APPENDIX D-2

GEOTECHNICAL REPORT ADDENDUM NO. 1

TETRA TECH

Geotechnical Engineering ● Engineering Geology

Geotechnical Design Report

FURMAN PARK
STORMWATER CAPTURE, STORAGE AND INFILTRATION FACILITIES
10419 Rives Avenue
Downey, California



Prepared for:

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July 15, 2022
Project No. CWT 22-06E

TETRA TECH

Project No. CWT 22-06E
July 15, 2022

Mr. Oliver Galang
Craftwater Engineering Inc.
10711 Oakbend Dr.
San Diego, CA 92131-2610

Subject: **GEOTECHNICAL DESIGN REPORT**
FURMAN PARK
STORMWATER CAPTURE, STORAGE AND INFILTRATION FACILITIES
10419 Rives Avenue
Downey, California

Dear Mr. Galang:

Presented herein is Tetra Tech's geotechnical design report for the proposed stormwater capture, storage and infiltration facilities at Furman Park located at 10419 Rives Avenue in the City of Downey, 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. 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.

Respectfully submitted,
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1. INTRODUCTION AND PROJECT DESCRIPTION

This report presents the results of Tetra Tech's geotechnical engineering evaluation for the proposed stormwater capture, storage and infiltration facilities at Furman Park (see Figure 1 – Project Location Map) located at 10419 Rives Avenue, in the City of Downey, California. The appendices to this report include logs of borings, results of laboratory and field infiltration tests, a summary of seismic demand and liquefaction analyses.

The development of the Furman Park Stormwater Capture and Infiltration Project represents a major step to achieve pollutant load reductions for the Los Angeles River Watershed Management Program. The jurisdictions within the project area include the City of Downey with 394 acres and the City of Pico Rivera with 81 acres of watershed. The project is in the Montebello Forebay area which accounts for half of the total groundwater replenishment in the Los Angeles Central Basin. The park is currently occupied by several sport facilities and open fields used for recreation and several sports including tennis and baseball. The ground at the park is relatively level with a slight gradient of less than 1 percent descending from north to south.

The project is intended to intercept a storm runoff flow of about 50 cubic feet per second (cfs) from an RCP stormdrain line (BI 0020) running northwest-southeast under Florence Avenue and southwest-northeast under Horley Avenue to the north of the site. The stormdrain is a 96-inch reinforced concrete pipe (RCP) managed by the Los Angeles County Flood Control District (LACFCD). The diverted stormwater flow will be captured near the intersection between Florence Avenue and Horley Avenue located about 0.15 miles northeast of the park, and then conveyed through a new 36-inch stormdrain diversion pipe running southwest under Horley Avenue, southeast under Irwingrove Drive, and southwest under Rives Avenue before entering the park near the northeast corner, and then conveyed to an underground pre-treatment facility within the open field area. After pretreatment, the stormwater will be conveyed to a best management practice (BMP) underground storage and infiltration facility within the open field area with 8.4-acre-feet capacity (see Figure 2 – Site Layout and Boring Location Map). The invert of the storage facility will be at a depth of about 22 to 24 feet and the facility will be about 215 feet wide (southwest-northeast direction) by 468 feet long (northwest-southeast direction). The stored pre-treated stormwater will be pumped and conveyed through a new 18-inch outlet pipeline to either a stormwater harvesting unit or to an existing 33-inch RCP stormdrain line (BI 3151) located about 0.10 miles to the southwest of the park near the intersection between Quinn Street and Wiley Burke Avenue.

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

- stormwater storage and infiltration units with a bottom depth of 22 to 24 feet,
- storm drain diversion unit with an invert depth of about 16 to 18 feet,
- actuated valve with an invert depth of about 15 to 16 feet,
- pre-treatment unit with an invert depth of about 13 feet,
- pump station with an invert depth of about 24 feet,
- stormwater harvesting unit (no information was available regarding this structure at the time of writing, it is assumed to be at-grade within this report),
- diversion and outlet conveyance pipelines and several manholes at varying depths.

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) and from the City of Downey.
- Coordinate with LACDPH engineering staff, park personnel, City of Downey 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 conveyance pipelines and the storage and infiltration facilities will be located, including excavating, logging, and geotechnical sampling of 6 soil exploratory borings to a maximum depth of 56.5 feet.
- Excavate 3 percolation borings to depths of approximately 20 to 22 feet within the park's open 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 the subsurface soils for infiltration below the proposed infiltration units.
    - 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 subsurface soil and groundwater conditions beneath the site were explored on June 1st and June 2nd, 2022 by Tetra Tech and included the drilling, logging, and sampling of 6 hollow stem auger exploratory borings denoted B-1 through B-6, and 3 percolation test borings denoted P-1 through P-3.

The hollow stem auger exploratory and percolation borings were excavated at the approximate locations indicated on Figure 2 using a CME-75 truck-mounted drill rig equipped with 8-inch diameter hollow stem augers. Borings B-1 and B-6 were advanced along the proposed outlet and diversion pipeline alignments, respectively, to a maximum depth of 21.5 feet; borings B-2 through B-5 were advanced within the park area to a maximum depth of 56.5 feet; and percolation boreholes P-1 through P-3 were advanced within the northern area of the park to depths of approximately 20 to 22 feet. The approximate latitude and longitude of the soil explorations, the approximate elevations, and depths of the 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 the park staff. A drilling permit was obtained from the Los Angeles County Department of Public Health (LACDPH) and from the City of Downey for all the subsurface explorations. Underground Service Alert, the LACDPH inspector, City of Downey personnel, 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 90.1 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.

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 A.

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 within the park area were disposed at locations specified by park personnel. The percolation boreholes were backfilled with tamped excavated soil cuttings. At boreholes B-1 and B-6 the asphalt surface was restored with an asphalt cold patch.

Table 1
Boring Information

Boring Number	General Location	Approximate			
		Latitude	Longitude	Ground Elevation (feet)*	Depth (feet)
B-1	Outlet stormdrain near intersection between Quinn Street and Wiley Burke Avenue	33.952567	-118.140558	119.3	21.5
B-2	Outlet stormdrain near southwest corner of park	33.953193	-118.138930	122.3	21.5
B-3	Within the Storage and Infiltration BMP footprint (west)	33.954304	-118.137657	122.9	51.5
B-4	Within the Storage and Infiltration BMP footprint (central)	33.953813	-118.136777	123.6	56.5
B-5	Within the Storage and Infiltration BMP footprint (east)	33.953717	-118.136062	125.8	31.5
B-6	Diversion stormdrain near intersection between Florence Avenue and Horley Avenue	33.955824	-118.136550	125.5	21.5
P-1	Within the Storage and Infiltration BMP footprint (west)	33.954449	-118.137639	122.9	22.1
P-2	Within the Storage and Infiltration BMP footprint (central)	33.954019	-118.137437	122.4	19.8
P-3	Within the Storage and Infiltration BMP footprint (east)	33.953776	-118.136776	123.5	19.8

*Surveyed by Tetra Tech on 6/2/2022
Cells highlighted in light blue correspond to percolation borings

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:

- 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;
- 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 Appendix B. For ease of referral to the soil profile, most of the laboratory results are also included on the boring logs in Appendix A.

5. FIELD INFILTRATION TESTING

Tetra Tech performed 3 small diameter borehole percolation tests denoted P-1 through P-3 located within the park field area, using the test procedure described in the LACDPW infiltration guidelines GS200.1(2021). The percolation tests were performed at a depth of about 20 to 22 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 2-inch-diameter slotted pipe (casing). 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 about 2 feet was maintained above the bottom of the borehole. The readings to determine the water depth were taken with a well sounder every 10 minutes 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 minimum of 6 readings were taken.

The raw field percolation rate was converted, as indicated in the infiltration test logs in Appendix C, 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 2 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.

Table 2
Infiltration Rates

Boring Percolation Test No.	Borehole Depth (feet)	Infiltration Rate (inches/hour)	
		Corrected (field)	Design
P-1	22.1	3.63	0.66
P-2	19.8	5.98	1.09
P-3	19.8	1.75	0.32

The percolation results from this exploration indicate that the design infiltration rate within the range of depths tested ranges between 0.32 and 1.09 inches/hour. These design infiltration rates are higher than the acceptable minimum of 0.3 inches/hour required by the LACDPW Guidelines (2021). It is recommended that a design infiltration rate of 0.5 inches/hour be used for the design of the project.

6. SUBSURFACE CONDITIONS

6.1. Regional Geology

The project site is located in the central portion of the greater Los Angeles Basin. 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

The project site is located approximately 0.5 miles south of Rio Hondo River, and approximately 2 miles east of the Los Angeles River. Regional geologic mapping published by Saucedo et al (2016) and Jennings (1962) (Figure 3 – Geologic Map) shows that the site is underlain by alluvial deposits of Holocene and late Pleistocene age consisting of poorly consolidated, poorly sorted, permeable flood-plain deposits consisting of soft clay, silt, and loose to moderately dense sand and silty sand. Generalized descriptions of subsurface materials reported in this investigation include artificial fill and native alluvial soils. Reported subsurface conditions are described in the following sections. Copies of the boring logs are included in Appendix A.

6.3. Artificial Fill

Artificial fill was encountered up to a depth of 5 feet in borings B-1 and B-6 drilled along the pipeline alignments. The fill materials consisted of medium dense fine silty sands.

Artificial fill was encountered up to a depth of 2.5 feet in the borings drilled within the park area. The fill materials consisted of medium dense fine-grained silty sands.

6.4. Native Alluvium (Qya_2)

Native alluvial (Qya_2) soil was encountered below the fill soils to the maximum explored depth of 56.5 feet below the ground surface and included fine- and coarse-grained soils. The fine-grained materials consisted of olive to olive gray silt and sandy silt. The coarse-grained materials consisted of olive gray silty sand to poorly-graded sand. The coarse-grained soils appear to be more prevalent at depths greater than about 20 feet.

Uncorrected SPT blowcounts in the native alluvium for the fine-grained (clay, silt) soils ranged from 5 to 23 indicating stiff to very stiff consistencies. SPT blowcounts in the native alluvial coarse-grained (sandy) soils ranged from 8 to larger than 50 indicating loose to very dense materials.

6.5. Groundwater

According to the State of California (CDMG, 1998), the historic high groundwater level near the site has been mapped within the South Gate 7.5-minute Quadrangle at a depth of about 8 feet (Figure 4 – Historic High Groundwater Map). Groundwater was not encountered during our field exploration to the maximum explored depth of 56.5 feet. 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.

LACDPW groundwater well data obtained from their website for wells within a 1-mile radius of the site (https://dpw.lacounty.gov/general/wells/) between 1949 and 2018 indicate groundwater elevations in the area ranging from - 20 to 114 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 50 years has been generally deeper than 40 feet i.e., below elevation 78 feet, with only a couple of instances above that elevation (see Diagram 1).

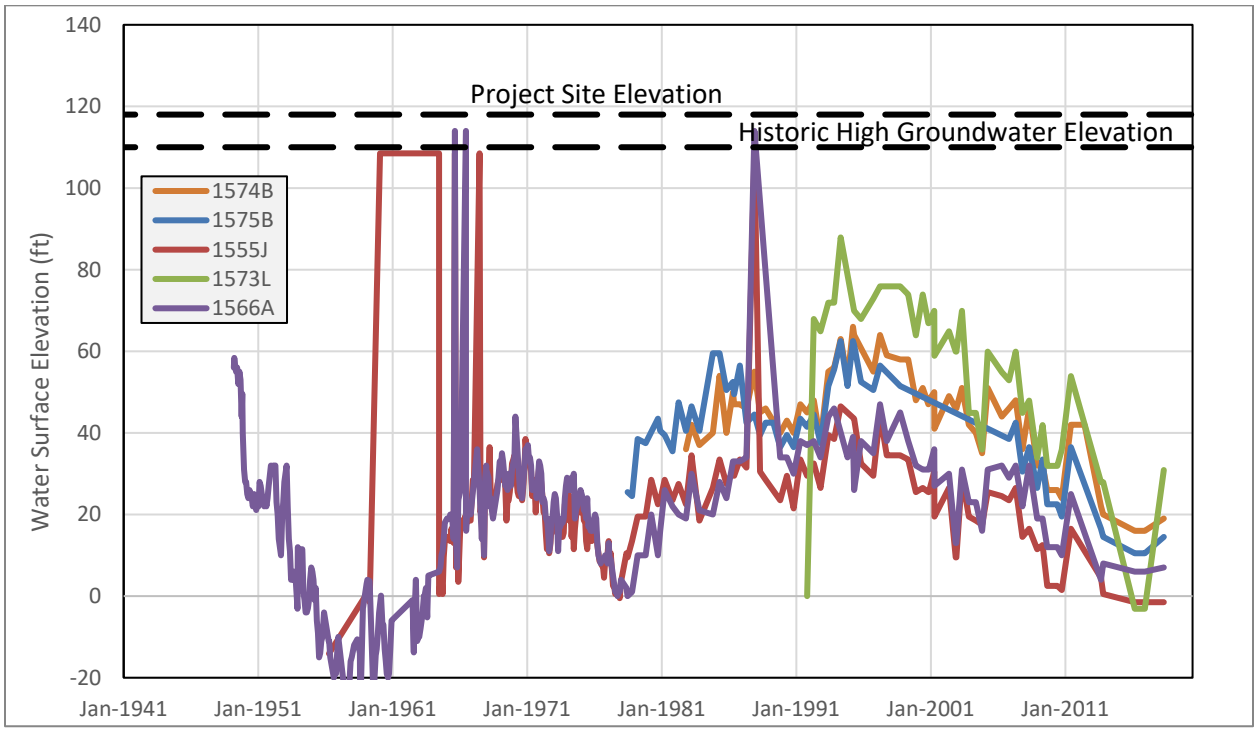


Diagram 1. Groundwater Elevations for Nearby LACDPW Groundwater Wells.

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 to be deeper than 50 feet from existing grade.

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 project sites are considered capable of generating ground shaking of engineering significance to the proposed construction. The project area is located in the general proximity of seismically active faults, as shown on Figure 5 – Regional Faults and Seismicity Map.

The closest seismically active faults to the site include the Puente Hills fault located approximately 0.9 miles north of the site, the Elsinore-Whittier fault located approximately 5.8 miles northeast of the site, and the Elysian Park fault (upper) located approximately 7.4 miles north of the site. The San Andreas fault, which is the most significant fault in the area in terms of generating a large earthquake, is located approximately 38 miles southwest of the site. Table 3 summarizes known seismically active faults within a distance of approximately 38 miles from the project site as identified by the USGS Quaternary Fault Database and in the 2008 National Seismic Hazard Maps (https://usgs.maps.arcgis.com and https://earthquake.usgs.gov).

Potential seismic sources of significance to the project are listed in Table 3 and include faults that break at the ground surface and faults that are not known to break the ground surface but are considered active. This latter group of faults is generally referred as 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. A brief description of the buried thrust faults that significantly contribute to the seismic hazard at the site follows.

- 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 3 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 7.0.

Table 3
Nearby Known Active Faults
Referenced Site Latitude and Longitude: 33.954076°, -118.137236°

Fault Name	Approximate Fault Distance to Site (miles)	Slip sense	Maximum Moment Magnitude (Mmax)
Puente Hills (LA)	0.9	thrust	7
Elsinore-Whittier	5.8	strike slip	7.9
Elysian Park (Upper)	7.4	reverse	6.7
Newport Inglewood	8.4	strike slip	7.5
Raymond	11.3	strike slip	6.8
Verdugo	12.3	reverse	6.9
Hollywood	12.6	strike slip	6.7
Santa Monica	13.7	strike slip	7.4
Compton	13.8	thrust	7.4
Palos Verdes	15.7	strike slip	7.7
Sierra Madre	15.8	reverse	7.3
San Jose	15.9	strike slip	6.7
Clamshell-Sawpit	17.3	reverse	6.7
San Joaquin Hills	21.3	thrust	7.1
Malibu Coast	22.9	strike slip	6.7
Chino	23.1	strike slip	6.8
Anacapa-Dume	24.3	thrust	7.2
Cucamonga	26.2	thrust	6.7
San Gabriel	26.4	strike slip	7.3
Northridge	28.2	thrust	6.9
Santa Susana	32.8	reverse	6.9
S. San Andreas	37.5	strike slip	8.02

[1] Information from https://maps.conservation.ca.gov/cgs/EQZApp/app/

- Upper Elysian Park thrust fault is an active blind thrust fault located in central Los Angeles, California. The fault lies underneath the Elysian Park Anticline, a fold that stretches from about El Monte and arcs northwest through East Los Angeles, Lincoln Heights, Elysian Park and Silver Lake. On the surface, the anticline manifests itself as the hills of City Terrace as well as those of Elysian Heights and Silver Lake near the Los Angeles River. Oskin et al. (2000) referred to a northeast-dipping reverse fault above Puente Hills blind thrust system as Elysian Park fault which is referred to in current literature as Upper Elysian Park fault (blind). The Elysian Park anticline is an active 20-km-long structure. From the style and rates of deformation of parasitic folds on the southern limb of the anticline, Oskin (2000) estimates a contraction rate of 0.6–1.1 mm/yr. This rate provides a basis for estimating a rate of contraction of the entire Elysian Park anticline, which in turn allows Oskin (2000) to estimate a 0.8 to 2.2 mm/yr time-averaged rate of slip on the underlying fault. At this rate of slip, rupture of the Elysian Park fault could produce a nominal Mw 6.2 to 6.7 earthquake every 500 to 1,300 years.

- The Compton blind-thrust fault is active and has generated at least six large-magnitude earthquakes (Mw 7.0-7.4) during the past 14,000 years (Leon et al., 2009). This large, concealed fault runs almost parallel to the Palos Verdes fault underlying the Los Angeles metropolitan area, and thus poses one of the largest sources of seismic risk to the area. Deformed Holocene strata record recent activity on the Compton thrust and are marked by discrete sequences that thicken repeatedly across a series of buried fold scarps. The intervals of growth have been interpreted as occurring after the formation of now-buried paleo-fold scarps that formed during uplift events on the underlying Compton thrust ramp. Minimum uplift in each of the scarp-forming events, which occurred at 0.7-1.75 ka [Event 1], 0.7-3.4 ka or 1.9-3.4 [Event 2], 5.6-7.2 ka [Event 3], 5.4-8.4 ka [Event 4], 10.3-12.5 ka [Event 5], and 10.3-13.7 ka [Event 6], ranged from ~0.6 to ~1.9 m, indicating minimum thrust displacements of ≥1.3 to 4.2 m. Such large displacements are consistent with the occurrence of large-magnitude earthquakes (Mw >7).

7.2. Historical Earthquakes

A large amount of seismic activity and associated events with their epicenters have been recorded surrounding the project site with magnitude of M_w 5 or greater. Table 4 summarizes historic earthquakes with a magnitude equal or greater than M_w 6.2 within a distance of approximately 60 miles from the project site obtained using the USGS Earthquake Catalog (https://earthquake.usgs.gov/earthquakes/search/). The most significant historic earthquakes near the project site are the 1857 Fort Tejon earthquake and the 1812 San Bernardino earthquake.

Table 4
Historic Earthquakes in Southern California
Referenced Site Latitude and Longitude: 33.954076°, -118.137236°

Earthquake Name	Year	Fault and Fault Type	Earthquake Magnitude*	Epicenter	
				Latitude	Longitude
Northridge	1994	Northridge Thrust (Blind Thrust) (a.k.a. Pico Thrust)	6.7 M_w	34.21°N	118.54°W
San Fernando	1971	San Fernando Fault (thrust)	6.5-6.7 M_w	34.42°N	118.37°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_w	34.00°N	117.24°W
Oxnard	1922	Simi Santa Rosa (reverse)	6.53 M_w	34.209°N	-119.143°W
Cajon Pass	1899	San Andreas Fault (right- lateral strike-slip)	6.36 M_w	34.3°N	-117.5°W
San Bernardino	1894	San Andreas Fault (right- lateral strike-slip)	6.2 M_w	34.3°N	-117.6°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
San Bernardino	1812	San Andreas Fault (right- lateral strike-slip)	7.5 M_w	34.37°N	-117.65°W

*M_w refers to Moment Magnitude scale
M_L refers to Local Magnitude scale

7.3. 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 Elsinore-Whittier Fault Zone approximately 5.8 miles northeast 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.

7.4. 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.

7.5. Liquefaction Potential and Dynamic Settlement Assessments

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.

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.

7.5.1. Groundwater Level for Liquefaction and Seismically Induced Settlement Assessment

Groundwater was not encountered during the field exploration. However, the historic high groundwater at the site was mapped by CDMG (South Gate Quadrangle) at a depth of about 8 feet. Based on CGS Special Publication SPT 117A (2008) and the Administrative Manual GS045 (2014) the historic groundwater level should be used in the analysis unless a shallower level (higher elevation than the historical high) is determined to be appropriate. Therefore, a groundwater depth of 8 feet was assumed for the liquefaction and seismically induced settlement assessments.

7.5.2. Seismic Demand for Liquefaction and Dynamic Settlement Assessments

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 33.954076°, -118.137236°, 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 0.822g. From the USGS Seismic Hazard Interactive Deaggregation website (https://earthquake.usgs.gov/hazards/interactive/) this ground motion corresponds to a predominant earthquake with a magnitude of approximately M_w 7.3. The largest contributors to the seismic hazard at the site are the Puente Hills Blind Thrust fault and the Compton Blind Thrust faults. These ground motion parameters were used in the liquefaction and seismically induced settlement assessments. A summary of the seismic demand parameters is presented in Appendix D.

7.5.3. Evaluation of Liquefaction Potential and Seismic 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-3 through B-5 and on laboratory test data.

The analyses utilized SPT blowcounts and considered the energy ratio correction factor C_E of 1.5, which was based 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) and included in the liquefaction potential and settlement analysis;

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 included in additional evaluation 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. The analyses based on SPT data from the soil borings indicates that the on-site sandy silts, silty sands and poorly-graded sands at depth intervals of 8 to 15 feet, 22.5 to 30 feet, and 30 to 50 feet are susceptible to liquefaction.

Analyses of the sensitivity of the saturated fine-grained soils to ascertain the potential for loss of strength were not performed because the Plasticity Index fell below the threshold level of 18 indicating that the respective fine-grained soils are not susceptible to cyclic softening.

7.5.4. Seismically Induced Settlement Assessment

Seismically induced settlement can occur in saturated materials due to liquefaction or in unsaturated soils due to densification of the soil matrix. The liquefaction induced settlement of the saturated soils was evaluated using SPT data using procedures outlined by Yoshimine et al (2006). The calculated settlements 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 1.8 and 5.3 inches at the ground surface. The differential seismic settlement is calculated to be about 2.6 inches over a horizontal distance of 30 feet. For structures that are embedded or buried at least 20 feet the total seismic settlement is calculated to be 1.2 to 3.4 inches with an upper estimate of differential settlement of about 1.7 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

Exploration 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]
B-3	8	10-15 17.5-20 22.5-25 35-40 45-50	0.5 0.2 0.7 0.2 0.2	5.3	none	5.3
B-4		8-10 12.5-15 25-30 55-56.5	0.4 0.8 0.6 0.7	2.0	none	2.0
B-5		8-10 30-35	0.5 0.3	1.8	- none	1.8

7.6. 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 at or in the immediate vicinity of the site. Therefore, an earthquake-induced landslide is not considered to be hazard at the site.

7.7. 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.

7.8. 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. Damage caused by subsidence can be visible cracks, fissures, or surface depression.

The site is not mapped within an area of land subsidence in California (https://ca.water.usgs.gov/land_subsidence/california-subsidence-areas.html) due to groundwater pumping. Therefore, subsidence is not considered a hazard at the site.

7.9. 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 sandy silts and medium dense silty sands to sands. Such deposits are generally not susceptible to collapse. Based on the consolidation testing of 3 soil samples it was observed that all the samples exhibited a collapse during wetting of less than 1 percent. Therefore, collapsible soils are not considered a hazard to the site.

7.10. 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 Indexes of two selected surficial samples taken within the upper 7.5 feet were 1 and 3, 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.

7.11. 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/).

7.12. 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).

7.13. 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/).

7.14. 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 118 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 over 5 inches are calculated which would likely cause shifting, misalignment, or cracking distress of the structures and could require repairs or result in a loss of function. Consequently, recommendations are herein provided for augmentation of the foundation subgrade with a geogrid-reinforced raft for all structures except for conveyance pipelines and manholes in order to mitigate against differential settlements. However, the recommended geogrid-reinforced raft may not be deemed necessary for unoccupied infrastructure facilities where the calculated liquefaction displacements can either be tolerated or will not immediately impact their functionality. Tetra Tech can provide additional recommendations under separate task and scope, for alternative subgrade preparation recommendations and allowable bearing pressures, in case the recommended geogrid-reinforced raft is not deemed necessary.

Additional specific geotechnical design considerations include:

- Excavation and shoring design;
- Evaluation of lateral earth pressures for underground structures.

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

- All major structures, except for stormwater conveyance pipelines and manholes, should be founded on a 3-foot-thick geogrid-reinforced select granular fill raft (described in the following section) placed on firm and unyielding approved native soils subgrade. No overexcavation is required, however, note that different subgrade preparation, as described in the following section of this report, is recommended for foundations where infiltration through the subgrade

is needed. To the extent practicable, the subgrade preparation and the geogrid-reinforced raft should extend outside the structure footprint for a horizontal distance of at least 3 feet.

- For other minor subterranean structures like manholes, overexcavation should be performed to a depth of at least 2 feet below the bottom of the structure. To the extent practicable, the overexcavation should extend outside the structure footprint for a horizontal distance of at least 3 feet. The overexcavation subgrade should be scarified to a depth of 4 inches. Fine-grained soils should be moisture conditioned to a minimum of 125 percent of optimum moisture content and compacted to at least 90 percent of the maximum dry density as evaluated by the latest version of 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 of the maximum dry density.

- In areas where existing pavements will be replaced with new pavements the removals should extend to the depth of the new pavement subgrade elevation.

- Flatwork and new pavement areas 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.

8.4. Geogrid-Reinforced Raft

After completion of the removals to the bottom of the geogrid-reinforced select fill raft, 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.

In the areas where infiltration will be taking place the subgrade should be left in its natural state as long as the dry density of the native subgrade is at least 83 percent relative compaction per ASTM D1557. Otherwise, the subgrade should be scarified to a depth of 4 inches and lightly recompacted wet of optimum moisture content to between 83 and 87 percent of relative compaction per ASTM D1557. No heavy equipment/vehicles should be allowed to travel in the area where infiltration will take place, so that the natural infiltration capacity of the soils is preserved.

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.
Larger areas where the soils are excessively 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.

The geogrid-reinforced select granular 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.

The geogrid-reinforced select granular fill raft should be constructed in the following sequence (refer to the "Select Granular Fill in Infiltration Areas", "Select Granular Fill in Other Areas", and "Geogrids" sections of this report for more details):

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.

8.5. Grading Materials

8.5.1. 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.

8.5.2. Select Granular Fill in Infiltration Areas

The material for the geogrid-reinforced select granular fill raft in the infiltration areas should generally be compliant with 1-inch or ¾-inch crushed rock specification per Greenbook Section 200-1.2, vibrated in place to maximum achievable density as judged by the Geotechnical Engineer.

In the infiltration areas, the crushed rock needs to be separated from the underlying subgrade native soils by a geotextile, e.g., Mirafi 140NL or equivalent. Furthermore, to control clogging of the gravel raft, it is recommended that another layer of geotextile be placed immediately under the infiltration units on top of the gravel raft. This configuration will facilitate collection and removal of any sediments accumulated over time.

8.5.3. Select Granular Fill in Other Areas

The material for the geogrid-reinforced select granular fill raft in the infiltration areas should generally be compliant with fine crushed miscellaneous base (CMB) specification per Greenbook Section 200-2.4, moisture-conditioned wet of optimum moisture content and compacted to at least 93 percent relative compaction per ASTM D1557.

8.5.4. Geogrids

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

8.5.5. 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.

8.6. Temporary Excavations

It is anticipated that all excavations to reach subgrade level for the underground stormwater storage and infiltration 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 Shored Excavations" 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.

8.7. Temporary Shored Excavations

Excavation depths for the construction of the proposed project are anticipated to be between 10 and 25 feet. If sloped excavations are not feasible because of lack of space, then shoring will be needed. A combined system of sloped excavations along certain sides of the excavation and shoring along the other sides of the excavations can also be considered.

Cantilevered shoring systems are typically suitable and practicable for retained heights less than about 15 feet. Alternatively, a restrained shoring system 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.

8.7.1. Shoring Design

Either sheet pile or soldier pile and lagging systems may be utilized for the proposed construction. Tiebacks or internal bracing are usually required for excavation depths greater than about 15 feet. Table 7 below summarizes the governing geotechnical design parameters and loading diagrams for a shoring system for both cantilever and internal bracing/tieback 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.

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 0.29 and 0.38, 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. If a different surcharge configuration is present, this office should be contacted to provide specific recommendations.

Continuous timber lagging or steel plates may be used between the soldier piles. For the design of the lagging the lateral earth pressures may be reduced by a factor of 0.6 to account for soil arching. The lagging design earth pressure diagram will be the sum of the lateral pressure due to soil loading as defined in Table 7 reduced for soil arching by a factor of 0.6 (not need to exceed 400 psf) and the lateral pressure due to surcharge loads as defined above. The lagging should be installed behind the front flange (closest to the excavation) and not behind the back flange (closest to the retained soil). Lagging should be removed at the time of backfilling.

The required penetration depth of the soldier or sheet pile wall below the excavation bottom may be calculated based on the allowable passive soil resistance 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. The passive resistance for soldier piles in Table 7 was increased to account for soil arching and is applicable for soldier piles that are spaced no closer than the arching capability times the pile width/diameter. For closer spacing the passive resistance would need to be reduced.

Table 7
Geotechnical Design Parameters
Cantilevered and Restrained Temporary Shoring

Excavation bottom depth (including overexcavation/geogrid-reinforced raft depth)	Up to 25 feet	
Subsurface materials	**Alluvial Soils** Mostly firm to stiff silts and medium dense silty sands and poorly-graded sands to a depth of 40 feet	
Soil unit weight, γ	125 pcf	
Design friction angle, ϕ	33°	
Design cohesion, c	0 psf	
K_a … active pressure coefficient	0.29	
LOADING DIAGRAM ON SHORING		
SHORING SYSTEM	**Cantilevered shoring** (excavation depth less than about 12 to 15 feet)	**For restrained shoring systems** Soldier pile or sheet pile wall with tieback(s) or with internal bracing
Equivalent fluid density, EFD	37 pcf	n/a
Loading Diagram	Triangular distribution	Trapezoidal load distribution (see Diagram 2 below)
ALLOWABLE PASSIVE RESISTANCE BELOW EXCAVATION BOTTOM		
Design friction angle, ϕ	33°	
K_p … passive pressure coefficient	3.4	
Arching capability *	2.6 (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: 280 Soldier pile system (includes arching): 735	

* Per Caltrans Trenching and Shoring Manual (2011)
** Valid without reduction for soldier pile spacing > 2.6 times the effective pile width

The anulus of the soldier pile beam borehole below the excavation bottom should be backfilled with concrete. 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. 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. Pea gravel may be used to backfill the hole from the excavation bottom to the top grade.

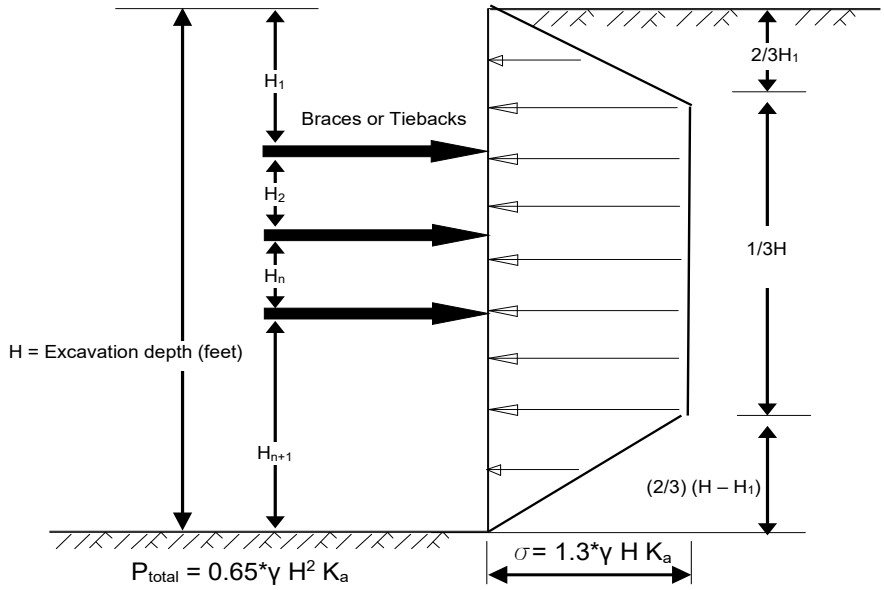


Diagram 2. Lateral pressure loading diagram for restrained shoring

8.7.2. Tieback Design

Tiebacks may be utilized to assist with the lateral restraint of the shoring and to reduce soil movement behind the shoring wall. Straight shaft, pressure-grouted tiebacks may be initially designed for an allowable bond stress of 2,000 psf. The allowable bond stress includes a Factor of Safety of 2 and assumes that the center of the anchor bond zone should be a minimum of 15 feet below the ground surface. The allowable bond stress may need to be adjusted for shallower tieback depth.

The allowable bond stress should be verified by pre-production testing at the beginning of the construction. To evaluate the global stability of the tieback system the soil strength parameters provided in Table 7 can be used.

The tieback bonded zone must not encroach inside the active zone. Only the resistance developed in the bonded zoned beyond the active wedge may be relied upon in resisting lateral loads. It may be assumed that the active wedge is defined by a plane drawn at 29 degrees from the vertical through the bottom of the excavation. The unbonded length of the anchor should extend either a minimum of distance of H/5, where H is the height of the wall, or 5 feet behind the surface defined by the active wedge, whichever is less. If the anchors are spaced at least 6 feet on centers, no reduction due to group action in the capacity of the anchors needs to be considered.

It should be specified that all or at least the upper row of anchors be de-tensioned after completion of the underground stormwater storage facility construction.

8.7.2.1. Tieback Testing

The bond stress and capacities of anchors should be verified by testing during construction. The tieback proof and performance testing program should be in compliance with the latest (4th edition) Post-Tensioning Institute (PTI) guidelines "Recommendations for Prestressed Rock and Soil Anchors". This office should review and approve the actual testing program and observe and interpret the execution of the testing program.

8.7.2.2. Tieback Installation

The anchors should be installed at angles of 15 to 30 degrees below the horizontal. Caving of the anchor holes at certain locations should be anticipated and provisions should be made available to minimize such caving. The anchors should be filled with grout placed by pumping from the tip out, and the grout should extend from the tip of the anchor to the active wedge. To minimize the potential for caving, we suggest that the portion of the anchor shaft within the active wedge be backfilled with sand before testing the anchor. This portion of the shaft should be filled tightly and flush with the face of the excavation. The sand backfill may contain a small amount of cement to allow the sand to be placed by pumping. For post-grouted anchors the anchor may be filled with grout to the face of the shoring provided the tieback strands are enclosed in plastic sheathing.

8.7.3. 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 ½-inch when there are nearby structures present, and 1-inch otherwise. 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 at the design stage, a greater lateral earth pressure could be used.

8.7.4. Internal Bracing

As an alternative to the tiebacks, internal bracing can be utilized 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 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.

8.7.5. 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 or tiebacks. 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 or tiebacks, the excavation then proceeds, and each level of internal bracing or tiebacks should be installed as soon as practicable. The shoring designer should analyze each stage of internal bracing or tieback 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.

8.7.6. 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.

8.8. 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 0.5 sec, 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. Alternatively, 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 *Cs* determined by Equation (12.8-2) is used for values of $T <= 1.5\ T_s$ and taken as equal to 1.5 times the value computed in accordance

with either Equation (12.8-3) for $T_L >= T > 1.5\ T_s$ 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: 33.954076°, -118.137236°

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	1.742*
	1 Second Period, S_1	0.624*
Coefficients for the Maximum Considered Earthquake, MCE_R (Site Modified)	Short Period (0.2 seconds), S_{MS}	1.742*
	1 Second Period, S_{M1}	1.061**
Coefficients for the Design Earthquake	Short Period (0.2 seconds), S_{DS}	1.161*
	1 Second Period, S_{D1}	0.707**
Design PGA (risk-targeted maximum rotated direction)		0.464g*
Site Modified Peak Ground Acceleration PGAM (geometric mean)		0.822g*

* 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.
** See requirements for site-specific ground motions in Section 11.4.8 of ASCE 7-16.

8.9. Foundations

Underground stormwater facilities and other buried structures may be supported on footing pads or mat foundations, supported on a geogrid-reinforced select granular 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 2019 California Building Code.

8.9.1. Foundations for at-grade Structures

Table 9 presents the geotechnical design parameters for the stormwater harvesting unit structure which has been assumed to be at-grade, and for any other at-grade structure.

Table 9
Geotechnical Design Parameters

Continuous Strip Footings	
Dimensions	• 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 (with geogrid-reinforced raft)	• 3,000 psf
Spread Footings	
Dimensions	• 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 (with geogrid-reinforced raft)	• 5,000 psf
All Foundations	
Geogrid-Reinforced Raft	• Structures founded on a 3-foot-thick geogrid-reinforced select fill or gravel raft
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 Static 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)	• 0.40 for mass concrete on geogrid-reinforced raft • 0.30 for formed concrete on geogrid-reinforced raft
Allowable Lateral Passive Resistance (incorporates Factor of Safety of 2)	• 220 pcf (EFD, equivalent fluid density, for compacted fill with ϕ=34°) • 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

8.9.2. Foundations for Underground Structures/Vaults More than 10 feet Deep

Since all the underground structures will be at least 10 feet below ground surface and given the wide range in foundation depths, Table 10 presents recommendations for structures founded on a geogrid-reinforced mat foundation, and two sets of recommendations are provided: one set for structures founded at a minimum depth of 10 feet but not to exceed 17 feet e.g., pretreatment unit, actuated valve, etc., and another set for structures founded at depths larger than 17 feet e.g., pump station, underground storage and infiltration units, etc.

Table 10
Geotechnical Design Parameters

Mat like Foundations			
Dimensions	• Up to 5 feet x 5 feet	• Up to 10 feet x 10 feet	• Up to 15 feet x 15 feet
Allowable Bearing Pressure (with geogrid-reinforced raft) (Structures at least 10 feet deep)	• 5,000 psf	• 3,100 psf	• 2,600 psf
Allowable Bearing Pressure (with geogrid-reinforced raft) (Structures at least 17 feet deep)	• 5,500 psf	• 3,900 psf	• 3,400 psf
Modulus of Subgrade Reaction	• For design of large footings which behave like mat foundations supported on a geogrid-reinforced select fill or gravel raft a reference modulus of subgrade reaction k_1 of 115 pci may be used (considers submerged conditions). 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 Raft	• Structures founded on a 3-foot-thick geogrid-reinforced select fill or gravel raft		
Allowable Bearing Pressure for Transient Loads	• The allowable bearing value may be increased by one-third for transient live loads from wind or seismicity.		
Estimated Static Settlement	• 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 ϕ=34°) • The passive resistance derived from the upper 12 inches should be neglected.		
Allowable Coefficient of Friction (incorporates Factor of Safety of 1.5)	• 0.40 for mass concrete on geogrid-reinforced raft • 0.30 for formed concrete on geogrid-reinforced raft • 0.30 for concrete 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		

8.9.3. 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. If any piping needs to be placed within this 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.

8.10. 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.

8.11. Lateral Earth Pressures on Permanent Subterranean Walls

The lateral pressures acting on buried walls should be calculated based on the recommendations provided in Table 11. Lateral earth pressures presented in this table are for level backfill for an active, at rest, and passive lateral pressure condition. 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, Q, (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.

Based on the 2019 CBC the design of retaining walls higher than 6 feet, as measured from the bottom of the footing, requires the inclusion of not only static lateral pressures but also inclusion of an additional seismically induced lateral earth pressure increment. 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), the PGA from the Design Response Spectrum at the site is approximately 0.464g as indicated in Table 8.

The on-site soils are considered susceptible to liquefaction between the depths of 8 and 15 feet and between the depths of 17.5 and 20 feet. Therefore, the designer should evaluate the stability of the structure also for the temporary liquefaction lateral pressure at these depth intervals. When

considering the liquefaction pressures, the seismic pressure increments do not need to be considered.

Table 11
Lateral Pressures on Subterranean Walls

Active Pressure for Yielding Walls		
Active static pressure (psf)	above groundwater, i.e., z < 8 ft	$36z + 0.29Q$
	below groundwater, i.e, z ≥ 8 ft	$36z_w + 81(z - z_w) + 0.29Q$
Active liquefaction pressure (psf)	below groundwater, i.e, z ≥ 8 ft	$125z_w + 125(z - z_w) + Q$ $- 2*0.09[125z_w + 63(z - z_w)]$
Active seismic pressure increment (psf)		$24z$
At rest Pressure for Non-yielding Walls		
At-rest static pressure (psf)	above groundwater, i.e., z < 8 ft	$57z + 0.46Q$
	below groundwater, i.e, z ≥ 8 ft	$57z_w + 91(z - z_w) + 0.46Q$
At-rest liquefaction pressure (psf)	below groundwater, i.e, z ≥ 8 ft	$125z_w + 125\ (z - z_w) + Q$
At-rest seismic pressure increment (psf)		$40z$
Passive Pressure		
Allowable passive static pressure (psf) Includes Factor of Safety of 2	above groundwater, i.e., z < 8 ft	$212z_1$
	below groundwater, i.e, z ≥ 8 ft	$212z_w + 106(z_1 - z_w)$
Ultimate total passive seismic pressure (psf) No Factor of Safety	above groundwater, i.e., z < 8 ft	$300z_1$
	below groundwater, i.e, z ≥ 8 ft	$300z_w + 150(z_1 - z_w)$
Ultimate passive liquefaction pressure (psf) No Factor of Safety	below groundwater, i.e, z ≥ 8 ft	$125z_w + 63(z_1 - z_w)$ $+ 2*0.09[125z_w + 63(z - z_w)]$

Notes:
- Lateral Pressures due to seismic loading are based on a PGA=0.464g for a design response spectrum taken as 2/3 MCE_R 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 be 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 $\phi = 33^o$, c = 0 psf, $\gamma_t = 125$ pcf (above groundwater), $\gamma_t = 125$ pcf (below groundwater)
- The 2019 CBC requires that basement walls be designed for at rest earth pressures for static conditions. It is our interpretation of the Code that this requirement also applies to other subterranean walls that have some type of internal lateral structural restraint, such as vaults, concrete boxes, etc. or are considered rigid enough for the walls to deflect only minimally.
- Liquefaction residual strength (S_r) was assumed per Idriss and Boulanger (2015) for $N_{1,60,cs} = 10$ to be equal to 0.09 the effective normal stress.

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

The seismic active and at-rest pressure increments were calculated using the method recommended by Agusti and Sitar (2013) as required by the County of Los Angeles Administrative Manual S004 dated January 6, 2020 regarding Seismic Earth Pressures on Retaining Walls. The combined ultimate total static and seismic passive pressure was computed as recommended by Taciroglu (2013). The lateral earth pressures for the soil intervals where liquefaction is considered were computed using Rankine earth pressures for the active and passive cases for a cohesive soil with an angle of friction $\phi = 0$ and an undrained shear strength computed based on a normalized blowcount $N_{1,60,cs}$ of 10 and using the methodology proposed by Idriss and Boulanger (2015). For the at-rest condition, for the soil intervals where liquefaction is considered, the lateral earth pressures exerted by the liquefied soils were computed assuming that the horizontal stress would be equal to the vertical stress.

8.12. Liquefaction-Induced Downdrag on Subterranean Structures

Structures founded below a depth of 8 feet will be subject to downdrag forces on the perimeter walls due to liquefaction potential identified at depths from 8 to15 feet and from 17.5 to 20 feet (Table 5).

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 calculated using Table 11 multiplied by the ultimate coefficient of friction of 0.5 and 0.1 for the non-liquefied and liquefied material, respectively. These forces should be added to the structural loads and verified against the allowable bearing pressures presented in Table 10.

8.13. Concrete Slabs-On-Grade

The recommendations provided in the "Site Preparation" and "Geogrid-Reinforced Raft" sections 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.

8.13.1. 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 1-foot x 1-foot plate 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.

8.13.2. 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.

8.13.3. 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).

8.14. 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 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.

8.14.1. 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.

8.14.2. 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.

8.15. Utility Trenches

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

8.15.1. 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.

8.15.2. 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.

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.

8.16. Infiltration Suitability

Based on the field testing the on-site soils are suitable for the infiltration at the proposed depth of 20 to 25 feet. A design infiltration rate of 0.5 inches/hour is recommended for this project as discussed in the "Field Infiltration Testing" section of this report.

The historic high groundwater has been mapped at a depth of about 8 feet, however, the groundwater level in the project area has been at least 40 feet deep within the last 50 years with only a couple of instances during that period where the levels were shallower as shown on Diagram 1. Groundwater was not encountered during our field exploration to the maximum explored depth of 56.5 feet. Therefore, for the invert of the proposed infiltration facilities at a depth of 20 to 25 feet, the 10-foot vertical separation required by the GS200.1 (2021) from the groundwater is achieved based on the recent historical groundwater depth data.

8.17. Pavement Design for Pipeline Trenches

Pavement repairs, e.g., over trenches, should follow the City of Downey Standard Plans for "Trench Paving Section Pipe Details" (https://www.downeyca.org/our-city/departments/public-works/standard-drawings). The plans require that the new pavement have a thickness equal to that of the existing pavement plus 1 inch but no less than 8 inches. The plans also show the required lateral extent of the pavement repairs. In the case that pavement replacement is preferred, such recommendations are provided below.

8.17.1. Pavement Subgrade Preparation

The subgrade preparation and fill placement in all paved areas should follow the recommendations provided in the "Site Preparation" section of this report. Paved areas should be properly sloped, and surface drainage established to reduce water infiltration into the pavement subgrade. Curbing located adjacent to paved areas should be founded in the soil subgrade in order to provide a cutoff to reduce water infiltration into the base course.

8.17.2. Asphalt Concrete Pavement Design

Flexible pavement sections have been designed in general accordance with the Caltrans Highway Design Manual method for flexible pavement design using the 20-year design life period. Laboratory R-values obtained from 2 soil samples from the expected subgrade materials, were 59 and 63, however, the maximum allowable subgrade R-value of 40 was used for the pavement design. The recommended pavement sections for several assumed Traffic Indices are presented in Table 12. The appropriate design Traffic Index should be selected based on the project requirements.

Table 12
Flexible Pavement Sections

Location	Traffic Index	Composite Section		Full Depth Asphalt Concrete Alternative (inches)
		Asphalt Concrete (inches)	Aggregate Base (inches)	
Parking / automobiles	5.5	3.5	5.0	6.0
Residential	6.5	3.5	6.5	7.0
Local Commercial and Minor Collectors	7.5	4.5	7.5	8.5
Local Industrial and Major Collector	8.5	5.0	9	9.5

Asphalt concrete and Aggregate Base (AB) should conform to the Specifications for Public Works Construction (Greenbook) Sections 203-6 and 200-2, respectively. The AB course should be compacted to 95 percent of relative compaction per ASTM D1557.

8.18. Corrosion Potential Evaluation

The corrosion potential of the on-site soils to buried steel and concrete was evaluated based on laboratory testing of 3 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 class of buried concrete to sulfate corresponds to S1 per

ACI 318-14, Table 19.3.1.1. Consequently, per Table 19.3.2.2, cement Type II with a minimum 28-day compressive strength of 4,000 psi should 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	R-1	Silty Sand	2.5	8.4	504	512/0.0512	1,637/0.1637
B-4	SK-1	Silty Sand	0-5	8.5	2,326	69/0.0069	95/0.0095
B-6	SK-1	Silty Sand	0-5	9.2	4,026	41/0.0041	60/0.0060

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 "corrosive" to "mildly 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.

8.19. 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" sections 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.

- 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, shoring installation, 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. Pavement Construction Observations

The preparation of the pavement subgrade and the placement of 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 pavement thickness is 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. 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 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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FURMAN PARK STORMWATER CAPTURE - DOWNEY, CA

SITE LOCATION MAP

JOB NO.	CWT 22-06E
DATE	JULY 2022
DRAWN BY	TAC
CHECKED BY	FC



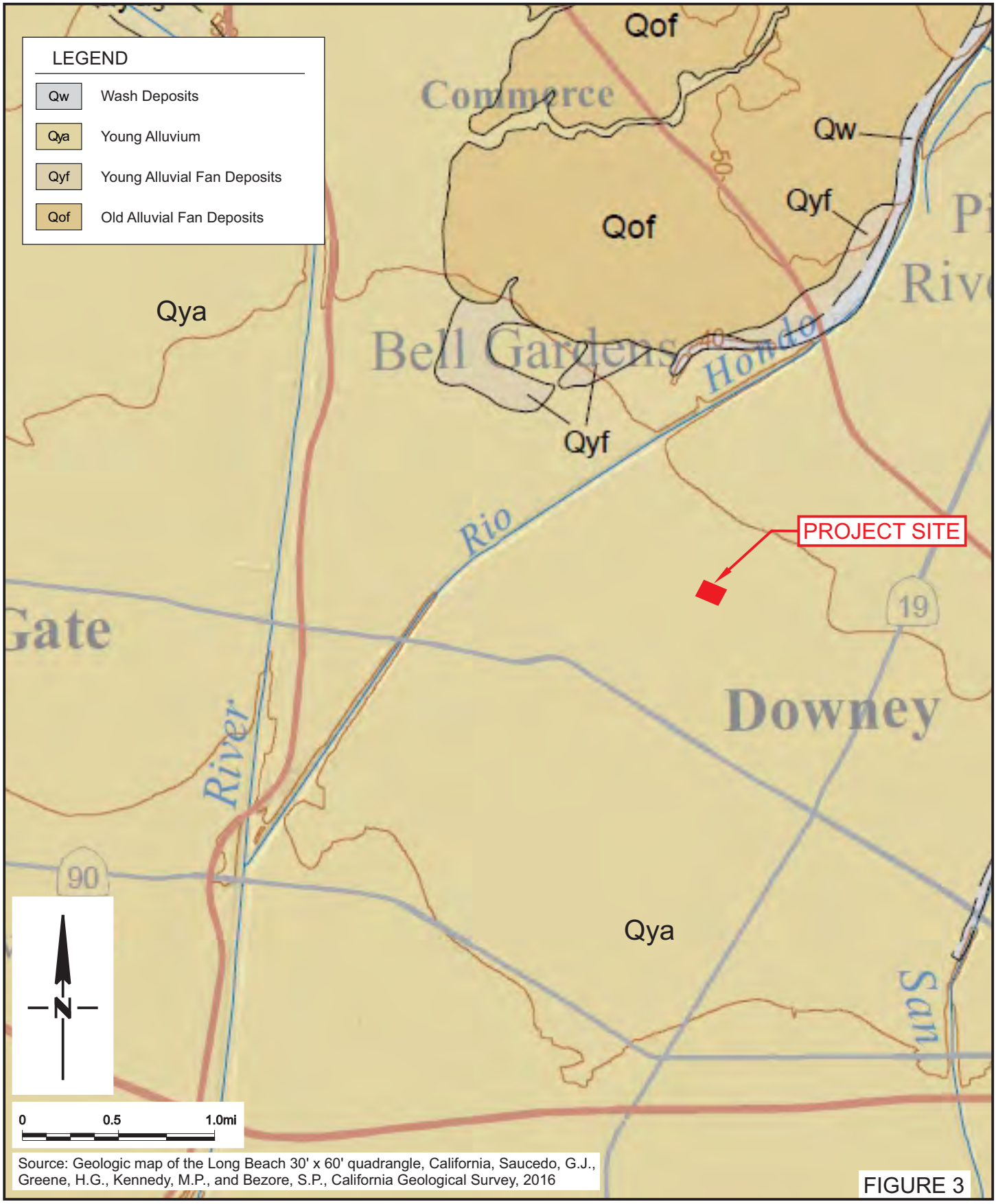
FIGURE 2

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FURMAN PARK STORMWATER CAPTURE - DOWNEY, CA

SITE LAYOUT AND BORING LOCATION MAP

JOB NO.	CWT 22-06E
DATE	JULY 2022
DRAWN BY	TAC
CHECKED BY	FC



Source: Geologic map of the Long Beach 30' x 60' quadrangle, California, Saucedo, G.J., Greene, H.G., Kennedy, M.P., and Bezore, S.P., California Geological Survey, 2016

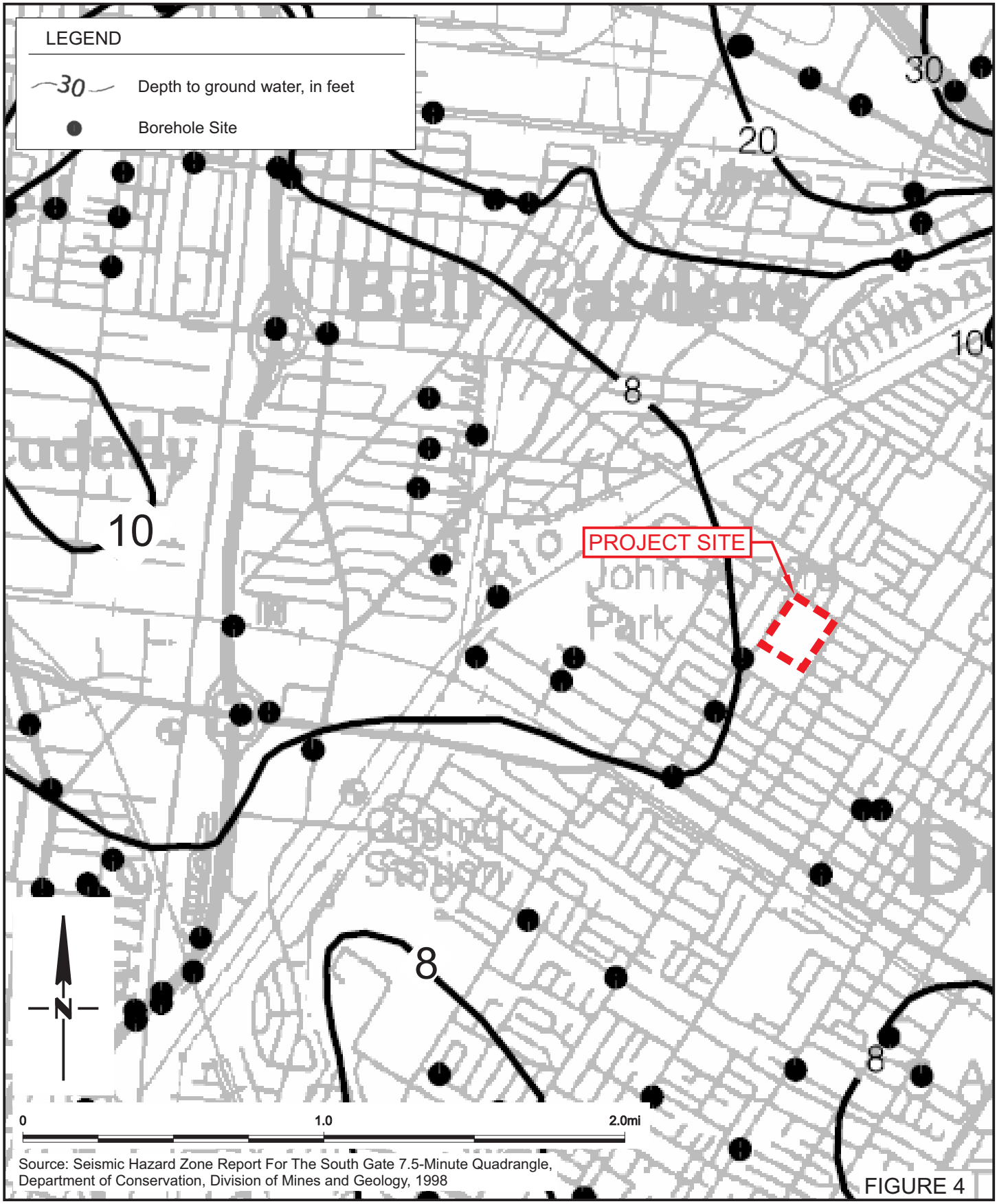
FIGURE 3

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FURMAN PARK STORMWATER CAPTURE - DOWNEY, CA

GEOLOGIC MAP

JOB NO.	CWT 22-06E
DATE	JULY 2022
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CHECKED BY	FC



Source: Seismic Hazard Zone Report For The South Gate 7.5-Minute Quadrangle, Department of Conservation, Division of Mines and Geology, 1998

FIGURE 4

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FURMAN PARK STORMWATER CAPTURE - DOWNEY, CA

HISTORIC HIGH GROUNDWATER MAP

JOB NO.	CWT 22-06E
DATE	JULY 2022
DRAWN BY	TAC
CHECKED BY	FC

EARTHQUAKE EPICENTERS

M 5.0 to 5.9

M 6.0 to 6.9

M 7.0 to 7.9

HISTORIC EARTHQUAKE EVENTS

1)	1812 Wrightwood	M7
2)	1827 Ventura County	M6
3)	1855 Los Angeles	M6?
4)	1858 San Bernardino	M6?
5)	1880 San Bernardino	M6
6)	1889 Monrovia	M5.5
7)	1893 San Fernando Valley	M5.9
8)	1894 n/o Pomona	M6
9)	1899 Cajon Pass	M6.5
10)	1899 San Jacinto	M6.4
11)	1907 San Bernardino	M6
12)	1910 Elsinore	M6
13)	1918 San Jacinto-Hemet	M6.8
14)	1920 Inglewood	M4.9
15)	1923 Loma Linda	M6
16)	1930 Santa Monica Bay	M5.2
17)	1933 Long Beach	M6.4
18)	1941 Gardena	M4.8
19)	1971 San Fernando	M6.7
20)	1973 Pt. Mugu	M5.9
21)	1979 Malibu	M5.2
22)	1981 Pt. Mugu	M6.0
23)	1987 Whittier-Narrows	M5.9
24)	1988 Pasadena	M5.0
25)	1989 Malibu	M5.0
26)	1990 Pomona	M5.3
27)	1991 Sierra Madre	M5.8
28)	1994 Northridge	M6.7
29)	2005 Chino Hills	M5.2

Map Reference: Excerpt of Jennings, C.W., and Bryant, W.A., 2010, Fault activity map of California; California Geologic Survey, Date Map No. 6, map scale ~ 1:750,00

Used in permission from California Geologic Survey.

Earthquake epicenters from SP 116

PROJECT SITE

EXPLANATION

Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION: ON LAND	DESCRIPTION: OFFSHORE	DESCRIPTION: MAP SYMBOLS
Quaternary	Late Quaternary	Historic				Displacement during historic time (e.g. San Andreas fault 1096). Includes areeas of known fault creep.		Triangle - termination portion data. Square - fault creep slippage. Hachure - linear extent of fault creep. (other symbols - see below)
			200					
		Holocene				Displacement during Holocene time.	Faults offsets sea floor sediments or strata of Holocene age.	Approximate located trace
			11,700					Location uncertain
		Pleistocene				Fault showing evidence of displacement during late Quaternary time.	Faults cuts strata of LatePleistocene age.	Bar and ball on dowthrown side
	Early Quaternary		700,000			Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,00 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Faults cuts strata of Quaternary age.	Arrows along fault indicate relative or apparent direction of lateral movement.
			1,600,000					Direction of fault dip
Pre-Quaternary						Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Faults cuts strata of Pliocene or older age.	Low angle fault (barbs on upper plate)
								Numbers refer to annotations listed in explanatory text of map.
								Subject Site Location

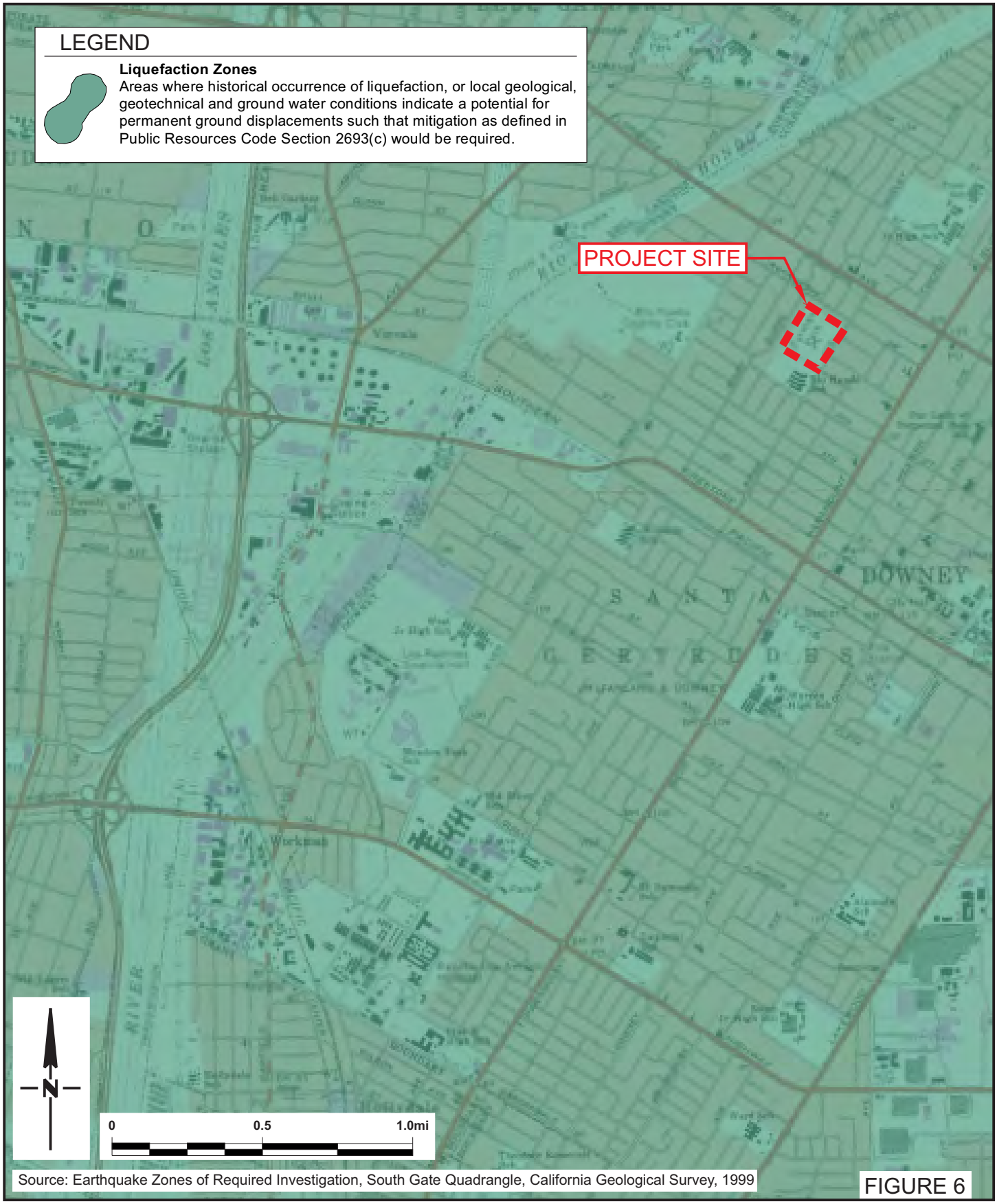
FIGURE 5

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FURMAN PARK STORMWATER CAPTURE - DOWNEY, CA

REGIONAL FAULTS AND SEISMICITY MAP

JOB NO.	CWT 22-06E
DATE	JULY 2022
DRAWN BY	TAC
CHECKED BY	FC



Source: Earthquake Zones of Required Investigation, South Gate Quadrangle, California Geological Survey, 1999

FIGURE 6

TETRA TECH
21700 Copley Drive, Suite 200, Diamond Bar, CA 91765
TEL 909.860.7777 www.tetratech.com

FURMAN PARK STORMWATER CAPTURE - DOWNEY, CA

SEISMIC HAZARD ZONES MAP

JOB NO.	CWT 22-06E
DATE	JULY 2022
DRAWN BY	TAC
CHECKED BY	FC

Appendix A

Logs of Exploratory Borings

Tetra Tech
21700 Copley Drive
Diamond Bar, CA 91765
909-860-7777

BORING B-1

PROJECT NAME Furman Park Stormwater Capture
CLIENT City of Downey
GROUND ELEVATION 116 ft
PROJECT NUMBER CWT 22-06E
PROJECT LOCATION Downey, CA
GROUNDWATER DEPTH Not encountered
DATES DRILLED 6/2/2022
DRILLING CONTRACTOR 2R Drilling Company
BOREHOLE DEPTH 21.5 ft
LOGGED/CHECKED BY SMS/FC
DRILLING METHOD Hollow Stem Auger, CME 75
BOREHOLE DIAMETER 8 in
COORDINATES 33.952568°, -118.140559°
HAMMER DATA Auto 140 lbs, 30" drop
BACKFILL Grout and cuttings
LOCATION DESCRIPTION Residential street (Wiley Burke Avenue)

Standard Penetration Test (SPT) | No Recovery | Shelby Tube
California-Type Ring Sample | No Recovery | Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOWCOUNT blows/6" (bpf)	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0	SK-1		SM	3" Asphalt [FILL] Artificial Fill (af) Silty SAND, brown (10YR 4/3), moist, fine grained			R-Value = 63	115
5	SPT-2	2-2-3 (5)	ML	[NATIVE] Alluvium (Qal) Sandy SILT, firm, olive (5Y 5/4), moist, fine grained sand, micaceous				110
	R-3	4-5-10 (15)	SM	Silty SAND, medium dense, light olive gray (5Y 6/2), moist, fine grained, micaceous	96.0	8.3	DS	
10	SPT-4	4-5-6 (11)		...(10.0 ft) as above, light brownish gray (2.5Y 6/2)				105
	R-5	7-14-21 (35)	SP	Poorly graded SAND, medium dense, light brownish gray (2.5Y 6/2), moist, fine grained	97.7	3.0		
15	SPT-6	6-10-12 (22)						100
	R-7	4-10-14 (24)	SM	Silty SAND, medium dense, gray (N 6/1), moist, trace fine grained sand, grades to a sandy SILT	97.0	9.6	ATT = NP	
20	SPT-8	3-4-7 (11)	ML	Sandy SILT, stiff, olive (5Y 5/4), moist, fine grained sand				95

Notes:
1) Total depth: 21.5 ft.
2) Groundwater not encountered.
3) Backfilled with cement-bentonite grout and tamped cuttings.
4) Location from handheld GPS, elevation from Google Earth.

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909-860-7777

BORING B-2

PROJECT NAME Furman Park Stormwater Capture
CLIENT City of Downey
GROUND ELEVATION 119 ft
PROJECT NUMBER CWT 22-06E
PROJECT LOCATION Downey, CA
GROUNDWATER DEPTH Not encountered
DATES DRILLED 6/2/2022
DRILLING CONTRACTOR 2R Drilling Company
BOREHOLE DEPTH 21.5 ft
LOGGED/CHECKED BY SMS/FC
DRILLING METHOD Hollow Stem Auger, CME 75
BOREHOLE DIAMETER 8 in
COORDINATES 33.953193°, -118.138931°
HAMMER DATA Auto 140 lbs, 30" drop
BACKFILL Grout and cuttings
LOCATION DESCRIPTION Baseball diamond stands

Standard Penetration Test (SPT) | No Recovery | Shelby Tube
California-Type Ring Sample | No Recovery | Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOWCOUNT blows/6" (bpf)	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0			SM	[FILL] Artificial Fill (af) Silty SAND, medium dense, brown (10YR 4/3), moist, fine grained				
	R-1	5-10-15 (25)	SM	[NATIVE] Alluvium (Qal) Silty SAND, medium dense, brown (10YR 4/3), moist, fine grained, micaceous	111.0	15.0		115
5	SPT-2	5-5-5 (10)	ML	Sandy SILT, stiff, olive (5Y 5/4), moist, fine grained sand, micaceous				
	R-3	4-10-24 (34)	ML	SILT, very stiff, light olive gray (5Y 6/2), moist, trace fine grained sand	96.0	24.8		110
10	SPT-4	5-7-9 (16)	SM	Silty SAND, medium dense, light olive gray (5Y 6/2), moist, fine grained, micaceous	91.8	8.1		
	R-5	11-19-29 (48)	SP	Poorly graded SAND, dense, light olive gray (5Y 6/2), moist, fine grained	103.7	5.7	DS	105
15	SPT-6	8-10-14 (24)		...(15.0 ft) as above, medium dense, laminated bedding of discolorations, moist				
	R-7	6-7-13 (20)	SM	Silty SAND, medium dense, olive (5Y 5/4), moist, fine grained	98.1	17.9	DS	100
20	SPT-8	3-5-7 (12)	ML	SILT with sand, stiff, olive (5Y 5/4), moist, fine grained sand				

Notes:
1) Total depth: 21.5 ft.
2) Groundwater not encountered.
3) Backfilled with cement-bentonite grout and tamped cuttings.
4) Location from handheld GPS, elevation from Google Earth.

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909-860-7777

BORING B-3

PROJECT NAME Furman Park Stormwater Capture
CLIENT City of Downey
GROUND ELEVATION 125 ft
PROJECT NUMBER CWT 22-06E
PROJECT LOCATION Downey, CA
GROUNDWATER DEPTH Not encountered
DATES DRILLED 6/1/2022
DRILLING CONTRACTOR 2R Drilling Company
BOREHOLE DEPTH 51.5 ft
LOGGED/CHECKED BY SMS/FC
DRILLING METHOD Hollow Stem Auger, CME 75
BOREHOLE DIAMETER 8 in
COORDINATES 33.954450°, -118.137640°
HAMMER DATA Auto 140 lbs, 30" drop
BACKFILL Grout and cuttings
LOCATION DESCRIPTION Furman Park, northwest corner

Standard Penetration Test (SPT) | No Recovery | Shelby Tube
California-Type Ring Sample | No Recovery | Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOWCOUNT blows/6" (bpf)	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0			SM	[FILL] Artificial Fill (af) Silty SAND, brown (10YR 4/3), moist, fine grained, rootlets				125
	R-1	5-11-15 (26)	SM	[NATIVE] Alluvium (Qal) Silty SAND, medium dense, brown (10YR 4/3), moist, fine grained, rootlets, porous			Corr: 504 ohm-cm, pH = 8.4, SO4 = 1637 ppm, Cl = 512 ppm, #200 = 38%	
5	SPT-2	5-6-7 (13)		...(5.0 ft) yellowish brown (10YR 5/6)			#200 = 22%	120
	SPT-3	9-14-14 (28)	SP	Poorly graded SAND, medium dense, olive (5Y 5/4), moist, micaceous				
10	R-4	5-7-9 (16)	SM	Silty SAND, medium dense, yellowish brown (10YR 5/6) to olive (5Y 5/4) mottling, fine grained, moist			DS	115
15	SPT-5	7-16-22 (38)		...(15.0 ft) dense, olive (5Y 5/4)			G/S/F/2μ = 1/64/34/8	110
	SPT-6	4-3-2 (5)		...(17.5 ft) loose			ATT = NP, #200 = 34%	
20	SPT-7	6-11-12 (23)		...(20.0 ft) medium dense, olive (5Y 5/4) to light yellowish brown (10YR 6/4) mottling				105
	R-8	7-11-13 (24)		...(22.5 ft) pale olive (5Y 6/4)	108.1	1.9		
25	SPT-9	12-14-12 (26)						100
	R-10	7-11-9 (20)		...(27.5 ft) light olive gray (5Y 6/2)	109.9	19.2	#200 = 38%	
30								95

Continued on next page

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909-860-7777

BORING B-3

PROJECT NAME Furman Park Stormwater Capture **CLIENT** City of Downey **GROUND ELEVATION** 125 ft

PROJECT NUMBER CWT 22-06E **PROJECT LOCATION** Downey, CA **GROUNDWATER DEPTH** Not encountered

Standard Penetration Test (SPT) | No Recovery | Shelby Tube
California-Type Ring Sample | No Recovery | Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
30									95
	SPT-11	8-13-21 (34)							
35									90
	R-12	6-9-12 (21)		SP	Poorly graded SAND, medium dense, light olive gray (5Y 6/2), moist, fine grained	90.2	9.7	G/S/F/2μ = 0/99/1/0	
40									85
	SPT-13	15-27-25 (52)			...(40.0 ft) very dense, fine to medium grained				
45									80
	R-14	6-6-10 (16)		SP-SM	Poorly graded SAND with Silt, medium dense, light olive gray (5Y 6/2), moist, fine grained	113.5	11.6		
50									75
	SPT-15	19-23-24 (47)		SP	Poorly graded SAND, dense, light olive gray (5Y 6/2), moist, fine to coarse grained, some fine gravel				

Notes:
1) Total depth: 51.5 ft.
2) Groundwater not encountered.
3) Backfilled with cement-bentonite grout and tamped cuttings.
4) Location from handheld GPS, elevation from Google Earth.

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909-860-7777

BORING B-4

Sheet 1 of 2

PROJECT NAME Furman Park Stormwater Capture
CLIENT City of Downey
GROUND ELEVATION 119 ft
PROJECT NUMBER CWT 22-06E
PROJECT LOCATION Downey, CA
GROUNDWATER DEPTH Not encountered
DATES DRILLED 6/1/2022
DRILLING CONTRACTOR 2R Drilling Company
BOREHOLE DEPTH 56.5 ft
LOGGED/CHECKED BY SMS/FC
DRILLING METHOD Hollow Stem Auger, CME 75
BOREHOLE DIAMETER 8 in
COORDINATES 33.953814°, -118.136777°
HAMMER DATA Auto 140 lbs, 30" drop
BACKFILL Grout and cuttings
LOCATION DESCRIPTION Furman Park, north side

Standard Penetration Test (SPT) | No Recovery | Shelby Tube
California-Type Ring Sample | No Recovery | Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOWCOUNT blows/6" (bpf)	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0								
	SK-1		SM	[FILL] Artificial Fill (af) Silty SAND, brown (10YR 4/3), moist, fine grained, rootlets				
	SPT-2	3-3-5 (8)	SM	[NATIVE] Alluvium (Qal) Silty SAND, medium dense, brown (10YR 4/3), moist, fine grained, rootlets			Corr: 2326 ohm-cm, pH = 8.5, SO4 = 95 ppm, Cl = 69 ppm, El	115
5	R-3	5-10-10 (20)			99.0	8.0		
	SPT-4	3-3-5 (8)	ML	SILT, firm, olive (5Y 5/4), moist, fine grained			El	110
10	R-5	9-12-19 (31)		...(10.0 ft) very stiff, olive (5Y 5/4) to yellowish brown (10YR 5/6) mottling, trace fine sand	97.9	13.0		
	SPT-6	5-8-10 (18)	SP	Poorly graded SAND, medium dense, olive (5Y 5/4), moist, fine to medium grained, trace rounded gravel, micaceous				105
15	R-7	8-15-21 (36)		...(15.0 ft) light olive gray (5Y 6/2)	103.6	2.4		
20	SPT-8	6-10-14 (24)					G/S/F/2μ = 0/96/4/0	100
							G/S/F/2μ = 0/83/17/2	95
25	R-9	10-9-15 (24)	ML	Sandy SILT, very stiff, brown (10YR 4/3), moist, fine grained sand			ATT: LL = 31, PL = 24, PI = 7, #200 = 50%	
30								90

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909-860-7777

BORING B-4

PROJECT NAME Furman Park Stormwater Capture **CLIENT** City of Downey **GROUND ELEVATION** 119 ft

PROJECT NUMBER CWT 22-06E **PROJECT LOCATION** Downey, CA **GROUNDWATER DEPTH** Not encountered

Standard Penetration Test (SPT) | No Recovery | Shelby Tube
California-Type Ring Sample | No Recovery | Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
30									
	SPT-10	7-8-12 (20)		SM	Silty SAND, medium dense, olive brown (2.5Y 4/3), moist, fine grained				85
35									
	R-11	20-45-50/5"			...(35.0 ft) very dense, yellowish brown (10YR 6/4), fine to coarse grained sand	98.6	1.7	Consol	
									80
40									
	SPT-12	18-19-21 (40)			...(40.0 ft) dense				
									75
45									
	R-13	8-9-24 (33)		ML	SILT, very stiff, olive (5Y 5/4), moist, some fine grained sand	84.9	33.0		
									70
50									
	SPT-14	11-17-21 (38)		SP	Poorly graded SAND, dense, light olive gray (5Y 6/2), moist, fine to coarse grained, trace fine gravel				
									65
55									
	R-15	10-16-24 (40)			...(55.0 ft) medium dense	89.1	21.6		

Notes:
1) Total depth: 56.5 ft.
2) Groundwater not encountered.
3) Backfilled with cement-bentonite grout and tamped cuttings.
4) Location from handheld GPS, elevation from Google Earth.

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909-860-7777

BORING B-5

PROJECT NAME Furman Park Stormwater Capture
CLIENT City of Downey
GROUND ELEVATION 121 ft
PROJECT NUMBER CWT 22-06E
PROJECT LOCATION Downey, CA
GROUNDWATER DEPTH Not encountered
DATES DRILLED 6/1/2022
DRILLING CONTRACTOR 2R Drilling Company
BOREHOLE DEPTH 31.5 ft
LOGGED/CHECKED BY SMS/FC
DRILLING METHOD Hollow Stem Auger, CME 75
BOREHOLE DIAMETER 8 in
COORDINATES 33.953717°, -118.136062°
HAMMER DATA Auto 140 lbs, 30" drop
BACKFILL Grout and cuttings
LOCATION DESCRIPTION Furman Park, northeast corner

Standard Penetration Test (SPT) | No Recovery | Shelby Tube
California-Type Ring Sample | No Recovery | Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOWCOUNT blows/6" (bpf)	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0			SM	[FILL] Artificial Fill (af) Silty SAND, medium dense, brown (10YR 4/3), moist, fine grained, rootlets				120
	SK-1 R-2	4-3-5 (8)	SM	[NATIVE] Alluvium (Qal) Silty SAND, medium dense, brown (10YR 4/3), moist, fine grained, rootlets	91.9	12.3		
5	SPT-3	2-3-4 (7)		...(5.0 ft) olive (5Y 5/4), moist, fine grained				115
	R-4	5-7-11 (18)	SP	Poorly graded SAND, medium dense, light olive gray (5Y 6/2), moist, fine grained, micaceous	90.5	9.2		
10	SPT-5	4-5-7 (12)		...(10.0 ft) paper thin interbedded silt layer				110
	R-6	8-12-17 (29)		...(12.5 ft) fine to medium grained, oxidized	93.9	4.8		
15	SPT-7	5-10-11 (21)		...(15.0 ft) fine grained, micaceous				105
20	R-8	13-19-25 (44)		...(20.0 ft) fine to medium grained	89.5	1.7	Consol, G/S/F/2μ = 0/96/4/0	100
25	SPT-9	8-11-15 (26)					G/S/F/2μ = 0/95/5/0	95
30								

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Tetra Tech
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BORING B-5

PROJECT NAME Furman Park Stormwater Capture **CLIENT** City of Downey **GROUND ELEVATION** 121 ft

PROJECT NUMBER CWT 22-06E **PROJECT LOCATION** Downey, CA **GROUNDWATER DEPTH** Not encountered

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOWCOUNT blows/6" (bpf)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
30					Standard Penetration Test (SPT); No Recovery; Shelby Tube; California-Type Ring Sample; No Recovery; Grab/Bulk Sample				
	R-10	4-7-10 (17)		ML	Sandy SILT, stiff, olive (5Y 5/4), moist, fine grained sand	97.9	19.3	Consol, #200 = 52.8%	90

Notes:
1) Total depth: 31.5 ft.
2) Groundwater not encountered.
3) Backfilled with cement-bentonite grout and tamped cuttings.
4) Location from handheld GPS, elevation from Google Earth.

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Tetra Tech
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909-860-7777

BORING B-6

PROJECT NAME Furman Park Stormwater Capture
CLIENT City of Downey
GROUND ELEVATION 121 ft
PROJECT NUMBER CWT 22-06E
PROJECT LOCATION Downey, CA
GROUNDWATER DEPTH Not encountered
DATES DRILLED 6/2/2022
DRILLING CONTRACTOR 2R Drilling Company
BOREHOLE DEPTH 21.5 ft
LOGGED/CHECKED BY SMS/FC
DRILLING METHOD Hollow Stem Auger, CME 75
BOREHOLE DIAMETER 8 in
COORDINATES 33.955825°, -118.136550°
HAMMER DATA Auto 140 lbs, 30" drop
BACKFILL Grout and cuttings
LOCATION DESCRIPTION Residential street (Horley Avenue)

Standard Penetration Test (SPT) | No Recovery | Shelby Tube
California-Type Ring Sample | No Recovery | Grab/Bulk Sample

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOWCOUNT blows/6" (bpf)	USCS	MATERIAL DESCRIPTION	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	Notes	ELEVATION (ft)
0				2.5" Asphalt				120
	SK-1		SM	[FILL] Artificial Fill (af) Silty SAND, medium dense, brown (10YR 4/3), moist, fine grained			R-value = 59	
	R-2	9-13-15 (28)			100.1	7.4	Corr: 4026 ohm-cm, pH = 9.2, SO4 = 60 ppm, Cl = 41 ppm	
5								
	SPT-3	4-4-7 (11)	SM	[NATIVE] Alluvium (Qal) ...(5.0 ft) light brownish gray (2.5Y 6/2)				115
	R-4	7-9-14 (23)		...(7.5 ft) olive (5Y 5/4)	90.1	26.7		
10								
	SPT-5	6-11-12 (23)						110
	R-6	13-20-25 (45)	SP	Poorly graded SAND, medium dense, pale yellow (2.5Y 7/3), fine grained	106.2	3.2		
15								
	SPT-7	7-10-14 (24)		...(15.0 ft) olive (5Y 5/4), fine to coarse grained				105
				...(17.5 ft) fine grained				
20								
	R-8	16-25-30 (55)		...(20.0 ft) dense, fine to medium grained	103.3	2.0		100

Notes:
1) Total depth: 21.5 ft.
2) Groundwater not encountered.
3) Backfilled with cement-bentonite grout and tamped cuttings.
4) Location from handheld GPS, elevation from Google Earth.

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Tetra Tech
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Diamond Bar, CA 91765
909-860-7777

WELL P-1

PROJECT NAME Furman Park Stormwater Capture
CLIENT City of Downey
GROUND ELEVATION 125 ft
PROJECT NUMBER CWT 22-06E
PROJECT LOCATION Downey, CA
GROUNDWATER DEPTH Not encountered
DATES DRILLED 6/1/2022
DRILLING CONTRACTOR 2R Drilling Company
BOREHOLE DEPTH 22 ft
LOGGED/CHECKED BY SMS/FC
DRILLING METHOD Hollow Stem Auger, CME 75
BOREHOLE DIAMETER 8 in
COORDINATES 33.954294°, -118.137859°
HAMMER DATA Auto 140 lbs, 30" drop
BACKFILL Percolation well
LOCATION DESCRIPTION Furman Park, northwest corner

DEPTH (ft)	SAMPLE TYPE NUMBER	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	WELL DIAGRAM
0		SM		[FILL] Artificial Fill (af) Silty SAND, brown (10YR 4/3), moist, fine grained, rootlets	
2.5					122.5
		SM		[NATIVE] Alluvium (Qal) Silty SAND, medium dense, brown (10YR 4/3), moist, fine grained, rootlets, porous	
5		SM		...(5.0 ft) yellowish brown (10YR 5/6)	
7.5					117.5
		SP		Poorly graded SAND, medium dense, olive (5Y 5/4), moist, micaceous	
10					
10.0					115.0
		SM		Silty SAND, medium dense, yellowish brown (10YR 5/6) to olive (5Y 5/4) mottling, fine grained, moist	2" diameter schedule 40 PVC with 0.020 inch slotted screen
15					
20					
22.0					103.0
					Gravel backfill

Notes:
1) Total depth: 22.0 ft.
2) Groundwater not encountered.
3) Backfilled with percolation test setup, grout and cuttings after testing.
4) Location from handheld GPS, elevation from Google Earth.

Bottom of borehole at 22.0 feet.

GENERAL BH / TP / WELL - GINT STD US LAB.GDT - L:\03 - TECHNICAL RESOURCES\02 SOFTWARE LIBRARY\GINT\PROJECTS\197-4703-0006 CWT 22-06 FURMAN PARK.GPJ

Printed on: 7/13/22 17:05

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21700 Copley Drive
Diamond Bar, CA 91765
909-860-7777

WELL P-2

PROJECT NAME Furman Park Stormwater Capture
CLIENT City of Downey
GROUND ELEVATION 123 ft
PROJECT NUMBER CWT 22-06E
PROJECT LOCATION Downey, CA
GROUNDWATER DEPTH Not encountered
DATES DRILLED 6/1/2022
DRILLING CONTRACTOR 2R Drilling Company
BOREHOLE DEPTH 19.8 ft
LOGGED/CHECKED BY SMS/FC
DRILLING METHOD Hollow Stem Auger, CME 75
BOREHOLE DIAMETER 8 in
COORDINATES 33.954020°, -118.137438°
HAMMER DATA Auto 140 lbs, 30" drop
BACKFILL Percolation well
LOCATION DESCRIPTION Furman Park, north side

DEPTH (ft)	SAMPLE TYPE NUMBER	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION		WELL DIAGRAM
0		SM		[FILL] Artificial Fill (af) Silty SAND, brown (10YR 4/3), moist, fine grained, rootlets		
2.5					120.5	
5		SM		[NATIVE] Alluvium (Qal) Silty SAND, medium dense, brown (10YR 4/3), moist, fine grained, rootlets, porous ...(5.0 ft) yellowish brown (10YR 5/6)		
7.5					115.5	
10		SP		Poorly graded SAND, medium dense, olive (5Y 5/4), moist, micaceous		2" diameter schedule 40 PVC with 0.020 inch slotted screen
10.0					113.0	
15		SM		Silty SAND, medium dense, yellowish brown (10YR 5/6) to olive (5Y 5/4) mottling, fine grained, moist		
19.8					103.2	Gravel backfill

Notes:
1) Total depth: 19.8 ft.
2) Groundwater not encountered.
3) Backfilled with percolation test setup, grout and cuttings after testing.
4) Location from handheld GPS, elevation from Google Earth.

Bottom of borehole at 19.8 feet.

GENERAL BH / TP / WELL - GINT STD US LAB.GDT - L:\03 - TECHNICAL RESOURCES\02 SOFTWARE LIBRARY\GINT\PROJECTS\197-4703-0006 CWT 22-06 FURMAN PARK.GPJ

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Diamond Bar, CA 91765
909-860-7777

WELL P-3

PROJECT NAME Furman Park Stormwater Capture
CLIENT City of Downey
GROUND ELEVATION 123 ft
PROJECT NUMBER CWT 22-06E
PROJECT LOCATION Downey, CA
GROUNDWATER DEPTH Not encountered
DATES DRILLED 6/1/2022
DRILLING CONTRACTOR 2R Drilling Company
BOREHOLE DEPTH 20 ft
LOGGED/CHECKED BY SMS/FC
DRILLING METHOD Hollow Stem Auger, CME 75
BOREHOLE DIAMETER 8 in
COORDINATES 33.953881°, -118.136946°
HAMMER DATA Auto 140 lbs, 30" drop
BACKFILL Percolation well
LOCATION DESCRIPTION Furman Park, north side

DEPTH (ft)	SAMPLE TYPE NUMBER	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION		WELL DIAGRAM
0		SM		[FILL] Artificial Fill (af) Silty SAND, brown (10YR 4/3), moist, fine grained, rootlets		
2.5					120.5	
		SM		[NATIVE] Alluvium (Qal) Silty SAND, medium dense, brown (10YR 4/3), moist, fine grained, rootlets		
5						
7.5					115.5	
		ML		SILT, firm, olive (5Y 5/4), moist, fine grained		
10				...(10.0 ft) very stiff, olive (5Y 5/4) to yellowish brown (10YR 5/6) mottling, trace fine sand		2" diameter schedule 40 PVC with 0.020 inch slotted screen
12.5					110.5	
		SP		Poorly graded SAND, medium dense, olive (5Y 5/4), moist, fine to medium grained, trace rounded gravel, micaceous		
15						
20						Gravel backfill
20.0					103.0	

Notes:
1) Total depth: 20.0 ft.
2) Groundwater not encountered.
3) Backfilled with percolation test setup, grout and cuttings after testing.
4) Location from handheld GPS, elevation from Google Earth.

Bottom of borehole at 20.0 feet.

GENERAL BH / TP / WELL - GINT STD US LAB.GDT - L:\03 - TECHNICAL RESOURCES\02 SOFTWARE LIBRARY\GINT\PROJECTS\197-4703-0006 CWT 22-06 FURMAN PARK.GPJ

Printed on: 7/13/22 17:05

Appendix B

Results of Laboratory Testing

TETRA TECH

MOISTURE CONTENT AND DENSITY

ASTM D7263

Job Name:	**Furman Park Storm water Capture**	**Date Sampled:**	6/17/2022
Job Number:	**CWT-22-06E**	**Date Completed:**	7/1/2022
Tested By:	**MG**	**Note:**	Page 1 of 3

Boring / Test Pit / Trench		**B-1**	**B-1**	**B-2**	**B-2**	**B-3**	**B-3**	**B-3**	**B-3**	
Sample Number		**R-5**	**R-7**	**R-1**	**R-3**	**R-8**	**R-10**	**R-12**	**R-14**	
Sample Depth	*ft*	**12.5**	**17.5**	**2.5**	**7.5**	**22.5**	**27.5**	**35**	**45**	
USCS Soil Description		Light Gray SP, Native	Olive Brown CL, Native	Olive Brown SM, Fill	Olive Gray ML, Native	Olive Gray SP, Native	Olive Gray SM, Native	Olive Gray SP, Native	Olive Gray SP-SM, Native	
Number of Rings		6	6	6	6	6	6	6	6	
Total Weight Rings + Soil	*grams*	995.60	1036.40	1191.20	1134.20	1064.20	1215.20	983.20	1183.60	
Volume of Rings	ft^3	0.0159	0.0159	0.0159	0.0159	0.0159	0.0159	0.0159	0.0159	
Weight of Rings	*grams*	267.60	267.60	267.60	267.60	267.60	267.60	267.60	267.60	
Weight of Soil	*grams*	728.00	768.80	923.60	866.60	796.60	947.60	715.60	916.00	
Wet Density	*pcf*	100.66	106.30	127.70	119.82	110.14	131.02	98.94	126.65	
	Container ID	X21	X63	X61	X16	X52	X25	X27	X35	
Tare	*grams*	10.5	10.5	10.5	10.5	10.5	10.5	10.5	10.5	
Wet Soil + Tare	*grams*	207	251.6	314.2	322.2	218.9	317.7	229.5	314.8	
Dry Soil + Tare	*grams*	201.3	230.4	274.5	260.3	215	268.2	210.2	283.2	
Weight of Water	*grams*	5.7	21.2	39.7	61.9	3.9	49.5	19.3	31.6	

Dry Density	***pcf***	**97.7**	**97.0**	**111.0**	**96.0**	**108.1**	**109.9**	**90.2**	**113.5**	
Moisture Content	***%***	**3.0**	**9.6**	**15.0**	**24.8**	**1.9**	**19.2**	**9.7**	**11.6**	

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TETRA TECH

MOISTURE CONTENT AND DENSITY

ASTM D7263

Job Name:	**Furman Park Storm Water Capture**	**Date Sampled:**	6/17/2022
Job Number:	**CWT-22-06E**	**Date Completed:**	7/1/2022
Tested By:	**MG**	**Note:**	Page 2 of 3

Boring / Test Pit / Trench		**B-4**	**B-4**	**B-4**	**B-4**	**B-4**	**B-5**	**B-5**	**B-5**	
Sample Number		**R-3**	**R-5**	**R-7**	**R-13**	**R-15**	**R-2**	**R-4**	**R-6**	
Sample Depth	*ft*	**5**	**10**	**15**	**45**	**55**	**2.5**	**7.5**	**12.5**	
USCS Soil Description		Olive Brown SM, Native	Olive Brown ML, Native	Olive Gray SP, Native	Brownish Gray ML, Native	Olive Gray SP, Native	Olive Brown SM, Native	Olive Gray SP, Native	Olive Yellow SP, Native	
Number of Rings		6	6	6	6	6	6	6	6	
Total Weight Rings + Soil	*grams*	1040.80	1088.60	1035.00	1084.40	1051.30	1014.00	982.20	979.20	
Volume of Rings	ft^3	0.0159	0.0159	0.0159	0.0159	0.0159	0.0159	0.0159	0.0159	
Weight of Rings	*grams*	267.60	267.60	267.60	267.60	267.60	267.60	267.60	267.60	
Weight of Soil	*grams*	773.20	821.00	767.40	816.80	783.70	746.40	714.60	711.60	
Wet Density	*pcf*	106.91	113.52	106.11	112.94	108.36	103.20	98.81	98.39	
	Container ID	X32	X48	X30	X34	X22	X12	X26	X24	
Tare	*grams*	10.5	10.5	10.5	10.5	10.5	10.5	10.5	10.5	
Wet Soil + Tare	*grams*	283.3	322.4	242.2	298.9	262.3	256.3	251	244.9	
Dry Soil + Tare	*grams*	263.2	279.4	236.8	227.4	217.6	229.3	230.8	234.2	
Weight of Water	*grams*	20.1	43	5.4	71.5	44.7	27	20.2	10.7	

Dry Density	***pcf***	**99.0**	**97.9**	**103.6**	**84.9**	**89.1**	**91.9**	**90.5**	**93.9**	
Moisture Content	***%***	**8.0**	**16.0**	**2.4**	**33.0**	**21.6**	**12.3**	**9.2**	**4.8**	

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TETRA TECH

MOISTURE CONTENT AND DENSITY

ASTM D7263

Job Name:	**Furman Park Storm Water Capture**	**Date Sampled:**	6/17/2022
Job Number:	**CWT-22-06E**	**Date Completed:**	7/1/2022
Tested By:	**MG**	**Note:**	Page 3 of 3

Boring / Test Pit / Trench		**B-6**	**B-6**	**B-6**	**B-6**					
Sample Number		**R-2**	**R-4**	**R-6**	**R-8**					
Sample Depth	*ft*	**2.5**	**7.5**	**12.5**	**20**					
USCS Soil Description		Olive Brown SM, Fill	Olive Brown SM, Native	Olive Gray SP, Native	Light Gray SP, Native					
Number of Rings		6	6	6	6					
Total Weight Rings + Soil	*grams*	1045.20	1093.20	1060.40	1030.00					
Volume of Rings	ft^3	0.0159	0.0159	0.0159	0.0159					
Weight of Rings	*grams*	267.60	267.60	267.60	267.60					
Weight of Soil	*grams*	777.60	825.60	792.80	762.40					
Wet Density	*pcf*	107.52	114.15	109.62	105.41					
Container ID		X33	X50	X38	X44					
Tare	*grams*	10.5	10.5	10.5	10.5					
Wet Soil + Tare	*grams*	261.3	281.5	254.4	253.6					
Dry Soil + Tare	*grams*	244	224.4	246.9	248.8					
Weight of Water	*grams*	17.3	57.1	7.5	4.8					

Dry Density	***pcf***	**100.1**	**90.1**	**106.2**	**103.3**					
Moisture Content	***%***	**7.4**	**26.7**	**3.2**	**2.0**					

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TETRA TECH

PERCENT PASSING # 200 SIEVE

ASTM D1140

Job Name:	Furman Park Stormwater Capture	**Date Completed:**	6/17/2022
Job Number:	CWT-22-06E	**Date Sampled:**	July 2, 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-3	**SPT-2**	5.0	356.5	278.3	22%	SM
B-3	**SPT-6**	17.5	298.2	197.2	34%	SM
B-3	**R-1**	27.5	391.7	243.5	38%	SM
B-4	**R-9**	25.0	244	122.2	50%	ML

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PERCENT PASSING NO. 200 SIEVE

ASTM D1140

Client:	Tetra Tech	AP Lab No.:	22-0659
Project Name:	Furman Park Stormwater Capture	Test Date:	06/23/22
Project Number:	CWT 22-06E		

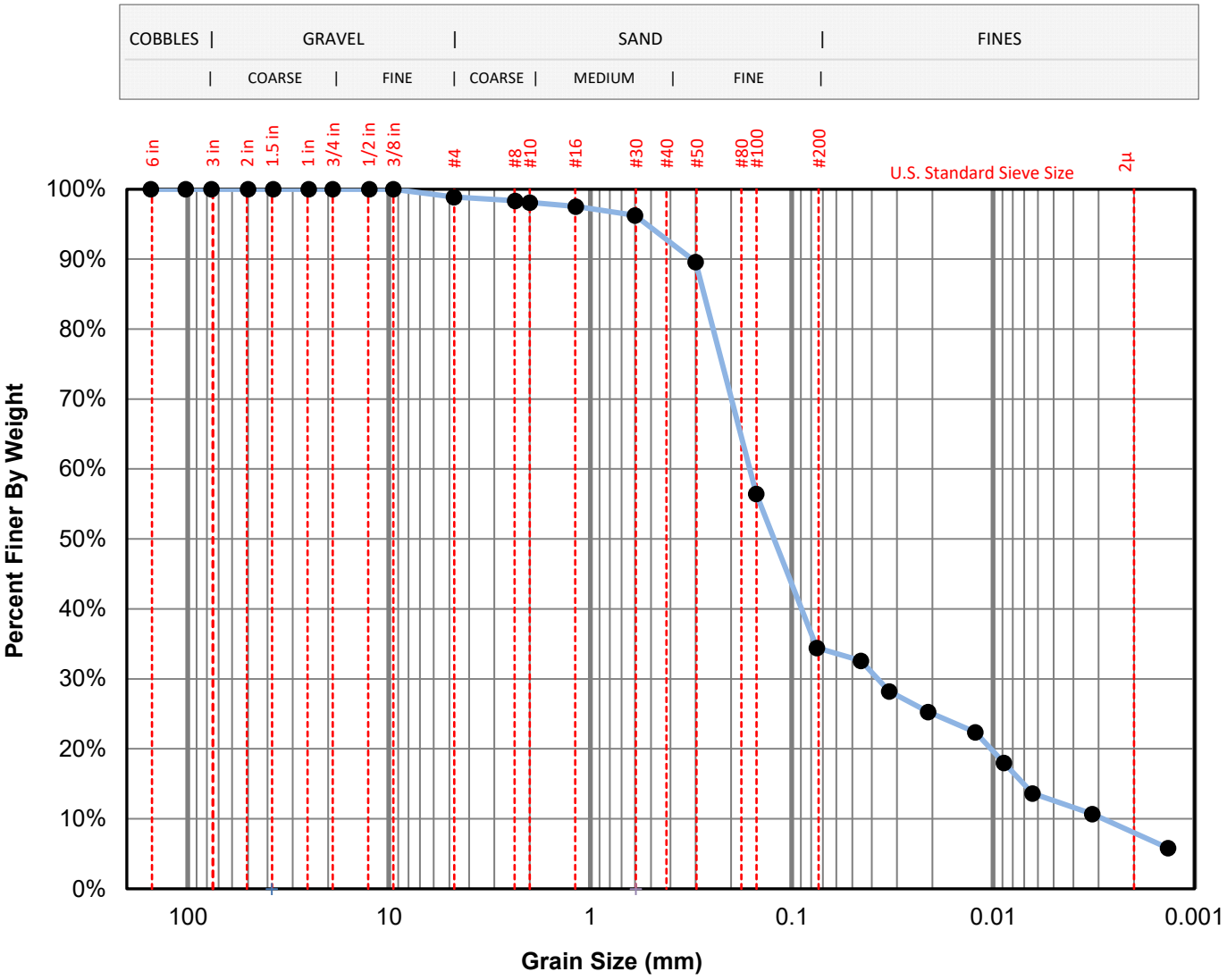
Boring No.	Sample No.	Depth (ft)	Percent Fines (%)
B-5	R-10	30	52.8

TETRA TECH

GRAIN SIZE DISTRIBUTION ANALYSIS

ASTM C136/C117/D7928/D6913

Job Name:	Furman Park Storm water	**Date Sampled:**	6/17/2022
Job Number:	CWT-22-06E	**Date Completed:**	7/2/2022
Tested By:	MG	**Sample Identification:**	B-3, SPT-5
Note:		**Sample Depth:**	15ft
Sample Description:	Olive Brown Silty SAND, SM, Native		



Symbol	Boring No.	Sample No.	Depth	LL	PI	USCS	Cobbles	Gravel	Sand	Fines	2µ
●	B-3	SPT-5	15ft			SM	**0%**	**1%**	**64%**	**34%**	**8%**

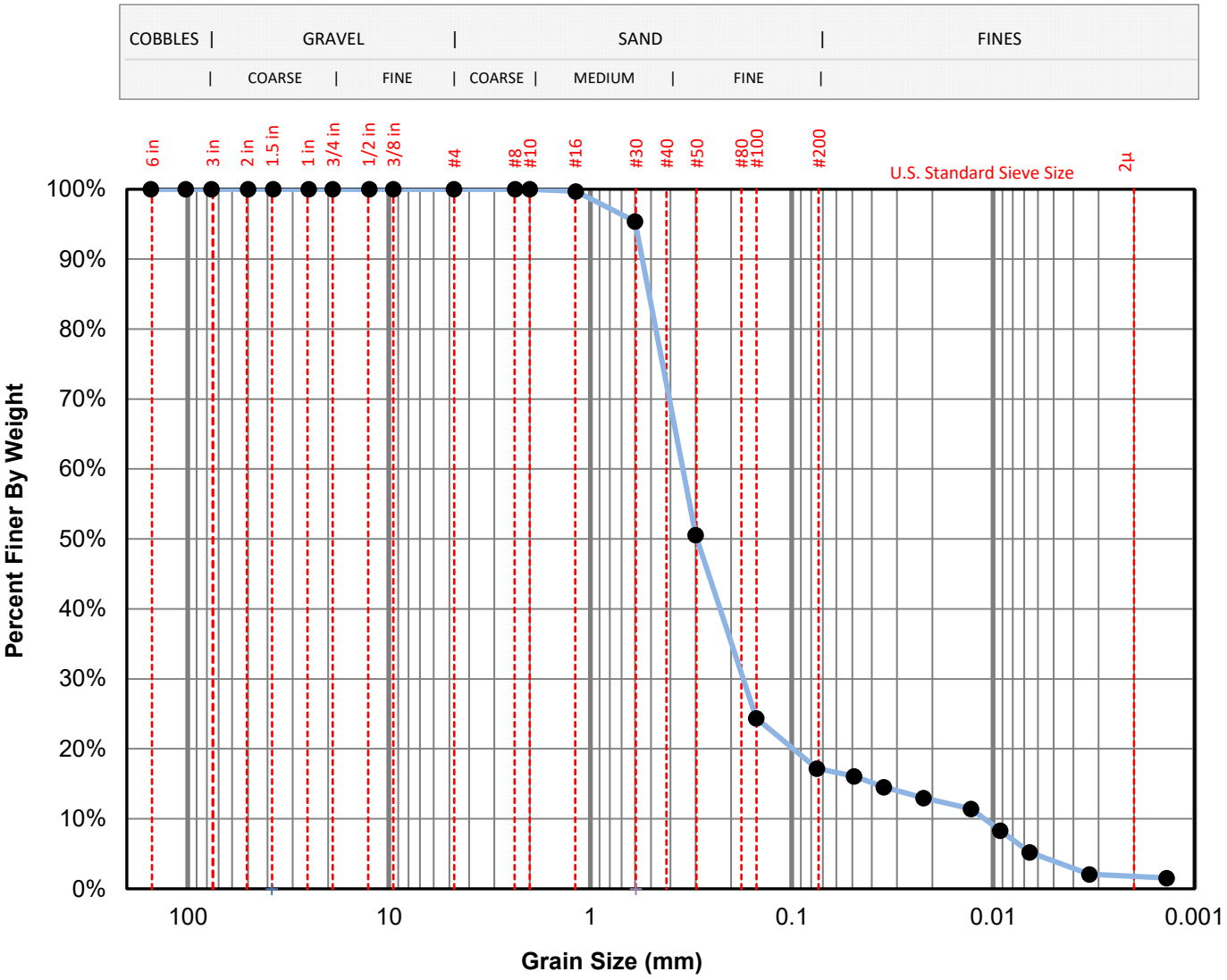
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TETRA TECH

GRAIN SIZE DISTRIBUTION ANALYSIS

ASTM C136/C117/D7928/D6913

Job Name:	Furman Park Storm water	**Date Sampled:**	6/17/2022
Job Number:	CWT-22-06E	**Date Completed:**	7/2/2022
Tested By:	MG	**Sample Identification:**	B-3, R-8
Note:		**Sample Depth:**	22.5 ft
Sample Description:	Olive Gray Silty SAND, SM, Native		



Symbol	Boring No.	Sample No.	Depth	LL	PI	USCS	Cobbles	Gravel	Sand	Fines	2µ
●	B-3	R-8	22.5 ft			SM	**0%**	**0%**	**83%**	**17%**	**2%**

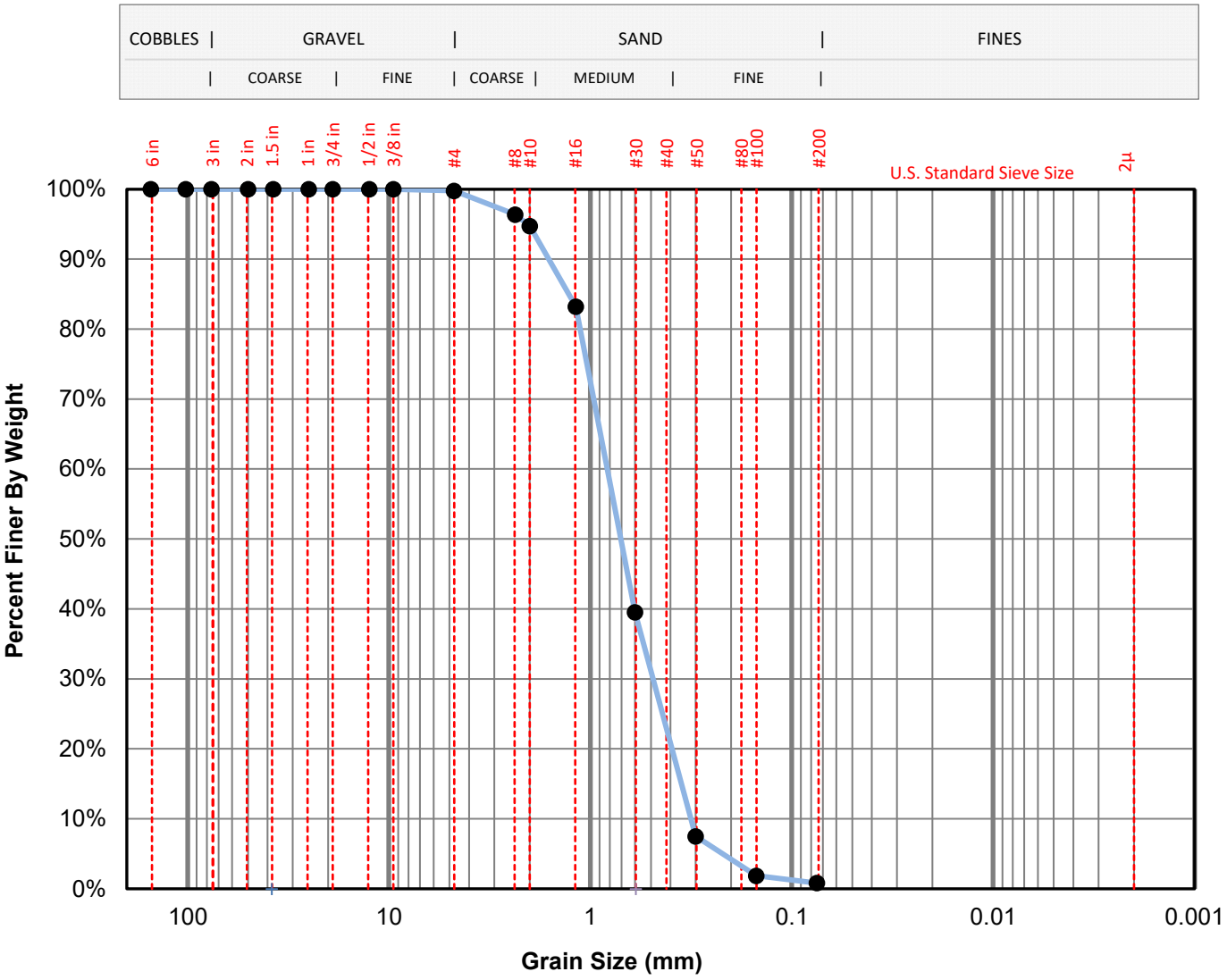
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GRAIN SIZE DISTRIBUTION ANALYSIS

ASTM C136/C117/D7928/D6913

Job Name:	Furman Park Storm water	**Date Sampled:**	6/17/2022
Job Number:	CWT-22-06E	**Date Completed:**	7/2/2022
Tested By:	MG	**Sample Identification:**	B-3, R-12
Note:		**Sample Depth:**	35 ft
Sample Description:	Olive Gray Poorly-graded SAND, SP, Native		



Symbol	Boring No.	Sample No.	Depth	LL	PI	USCS	Cobbles	Gravel	Sand	Fines	2µ
●	B-3	R-12	35 ft			SP	**0%**	**0%**	**99%**	**1%**	**0%**

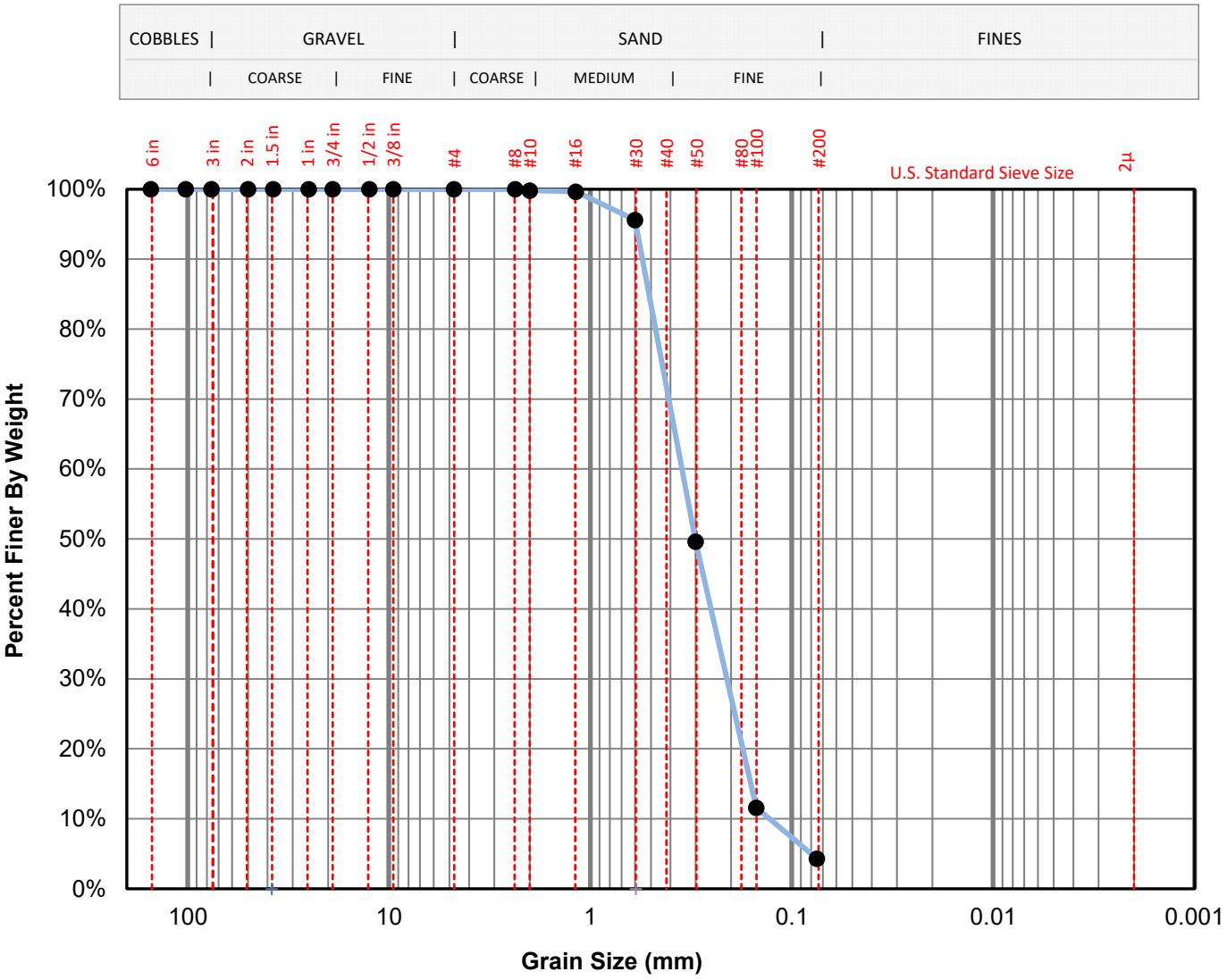
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TETRA TECH

GRAIN SIZE DISTRIBUTION ANALYSIS

ASTM C136/C117/D7928/D6913

Job Name:	Furman Park Storm water	**Date Sampled:**	6/17/2022
Job Number:	CWT-22-06E	**Date Completed:**	7/2/2022
Tested By:	MG	**Sample Identification:**	B-4, SPT-8
Note:		**Sample Depth:**	20 ft
Sample Description:	Olive Gray poorly-graded SAND, SP, Native		



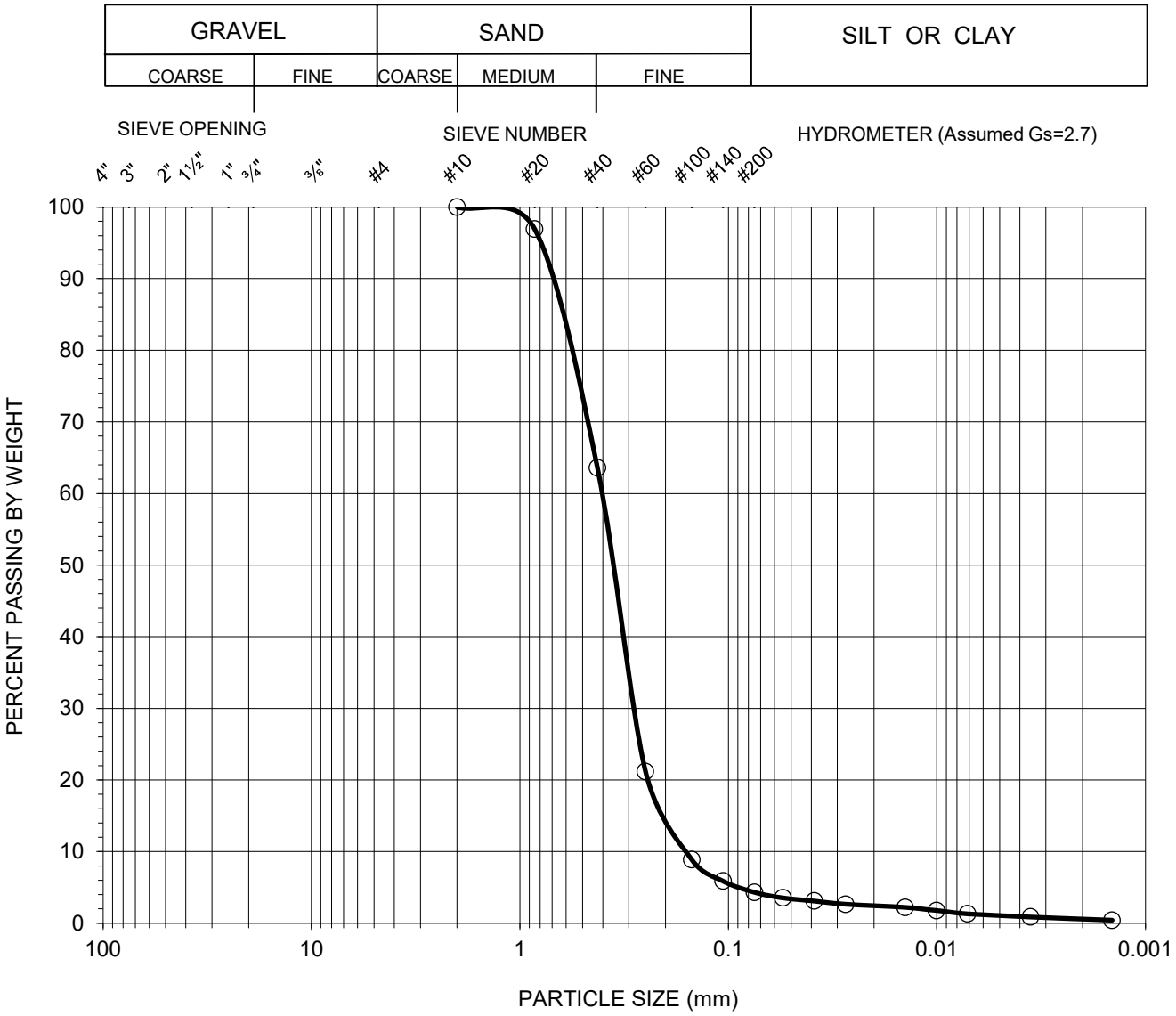
Symbol	Boring No.	Sample No.	Depth	LL	PI	USCS	Cobbles	Gravel	Sand	Fines	2µ
●	B-4	SPT-8	20ft			SP	**0%**	**0%**	**96%**	**4%**	**0%**

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GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913 & D 7928

Client Name:	Tetra Tech	Tested by:	TV	Date:	06/29/22
Project Name:	Furman Park Stormwater Capture	Computed by:	JP	Date:	06/29/22
Project No.:	CWT 22-06E	Checked by:	AP	Date:	06/29/22



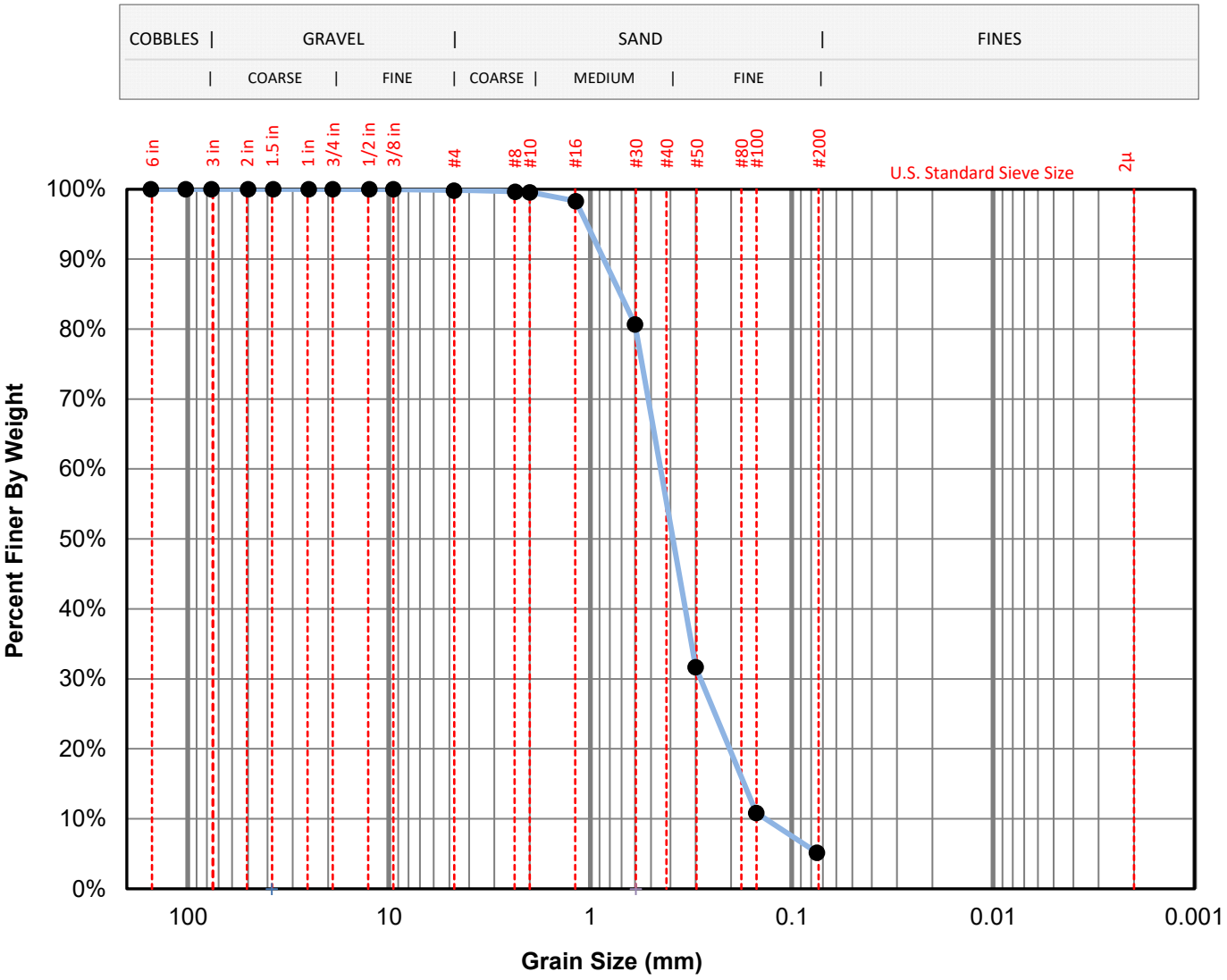
Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	B-5	R-8	20	0	96	4	N/A	SP

TETRA TECH

GRAIN SIZE DISTRIBUTION ANALYSIS

ASTM C136/C117/D7928/D6913

Job Name:	Furman Park Storm water	**Date Sampled:**	6/17/2022
Job Number:	CWT-22-06E	**Date Completed:**	7/2/2022
Tested By:	MG	**Sample Identification:**	B-5, SPT-9
Note:		**Sample Depth:**	25 ft
Sample Description:	Olive Gray Poorly-graded SAND with Silt, SP-SM, Native		



Symbol	Boring No.	Sample No.	Depth	LL	PI	USCS	Cobbles	Gravel	Sand	Fines	2µ
●	B-5	SPT-9	25 ft			SP-SM	**0%**	**0%**	**95%**	**5%**	**0%**

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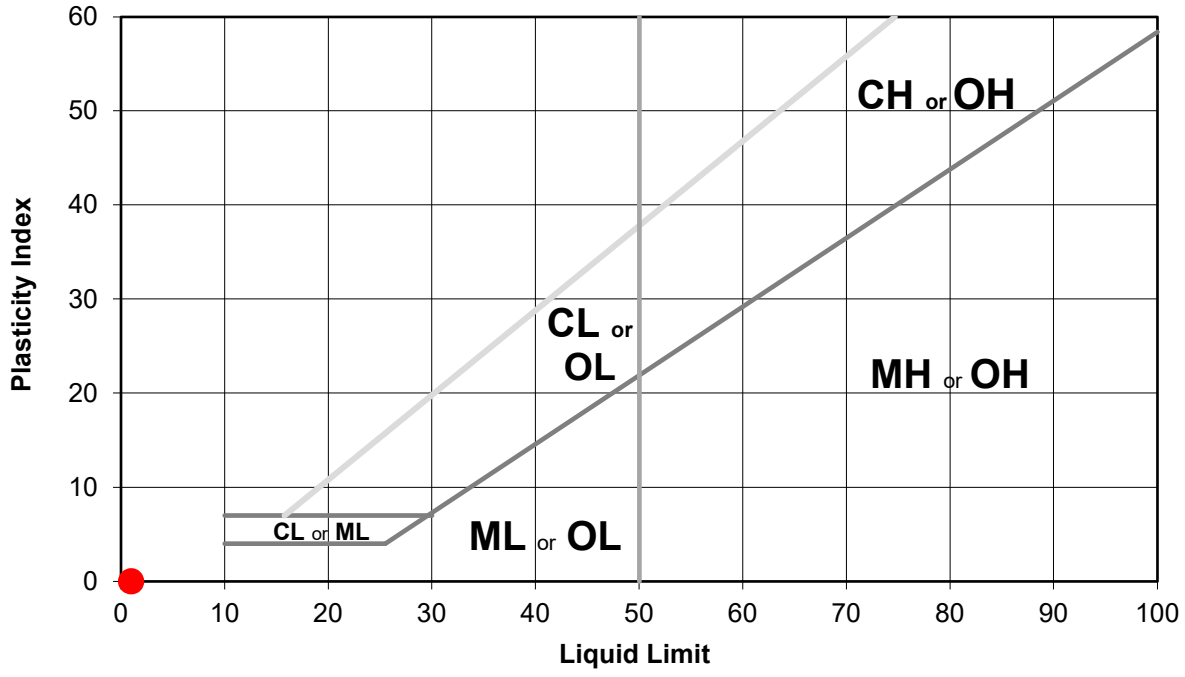
TETRA TECH

ATTERBERG LIMITS

ASTM D4318

Job Name:	Furman Park	**Date Sampled:**	6/17/2022
Job Number:	CWT-22-06E	**Date Completed:**	7/2/2022
Tested By:	MG	**Sample Identification:**	B-1, R-7
Note:		**Sample Depth:**	17.5 ft
Sample Description:	Brown Silty SAND, SM		

		Plastic Limit		Liquid Limit			
Test No.		**1**	**2**	**1**	**2**	**3**	**4**
Number of Blows							
Container ID							
Wet Weight of Soil + Cont.	*grams*	NP	NP	NP	NP	NP	NP
Dry Weight of Soil + Cont.	*grams*						
Weight of Container	*grams*						
Moisture Weight	*grams*	0.00	0.00	0.00	0.00	0.00	0.00
Weight of Dry Soil	*grams*	0.00	0.00	0.00	0.00	0.00	0.00
Moisture Content	%	**NP**	**NP**	**NP**	**NP**	**NP**	**NP**



Plastic Limit	**NP**
Liquid Limit	**NP**
Plasticity Index	**NP**

USCS Classification	**SM**

Based on Atterberg Limits only

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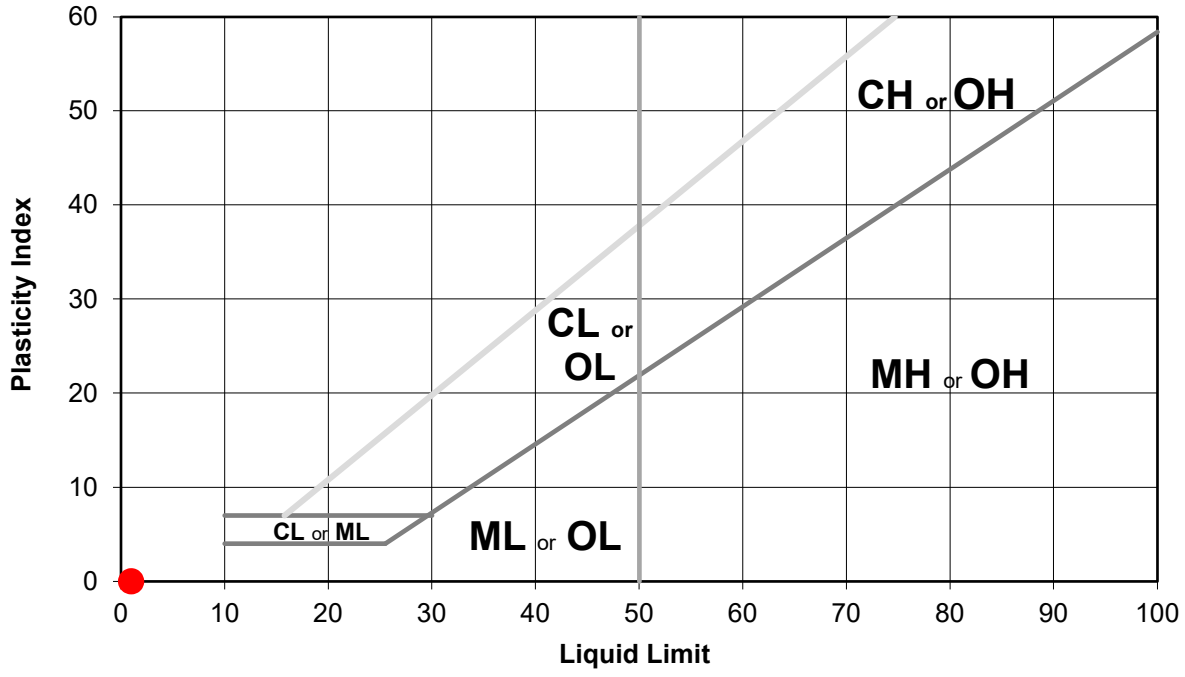
TETRA TECH

ATTERBERG LIMITS

ASTM D4318

Job Name:	Furman Park	**Date Sampled:**	6/17/2022
Job Number:	CWT-22-06E	**Date Completed:**	7/2/2022
Tested By:	MG	**Sample Identification:**	B-3 SPT-6
Note:		**Sample Depth:**	17.5ft
Sample Description:	Olive Gray SM, Native		

		Plastic Limit		Liquid Limit			
Test No.		**1**	**2**	**1**	**2**	**3**	**4**
Number of Blows							
Container ID							
Wet Weight of Soil + Cont.	*grams*	NP	NP	NP	NP	NP	NP
Dry Weight of Soil + Cont.	*grams*						
Weight of Container	*grams*						
Moisture Weight	*grams*	0.00	0.00	0.00	0.00	0.00	0.00
Weight of Dry Soil	*grams*	0.00	0.00	0.00	0.00	0.00	0.00
Moisture Content	%	**NP**	**NP**	**NP**	**NP**	**NP**	**NP**



Plastic Limit	**NP**
Liquid Limit	**NP**
Plasticity Index	**NP**

USCS Classification	**SM**

Based on Atterberg Limits only

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TETRA TECH

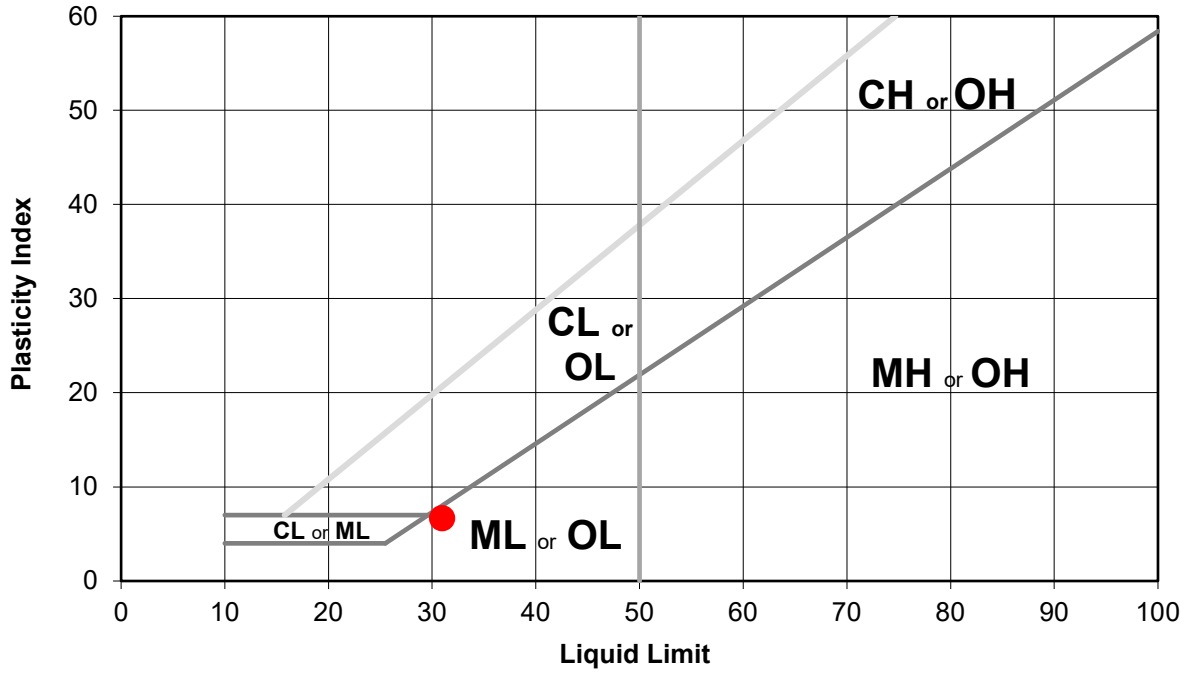
ATTERBERG LIMITS

ASTM D4318

Job Name:	Furman Park	**Date Sampled:**	6/17/2022
Job Number:	CWT-22-06E	**Date Completed:**	7/2/2022
Tested By:	MG	**Sample Identification:**	B-4 R-9
Note:		**Sample Depth:**	25ft
Sample Description:	Olive Gray ML, Native		

		Plastic Limit	
Test No.		**1**	**2**
Number of Blows			
Container ID		F8	P5
Wet Weight of Soil + Cont.	*grams*	20.90	21.80
Dry Weight of Soil + Cont.	*grams*	19.20	20.00
Weight of Container	*grams*	12.40	12.40
Moisture Weight	*grams*	1.70	1.80
Weight of Dry Soil	*grams*	6.80	7.60
Moisture Content	*%*	**25.0**	**23.7**

Liquid Limit			
1	**2**	**3**	**4**
15	25	32	
P26	M14	M7	
57.60	49.80	60.40	
49.60	44.10	52.50	
25.20	25.40	25.00	
8.00	5.70	7.90	
24.40	18.70	27.50	
32.8	**30.5**	**28.7**	



Plastic Limit	**24**
Liquid Limit	**31**
Plasticity Index	**7**

USCS Classification **ML**

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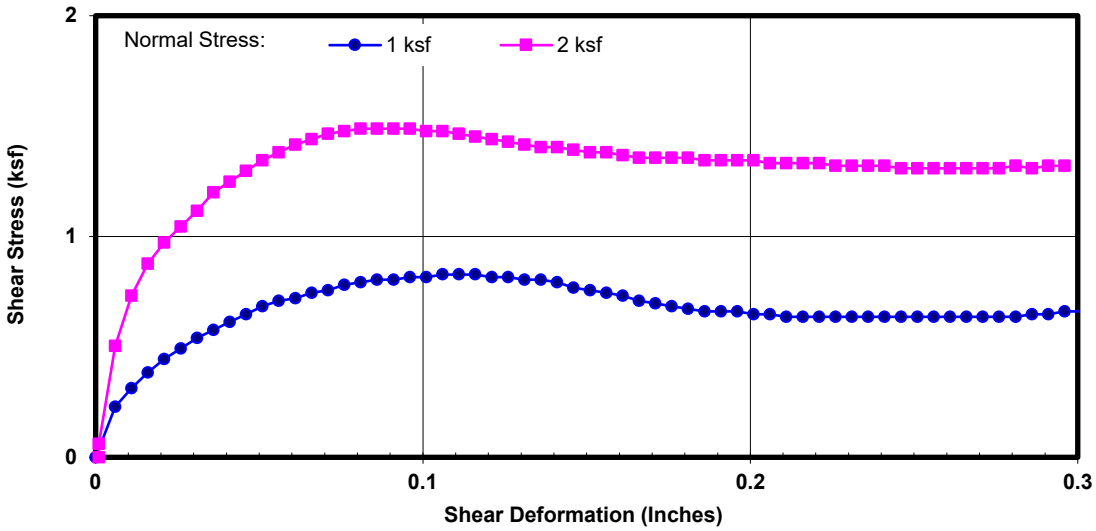
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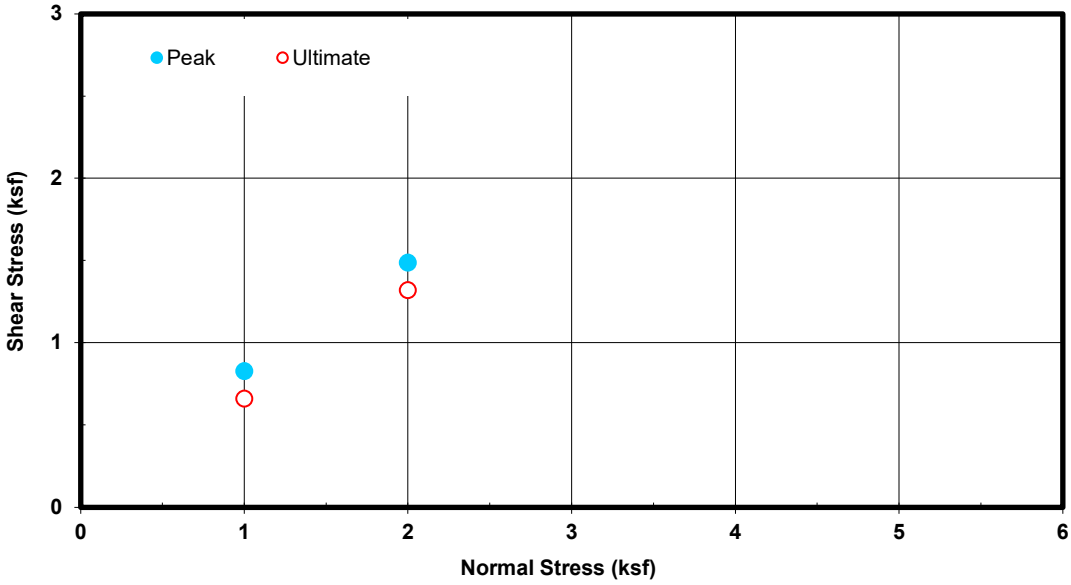
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DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client:	Tetra Tech		**Tested By:**	VT	**Date:** 06/23/22
Project Name:	Furman Park Stormwater Capture		**Computed By:**	NR	**Date:** 06/27/22
Project No.:	CWT 22-06E		**Checked by:**	AP	**Date:** 06/29/22
Boring No.:	B-1				
Sample No.:	R-3	**Depth (ft):** 7.5			
Sample Type:	Mod. Cal.				
Soil Description:	Silty Sand				
Test Condition:	Inundated	**Shear Type:** Regular			

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)
104.0	96.0	8.3	25.5	30	91	1	0.828	0.660
						2	1.488	1.320



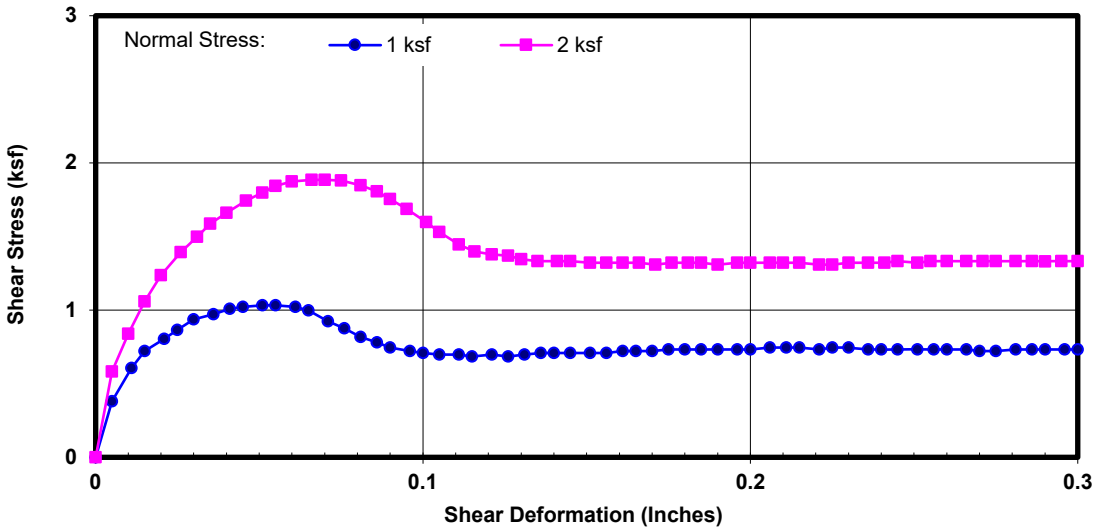


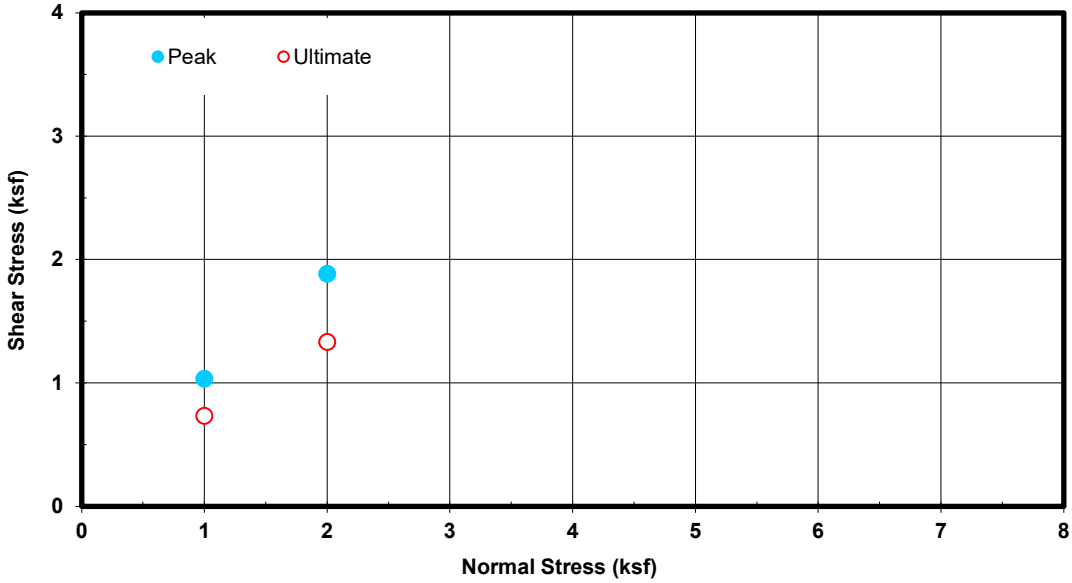
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DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client:	Tetra Tech		**Tested By:**	VT	**Date:** 06/23/22
Project Name:	Furman Park Stormwater Capture		**Computed By:**	NR	**Date:** 06/27/22
Project No.:	CWT 22-06E		**Checked by:**	AP	**Date:** 06/29/22
Boring No.:	B-2				
Sample No.:	R-5	**Depth (ft):** 12.5			
Sample Type:	Mod. Cal.				
Soil Description:	Sand w/silt				
Test Condition:	Inundated	**Shear Type:** Regular			

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)
109.6	103.7	5.7	22.8	25	99	1	1.032	0.732
						2	1.885	1.332



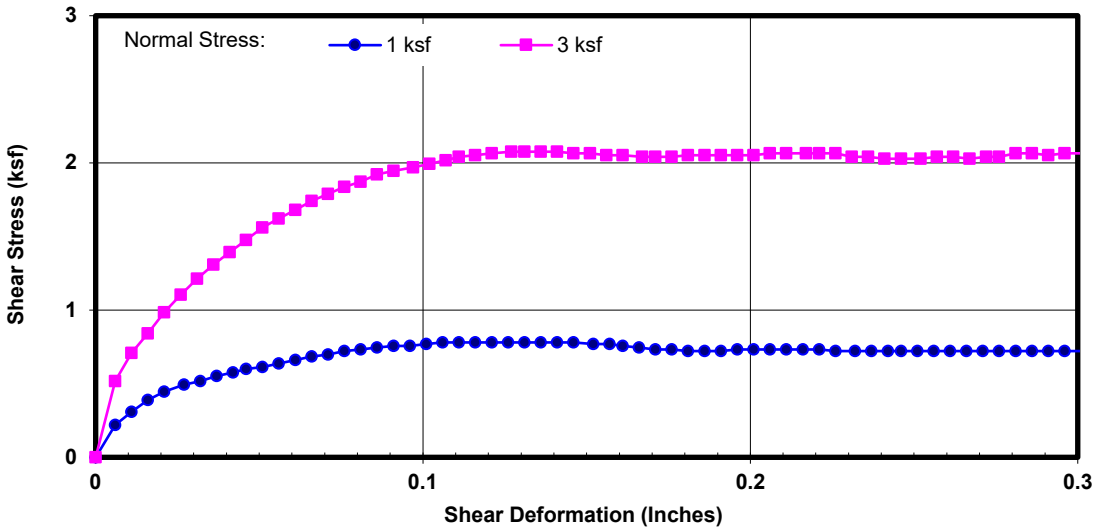


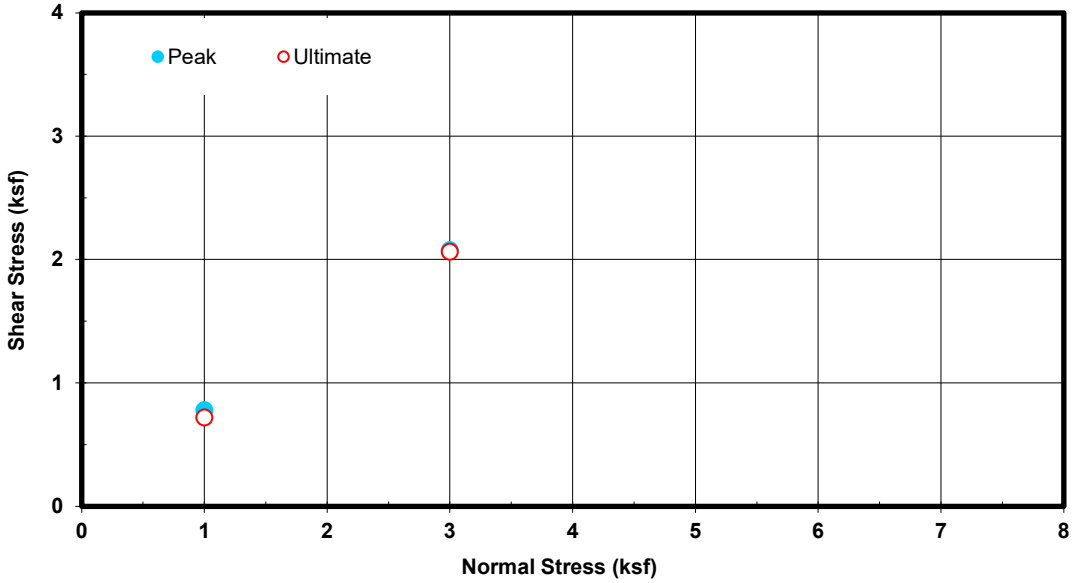
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DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client:	Tetra Tech		**Tested By:**	ST	**Date:** 06/24/22
Project Name:	Furman Park Stormwater Capture		**Computed By:**	NR	**Date:** 06/27/22
Project No.:	CWT 22-06E		**Checked by:**	AP	**Date:** 06/29/22
Boring No.:	B-2				
Sample No.:	R-7	**Depth (ft):** 17.5			
Sample Type:	Mod. Cal.				
Soil Description:	Silty Sand, fine-grained				
Test Condition:	Inundated	**Shear Type:** Regular			

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)
115.7	98.1	17.9	24.0	67	90	1	0.780	0.720
						3	2.076	2.064



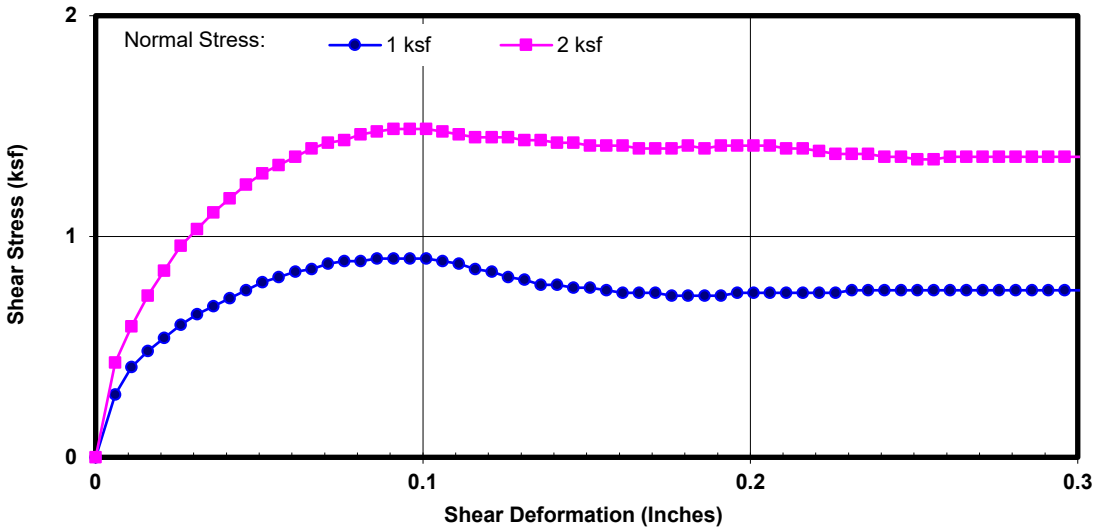


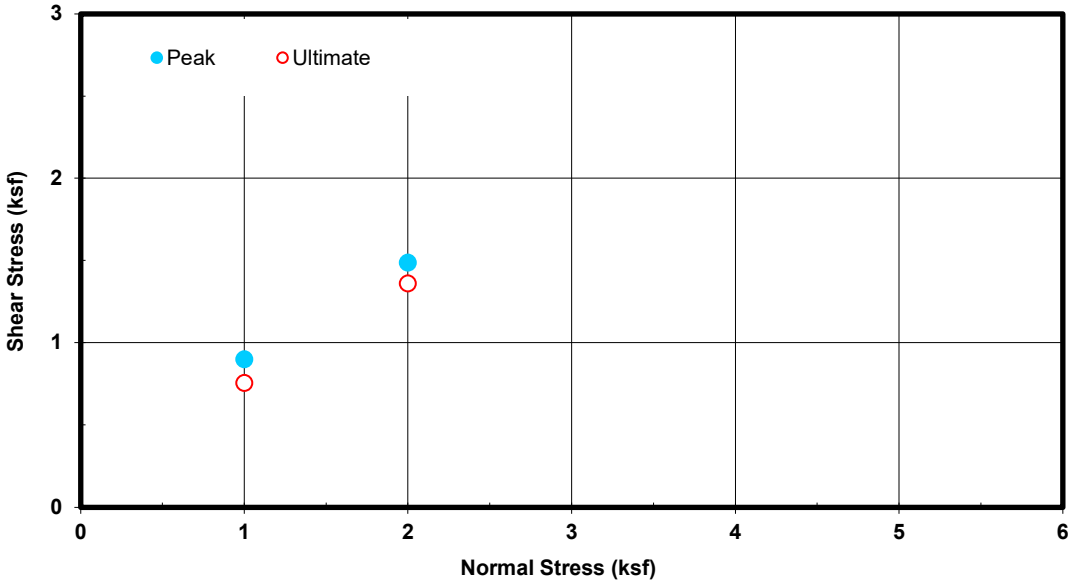
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DIRECT SHEAR TEST RESULTS
ASTM D 3080

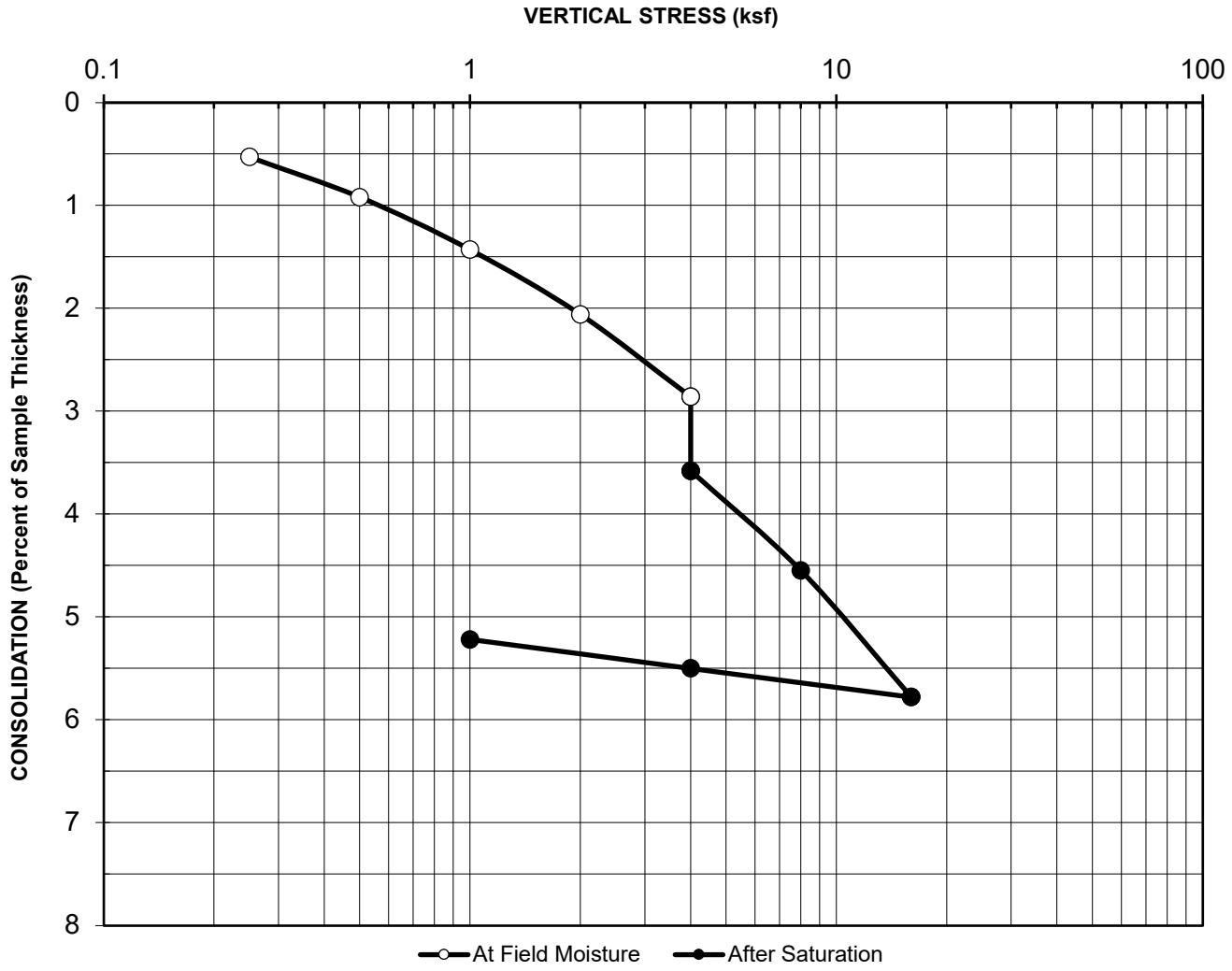
Client:	Tetra Tech		**Tested By:**	ST	**Date:** 06/24/22
Project Name:	Furman Park Stormwater Capture		**Computed By:**	NR	**Date:** 06/27/22
Project No.:	CWT 22-06E		**Checked by:**	AP	**Date:** 06/29/22
Boring No.:	B-3				
Sample No.:	R-4	**Depth (ft):** 10			
Sample Type:	Mod. Cal.				
Soil Description:	Silty Sand				
Test Condition:	Inundated	**Shear Type:** Regular			

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)
99.2	91.8	8.1	30.7	26	99	1	0.900	0.756
						2	1.487	1.361





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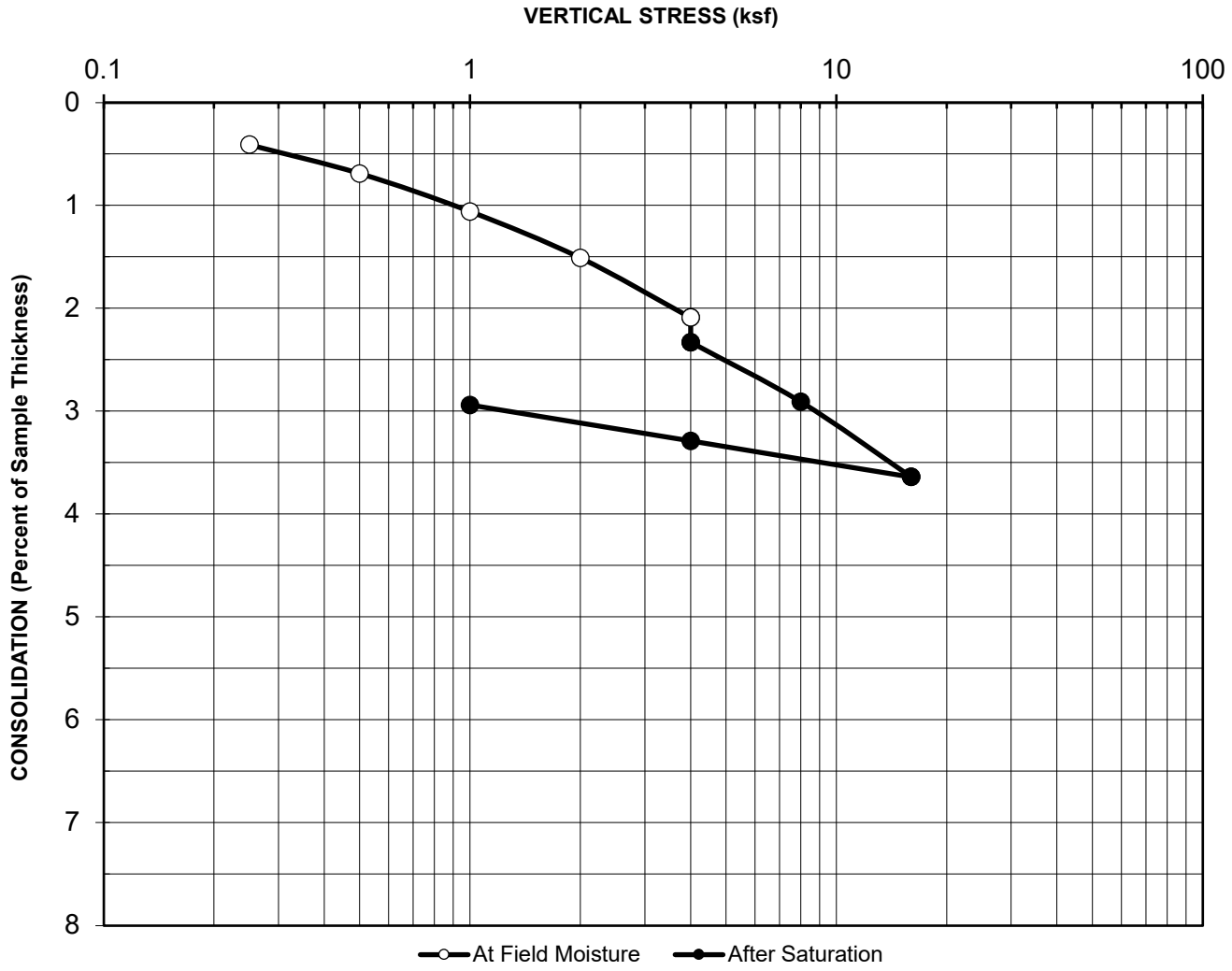


Boring No. :	B-4	Initial Dry Unit Weight (pcf):	98.6
Sample No.:	R-11	Initial Moisture Content (%):	1.7
Depth (feet):	35	Final Moisture Content (%):	16.7
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Sand w/silt & gravel	Initial Void Ratio:	0.71
Remarks:	Collapse= 0.72% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name:	Furman Park Stormwater Capture
Project No.:	CWT 22-06E
Date:	6/23/2022
AP No:	22-0659 **Sheet No:** 1

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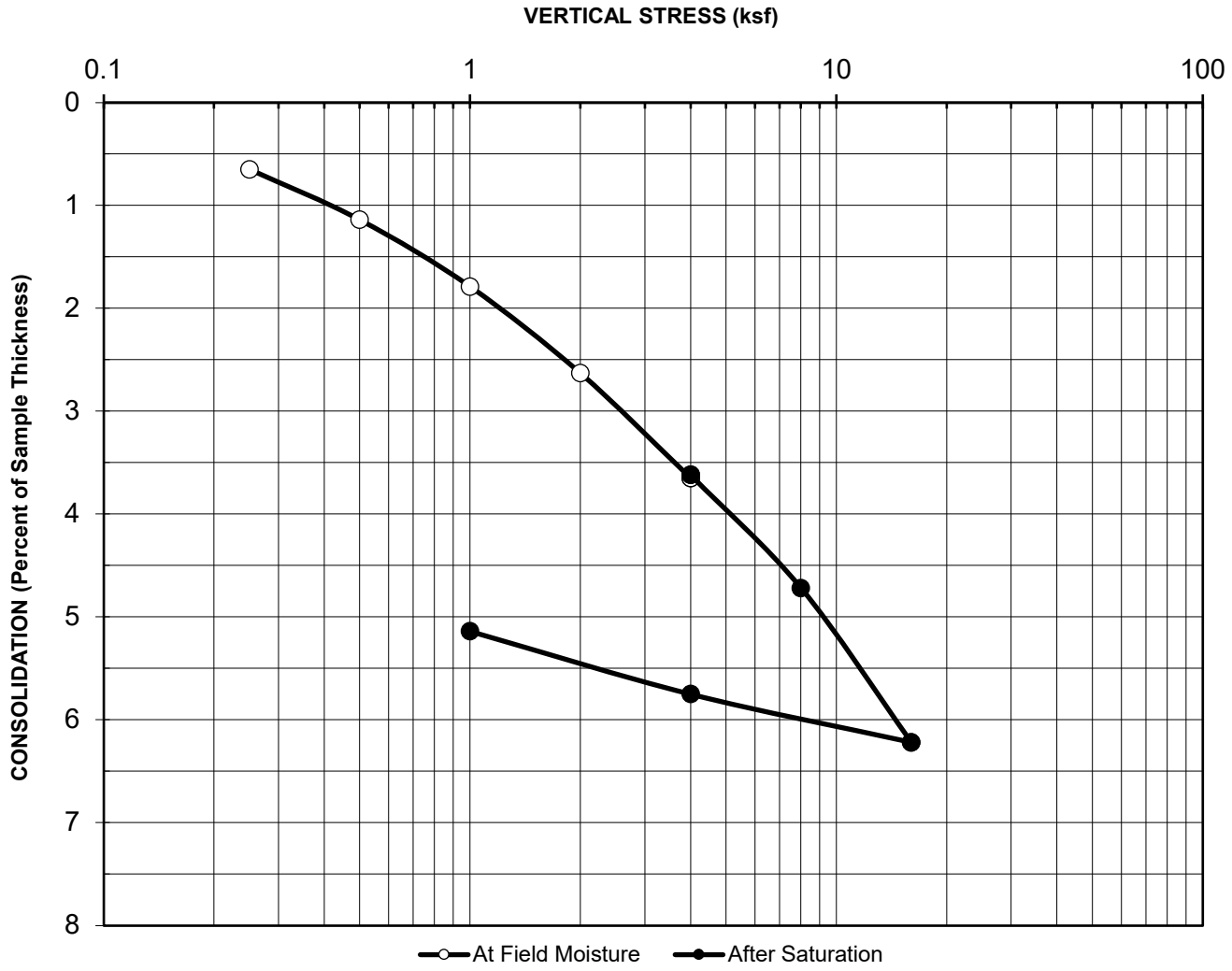


Boring No. :	B-5	Initial Dry Unit Weight (pcf):	89.5
Sample No.:	R-8	Initial Moisture Content (%):	1.7
Depth (feet):	20	Final Moisture Content (%):	22.5
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Poorly-Graded Sand	Initial Void Ratio:	0.88
Remarks:	Collapse= 0.24% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name:	Furman Park Stormwater Capture
Project No.:	CWT 22-06E
Date:	6/23/2022
AP No:	22-0659 **Sheet No:** 1

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Boring No. :	B-5	Initial Dry Unit Weight (pcf):	97.9
Sample No.:	R-10	Initial Moisture Content (%):	19.3
Depth (feet):	30	Final Moisture Content (%):	21.7
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Sandy Clay	Initial Void Ratio:	0.72
Remarks:	Swell= 0.03% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name:	Furman Park Stormwater Capture
Project No.:	CWT 22-06E
Date:	6/23/2022
AP No:	22-0659 **Sheet No:** 1

TETRA TECH

EXPANSION INDEX

ASTM D4829

Job Name:	Furman Park	**Date Sampled:**	6/17/2022
Job Number:	CWT-22-06E	**Date Completed:**	7/7/2022
Tested By:	MG	**Sample Identification:**	B-4 SPT-2
Note:		**Sample Depth:**	2.5ft
Sample Description:	Olive Gray SM, Native		

SAMPLE PROCESSING		
Percentage Passing #4 Sieve		
Total Air Dry Weight	*grams*	995.00
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 #		Z35	X27
Wet Weight of Soil + Tare	*grams*	94.8	427.0
Dry Weight of Soil + Tare	*grams*	86.7	346.7
Weight of Tare	*grams*	4.0	10.5
* Moisture Weight	*grams*	8.1	80.3
* Weight of Dry Soil	*grams*	82.7	336.2
***Moisture Content**	*%*	**9.8**	**23.9**

AFTER REMOLDING		INITIAL	FINAL
Weight of Ring and Sample	*grams*	604.4	604.4
Weight of Ring	*grams*	198.7	198.4
*Remolded Wet Weight	*grams*	405.7	406.0
*Wet Density	*pcf*	122.3	122.3
*Dry Density	*pcf*	111.4	98.8
Assumed/Measured Specific Gravity		2.7	2.7
***Degree of Saturation**	*%*	**51.6**	**91.3**

EXPANSION INDEX TEST			
DATE	**TIME**	**DIAL**	**Δ H %**
7/5/2022	11:30 AM	0.0140	0.0
7/5/2022	11:40 AM	0.0140	0.0
		0.0141	0.0
		0.0142	0.0
		0.0144	0.0
7/5/2022	11:30 AM	0.0145	0.0
7/5/2022	11:50 AM	0.0145	0.0
***Total**		**0.0145**	**0.0**

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 0

CORRECTED EXPANSION INDEX **1**

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

TETRA TECH

EXPANSION INDEX

ASTM D4829

Job Name:	Furman Park	**Date Sampled:**	6/17/2022
Job Number:	CWT-22-06E	**Date Completed:**	7/7/2022
Tested By:	MG	**Sample Identification:**	B-4 SPT-4
Note:		**Sample Depth:**	7.5ft
Sample Description:	Olive Gray ML, Native		

SAMPLE PROCESSING		
Percentage Passing #4 Sieve		
Total Air Dry Weight	*grams*	995.00
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 #		Z33	X27
Wet Weight of Soil + Tare	*grams*	103.2	423.5
Dry Weight of Soil + Tare	*grams*	93.8	339.7
Weight of Tare	*grams*	4.0	10.5
* Moisture Weight	*grams*	9.4	83.8
* Weight of Dry Soil	*grams*	89.8	329.2
*** Moisture Content**	*%*	**10.5**	**25.5**

AFTER REMOLDING		INITIAL	FINAL
Weight of Ring and Sample	*grams*	612.4	612.4
Weight of Ring	*grams*	202.2	202.2
*Remolded Wet Weight	*grams*	410.2	410.2
*Wet Density	*pcf*	123.7	123.6
*Dry Density	*pcf*	112.0	98.5
Assumed/Measured Specific Gravity		2.7	2.7
***Degree of Saturation**	*%*	**56.0**	**96.8**

EXPANSION INDEX TEST			
DATE	**TIME**	**DIAL**	**Δ H %**
7/5/2022	11:30 AM	0.0430	0.0
7/5/2022	11:40 AM	0.0430	0.0
		0.0431	0.0
		0.0431	0.0
		0.0435	0.0
7/5/2022	11:30 AM	0.0437	0.1
7/5/2022	11:50 AM	0.0437	0.1
***Total**		**0.0437**	**0.1**

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 1

CORRECTED EXPANSION INDEX **3**

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

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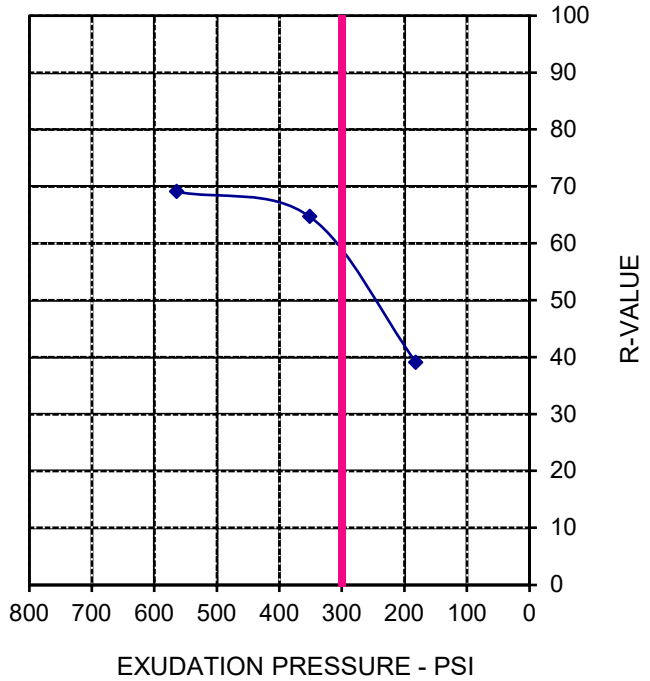
R-VALUE TEST DATA

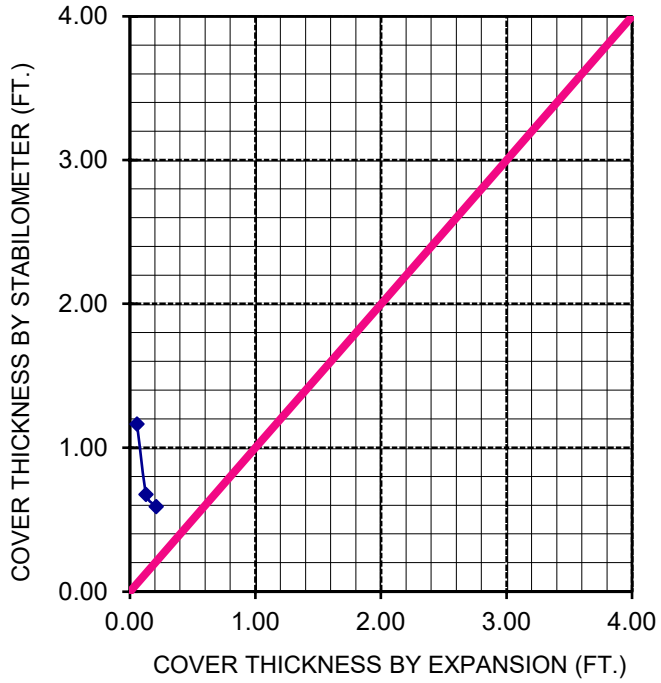
ASTM D2844

Project Name:	Furman Park Stormwater Capture		Tested By:	ST	Date: 06/23/22
Project Number:	CWT 22-06E		Computed By:	KM	Date: 06/24/22
Boring No.:	B-6		Checked By:	AP	Date: 06/29/22
Sample No.:	SK-1	Depth (ft.): 0			
Location:	N/A				
Soil Description:	Silty Sand, fine-grained				

Mold Number	C	A	B	
Water Added, g	21	31	44	
Compact Moisture(%)	13.5	14.5	15.8	
Compaction Gage Pressure, psi	250	250	250	
Exudation Pressure, psi	564	351	182	
Sample Height, Inches	2.4	2.4	2.4	
Gross Weight Mold, g	2979	2989	2987	
Tare Weight Mold, g	1968	1967	1966	
Net Sample Weight, g	1011	1023	1021	
Expansion, inchesx10^{-4}	63	39	17	
Stability 2,000 (160 psi)	16/27	17/32	32/55	
Turns Displacement	5.06	4.97	6.59	
R-Value Uncorrected	71	67	42	
R-Value Corrected	69	65	39	
Dry Density, pcf	112.4	112.8	111.3	
Traffic Index	8.0	8.0	8.0	
G.E. by Stability	0.59	0.67	1.16	
G.E. by Expansion	0.21	0.13	0.06	

R-VALUE		
	By Exudation:	59
	By Expansion:	*N/A
	At Equilibrium: (by Exudation)	59
Remarks	Gf = 1.34, and 0.0 % Retained on the ¾" *Not Applicable	





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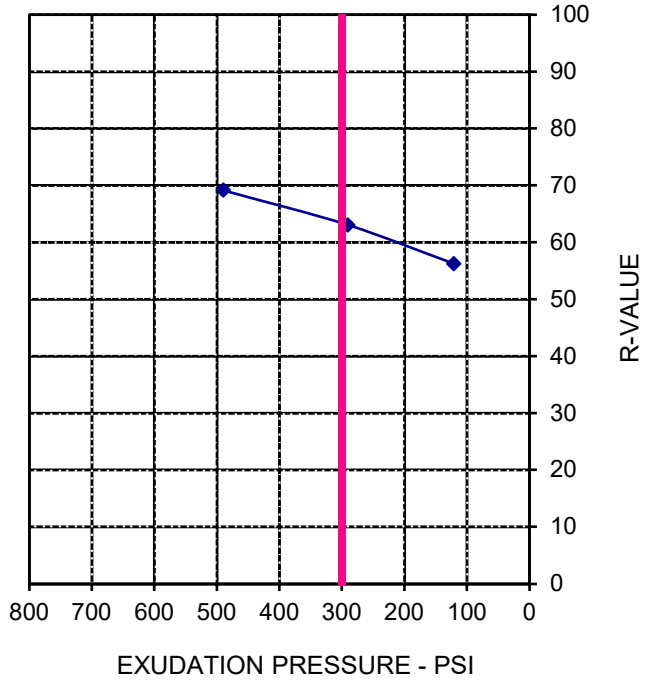
R-VALUE TEST DATA

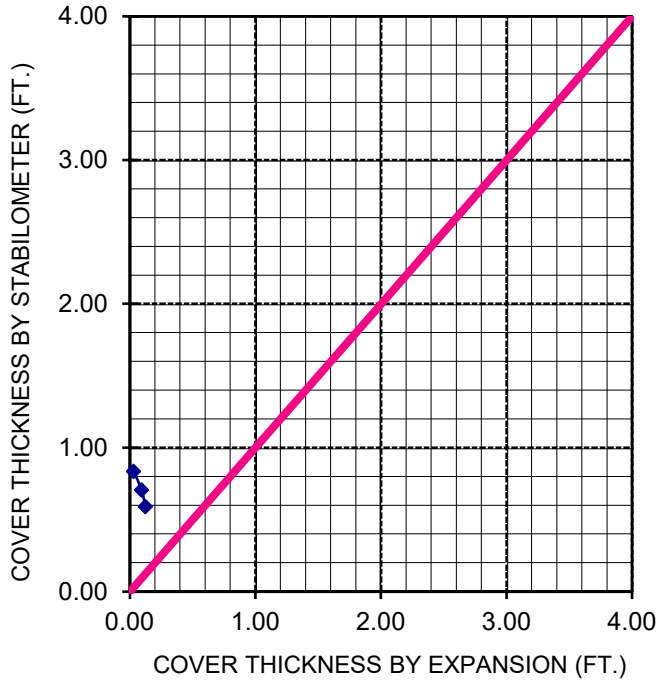
ASTM D2844

Project Name:	Furman Park Stormwater Capture		Tested By:	ST	Date: 06/23/22
Project Number:	CWT 22-06E		Computed By:	KM	Date: 06/24/22
Boring No.:	B-1		Checked By:	AP	Date: 06/29/22
Sample No.:	SK-1	Depth (ft.): 0			
Location:	N/A				
Soil Description:	Silty Sand, fine-grained				

Mold Number	D	E	F	
Water Added, g	50	26	11	
Compact Moisture(%)	13.4	10.9	9.4	
Compaction Gage Pressure, psi	250	250	250	
Exudation Pressure, psi	121	290	490	
Sample Height, Inches	2.6	2.6	2.6	
Gross Weight Mold, g	3054	3043	2958	
Tare Weight Mold, g	1964	1955	1869	
Net Sample Weight, g	1090	1088	1089	
Expansion, inchesx10^{-4}	9	28	37	
Stability 2,000 (160 psi)	24/46	19/38	15/30	
Turns Displacement	5.36	5.13	5.36	
R-Value Uncorrected	54	61	67	
R-Value Corrected	56	63	69	
Dry Density, pcf	112.0	114.3	116.0	
Traffic Index	8.0	8.0	8.0	
G.E. by Stability	0.84	0.71	0.59	
G.E. by Expansion	0.03	0.09	0.12	

R-VALUE		
	By Exudation:	63
	By Expansion:	*N/A
	At Equilibrium: (by Exudation)	63
Remarks	Gf = 1.34, and 0.0 % Retained on the ¾" *Not Applicable	





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CORROSION TEST RESULTS

Client Name: Tetra Tech
Project Name: Furman Park Stormwater Capture
Project No.: CWT 22-06E

AP Job No.: 22-0659
Date: 06/27/22

Boring No.	Sample No.	Depth (feet)	Soil Description	Minimum Resistivity (ohm-cm)	pH	Sulfate Content (ppm)	Chloride Content (ppm)
B-3	R-1	2.5	Sandy Silt	504	8.4	1637	512
B-4	SK-1	0-5	Sandy Silt	2,326	8.5	95	69
B-6	SK-1	0	Silty Sand, fine-grained	4,026	9.2	60	41

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

Results of Infiltration Testing

TETRA TECH

PERCOLATION DATA SHEET

Project:	Furman Park BMP - Downey		Job No:	CWT22-06E	
Test Hole No:	**P-1**		Date Excavated:	Thursday, June 2, 2022	
Test Hole Depth (feet):	22.1	feet	Soil Classification:	SP	
Stick up Length (feet):	0.4	foot	Sand Porosity:	0.30	
Sandy Soil Criteria Tested By:	SMS		Presoak Date:	Wednesday, June 1, 2022	Presoak: 02:39
Actual Percolation Tested By:	TA		Test Date:	Thursday, June 2, 2022	
Test Hole Diameter (inches):	8	inches	Latitude:	33.9544	Longitude: -118.1375
Casing Diameter (inches):	2	inches	Elevation:	119 feet	

PERCOLATION TEST (presoaking for at least 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)
8:35 AM	5	20.80	Empty	1.70	-	-	-
8:40 AM							
8:55 AM	10	20.80	Empty	1.70	-	-	-
9:05 AM							
11:43 AM	10	20.50	21.80	2.00	0.70	15.60	3.536
11:53 AM							
11:56 AM	10	20.50	21.78	2.00	0.72	15.36	3.459
12:06 PM							
12:08 PM	10	20.45	21.79	2.05	0.71	16.08	3.574
12:18 PM							
12:20 PM	10	20.50	21.80	2.00	0.70	15.60	3.536
12:30 PM							
12:32 PM	10	20.55	21.87	1.95	0.63	15.84	3.738
12:42 PM							
12:44 PM	10	20.50	21.82	2.00	0.68	15.84	3.614
12:54 PM							

TETRA TECH

PERCOLATION DATA SHEET

Project:	Furman Park BMP - Downey		Job No:	CWT22-06E	
Test Hole No:	**P-2**		Date Excavated:	Thursday, June 2, 2022	
Test Hole Depth (feet):	19.8	feet	Soil Classification:	SP	
Stick up Length (feet):	0.3	foot	Gravel Porosity:	0.35	
Sandy Soil Criteria Tested By:	SMS		Presoak Date:	Wednesday, June 1, 2022	Presoak: 03:39
Actual Percolation Tested By:	TA		Test Date:	Thursday, June 2, 2022	
Test Hole Diameter (inches):	8	inches	Latitude:	33.9540	Longitude: -118.1375
Casing Diameter (inches):	2	inches	Elevation:	118 feet	

PERCOLATION TEST (presoaking for at least 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)
8:10 AM 8:15 AM	5	18.10	Empty	2.00	-	-	-
10:39 AM 10:42 AM	3	18.00	Empty	2.10	-	-	-
10:45 AM 10:49 AM	4	18.25	Empty	1.85	-	-	-
10:50 AM 11:00 AM	10	18.25	19.90	1.85	0.20	19.80	6.490
11:02 AM 11:12 AM	10	18.10	19.70	2.00	0.40	19.20	5.488
11:13 AM 11:23 AM	10	18.00	19.74	2.10	0.36	20.88	5.840
11:24 AM 11:34 AM	10	18.10	19.79	2.00	0.31	20.28	5.994
11:35 AM 11:45 AM	10	18.05	19.77	2.05	0.33	20.64	5.943
11:46 AM 11:56 PM	10	18.10	19.79	2.00	0.31	20.28	5.994

Reduction Factor Method (1 to 3)	2.00
Reduction Factor Site Variability (1 to 3)	1.50
Reduction Factor Long Term Siltation (1 to 3)	2.00

Observed short-term infiltration rate (in/hr)= 5.977

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

Corrected Percolation Rate = 1.09 in/hour

TETRA TECH

PERCOLATION DATA SHEET

Project:	Furman Park BMP - Downey		Job No:	CWT22-06E	
Test Hole No:	**P-3**		Date Excavated:	Thursday, June 2, 2022	
Test Hole Depth (feet):	19.8	feet	Soil Classification:	SP	
Stick up Length (feet):	0.4	foot	Gravel Porosity:	0.35	
Sandy Soil Criteria Tested By:	SMS		Presoak Date:	Wednesday, June 1, 2022	Presoak: 11:45
Actual Percolation Tested By:	TA		Test Date:	Thursday, June 2, 2022	
Test Hole Diameter (inches):	8	inches	Latitude:	33.9539	Longitude: -118.1369
Casing Diameter (inches):	2	inches	Elevation:	119 feet	

PERCOLATION TEST (presoaking for at least 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)
8:02 AM 8:04 AM	2	18.40	Empty	1.80	-	-	-
12:22 PM 12:32 PM	10	15.95	16.92	4.25	3.28	11.64	1.156
12:33 PM 12:43 PM	10	17.01	17.94	3.19	2.26	11.16	1.508
12:44 PM 12:54 PM	10	17.98	18.75	2.22	1.45	9.24	1.803
12:57 PM 1:07 PM	10	18.00	18.72	2.20	1.48	8.64	1.682
1:11 PM 1:21 PM	10	17.80	18.57	2.40	1.63	9.24	1.654
1:22 PM 1:32 PM	10	18.00	18.73	2.20	1.47	8.76	1.710
1:33 PM 1:43 PM	10	18.01	18.76	2.19	1.44	9.00	1.774
1:45 PM 1:55 PM	10	18.00	18.75	2.20	1.45	9.00	1.765

Reduction Factor Method (1 to 3)	2.00	Observed short-term infiltration rate (in/hr)=	1.750
Reduction Factor Site Variability (1 to 3)	1.50		
Reduction Factor Long Term Siltation (1 to 3)	2.00		

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

Corrected Percolation Rate = 0.32 in/hour

Appendix D

Seismic Demand

Furman Park

Latitude, Longitude: 33.954076, -118.137236



Date	6/22/2022, 3:10:42 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.742	MCE_R ground motion. (for 0.2 second period)
S_1	0.624	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.742	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.161	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	0.747	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.822	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.742	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.923	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.372	Factored deterministic acceleration value. (0.2 second)
S1RT	0.624	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.692	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.794	Factored deterministic acceleration value. (1.0 second)
PGAd	0.957	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.906	Mapped value of the risk coefficient at short periods
C_{R1}	0.902	Mapped value of the risk coefficient at a period of 1 s

U.S. Geological Survey - Earthquake Hazards Program

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 (up…

Spectral Period

Peak Ground Acceleration

Latitude

Decimal degrees

33.954076

Time Horizon

Return period in years

2475

Longitude

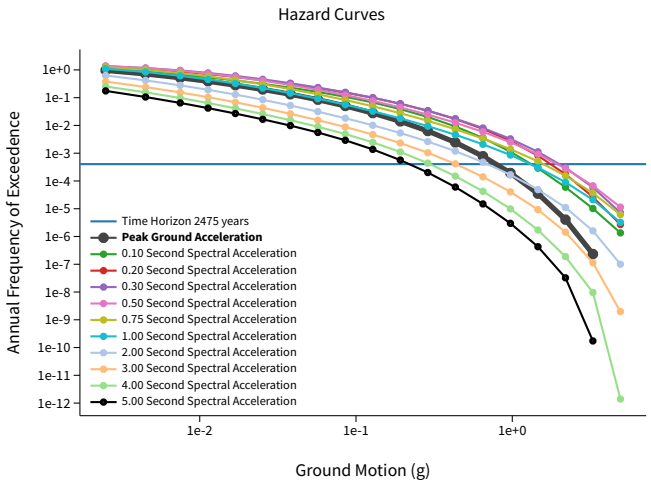
Decimal degrees, negative values for western longitudes

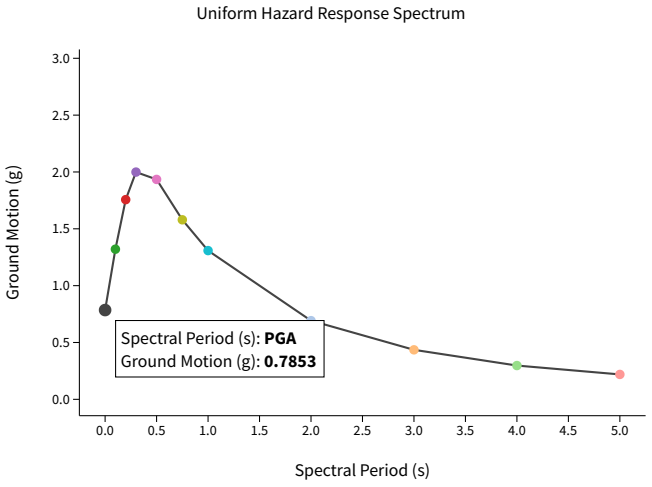
-118.137236

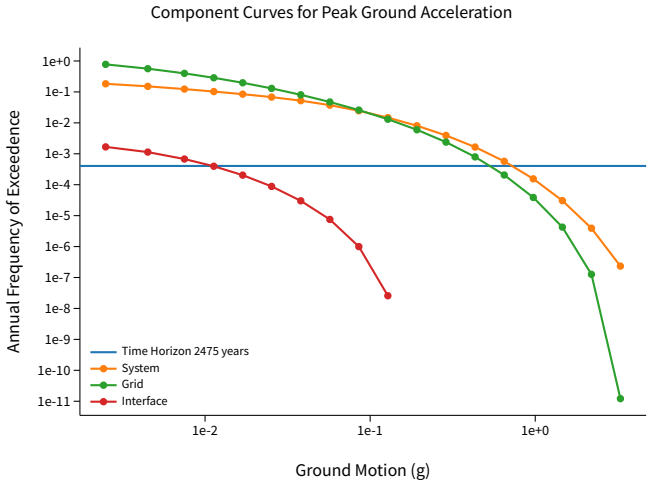
Site Class

259 m/s (Site class D)

Hazard Curve





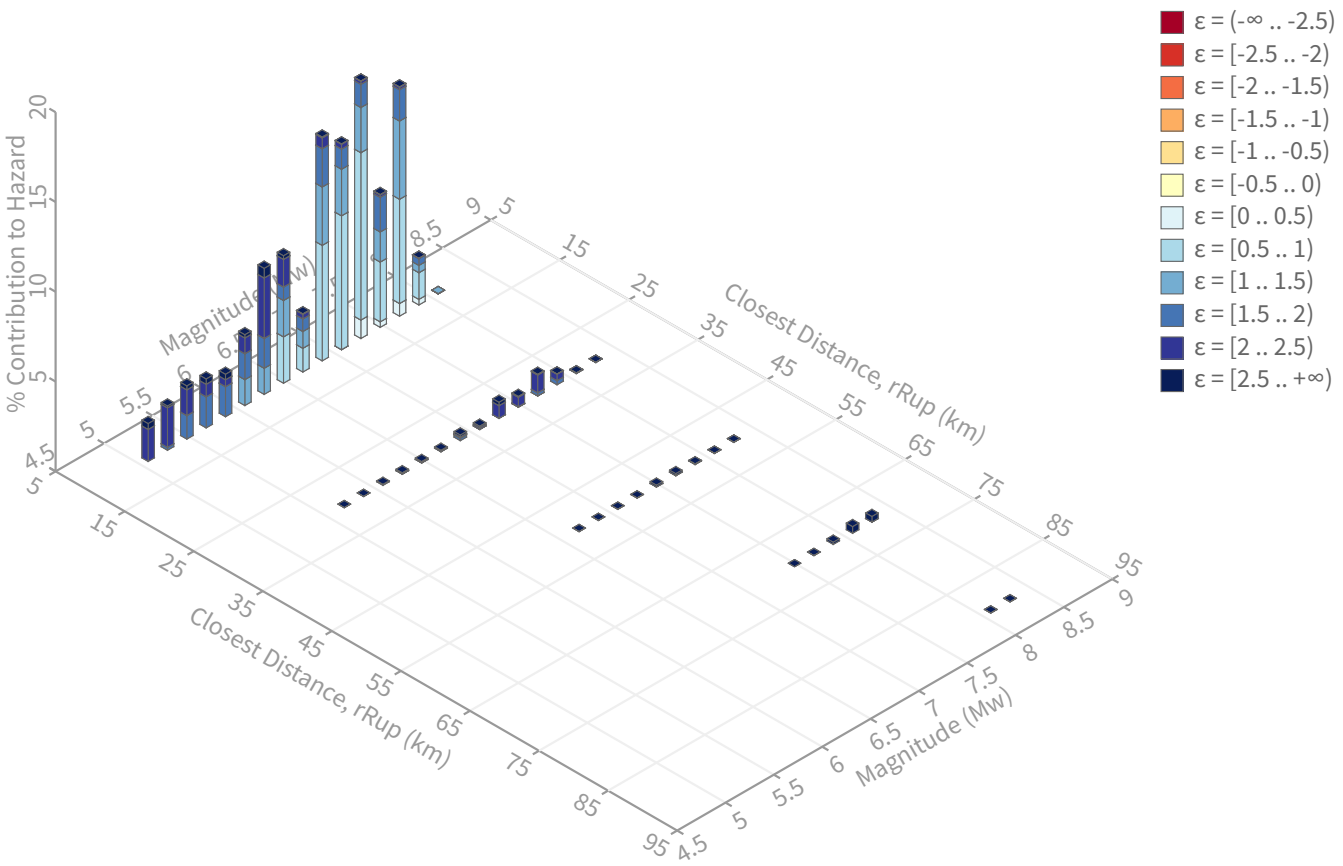


View Raw Data

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.78528676 g

Recovered targets

Return period: 2926.2788 yrs
Exceedance rate: 0.00034173094 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.08 %

Mean (over all sources)

m: 6.91
r: 10.85 km
ε_0: 1.35 σ

Mode (largest m-r bin)

m: 7.3
r: 10.41 km
ε_0: 0.89 σ
Contribution: 14.33 %

Mode (largest m-r-ε_0 bin)

m: 7.29
r: 10.06 km
ε_0: 0.71 σ
Contribution: 9.32 %

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							42.37
Puente Hills (Santa Fe Springs) [1]		4.96	7.01	0.82	118.134°W	33.928°N	173.57	11.12
Compton [1]		12.66	7.30	0.76	118.247°W	33.801°N	210.82	7.25
Puente Hills (LA) [0]		4.98	7.17	0.95	118.148°W	33.986°N	343.72	6.75
Compton [2]		13.09	7.47	0.93	118.286°W	33.817°N	222.02	2.88
Puente Hills (Coyote Hills) [1]		10.13	7.35	1.34	118.044°W	33.915°N	117.14	1.91
Puente Hills (Santa Fe Springs) [0]		6.71	7.74	0.91	118.083°W	33.935°N	112.47	1.71
Whittier alt 2 [6]		9.81	7.12	1.59	118.046°W	33.998°N	59.68	1.62
Elysian Park (Upper) [0]		13.60	6.83	1.94	118.127°W	34.070°N	4.17	1.18
Anaheim [2]		11.74	7.25	1.03	118.063°W	33.881°N	139.84	1.14
Elysian Park (Upper) [1]		14.11	7.17	1.80	118.169°W	34.072°N	347.62	1.09
UC33brAvg_FM31	System							35.65
Puente Hills [2]		6.01	7.32	0.90	118.134°W	33.968°N	11.42	7.50
Compton [1]		12.66	7.25	0.77	118.247°W	33.801°N	210.82	6.97
Compton [2]		13.09	7.51	0.92	118.286°W	33.817°N	222.02	3.33
Whittier alt 1 [7]		9.84	6.93	1.67	118.046°W	33.999°N	59.43	3.25
Puente Hills [3]		6.17	6.82	1.05	118.143°W	33.972°N	346.04	2.93
Elysian Park (Upper) [0]		13.60	6.52	2.11	118.127°W	34.070°N	4.17	2.11
Newport-Inglewood alt 1 [6]		14.07	7.33	1.70	118.265°W	33.886°N	237.49	1.25
Anaheim [2]		11.74	7.20	1.05	118.063°W	33.881°N	139.84	1.10
UC33brAvg_FM32 (opt)	Grid							11.31
PointSourceFinite: -118.137, 34.013		7.94	5.75	1.78	118.137°W	34.013°N	0.00	2.50
PointSourceFinite: -118.137, 34.013		7.94	5.75	1.78	118.137°W	34.013°N	0.00	2.50
PointSourceFinite: -118.137, 34.022		8.45	5.84	1.81	118.137°W	34.022°N	0.00	1.38
PointSourceFinite: -118.137, 34.022		8.45	5.84	1.81	118.137°W	34.022°N	0.00	1.38
UC33brAvg_FM31 (opt)	Grid							10.66
PointSourceFinite: -118.137, 34.013		7.95	5.74	1.78	118.137°W	34.013°N	0.00	2.41
PointSourceFinite: -118.137, 34.013		7.95	5.74	1.78	118.137°W	34.013°N	0.00	2.41
PointSourceFinite: -118.137, 34.022		8.49	5.82	1.83	118.137°W	34.022°N	0.00	1.21
PointSourceFinite: -118.137, 34.022		8.49	5.82	1.83	118.137°W	34.022°N	0.00	1.21

Appendix E

Liquefaction Analyses

Summary of Liquefaction and Earthquake-Induced Settlement Analysis

Project:	Furman Park	Boring:	**B-3**	Engineer:	FC	Date:	6/30/2022

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 Kσ	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	51.5	feet
Profile thickness susceptible to liquefaction	22.5	feet
Number of evaluated intervals	15	
Number of potentially liquefiable intervals	13	
Average Factor of Safety of sandy intervals	**1.87**	
Dry sand settlement	0.00	inches
Liquefaction settlement	5.34	inches
Total earthquake-induced settlement	**5.34**	**inches**

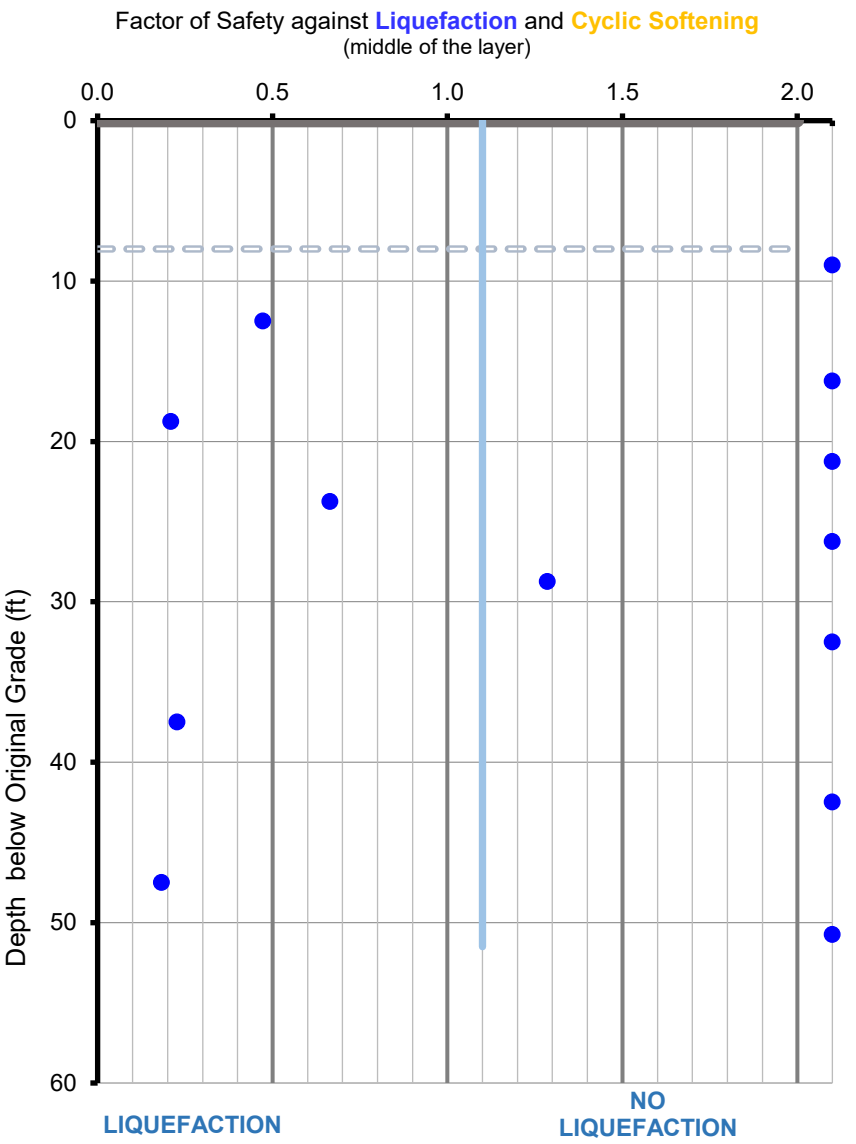
Profile			Earthquake loading	
In-Situ Groundwater depth	no GW	feet	M	7.3
DESIGN Groundwater depth	8.00	feet	PGA	0.822
DESIGN Excavation depth	0.00	feet		
DESIGN Surcharge (fill)	0.00	feet		

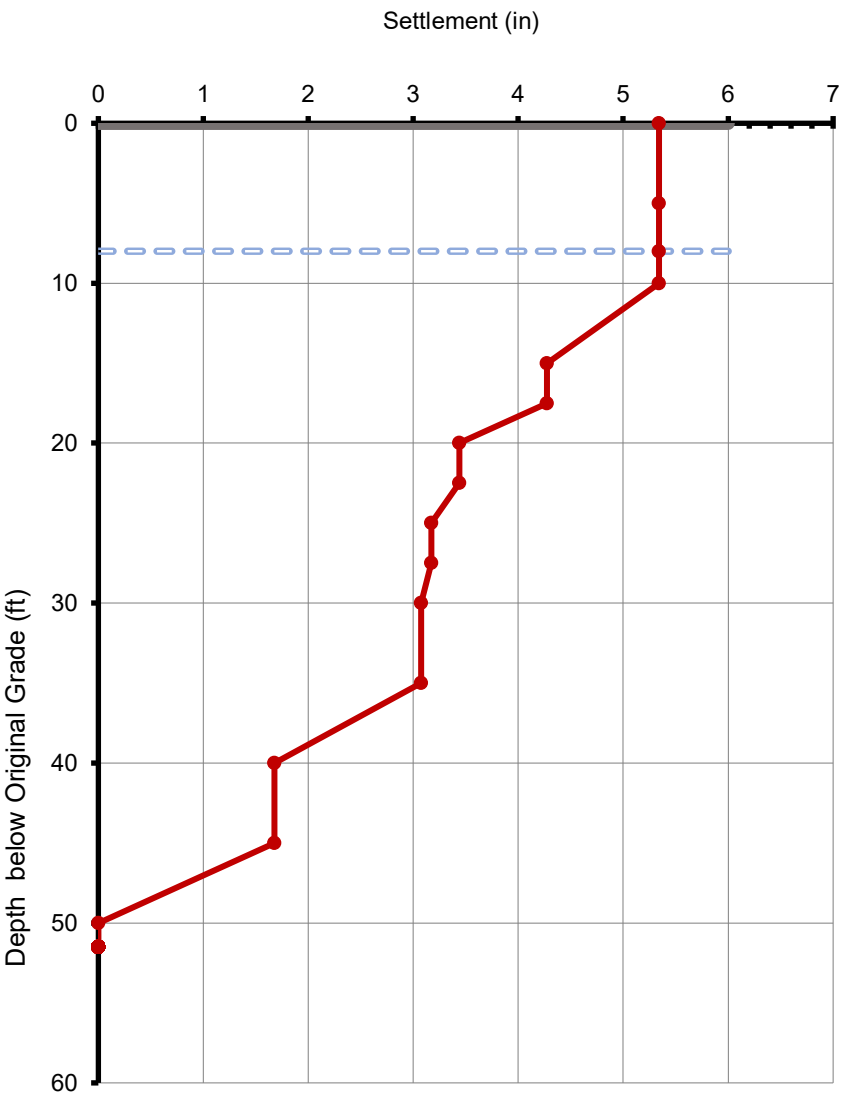
Checks	
Groundwater depth check	OK
Design groundwater/excavation depth check	OK
Fines correction method compatibility	OK
Idris & Boulanger, 2004 method for C_N	not used
Cetin 2009 settlement method	not used

Version v2 2022-03

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	126.5	121.0	38	n/plastic	26.0	59.7	65.2	–	–	- no groundwater	0.00	5.34
5	3	126.5	120.0	35	n/plastic	8.7	17.3	22.8	–	–	- no groundwater	0.00	5.34
8	2	126.5	120.0	5	n/plastic	28.0	45.3	45.3	4.28	–	- too dense - (N1)60,CS > 32	0.00	5.34
10	5	126.5	120.0	35	n/plastic	10.7	18.4	23.9	0.47	–	- liquefieable (FS < 1.1)	1.07	5.34
15	2.5	126.5	120.0	34	n/plastic	38.0	58.4	63.9	3.43	–	- too dense - (N1)60,CS > 32	0.00	4.27
17.5	2.5	126.5	120.0	34	n/plastic	5.0	8.0	13.5	0.21	–	- liquefieable (FS < 1.1)	0.83	4.27
20	2.5	126.5	120.0	34	n/plastic	23.0	36.5	41.9	3.09	–	- too dense - (N1)60,CS > 32	0.00	3.44
22.5	2.5	126.5	120.0	35	n/plastic	16.1	24.1	29.6	0.66	–	- liquefieable (FS < 1.1)	0.27	3.44
25	2.5	126.5	120.0	38	n/plastic	26.0	38.7	44.2	2.86	–	- too dense - (N1)60,CS > 32	0.00	3.17
27.5	2.5	126.5	120.0	38	n/plastic	20.1	28.5	34.1	1.29	–	- too dense - (N1)60,CS > 32	0.10	3.17
30	5	126.5	120.0	38	n/plastic	34.0	49.9	55.5	2.68	–	- too dense - (N1)60,CS > 32	0.00	3.07
35	5	126.5	120.0	5	n/plastic	14.1	17.3	17.3	0.23	–	- liquefieable (FS < 1.1)	1.40	3.07
40	5	126.5	120.0	5	n/plastic	52.0	70.5	70.5	2.52	–	- too dense - (N1)60,CS > 32	0.00	1.68
45	5	126.5	120.0	12	n/plastic	10.7	11.3	13.4	0.18	–	- liquefieable (FS < 1.1)	1.68	1.68
50	1.5	126.5	120.0	5	n/plastic	47.0	60.4	60.4	2.45	–	- too dense - (N1)60,CS > 32	0.00	0.00

B-3		
	Design excavation depth	0.00 feet
	Design groundwater depth	8.00 feet





Summary of Liquefaction and Earthquake-Induced Settlement Analysis

Project:	Furman Park	Boring:	**B-4**	Engineer:	FC	Date:	6/30/2022

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 Kσ	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	56.5	feet
Profile thickness susceptible to liquefaction	21	feet
Number of evaluated intervals	14	
Number of potentially liquefiable intervals	12	
Average Factor of Safety of sandy intervals	**1.87**	
Dry sand settlement	0.00	inches
Liquefaction settlement	2.03	inches
Total earthquake-induced settlement	**2.03**	**inches**

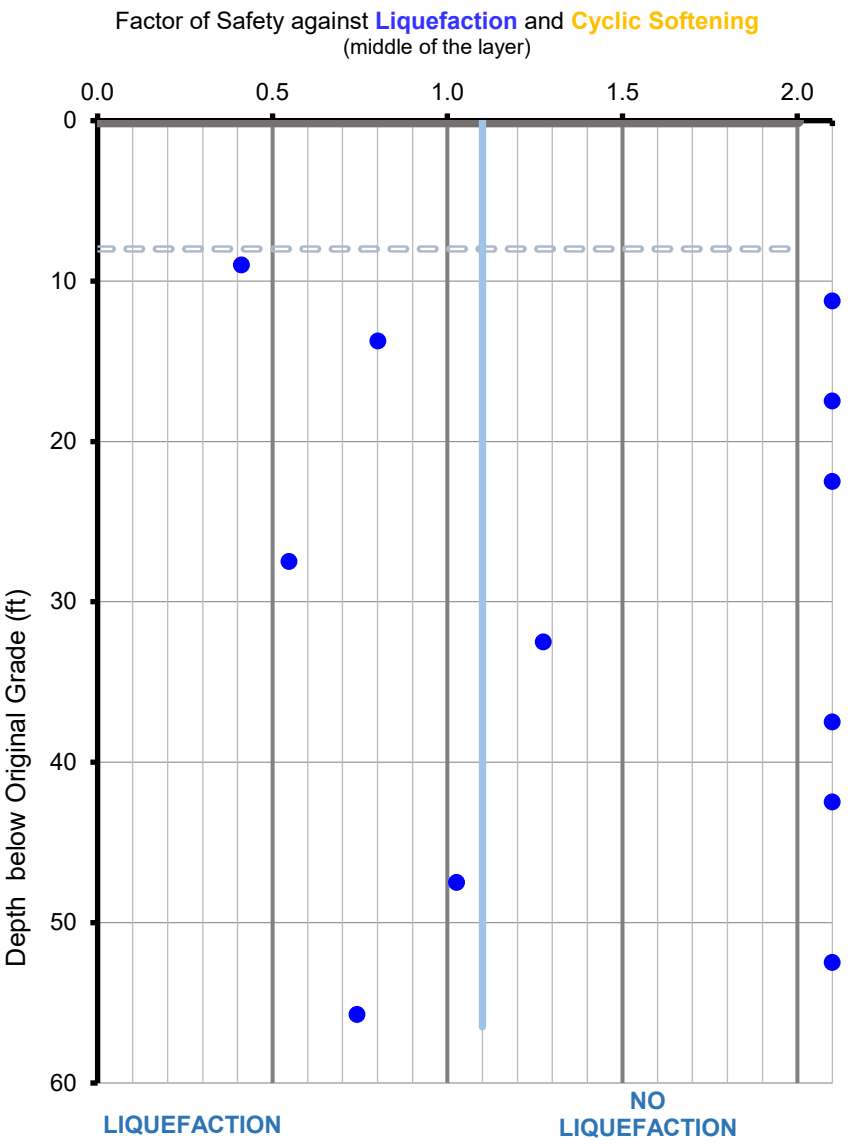
Profile			Earthquake loading	
In-Situ Groundwater depth	no GW	feet	M	7.3
DESIGN Groundwater depth	8.00	feet	PGA	0.822
DESIGN Excavation depth	0.00	feet		
DESIGN Surcharge (fill)	0.00	feet		

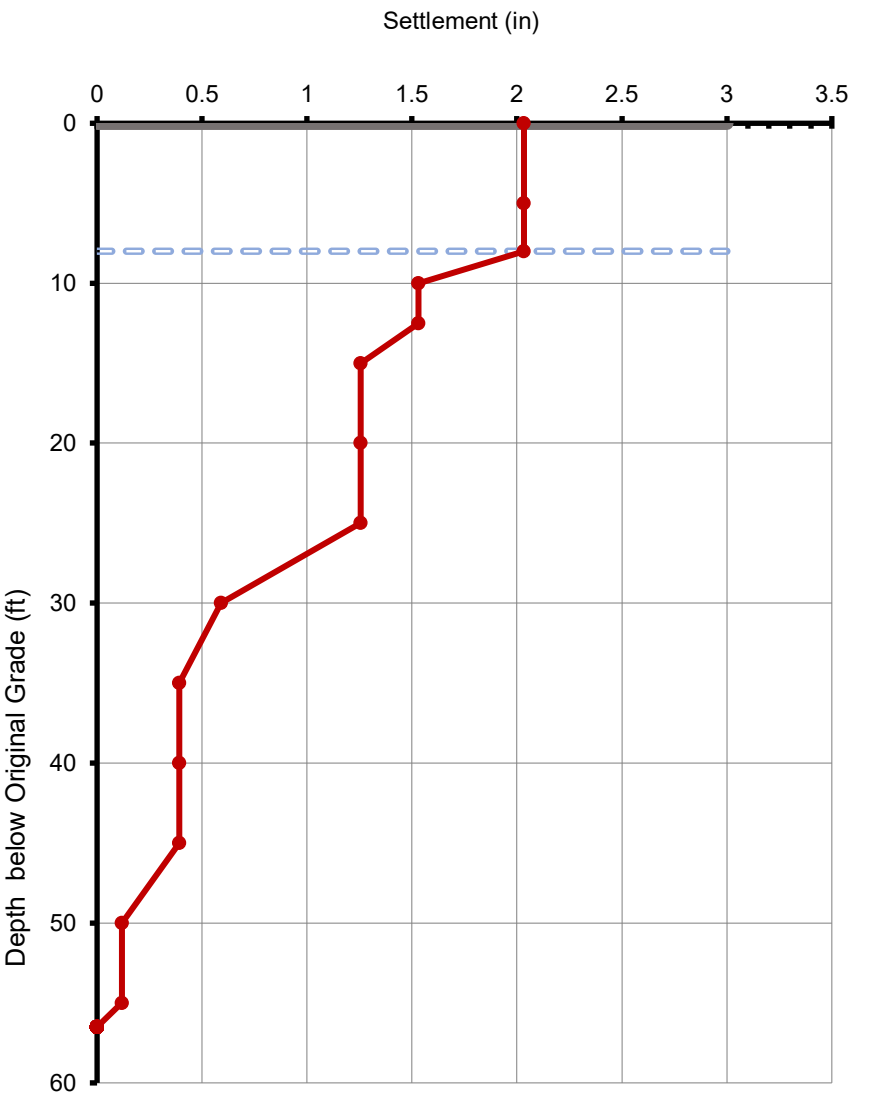
Checks	
Groundwater depth check	OK
Design groundwater/excavation depth check	OK
Fines correction method compatibility	OK
Idris & Boulanger, 2004 method for C_N	not used
Cetin 2009 settlement method	not used

Version v2 2022-03

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	126.5	121.0	35	n/plastic	8.0	18.4	23.9	–	–	- no groundwater	0.00	2.03
5	3	126.5	120.0	35	n/plastic	19.4	34.7	40.2	–	–	- no groundwater	0.00	2.03
8	2	126.5	120.0	65	10	8.0	14.1	19.7	0.41	–	- liquefieable (FS < 1.1)	0.50	2.03
10	2.5	126.5	120.0	65	10	20.8	35.7	41.2	3.89	–	- too dense - (N1)60,CS > 32	0.00	1.53
12.5	2.5	126.5	120.0	5	n/plastic	18.0	29.5	29.5	0.80	–	- liquefieable (FS < 1.1)	0.27	1.53
15	5	126.5	120.0	5	n/plastic	24.1	40.5	40.5	3.35	–	- too dense - (N1)60,CS > 32	0.00	1.26
20	5	126.5	120.0	5	n/plastic	24.0	37.2	37.2	2.80	–	- too dense - (N1)60,CS > 32	0.00	1.26
25	5	126.5	120.0	65	7	16.1	22.7	28.3	0.55	–	- liquefieable (FS < 1.1)	0.67	1.26
30	5	126.5	120.0	35	n/plastic	20.0	28.7	34.2	1.27	–	- too dense - (N1)60,CS > 32	0.20	0.59
35	5	126.5	120.0	35	n/plastic	46.9	66.0	71.5	2.58	–	- too dense - (N1)60,CS > 32	0.00	0.39
40	5	126.5	120.0	35	n/plastic	40.0	54.2	59.7	2.52	–	- too dense - (N1)60,CS > 32	0.00	0.39
45	5	126.5	120.0	65	7	22.1	27.8	33.4	1.03	–	- too dense - (N1)60,CS > 32	0.27	0.39
50	5	126.5	120.0	5	n/plastic	38.0	48.3	48.3	2.44	–	- too dense - (N1)60,CS > 32	0.00	0.12
55	1.5	126.5	120.0	5	n/plastic	26.8	31.4	31.4	0.74	–	- liquefieable (FS < 1.1)	0.12	0.12

B-4		
	Design excavation depth	0.00 feet
	Design groundwater depth	8.00 feet





Summary of Liquefaction and Earthquake-Induced Settlement Analysis

Project:	Furman Park	Boring:	**B-5**	Engineer:	FC	Date:	6/30/2022

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 Kσ	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	35	feet
Profile thickness susceptible to liquefaction	9.5	feet
Number of evaluated intervals	9	
Number of potentially liquefiable intervals	7	
Average Factor of Safety of sandy intervals	**1.93**	
Dry sand settlement	0.00	inches
Liquefaction settlement	1.83	inches
Total earthquake-induced settlement	**1.83**	**inches**

Profile			Earthquake loading	
In-Situ Groundwater depth	no GW	feet	M	7.3
DESIGN Groundwater depth	8.00	feet	PGA	0.822
DESIGN Excavation depth	0.00	feet		
DESIGN Surcharge (fill)	0.00	feet		

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

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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	126.5	121.0	35	n/plastic	4.7	10.8	16.3	–	–	- no groundwater	0.00	1.83
5	3	126.5	120.0	35	n/plastic	7.0	14.3	19.8	–	–	- no groundwater	0.00	1.83
8	2	126.5	120.0	5	n/plastic	12.1	21.1	21.1	0.45	–	- liquefieable (FS < 1.1)	0.47	1.83
10	2.5	126.5	120.0	5	n/plastic	23.0	39.5	39.5	3.89	–	- too dense - (N1)60,CS > 32	0.00	1.36
12.5	2.5	126.5	120.0	5	n/plastic	19.4	31.7	31.7	1.11	–	- NOT liquefiable (FS > 1.1)	0.15	1.36
15	5	126.5	120.0	5	n/plastic	21.0	35.2	35.2	1.93	–	- too dense - (N1)60,CS > 32	0.01	1.21
20	5	126.5	120.0	5	n/plastic	29.5	45.9	45.9	3.02	–	- too dense - (N1)60,CS > 32	0.00	1.20
25	5	126.5	120.0	5	n/plastic	26.0	37.9	37.9	2.82	–	- too dense - (N1)60,CS > 32	0.00	1.20
30	5	126.5	120.0	53	10	11.4	15.3	20.9	0.29	–	- liquefieable (FS < 1.1)	1.20	1.20

B-5		
	Design excavation depth	0.00 feet
	Design groundwater depth	8.00 feet

