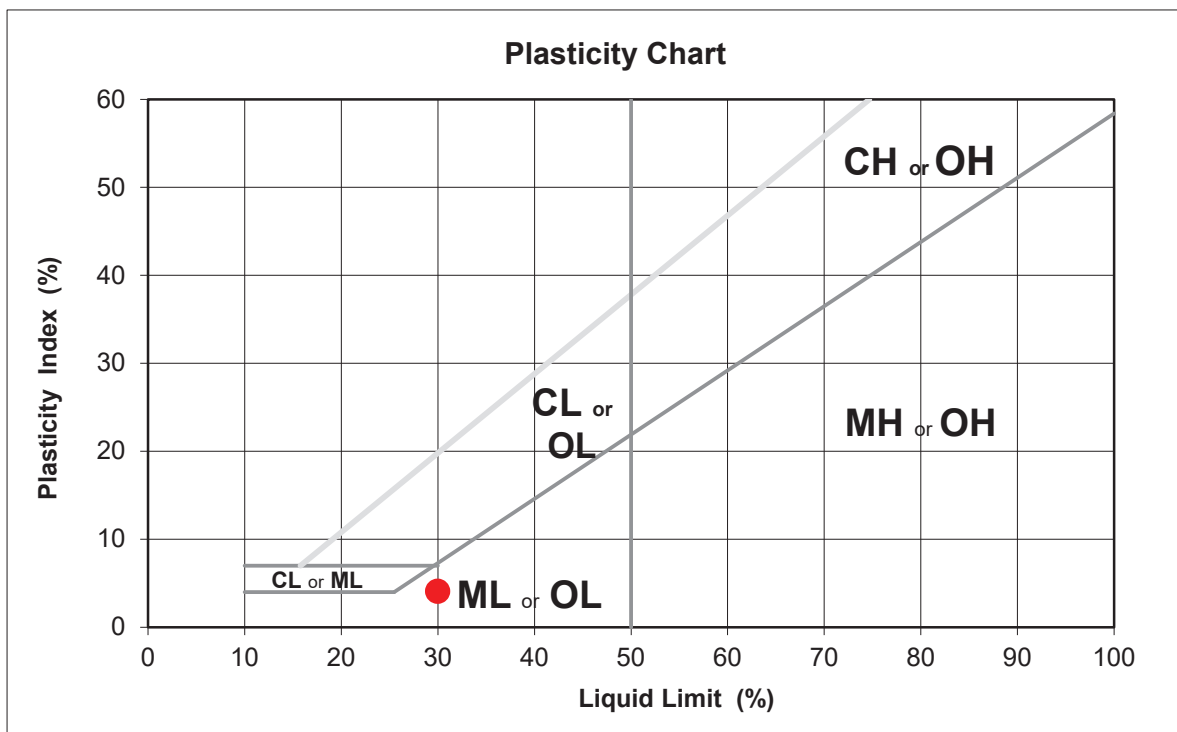
ASTM D4318

Job Name: Simms Park Bellflower
Job Number: 197-4703-0001
Tested By: MG
Note:
Sample Description: Olive Sandy SILT

Date Sampled: 4/27/2020
Date Completed: 5/3/2020
Sample Identification: B-2, R-3
Sample Depth: 5ft

		PLASTIC LIMIT	
Test No.		1	2
Number of Blows			
Container ID		P41	P9
Wet Weight of Soil + Cont.	grams	21.60	24.60
Dry Weight of Soil + Cont.	grams	19.70	22.10
Weight of Container	grams	12.40	12.40
* Moisture Weight	grams	1.90	2.50
* Weight of Dry Soil	grams	7.30	9.70
* Moisture Content	%	26.0	25.8

LIQUID LIMIT			
1	2	3	4
32	25	15	
P39	T23	T18	
61.20	58.10	44.50	
53.20	50.70	39.60	
25.30	26.00	25.60	
8.00	7.40	4.90	
27.90	24.70	14.00	
28.7	30.0	35.0	



Plastic Limit 26
Liquid Limit 30
Plasticity Index 4

USCS Classification ML

EXPANSION INDEX

ASTM D4829

Job Name: Simms Park Bellflower
Job Number: 197-4703-0001
Tested By: MG
Note:
Sample Description: Olive Brown Sandy Silt

Date Sampled: 4/27/2020
Date Completed: 5/3/2020
Sample Identification: B-1, SK-1
Sample Depth: 0-5ft

SAMPLE PROCESSING		
Percentage Passing #4 Sieve		
Total Air Dry Weight	2500.00	grams
Weight Retained on #4	1.00	grams
*% Retained	0.04	%
*% Passing # 4 Sieve	100.0	%

SAMPLE DIMENSIONS		
Sample Height	1.00	inches
Sample Diameter	4.01	inches

MOISTURE CALCS	INITIAL	FINAL
Tare ID or #	Z15	5A
Wet Weight of Soil + Tare	grams 173.50	617.30
Dry Weight of Soil + Tare	grams 156.30	546.00
Weight of Tare	grams 4.00	191.10
* Moisture Weight	grams 17.20	71.30
* Weight of Dry Soil	grams 152.30	354.90
* Moisture Content	% 11.3	20.1

AFTER REMOLDING	INITIAL	FINAL
Weight of Ring and Sample	grams 609.2	609.2
Weight of Ring	grams 202.2	202.2
*Remolded Wet Weight	grams 407.0	407.0
*Wet Density	pcf 122.7	122.1
*Dry Density	pcf 110.3	101.7
Assumed/Measured Specific Gravity	2.7	2.7
*Degree of Saturation	% 57.7	82.6

EXPANSION INDEX TEST			
DATE	TIME	DIAL	Δ H %
5/1/2020	11:50 AM	0.0345	0.0
		0.0345	0.0
		0.0348	0.0
		0.0386	0.4
		0.0387	0.4
5/2/2020	1:00 PM	0.0391	0.5
5/2/2020	1:30 PM	0.0391	0.5
*Total			0.5

UNCORRECTED EXPANSION INDEX 5
CORRECTED EXPANSION INDEX 8
 For degrees of Saturation ≠ 50%, >40% and <60%

very Low

EXPANSION INDEX

ASTM D4829

Job Name: Simms Park Bellflower
Job Number: 197-4703-0001
Tested By: MG
Note:
Sample Description: Olive Brown Sandy Silt

Date Sampled: 4/27/2020
Date Completed: 5/3/2020
Sample Identification: B-2, SK-1
Sample Depth: 0-5ft

SAMPLE PROCESSING		
Percentage Passing #4 Sieve		
Total Air Dry Weight	2500.00	grams
Weight Retained on #4	1.00	grams
*% Retained	0.04	%
*% Passing # 4 Sieve	100.0	%

SAMPLE DIMENSIONS		
Sample Height	1.00	inches
Sample Diameter	4.01	inches

MOISTURE CALCS	INITIAL	FINAL
Tare ID or #	Z17	P-1
Wet Weight of Soil + Tare	grams 200.80	564.90
Dry Weight of Soil + Tare	grams 179.60	488.70
Weight of Tare	grams 4.00	149.80
* Moisture Weight	grams 21.20	76.20
* Weight of Dry Soil	grams 175.60	338.90
* Moisture Content	% 12.1	22.5

AFTER REMOLDING	INITIAL	FINAL
Weight of Ring and Sample	grams 592.9	592.9
Weight of Ring	grams 198.7	198.7
*Remolded Wet Weight	grams 394.2	394.2
*Wet Density	pcf 118.8	118.1
*Dry Density	pcf 106.0	96.4
Assumed/Measured Specific Gravity	2.7	2.7
*Degree of Saturation	% 55.4	81.2

EXPANSION INDEX TEST			
DATE	TIME	DIAL	Δ H %
5/1/2020	1:00 PM	0.0434	0.0
	1:10 PM	0.0434	0.0
		0.0448	0.1
		0.0450	0.2
		0.0487	0.5
5/2/2020	1:00 PM	0.0505	0.7
5/2/2020	1:30 PM	0.0500	0.7
*Total			0.7

UNCORRECTED EXPANSION INDEX

7

CORRECTED EXPANSION INDEX
9

For degrees of Saturation ≠ 50%, >40% and <60%

very Low



MOISTURE CONTENT AND DENSITY

ASTM D7263

Job Name:	Simms Park Belflower	Date Sampled:	5/9/2022
Job Number:	CWT 22-05	Date Completed:	5/16/2022
Tested By:	MG	Note:	Page 1 of 2

Boring / Test Pit / Trench		B-3	B-3	B-3	B-3	B-4	B-4	B-5	B-5	B-5
Sample Number		R-2	R-10	R-12	R-14	R-3	R-7	R-2	R-4	R-6
Sample Depth	ft	2.5	30	40	50	5	15	2.5	7.5	12.5
USCS Soil Description		Olive Gray Qal, SM	Light Olive Gray Qal, SM	Olive Gray Qal, SM	Olive Gray Qal, SM	Olive Brown Qal, SM	Olive Gray Qal, SM	Olive Gray Qal, SM	Olive Gray Qal, SM	Olive Gray Qal, SP
Number of Rings		6	6	6	6	6	6	6	6	6
Total Weight Rings + Soil	grams	1144.40	1063.20	1139.00	1221.60	1064.60	1235.70	1055.60	1091.80	1003.80
Volume of Rings	ft ³	0.0159	0.0159	0.0159	0.0159	0.0159	0.0159	0.0159	0.0159	0.0159
Weight of Rings	grams	256.46	256.46	256.46	256.46	256.46	256.46	256.46	256.46	256.46
Weight of Soil	grams	887.94	806.74	882.54	965.14	808.14	979.24	799.14	835.34	747.34
Wet Density	pcf	122.77	111.55	122.03	133.45	111.74	135.40	110.49	115.50	103.33
Container ID		X46	X31	X53	X38	X12	X21	E5	X24	X33
Tare	grams	10.5	10.5	10.5	10.5	10.5	10.5	29.2	10.5	10.5
Wet Soil + Tare	grams	327.9	243.1	294.9	355.2	260.7	342.4	529.2	282.2	234.9
Dry Soil + Tare	grams	271.2	234.2	232.5	295.7	237.5	305.4	478.2	249.8	216
Weight of Water	grams	56.7	8.9	62.4	59.5	23.2	37	51	32.4	18.9

Dry Density	pcf	100.8	107.3	95.3	110.4	101.4	120.3	99.2	101.7	94.6
Moisture Content	%	21.7	4.0	28.1	20.9	10.2	12.5	11.4	13.5	9.2



MOISTURE CONTENT AND DENSITY

ASTM D7263

Job Name:	Simms Park Belflower	Date Sampled:	5/9/2022
Job Number:	CWT 22-05	Date Completed:	5/16/2022
Tested By:	MG	Note:	Page 2 of 2

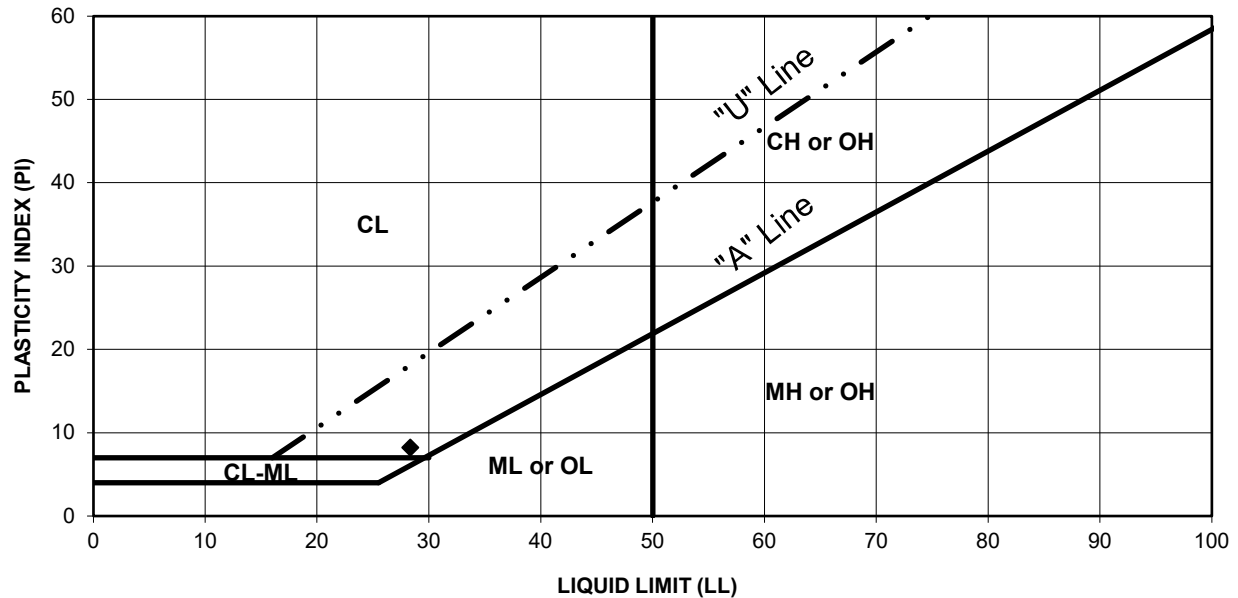
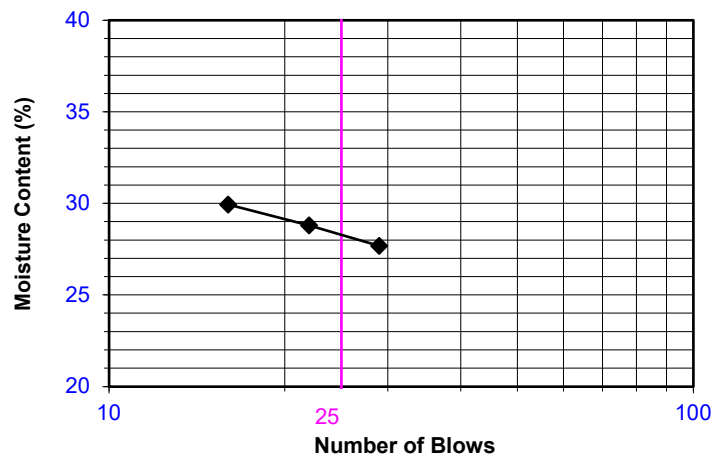
Boring / Test Pit / Trench		B-5	B-5							
Sample Number		R-8	R-10							
Sample Depth	ft	20	30							
USCS Soil Description		Olive Gray Qal, ML	Light Olive Gray Qal, SP							
Number of Rings		6	6							
Total Weight Rings + Soil	grams	1170.50	1178.90							
Volume of Rings	ft ³	0.0159	0.0159							
Weight of Rings	grams	256.46	256.46							
Weight of Soil	grams	914.04	922.44							
Wet Density	pcf	126.38	127.54							
Container ID		X30	X26							
Tare	grams	10.5	10.5							
Wet Soil + Tare	grams	309.2	298.2							
Dry Soil + Tare	grams	259	246.2							
Weight of Water	grams	50.2	52							

Dry Density	pcf	105.1	104.5							
Moisture Content	%	20.2	22.1							

**AP Engineering and Testing, Inc.**

DBE|MBE|SBE

2607 Pomona Boulevard | Pomona, CA 91768

t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com**ATTERBERG LIMITS
ASTM D 4318****Client Name:** Tetra Tech**Tested By:** LS**Date:** 05/20/22**Project Name:** Simms Park, Bellflower**Computed By:** NR**Date:** 05/23/22**Project No.:** CWT22-05**Checked By:** AP**Date:** 05/24/22**PROCEDURE USED**☐ Wet Preparation☒ Dry Preparation☒ Procedure A
Multipoint Test☐ Procedure B
One-point Test

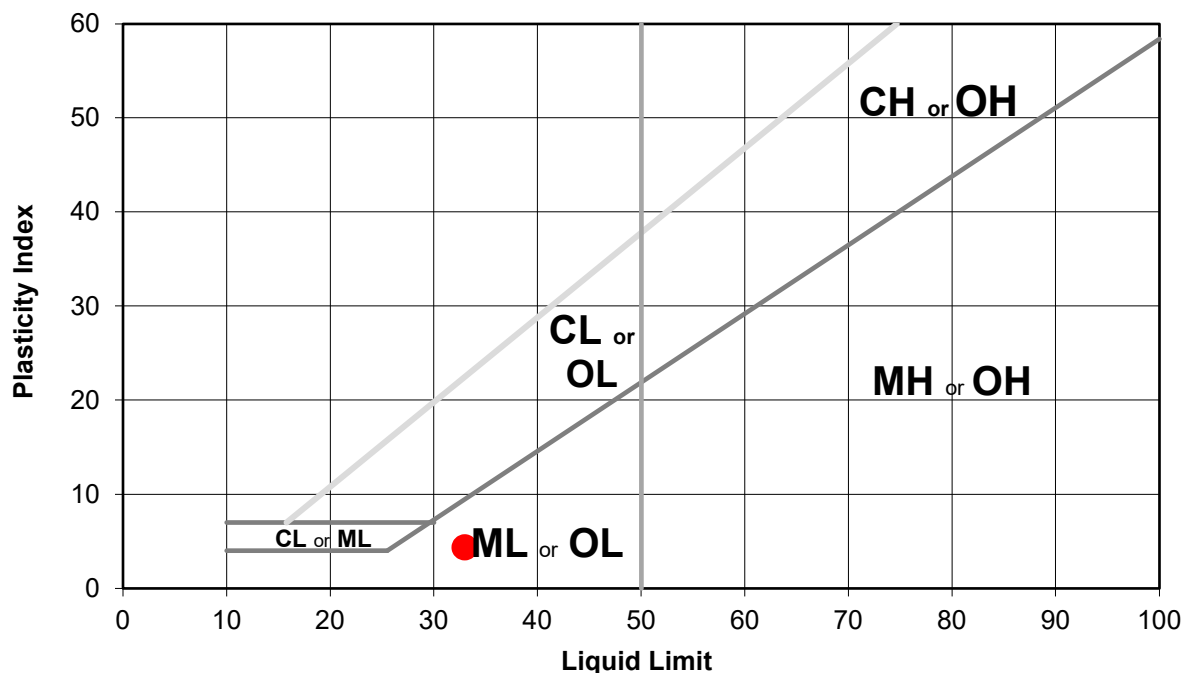
Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
◆	B-3	R-8	20	28	20	8	CL



Job Name:	Simms Park Belflower	Date Sampled:	5/9/2022
Job Number:	CWT22-05	Date Completed:	5/16/2022
Tested By:	MG	Sample Identification:	B-3, R-12
Note:		Sample Depth:	40ft
Sample Description:	Olive Gray ML Qal		

		Plastic Limit	
Test No.		1	2
Number of Blows			
Container ID		P1	P5
Wet Weight of Soil + Cont.	grams	22.40	22.60
Dry Weight of Soil + Cont.	grams	20.20	20.30
Weight of Container	grams	12.40	12.40
Moisture Weight	grams	2.20	2.30
Weight of Dry Soil	grams	7.80	7.90
Moisture Content	%	28.2	29.1

Liquid Limit			
1	2	3	4
16	23	32	
M14	P13	S12	
49.00	44.50	52.40	
42.60	39.70	46.30	
25.40	25.40	25.60	
6.40	4.80	6.10	
17.20	14.30	20.70	
37.2	33.6	29.5	

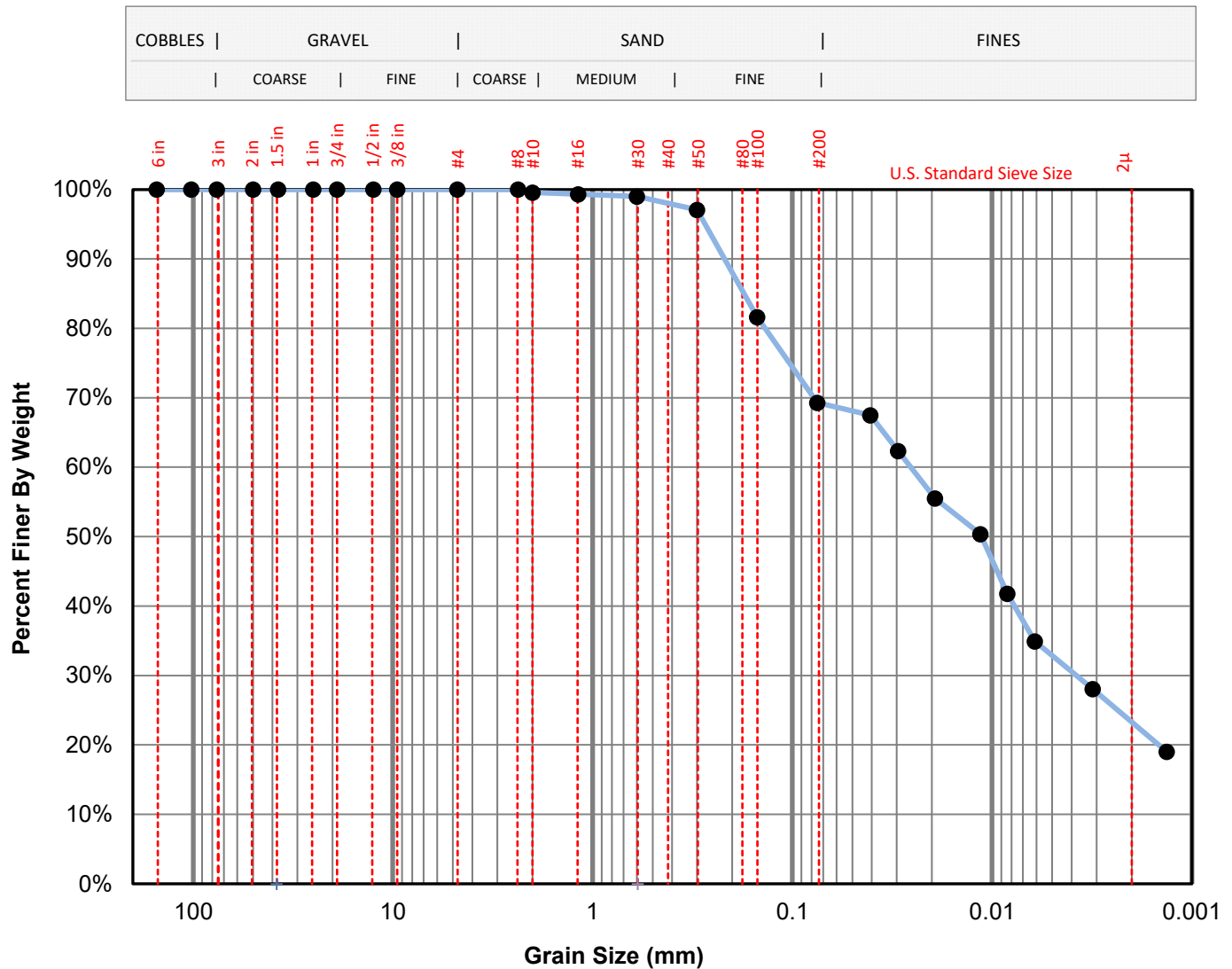


Plastic Limit **29**
 Liquid Limit **33**
 Plasticity Index **4**

USCS Classification ML
 Based on Atterberg Limits only



Job Name:	Simms Park Belflower	Date Sampled:	5/9/2022
Job Number:	CWT-22-05	Date Completed:	5/18/2022
Tested By:	MG	Sample Identification:	B-4 S-2
Note:		Sample Depth:	2.5ft
Sample Description:	Olive Gray Sandy SILT, ML		



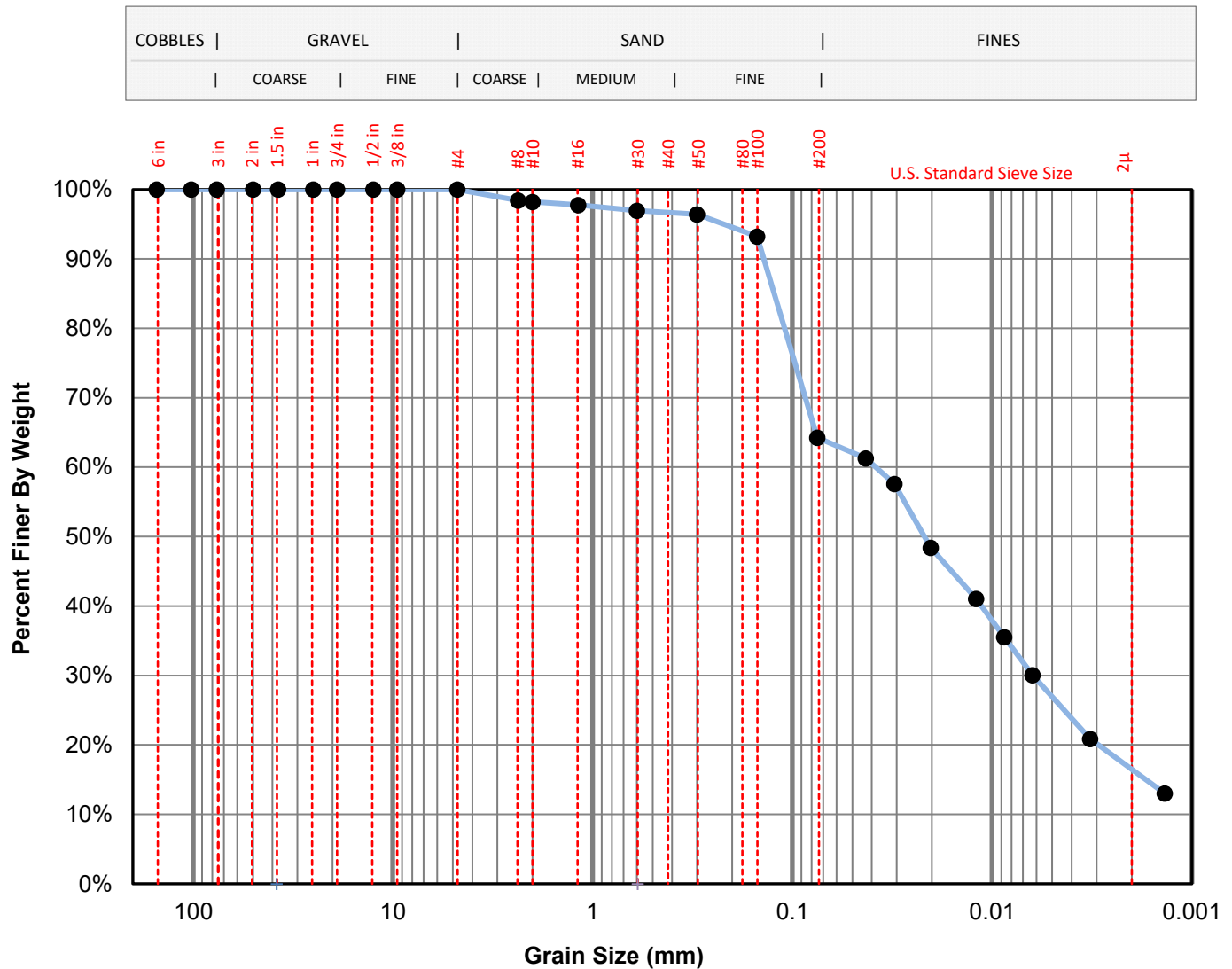
Symbol	Boring No.	Sample No.	Depth	LL	PI	USCS	Cobbles	Gravel	Sand	Fines	2 μ
●	B-4	S-2	2.5ft			ML	0%	0%	31%	69%	24%



GRAIN SIZE DISTRIBUTION ANALYSIS

ASTM C136/C117/D7928/D6913

Job Name:	Simms Park Belflower	Date Sampled:	5/9/2022
Job Number:	CWT-22-05	Date Completed:	5/18/2022
Tested By:	MG	Sample Identification:	B-5, R-2
Note:		Sample Depth:	2.5ft
Sample Description:	Olive Gray Sandy SILT, ML		



Symbol	Boring No.	Sample No.	Depth	LL	PI	USCS	Cobbles	Gravel	Sand	Fines	2 μ
●	B-5	R-2	2.5ft			ML	0%	0%	36%	64%	17%

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Client: Tetra Tech
 Project Name: Simms Park, Bellflower
 Project No.: CWT22-05
 Boring No.: B-5
 Sample No.: SK-1
 Visual Sample Description: Sandy Silt

AP Number: 22-0526
 Tested By: TV Date: 05/19/22
 Calculated By: NR Date: 05/20/22
 Checked By: AP Date: 05/24/22
 Depth (ft.): 0-5

METHOD

MOLD VOLUME (CU.FT)

A
0.0333

Compaction Method

Preparation Method

☒ ASTM D1557
☐ ASTM D698
☐ Moist
☒ Dry

Wt. Comp. Soil + Mold (gm.)	3847	3902	3824	3724		
Wt. of Mold (gm.)	1838	1838	1838	1838		
Net Wt. of Soil (gm.)	2009	2064	1986	1886		
Container No.						
Wt. of Container (gm.)	144.63	181.50	135.88	145.25		
Wet Wt. of Soil + Cont. (gm.)	772.32	831.81	770.40	789.59		
Dry Wt. of Soil + Cont. (gm.)	714.54	760.38	687.64	743.95		
Moisture Content (%)	10.14	12.34	15.00	7.62		
Wet Density (pcf)	132.84	136.47	131.35	124.74		
Dry Density (pcf)	120.61	121.48	114.22	115.90		

Maximum Dry Density (pcf)

122.0

Optimum Moisture Content (%)

11.7

Maximum Dry Density w/ Rock Correction (pcf)

N/A

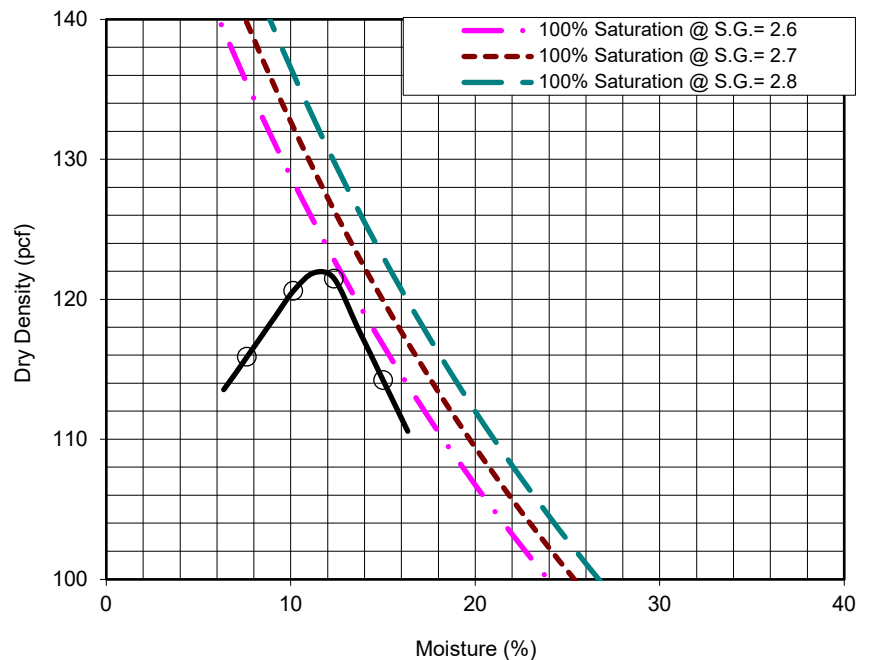
Optimum Moisture Content w/ Rock Correction (%)

N/A**PROCEDURE USED**

☒ **METHOD A: Percent of Oversize:** 5.0%
 Soil Passing No. 4 (4.75 mm) Sieve
 Mold: 4 in. (101.6 mm) diameter
 Layers: 5 (Five)
 Blows per layer: 25 (twenty-five)

☐ **METHOD B: Percent of Oversize:** N/A
 Soil Passing 3/8 in. (9.5 mm) Sieve
 Mold: 4 in. (101.6 mm) diameter
 Layers: 5 (Five)
 Blows per layer: 25 (twenty-five)

☐ **METHOD C: Percent of Oversize:** N/A
 Soil Passing 3/4 in. (19.0 mm) Sieve
 Mold: 6 in. (152.4 mm) diameter
 Layers: 5 (Five)
 Blows per layer: 56 (fifty-six)



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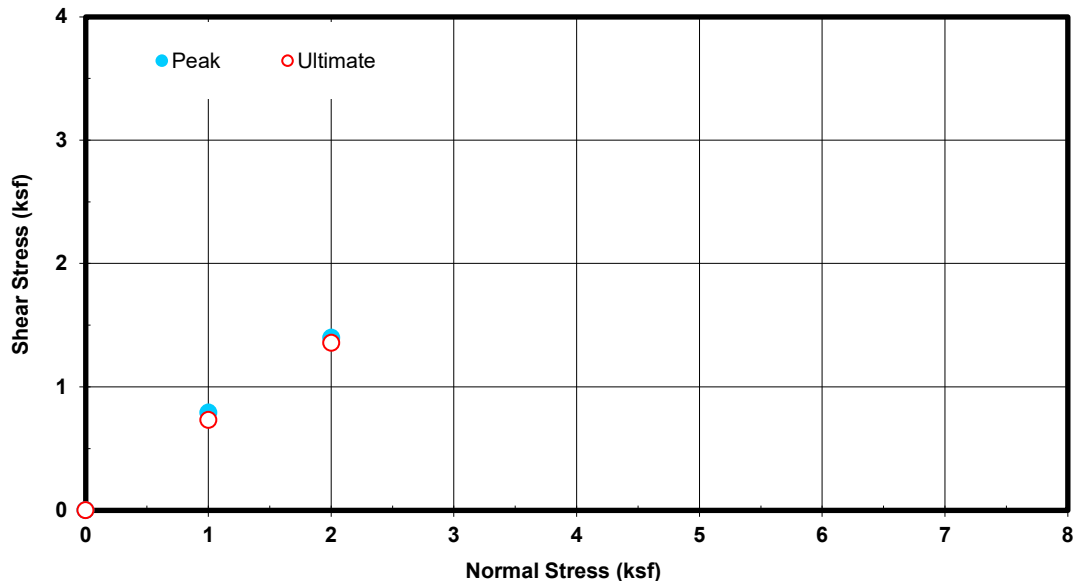
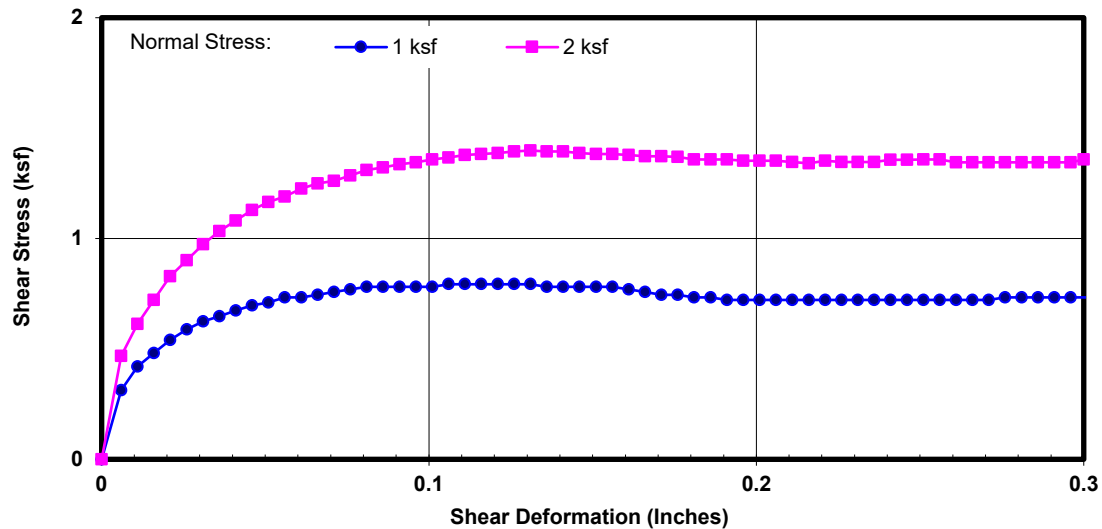
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t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com**DIRECT SHEAR TEST RESULTS**
ASTM D 3080

Client: Tetra Tech
Project Name: Simms Park, Bellflower
Project No.: CWT22-05
Boring No.: B-3
Sample No.: R-4 **Depth (ft):** 7.5
Sample Type: Mod. Cal.
Soil Description: Sandy Silt
Test Condition: Inundated **Shear Type:** Regular

Tested By: LS **Date:** 05/17/22
Computed By: NR **Date:** 05/19/22
Checked by: AP **Date:** 05/24/22

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
112.8	92.2	22.5	30.4	73	99	1	0.793	0.733
						2	1.398	1.357





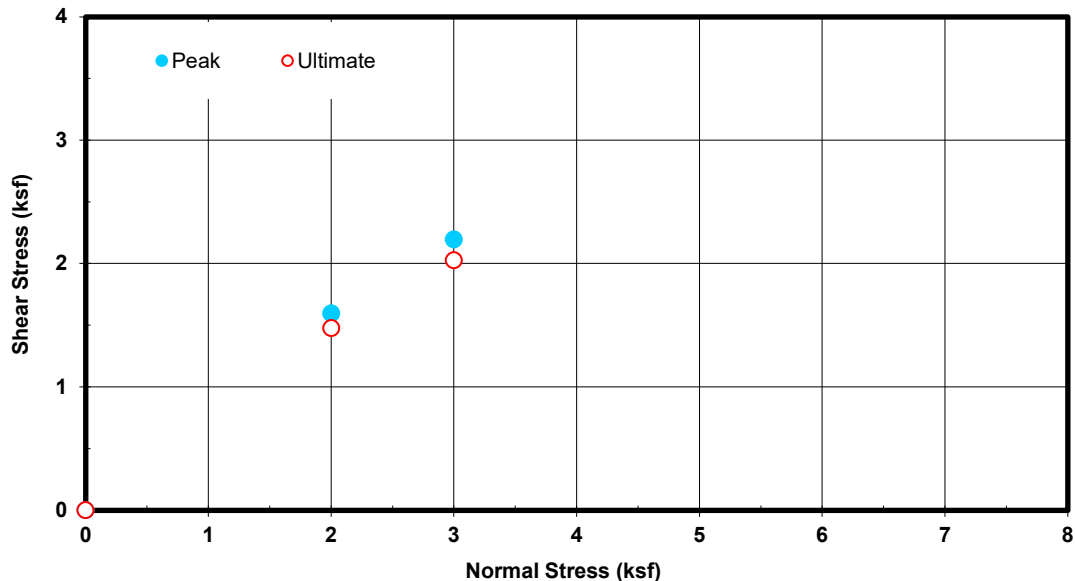
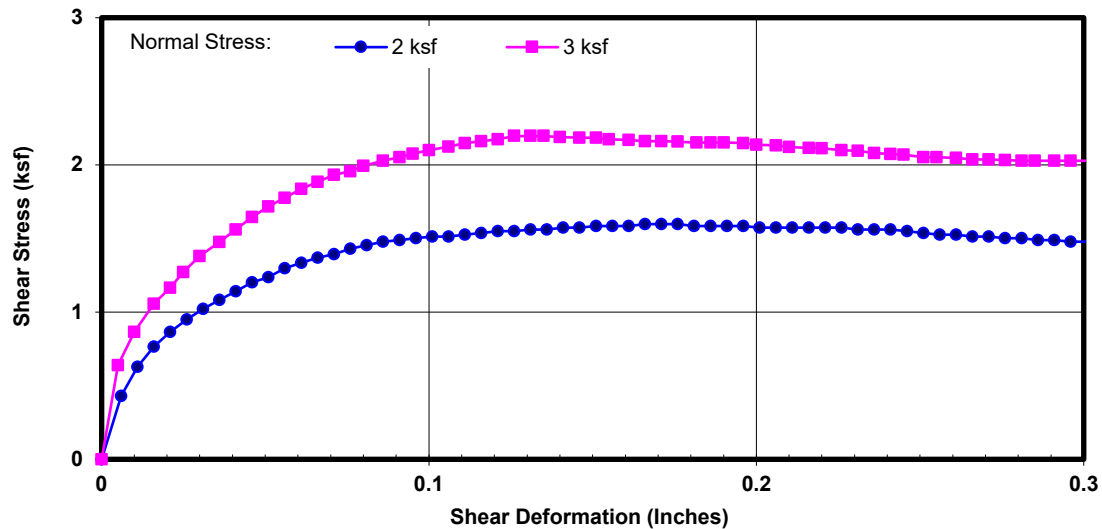
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DIRECT SHEAR TEST RESULTS ASTM D 3080

Client: Tetra Tech
Project Name: Simms Park, Bellflower
Project No.: CWT22-05
Boring No.: B-3
Sample No.: R-8 **Depth (ft):** 20
Sample Type: Mod. Cal.
Soil Description: Sandy Lean Clay
Test Condition: Inundated **Shear Type:** Regular

Tested By: LS **Date:** 05/17/22
Computed By: NR **Date:** 05/19/22
Checked by: AP **Date:** 05/24/22

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
129.4	106.7	21.3	21.6	99	100	2	1.597	1.477
						3	2.196	2.028



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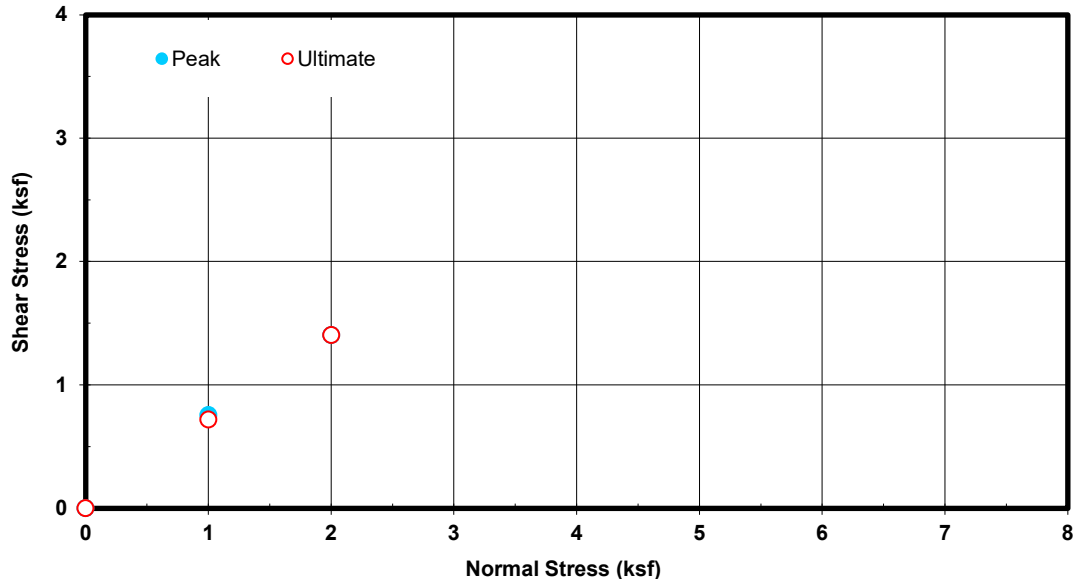
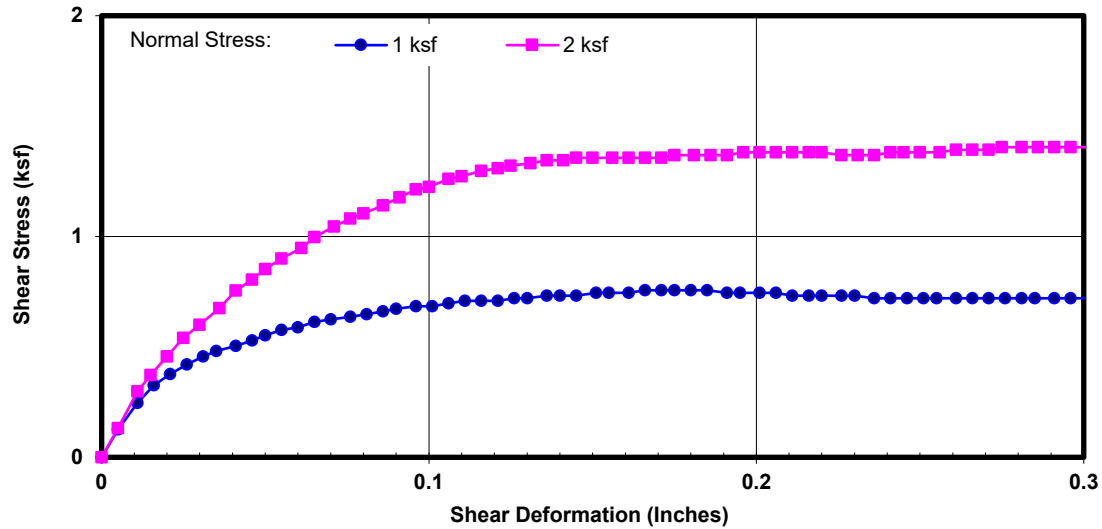
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t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com**DIRECT SHEAR TEST RESULTS**
ASTM D 3080

Client: Tetra Tech
Project Name: Simms Park, Bellflower
Project No.: CWT22-05
Boring No.: B-4
Sample No.: R-5 **Depth (ft):** 10
Sample Type: Mod. Cal.
Soil Description: Silty Sand, fine-grained
Test Condition: Inundated **Shear Type:** Regular

Tested By: LS **Date:** 05/17/22
Computed By: NR **Date:** 05/19/22
Checked by: AP **Date:** 05/24/22

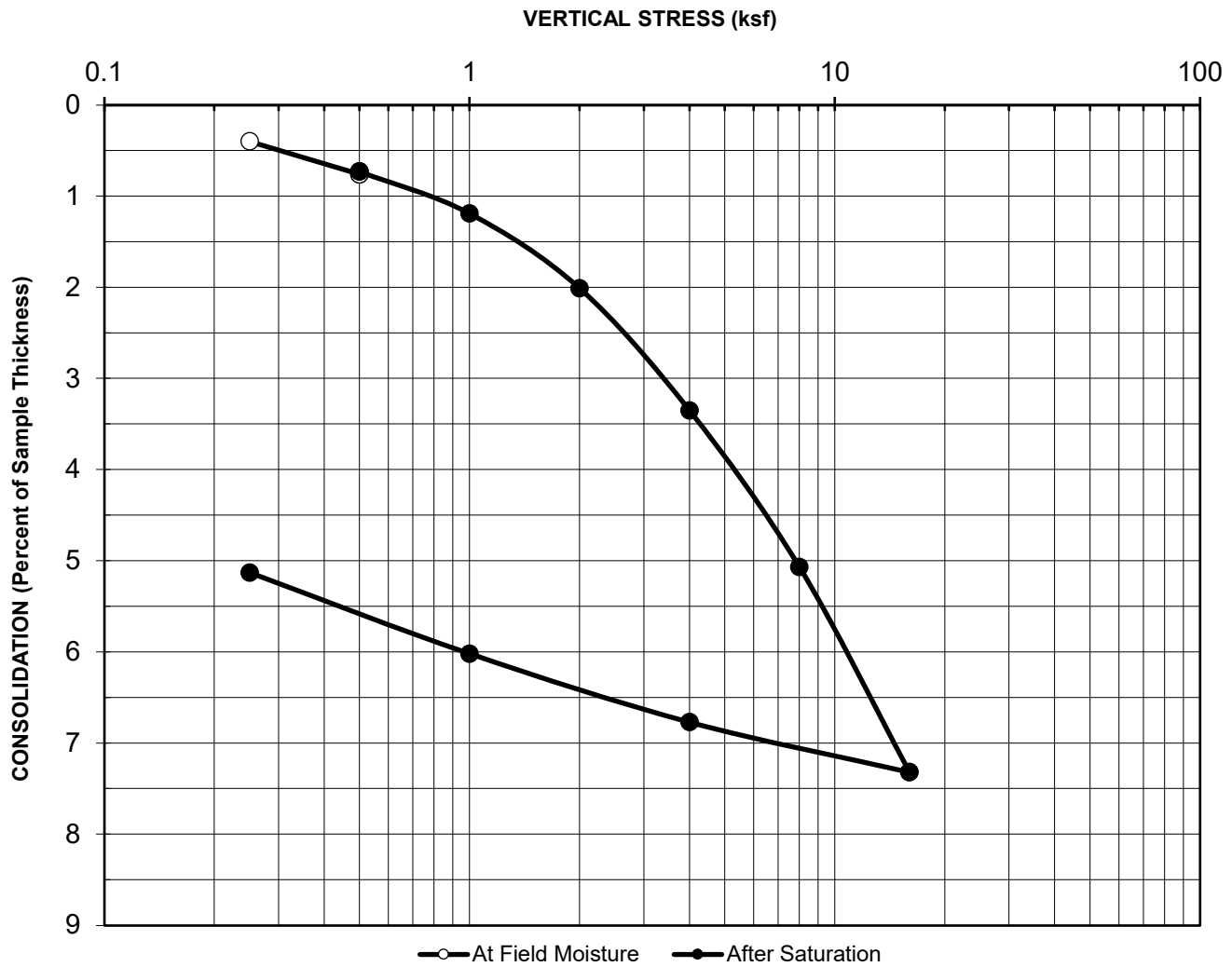
Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
101.0	91.3	10.6	31.3	34	100	1	0.756	0.720
						2	1.404	1.404



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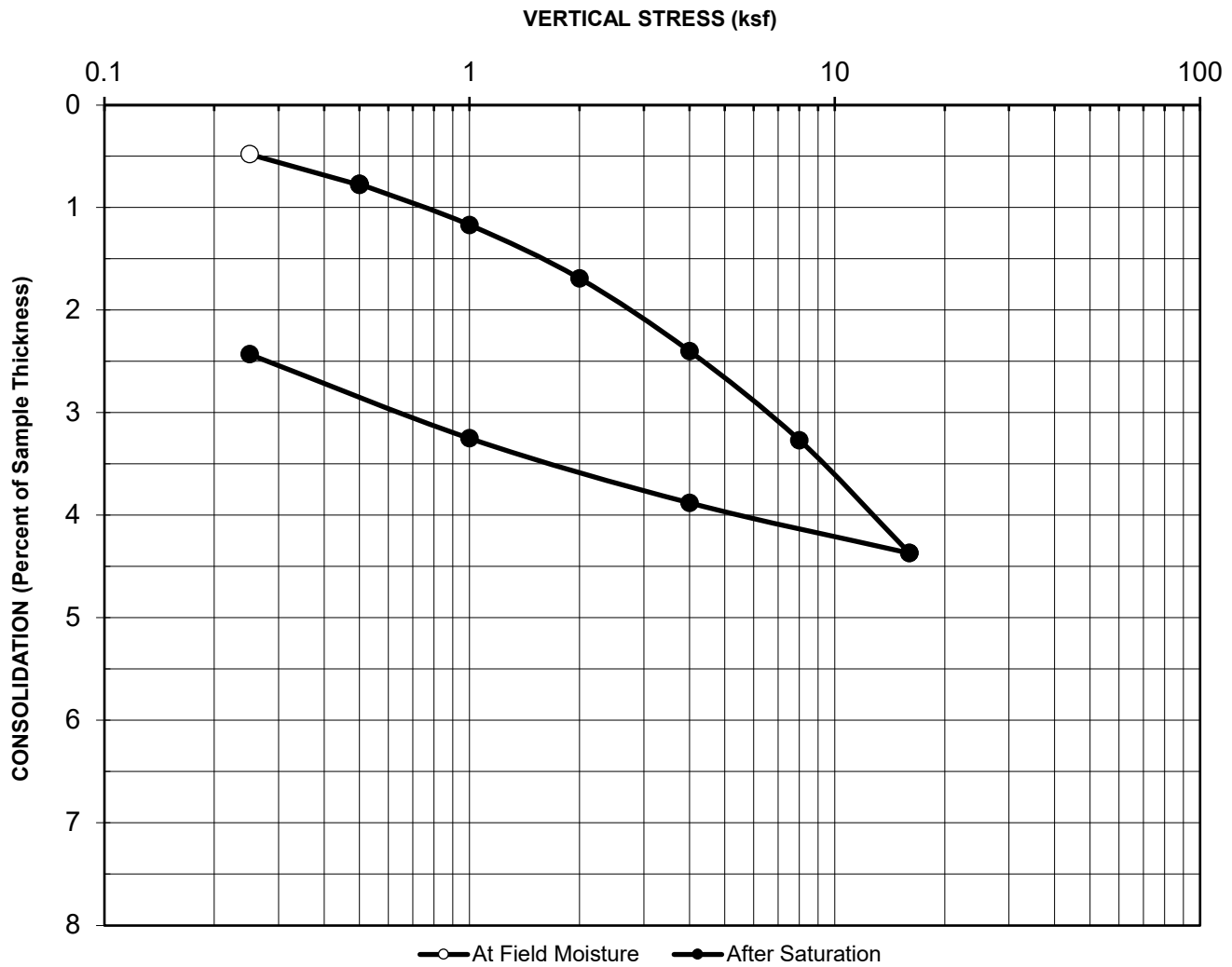
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t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.comBoring No. : B-3Initial Dry Unit Weight (pcf): 104.8Sample No.: R-8Initial Moisture Content (%): 21.3Depth (feet): 20Final Moisture Content (%): 21.5Sample Type: Mod CalAssumed Specific Gravity: 2.7Soil Description: Sandy Lean ClayInitial Void Ratio: 0.61Remarks: Swell= 0.03% upon inundation**CONSOLIDATION CURVE
ASTM D 2435**Project Name: Simms Park, BellflowerProject No.: CWT22-05Date: 5/16/2022AP No: 22-0526 Sheet No: 1

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t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.comBoring No. : B-4Initial Dry Unit Weight (pcf): 101.6Sample No.: R-9Initial Moisture Content (%): 16.7Depth (feet): 25Final Moisture Content (%): 22.4Sample Type: Mod CalAssumed Specific Gravity: 2.7Soil Description: Sandy SiltInitial Void Ratio: 0.66Remarks: Swell= 0.01% upon inundation**CONSOLIDATION CURVE
ASTM D 2435**Project Name: Simms Park, BellflowerProject No.: CWT22-05Date: 5/16/2022AP No: 22-0526 Sheet No: 1

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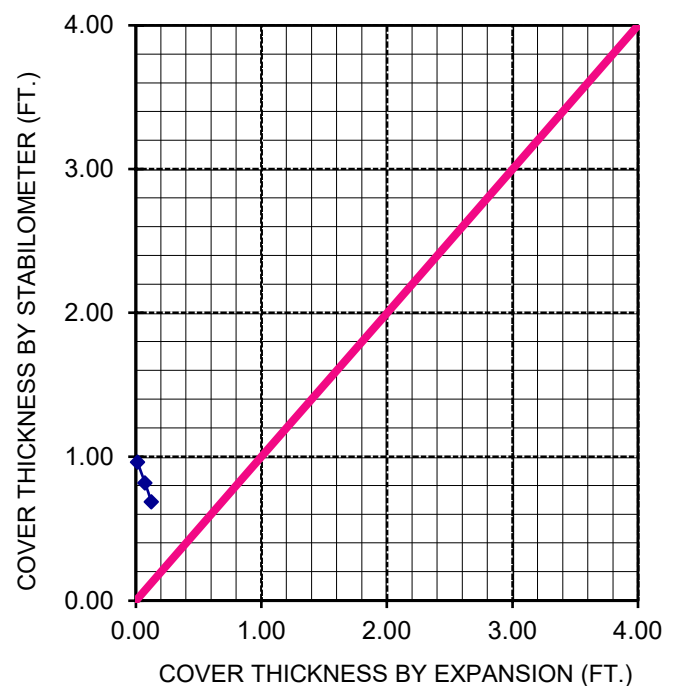
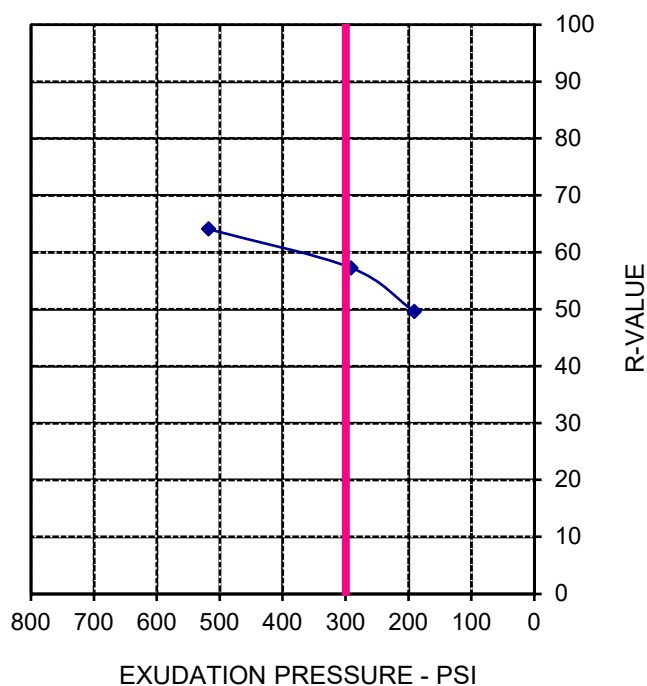
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t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com**R-VALUE TEST DATA**

ASTM D2844

Project Name: Simms Park, BellflowerProject Number: CWT22-05Boring No.: B-3Sample No.: SK-1Location: N/ASoil Description: Sandy SiltTested By: ST Date: 05/16/22Computed By: KM Date: 05/18/22Checked By: AP Date: 05/24/22Depth (ft.): 0-5

Mold Number	R4	R5	R6		R-VALUE	By Exudation:	58
Water Added, g	46	56	66			By Expansion:	*N/A
Compact Moisture(%)	16.3	17.4	18.3			At Equilibrium:	58
Compaction Gage Pressure, psi	250	250	250			(by Exudation)	
Exudation Pressure, psi	518	291	191		Remarks	Gf = 1.34, and 0.3 % Retained on the 3/4" *Not Applicable	
Sample Height, Inches	2.6	2.5	2.5				
Gross Weight Mold, g	3102	3044	3019				
Tare Weight Mold, g	2017	2012	2012				
Net Sample Weight, g	1085	1032	1007				
Expansion, inchesx10 ⁻⁴	37	22	4				
Stability 2,000 (160 psi)	16/34	18/39	22/46				
Turns Displacement	5.57	5.79	6.29				
R-Value Uncorrected	62	57	50				
R-Value Corrected	64	57	50				
Dry Density, pcf	108.7	106.5	103.1				
Traffic Index	8.0	8.0	8.0				
G.E. by Stability	0.69	0.82	0.96				
G.E. by Expansion	0.12	0.07	0.01				



**AP Engineering and Testing, Inc.**

DBE | MBE | SBE

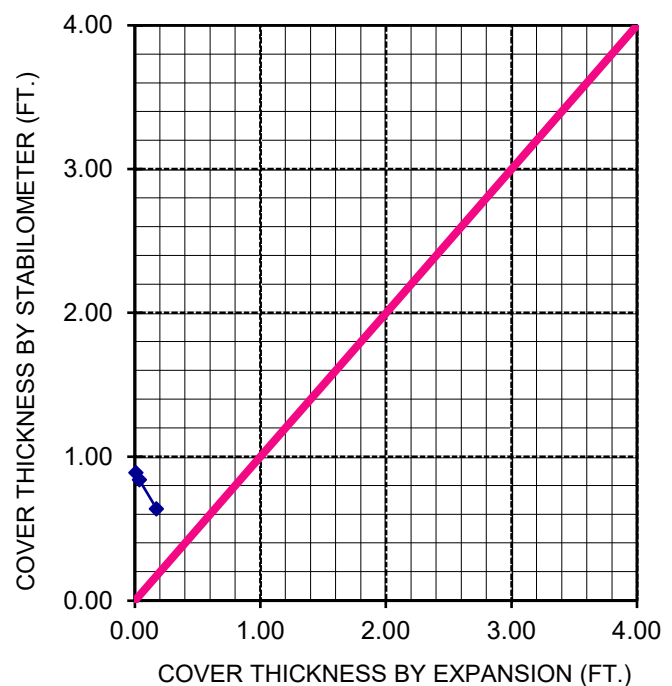
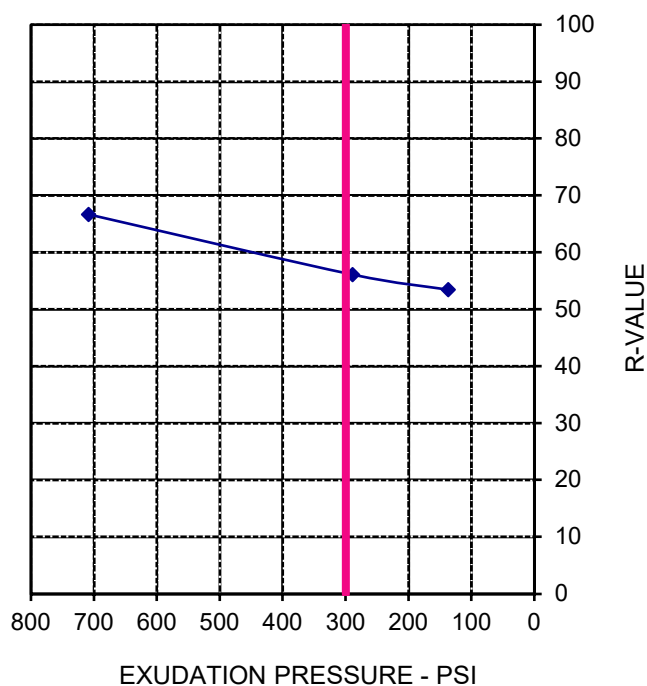
2607 Pomona Boulevard | Pomona, CA 91768

t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com**R-VALUE TEST DATA**

ASTM D2844

Project Name: Simms Park, BellflowerProject Number: CWT22-05Boring No.: B-5Sample No.: SK-1Location: N/ASoil Description: Sandy SiltTested By: ST Date: 05/16/22Computed By: KM Date: 05/18/22Checked By: AP Date: 05/24/22Depth (ft.): 0-5

Mold Number	D	F	E		R-VALUE	By Exudation:	56
Water Added, g	42	32	22			By Expansion:	*N/A
Compact Moisture(%)	15.1	14.1	13.1			At Equilibrium:	56
Compaction Gage Pressure, psi	250	250	250			(by Exudation)	
Exudation Pressure, psi	137	289	708		Remarks	Gf = 1.34, and 0.0 % Retained on the 3/4" *Not Applicable	
Sample Height, Inches	2.5	2.5	2.5				
Gross Weight Mold, g	3061	2928	3041				
Tare Weight Mold, g	1964	1869	1955				
Net Sample Weight, g	1096	1059	1086				
Expansion, inchesx10 ⁻⁴	3	11	52				
Stability 2,000 (160 psi)	20/44	19/42	14/30				
Turns Displacement	5.74	5.50	5.42				
R-Value Uncorrected	53	56	67				
R-Value Corrected	53	56	67				
Dry Density, pcf	115.5	112.5	116.4				
Traffic Index	8.0	8.0	8.0				
G.E. by Stability	0.89	0.84	0.64				
G.E. by Expansion	0.01	0.04	0.17				



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Boring No.	Sample No.	Depth (feet)	Soil Description	Minimum Resistivity (ohm-cm)	pH	Sulfate Content (ppm)	Chloride Content (ppm)
B-3	SK-1	0-5	Sandy Silt	471	9.1	871	579
B-4	SK-1	0	Sandy Clay	2,619	9.0	94	47

NOTES: Resistivity Test and pH: California Test Method 643

Sulfate Content : California Test Method 417

Chloride Content : California Test Method 422

ND = Not Detectable

NA = Not Sufficient Sample

NR = Not Requested

Appendix C

Seismic Demand



Youth and Senior Citizen Center

Latitude, Longitude: 33.884412, -118.133623



Date	10/26/2022, 10:35:21 AM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.594	MCE_R ground motion. (for 0.2 second period)
S_1	0.571	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.594	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.063	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.681	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.749	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
S_{sRT}	1.594	Probabilistic risk-targeted ground motion. (0.2 second)
S_{sUH}	1.753	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
S_{sD}	2.452	Factored deterministic acceleration value. (0.2 second)
S_{1RT}	0.571	Probabilistic risk-targeted ground motion. (1.0 second)
S_{1UH}	0.633	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S_{1D}	0.825	Factored deterministic acceleration value. (1.0 second)
$PGAd$	0.991	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.681	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.91	Mapped value of the risk coefficient at short periods

Type	Value	Description
C _{R1}	0.903	Mapped value of the risk coefficient at a period of 1 s
C _V	1.419	Vertical coefficient

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Unified Hazard Tool

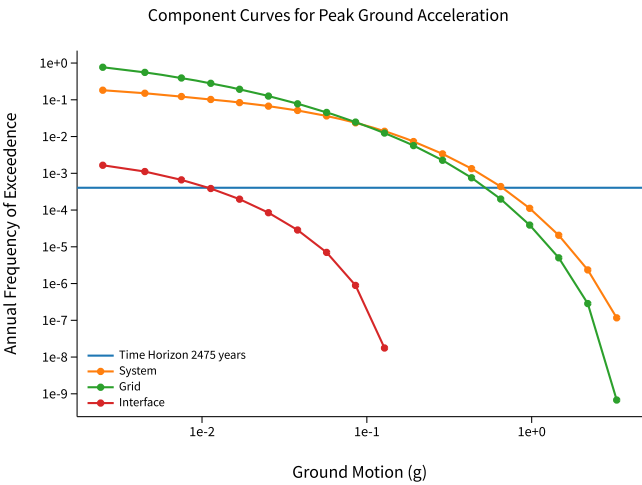
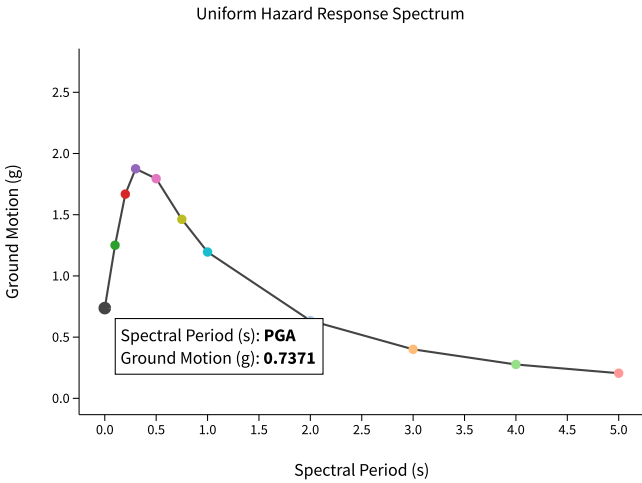
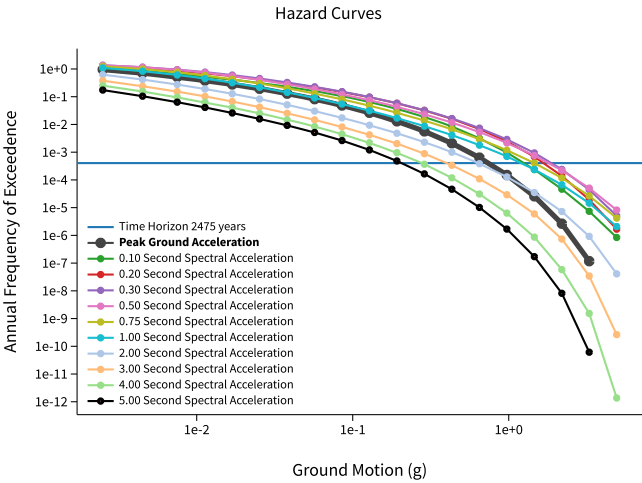


Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the [U.S. Seismic Design Maps web tools](#) (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

^ Input

Edition Dynamic: Conterminous U.S. 2014 (u...	Spectral Period Peak Ground Acceleration
Latitude Decimal degrees 33.884412	Time Horizon Return period in years 2475
Longitude Decimal degrees, negative values for western longitudes -118.133623	
Site Class 259 m/s (Site class D)	

^ Hazard Curve

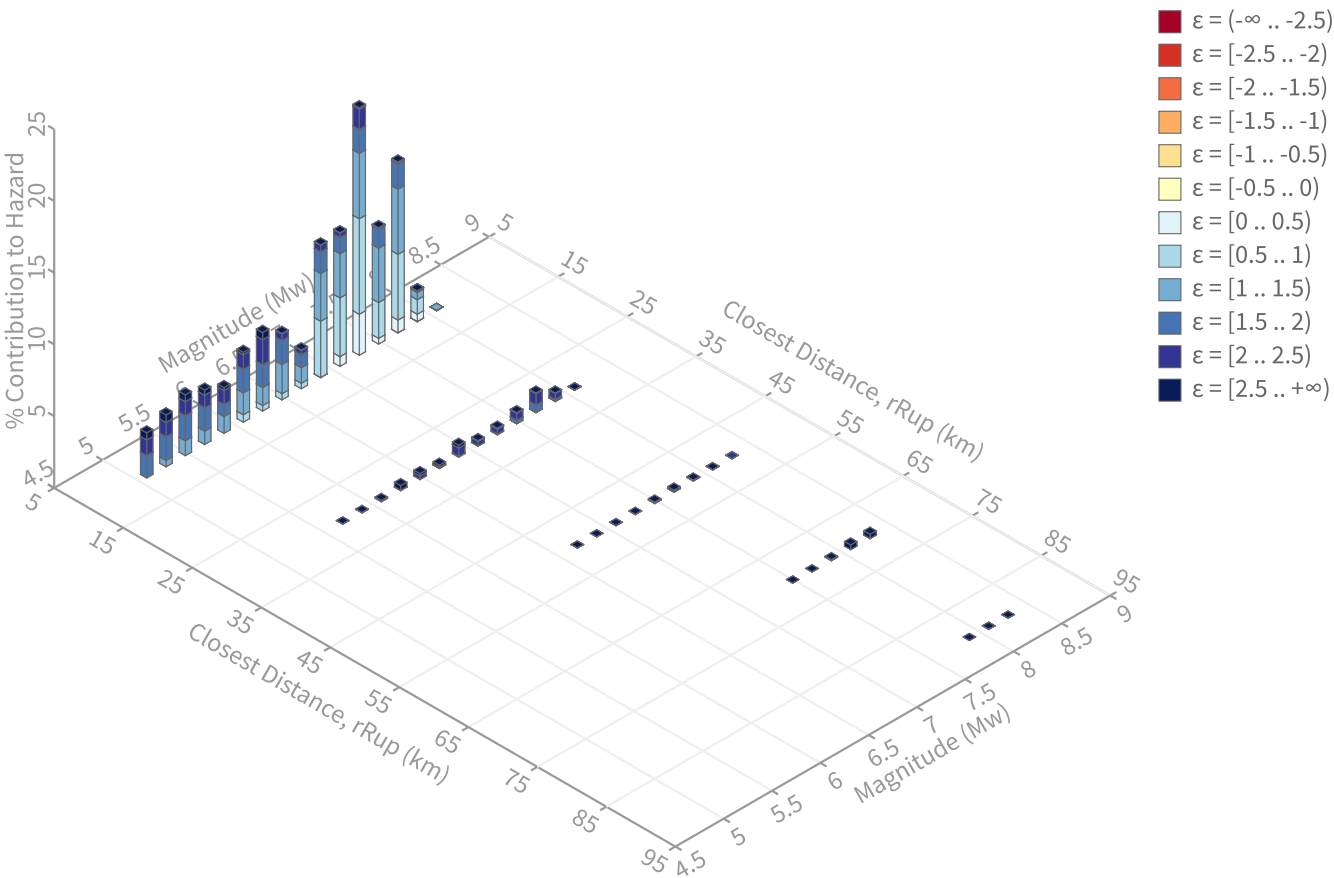


[View Raw Data](#)

^ Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr⁻¹
PGA ground motion: 0.73709924 g

Recovered targets

Return period: 2936.0493 yrs
Exceedance rate: 0.00034059374 yr⁻¹

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.05 %

Mean (over all sources)

m: 6.84
r: 11.48 km
ε₀: 1.38 σ

Mode (largest m-r bin)

m: 7.3
r: 11.18 km
ε₀: 0.98 σ
Contribution: 17.25 %

Mode (largest m-r-ε₀ bin)

m: 7.3
r: 9.46 km
ε₀: 0.67 σ
Contribution: 6.66 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

- ε0:** [-∞ .. -2.5)
- ε1:** [-2.5 .. -2.0)
- ε2:** [-2.0 .. -1.5)
- ε3:** [-1.5 .. -1.0)
- ε4:** [-1.0 .. -0.5)
- ε5:** [-0.5 .. 0.0)
- ε6:** [0.0 .. 0.5)
- ε7:** [0.5 .. 1.0)
- ε8:** [1.0 .. 1.5)
- ε9:** [1.5 .. 2.0)
- ε10:** [2.0 .. 2.5)
- ε11:** [2.5 .. +∞]

<https://earthquake.usgs.gov/hazards/interactive/>

5/5

Appendix D

Liquefaction Analyses

Summary of Liquefaction and Earthquake-Induced Settlement Analysis

Project:	ciBLF 22-01 Simms Park Youth and Senior Center	Boring:	B-6	Engineer:	FC	Date:	10/25/2022
----------	---	---------	------------	-----------	----	-------	------------

Liquefaction Evaluation Method				Liquefaction Analysis Statistics			
Correction for fines content		Idriss & Boulang. 2008, 2014		Total thickness of evaluated profile		56.5 feet	
Correction for overburden C_N		Idriss & Boulang. 2014 (N1)60cs		Profile thickness susceptible to liquefaction		29.5 feet	
Cyclic resistance ratio of soil CRR_{CS}		Idriss & Boulang. 2004, 2014		Number of evaluated intervals		15	
Correction for overburden K_σ		Idriss & Boulang. 2008, 2014		Number of potentially liquefiable intervals		13	
Stress reduction factor r_D		Idriss 1999, I&B 2008,2014		Average Factor of Safety of sandy intervals		1.16	
Magnitude scaling factor MSF		Idriss & Boulang. 2014					
Dry settlement		Pradel, 1998a,b		Dry sand settlement		0.00 inches	
Liquefaction settlement		Yoshimine et al., 2006 – w/ calibration		Liquefaction settlement		5.85 inches	
				Total earthquake-induced settlement			
Liquefaction behavior Plasticity Index threshold		less or equal to 7		5.85 inches			
Saturated settlement threshold		less or equal to 86% fines					
Dry settlement threshold		less or equal to 25% fines					
Cyclic softening Plasticity Index threshold		greater or equal to 18					
Profile		Earthquake loading		Checks			
In-Situ Groundwater depth	32.00 feet	M	7.3	Groundwater depth check		OK	
DESIGN Groundwater depth	8.00 feet	PGA	0.749	Design groundwater/excavation depth check		OK	
DESIGN Excavation depth	0.00 feet			Fines correction method compatibility		OK	
DESIGN Surcharge (fill)	0.00 feet			Idris & Boulanger, 2004 method for C_N		not used	
				Cetin 2009 settlement method		not used	
Version v2 2022-03							

[illegible]

B-6

Design excavation depth 0.00 feet
Design groundwater depth 8.00 feet

