Compton Community College District

Serving the Communities of
Compton, Lynwood, Paramount and Willowbrook, as well as portions of Athens, Bellflower, Carson, Downey, Dominguez, Lakewood, Long Beach, and South Gate

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April 5, 2022

Hon. Toni Atkins, President Pro Tem
California Senate
1021 O Street, Suite 8518
Sacramento, CA 95814

Hon. Anthony Rendon, Speaker
California Assembly
State Capitol, Room 219
Sacramento, CA 95814

Senator Nancy Skinner, Chair
Senate Budget Committee
1021 O Street, Suite 8630
Sacramento, CA 95814

Assemblymember Phil Ting, Chair
Assembly Budget Committee
P.O. Box 942849
Sacramento, CA 94249-0019

Re: Compton College Physical Education Complex Replacement Project Funding Request - Soil Mitigation Requirements

Dear Honorable Toni Atkins, Honorable Anthony Rendon, Honorable Nancy Skinner, and Honorable Phil Ting,

The Compton College Physical Education Complex Replacement Project involves the replacement of the existing physical education facilities with appropriate space to support modern instruction and learning methodologies.

The existing physical education facilities include the men's shower/locker and special services buildings constructed in 1953. The gymnasium and pool service buildings were built in the early 1960s. There has been no comprehensive renovation of these buildings. The facilities are currently configured as 'make-shift' instructional spaces. The women's shower and locker areas are locked and unused because the systems have failed, and the facility is inadequate to support any campus function. Third-party engineering evaluations indicated that mechanical, electrical, and plumbing systems are dying, and structural and life/safety systems do not conform to current standards.

The Compton College current gymnasium is used heavily by students and community members. This facility is a critical resource in the Compton community. The State approved the replacement of the facility with a budget of $45,576,000.00 ($23,082,000.00 State-funded - $22,494,000.00 District funded).

However, in the summer of 2021, during the development of the structural design and geotechnical investigation report, Compton College received notification from structural and geotechnical engineers that there was a soil issue under the Physical Education Complex Replacement project footprint, which has resulted in an increased cost of $5,800,000, not including inflation. The Geotechnical Investigation Report for Compton Community College District is dated July 7, 2021.

It was determined that a significant amount of soil mitigation needs to be done underneath the foundation of the new building and pool. Due to several conditions, including the new 2019 California Building Code requirements and the state of the soil under the location for the new gymnasium and pool, a much more rigorous and expensive method of soil mitigation, need to be done. When this project was submitted to the California Community Colleges Chancellor's Office, the 2019 California Building Code was not in place. The costs for this soil mitigation were not included in the original project budget.

Compton College requested additional funds for the project from the California Community Colleges Chancellor's Office on October 27, 2021, and the request was denied on November 19, 2021, due to funding.

Therefore, Compton College requests additional funding from the State from Proposition 98 funds of $5.8 million, not including inflation. The estimated cost includes the installation of stone columns under the gymnasium and deep soil mixing under the pool and pool house. The duration of this work is estimated at six months. In addition to the cost for the soil mitigation work, there will be additional soft costs (e.g., for a full-time soils inspector to monitor the contractor doing the work, an additional cost for the Division of State Architect project inspector, special testing, additional Division of State Architect fees, etc.).

For the above reasons, Compton College respectfully requests your support for additional funds from the 2022-2023 State of California Budget for $5.8 million for the Compton College Physical Education Complex Replacement Project. If you or your staff have any questions, please contact me at kcurry@compton.edu or (310) 900-1600, ext. 2000.

Sincerely,

Keith Curry
President/CEO
Compton College

cc: Compton Community College District Board of Trustees
Honorable Senator Steven Bradford, California State Senate – 35th District
Honorable Assemblymember Mike A. Gipson, California State Assembly – District 64
Honorable Nancy Skinner, Chair, Joint Legislative Budget Committee, Senate Budget, and Fiscal Review Committee
Honorable Anthony Portantino, Chair, Senate Appropriations Committee
Honorable John Laird, Chair, Senate Budget, and Fiscal Review Subcommittee No. 1
Honorable Phil Ting, Chair, Assembly Budget Committee
Honorable Chris Holden, Chair, Assembly Appropriations Committee
Honorable Kevin McCarty, Chair, Assembly Budget Subcommittee No. 2
Honorable Jim Nielsen, Vice Chair, Senate Budget, and Fiscal Review Committee
Honorable Vince Fong, Vice-Chair, Assembly Budget Committee
Gabriel Petek, Legislative Analyst (3)
Joe Stephenshaw, Staff Director, Senate Budget and Fiscal Review Committee
Kirk Feely, Fiscal Director, Senate Republican Fiscal Office
Christopher W. Woods, Senate President pro Tempore's Office (2)
Christian Griffith, Chief Consultant, Assembly Budget Committee
Joseph Shinstock, Fiscal Director, Assembly Republican Caucus, Office of Policy and Budget
Paul Dress, Caucus Co-Chief of Staff, Assembly Republican Leader's Office

Luigi Luciano, Legislative Director, Assembly Republican Leader's Office
Jason Sisney, Assembly Speaker's Office (2)
Mark McKenzie, Staff Director, Senate Appropriations Committee
Jay Dickenson, Chief Consultant, Assembly Appropriations Committee

Enc

GEOTECHNICAL INVESTIGATION REPORT

PHYSICAL EDUCATION COMPLEX REPLACEMENT
COMPTON COMMUNITY COLLEGE DISTRICT

Compton, CA

PREPARED FOR:

Compton Community College District
1111 East Artesia Boulevard
Compton, CA 90221

PREPARED BY:

Atlas Technical Consultants LLC
14457 Meridian Parkway
Riverside, CA 92518

July 7, 2021

ATLAS

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July 7, 2021

Atlas No. 10-57575PW
Report No. 1

Ms. LINDA OWENS, CHIEF FACILITIES OFFICER
COMPTON COMMUNITY COLLEGE DISTRICT
1111 EAST ARTESIA BOULEVARD COMPTON,
CA 90221

Subject: Geotechnical Investigation
Compton College PE Complex Replacement
Compton Community College District
1111 East Artesia Boulevard, Compton, CA 90221

Dear Ms. Owens:

Atlas Technical Consultants (formerly United Heider Inspection Group) is pleased to present this geotechnical investigation report for the proposed Physical Education Complex Replacement, Compton College located at 1111 East Artesia Boulevard in the city of Compton, California.

The purpose of our investigation was to explore the subsurface conditions with respect to the planned improvements, to evaluate the general soil characteristics, and to provide geotechnical recommendations for design and construction. This investigation is based on the plan provided by Struere, Inc. and our correspondences with the district and the project construction and design team.

Based upon our study and investigation, the proposed development is feasible from a geotechnical viewpoint, provided our recommendations are incorporated in the design and construction of the project. The most significant design considerations for this project are compressible soil at the near surface, liquefaction and seismic settlement, and seismic shaking. We have evaluated the appropriate foundation systems to support the proposed building and other improvements. This report presents our findings, conclusions, and geotechnical recommendations for the project.

If you have any questions, please call us at (951) 697-4777.

Respectfully submitted,
Atlas Technical Consultants LLC

REGISTERED PROFESSIONAL ENGINEER
MEHRAB JESMANI
GE 3175
EXP. 9/30/21
GEOTECHNICAL
STATE OF CALIFORNIA

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1. INTRODUCTION

1.1 Site Location and Description

The project site is located within the south portion of the Compton College Campus in the city of Compton, California. The project site is surrounded by landscaped areas to the north, a building and a landscaped area to the south, the Vo-Tech Building and the Stadium to the west, and the Theater and Health Buildings and a Courtyard area to the east. Figure 1 presents the site location. The project location, measured on a Google Earth map, has a latitude reading of North 33.87696° and longitude reading of West 118.21110°. These coordinate readings should be considered accurate only to within an approximately 50-foot radius as implied by the method used.

1.2 Proposed Development

We understand this project will include the demolition of the existing Physical Education Complex, and the design and construction of a new two-story Physical Education (PE) building, pool house, a new pool, and parking areas. The proposed PE building will have a footprint of approximately 43,000 square feet. Information provided by the Project Structural Engineer indicate that the building will have wide spans with an estimated maximum column load for Dead and Live load on the order of about 241 kips with an average of about 100 Kips and the maximum load including seismic load (Dead, Live and Earthquake) on the order of about 554 kips. Infiltration BMPs are also planned at depths of either approximately 3 to 5 feet below existing grade or approximately 25 feet below existing grade.

We anticipate that the new building will be designed and constructed under the 2019 California Building Code (CBC).

1.3 Purpose and Scope

The purpose of our investigation has been to evaluate general engineering characteristics of the earth materials with respect to the planned improvements for the proposed PE building and associate improvements, such as a new pool and parking lot, BMP, and infiltration system, and to provide geotechnical recommendations for design and construction of the proposed project.

Our scope of work included the following tasks:

- **Background Review** - A background review of readily available, relevant, local and regional geology maps, geohazard maps, geotechnical reports, and literature pertinent to the proposed improvements was performed.
- **Pre-Field Investigation Activities** - Prior to our drilling activities, we conducted a site reconnaissance to locate proposed boring locations for access and for coordination with Underground Service Alert (USA).

- **Field Investigation** - Our field investigation consisted of excavation, logging and sampling of 15 borings to depths ranging from about 5 feet to 61.5 feet below the ground surface within the project improvements. The borings were drilled using either a hand auger or a truck mounted hollow-stem auger drill rig. Each boring was logged by a qualified member of our technical staff. Relatively undisturbed soil samples were obtained at selected intervals within the borings using a California Ring Sampler. Standard Penetration Tests (SPTs) were also conducted at selected depths within the borings, and soil samples were obtained. Bulk samples of representative soil types were also obtained from the borings. Borings B-11, B-13, and B-14 were converted to and used as borehole percolation test points. Additionally, a fourth borehole percolation test point, P-4, was drilled using a hand auger. The borings were backfilled in accordance with regulatory requirements. Logs of the borings are presented in Appendix II. Boring locations are shown on Figure 2 (Boring Location Map).

- **Laboratory Tests** - Laboratory tests were performed on selected soil samples obtained during our field investigation. The laboratory testing program was designed to evaluate the physical and engineering characteristics of the on-site soils. Tests performed during this investigation include:

  — In situ moisture content and dry density of existing soils.
  — Particle Size Analysis to characterize the soil type according to USCS, and to assist in the evaluation of liquefaction susceptibility of granular soil.
  — Atterberg limit tests to classify and characterize of the engineering properties of soils.
  — Direct shear to evaluate the strength characteristics of the on-site materials.
  — Expansion Index test to evaluate the expansion potential of the on-site material.
  — R-Value.
  — #200 Wash.
  — Soil Corrosivity.
  — Collapse/Swell potential of soil.

  All laboratory tests were performed in general conformance with ASTM Standard Methods and California Test Methods. The results of the in-situ moisture and density tests are shown on the boring logs (Appendix I). Results of the other laboratory tests are provided in Appendix III.

- **Engineering Analysis** - The data obtained from our background review, field exploration, and laboratory testing program were evaluated and analyzed in order to develop the conclusions and recommendations for the site.

- **Report Preparation** - The results of this investigation have been summarized in this report, presenting our findings, conclusions and recommendations for the proposed project.

2. GEOLOGIC AND GEOTECHNICAL FINDINGS

2.1 Regional Geology

The site is mapped on the South Gate Quadrangle and is situated on the Downey Plain within the Los Angeles metropolitan region. The Downey Plain is located at the convergence of two major physiographic/geomorphic provinces, the Transverse Ranges and the Peninsular Ranges, and includes rugged mountains, hills, valleys, and alluvial plains. The east-west trending Transverse Ranges are irregular to the main northwest structural grain of California. The Transverse Ranges were uplifted along east to west–trending thrust faults and folds (Crowell, 1976; Wright, 1991; and Ingersoll and Rumelhart, 1999). The central Los Angeles basin is divided by a mountain range, the Santa Monica Mountains. The leading structure in the area is the north-dipping Santa Monica–Hollywood–Raymond fault system, located at the southern boundary of the Transverse Ranges. The Los Angeles basin itself is part of the northern Peninsular Ranges Geomorphic Province, which extends southeastward into Baja California, Mexico. The Transverse Ranges are formed by mildly metamorphosed sedimentary and volcanic rocks of Jurassic age that have been infringed by mid-Cretaceous plutonic rocks of the southern California batholith and rimmed by Cenozoic sedimentary rocks (Gastil et al., 1981; Schoellhamer et al., 1981). The Los Angeles greater basin is also part of the onshore portion of the California continental borderland, characterized by northwest-trending offshore ridges and basins, formed primarily during early and middle Miocene time (Legg, 1991; Wright, 1991; and Crouch and Suppe, 1993). The thickness of the predominantly Neogene-age sedimentary fill in the central depression of the Los Angeles basin, a structural low between the Whittier and Newport–Inglewood faults, is estimated to be about 30,000 feet (Yerkes et al., 1965).

Major northwest-trending strike-slip faults such as the Whittier, Verdugo, Northridge, Sierra Madre, Newport–Inglewood, and Palos Verdes faults dominate the great basin. In addition to these surface faults, significant buried thrust faults in the general site vicinity in the Los Angeles basin include the lower and upper Elysian Park thrust faults, the Compton thrust, and the Puente Hills thrust (Shaw, et al., 2002; Bilodeau, et. al., 2007).

The youngest surficial deposits are Holocene sediments of modern alluvial fans, stream channels (i.e., Los Angeles and San Gabriel Rivers), and their flood plains. These debris-flow, sheet flood, and fluvial deposits consist of boulder, cobble, and pebble gravel lenses and sheets, interbedded with sand, silt, and clay derived from the surrounding highlands. Although the thickness of these sediments is usually less than 100 feet (30 m), they are locally as thick as 200 feet (60 m), and the fluvial sediments are roughly graded, with the lower parts containing coarser material. A narrow zone of well-sorted, fine to medium-grained dune sand, as thick as 70 feet (21 m), is located near the coast between Santa Monica and the Palos Verdes Hills (DWR, 1961; Yerkes et al., 1965). Since about 6 thousand years ago, when postglacial sea level had risen to near its present level, coastal estuaries and tidal marshes formed and became filled with organic-rich, fine-grained sediment that extended as far as 4 miles (6.4 km) inland from the mouths of the streams (Yerkes et al., 1965). Real estate development has now transformed most of these estuaries and marshes into marinas and residential areas (Bilodeau, et al., 2007).

Based on a review of the California Geologic Survey geologic maps of the Long Beach 30' x 60' Quadrangle (CGS, 2010; 2016), the site area is mapped as being underlain by younger alluvial deposits (or Young Alluvium, Unit 2), as shown on Figure 3 (Regional Geology Map). As shown on this geologic map, the project site and much of the project vicinity are underlain by Holocene to Late Pleistocene age Younger Alluvial Fan Deposits (Qyf), described by the California Geological Survey (2010) as "unconsolidated to slightly consolidated, unvisited to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon" as "Young alluvium, Unit 2" by the California Geological Survey (2016).

2.2 Subsurface Conditions

The site in unpaved areas generally is underlain by about ½ foot of grass/topsoil/surficial fill and young alluvial deposits of Holocene to late Pleistocene age (Qya_2) as shown on the geologic cross sections (Figures 7 and 8). The young alluvial deposits encountered at the site are predominantly comprised of inter-layered Silty SAND and Sandy SILT. In general, the near-surface sandy soils layers are mostly loose to medium dense, and sandy soils layers at depth are medium dense to dense in relative density. The near-surface, fine-grained soil layers are mostly firm to stiff and stiff to very stiff at depth in consistency.

Important geotechnical characteristics of the subsurface soils that are relevant for the proposed developments are discussed briefly in the following subsections.

2.2.1 Expansion Potential

Samples of the sub-surface soils within the project site that were tested had expansion indexes of 9 and 2, generally indicating very low to low expansion potential. The Geotechnical and Soil Investigation Report prepared by United Heider Inspection Group (UHIG, 2018) for the nearby project (Student Service Building) reported a medium expansion potential for the site (EI=56). Based on this finding and our experience with similar type of materials, generally the on-site soils are anticipated to contain a low expansion potential (per ASTM D4829).

2.2.2 Corrosivity Potential

In general, soil environments that are detrimental to concrete have high concentrations of soluble sulfates and/or pH values of less than 5.5. Section 19.3.2 of ACI 318 (ACI, 2014), as referred in the 2019 CBC, provides specific guidelines for the concrete mix-design when the soluble sulfate content of the soil exceeds 0.1% by weight or 1,000 parts per million (ppm). The County of Los Angeles (2013) recommends implementing mitigation measures to protect any concrete structures when soluble sulfate concentrations are equal to or greater than 2,000 ppm in soil and 1,000 ppm in groundwater.

Samples of the subsurface soil within the proposed buildings footprint were tested for water-soluble sulfate during the investigation and had a soluble sulfate contents of 20 and 50 ppm that are less than 0.1% by weight (1000 ppm), indicating negligible sulfate exposure. Therefore, no cement type restriction/concrete class restriction is necessary per ACI Table 19.3.2.1 for the consideration of soluble sulfate exposure, as well as no soil mitigation necessary for the site.

The minimum amount of chloride ions in the soil environment that are corrosive to steel, either in the form of reinforcement protected by concrete cover or plain steel substructures (such as steel pipes or piles) is 500 ppm per California Test 532. Soil corrosivity to ferrous metals can be estimated by the soil's pH level, electrical resistivity, and chloride content (County of Los Angeles, 2013). In general, soils are considered corrosive when the minimum resistivity is less than 1,000 ohm-centimeters. Soil with a chloride content of 500 ppm or more is considered corrosive.

As a screening for potentially corrosive soil, samples of the subsurface soil within the buildings sites were tested to determine minimum resistivity, chloride content, and pH level. The chloride content of the samples was 30 ppm and 40 ppm. The measured resistivity of tested samples was 2,940 and 2,970 ohm-cm. The pH values of the samples were 8.19 and 8.87.

Based on these results, the on-site soil is generally considered to be highly corrosive towards buried ferrous metals. This information should be provided to the underground utility subcontractors. Consideration should be given to retaining a corrosion consultant to obtain recommendations for the protection of metal components embedded in the site soil. Further interpretation of the corrosivity test results (resistivity value, pH and other test results and data), and providing corrosion design and construction recommendations for foundation and ferrous metals, are the purview of corrosion specialists/consultants.

The Geotechnical and Soil Investigation Report (UHIG, 2015) for the nearby project (Instructional Building #2) reported the following substantially conforming corrosion suite results as listed in Table 1.

Table 1 – Corrosion Results (UHIG, 2015)

Boring (Heider Inspection 2015)	Sample Depth (feet)	Sulfate (mg/kg)	Chloride (mg/kg)	Resistivity (ohm-cm)	pH
B-2	0-5	36	<10	2,700	7.3

2.2.3 Excavatability

Based on our investigation findings, subsurface soils within the anticipated maximum depth of excavation are expected to be readily excavatable by conventional heavy earthmoving equipment in good condition.

2.3 Groundwater

Groundwater was encountered in our soil borings B-4 at a depth of approximately 44 feet below the existing ground surface and in B-10 at a depth of approximately 52 feet below existing ground surface. Groundwater was encountered in Borings B-1 during the UHIG investigation (2018) for the Student Building at the depth of about 46 feet below ground surface. The depths of groundwater encountered in the previous borings, as well as estimated from the CPTs, ranged from about 46 to 48.5 feet below existing ground surface.

According to the California Geological Survey (CGS, 1998) seismic hazard zone report for the South Gate quadrangle, historically shallowest groundwater level is estimated to be on the order of 8 feet below existing grade. According to the California Department of Water Resources (DWR), available groundwater level data for Well 338872N1182432W001, the nearest well located approximately 2 miles northwest of the project site, a single measurement made on September 14, 1995 indicated the groundwater on that date to be at 122.45 feet below the existing local ground surface, corresponding to El. -32.5 feet (mean sea level datum).

Groundwater levels generally fluctuate between different locations, years, and seasons. Therefore, variations from our observations may occur in the future; historically, these appear to be on the order of a few feet.

3. FAULTING, SEISMICITY AND SEISMIC HAZARDS

3.1 Faulting and Primary Seismic Hazards

Our review of available in-house literature indicates that there are no known active or potentially active faults that traverse the site, and the site is not located within an Alquist-Priolo Earthquake Fault Zone, although such faults are in general proximity to the subject site (Hart and Bryant, 1999). The nearest mapped Alquist-Priolo Earthquake Fault Zone is the Newport- Inglewood Fault Zone, approximately 1.65 miles southwest of the site. In addition to this surface fault zone, two buried thrust faults, the Lower Elysian Park and Compton, are inferred to be located about 2.5 miles north and 8 miles south, respectively, from the site (Shaw, et al., 2002; Bilodeau, et. al., 2007).

The principal seismic hazard that could affect the site is ground shaking resulting from an earthquake occurring along nearby several major active or potentially active faults in southern California as shown in Figure 4 (Regional Fault Map). The known regional active and potentially active faults that could produce the most significant ground shaking and closer to the site include those faults listed (in order of increasing distance from the site) in Table 2.

Table 2 – Characteristics and Estimated Earthquakes for Regional Faults

Fault Name	Approximate Distance to Site (miles)[1]	Maximum Credible Earthquake (MCE) Magnitude[2]
Newport-Inglewood	2	7.1
Lower Elysian Park Thrust	2.5[3]	6.7
Compton Thrust	8[3]	6.8
Puente Hills Blind Thrust	7[3]	7.1
Palos Verdes	9	7.3
Upper Elysian Park Thrust	10[3]	6.4
Whittier	13	6.8
Hollywood	16	6.4
Raymond	17	6.5
Verdugo	17	6.9
Santa Monica	18	6.6
Malibu Coast	21	6.7
Sierra Madre	22	7.2
Newport-Inglewood (offshore)	26	7.1
San Fernando	28	6.7
Anacapa-Dume	29	7.5
Chino-Central Avenue	29	6.7
Northridge	29	7.0
San Gabriel	31	7.2
Santa Susana	34	6.7
Elsinore (Glen Ivey)	36	6.8
Simi-Santa Rosa	40	7.0
San Andreas (Mojave)	44	7.4
Oak Ridge	48	7.1
San Clemente	50	7.25[4]
San Cayetano	50	7.0
North Frontal Thrust (Western)	63	7.2
Pinto Mountain	86	7.2

(1) Fault distances estimated from measurements using the Fault Activity Map of California by C.W. Jennings and W.A. Bryant, California Geological Survey, Geologic Data Map No. 6, 2010.

(2) Maximum moment magnitude calculated from relationships (rupture area) derived from Wells and Coppersmith (1994; values listed in Appendix A of Cao, T., Bryant, W.A., Rowshandel, B., Branum, D., and Wills, C.J., 2003, The revised 2002 California probabilistic seismic hazard maps, June 2003: California Geological Survey, 12 p., Appendix A.

(3) Fault distances estimated from measurements using Puente Hills Blind-Thrust System, Los Angeles, California by Shaw and others (2002): Bulletin of the Seismological Society of America, vol. 92, no. 8, pp. 2946-2960 and Bilodeau, W.L., Bilodeau, S.W., Gath, E.M. Oborne, M., and Proctor, R.J., 2007, Geology of Los Angeles, California, United States of America: Environmental & Engineering Geoscience, Vol. XIII, No. 2, May 2007, pp. 99–160.

(4) Legg, M.R., Luyendyk, B.P., Mammerickx, J., and Tyce, R.C., 1989, Sea Beam Survey of an Active Strike-Slip Fault: The San Clemente Fault in the California Continental Borderland: Journal of Geophysical Research, v. 94, pp. 1727-1744.

3.1.1 Regional Seismicity

Evaluation of the historic seismicity related to the New Instructional Building #2 site was performed to show the significant past earthquakes. Figure 5 (Regional Seismicity Map) and the associated table show the recent regional seismicity with respect to the site. Significant past earthquakes from 1900 to 2018 with magnitudes 5 or greater were estimated using the USGS Earthquake database. This historical seismicity evaluation was performed within the 100-kilometer radius search from the project site, and the seismic events are listed in Appendix VII.

The chance of earthquake damage in Compton is near the California average and is much higher than the national average due to active earthquake faults in the region. Based on the online reports at the http://www.city-data.com, it appears no property damage and human losses were reported in the City of Compton area during the previous historic earthquakes. Summary of the major earthquakes and reported damages at the epicenter are summarized below:

- On 7/21/1952 at 11:52:14, a magnitude 7.7 (7.7 UK, Class: Major, Intensity: VIII - XII) earthquake occurred 88.2 miles away from the city center, causing $50,000,000 total damage on 6/28/1992 at 11:57:34, a magnitude 7.6 (6.2 MB, 7.6 MS, 7.3 MW, Depth: 0.7 mi) earthquake occurred 99.1 miles away from Compton center, causing 3 deaths (1 shaking death, 2 other deaths) and 400 injuries, causing $100,000,000 total damage and $40,000,000 insured losses.
- On 10/16/1999 at 09:46:44, a magnitude 7.4 (6.3 MB, 7.4 MS, 7.2 MW, 7.3 ML) earthquake occurred 111.0 miles away from the city center.
- On 11/4/1927 at 13:51:53, a magnitude 7.5 (7.5 UK) earthquake occurred 174.9 miles away from the city center.
- On 1/17/1994 at 12:30:55, a magnitude 6.8 (6.4 MB, 6.8 MS, 6.7 MW, Depth: 11.4 mi, Class: Strong, Intensity: VII - IX) earthquake occurred 26.9 miles away from Compton center, causing 60 deaths (60 shaking deaths) and 7,000 injuries.
- On 4/21/1918 at 22:32:30, a magnitude 6.8 (6.8 UK) earthquake occurred 45.5 miles away from the city center.

  ** Magnitude types: body-wave magnitude (MB), local magnitude (ML), surface-wave magnitude (MS), moment magnitude (MW).

3.2 Secondary Seismic Hazards

Secondary seismic hazards for this site, generally associated with severe ground shaking, include liquefaction, seismic settlement, landslide, tsunamis, and seiches.

3.2.1 Liquefaction

Liquefaction is the loss of soil strength or stiffness due to a buildup of pore-water pressure during severe ground shaking. Liquefaction is associated primarily with loose (low density), saturated, fine to medium-grained cohesionless soil. As the shaking action of an earthquake progresses, the soil grains are rearranged and the soil densifies within a short period of time. Rapid densification of the soil results in a buildup of pore-water pressure. When the pore-water pressure approaches the total overburden pressure, the soil reduces greatly in strength and temporarily behaves similarly to a fluid.

The site is mapped within an area shown as potentially susceptible to liquefaction on the California Geological Survey (CGS, 2016) seismic hazard zones for the South Gate Quadrangle as shown on Figure 6.

A site-specific liquefaction analysis was performed in accordance with the method of Boulanger and Idriss (2014) using LiqSVs 2.0.2.1 computer program developed by GEOLOGISMIKI Software. Seismically induced settlement analyses were performed based on the sub-surface conditions encountered in the deep borings B-4 and B-10 and peak ground acceleration values PGA corresponding to adjusted Peak Ground Acceleration PGA_M. For this analysis, we considered a historic high groundwater level at eight feet below ground surface as indicated on the CGS Seismic Hazards Report and considered depth reduction factor. The predominant earthquake magnitude was obtained from the USGS Interactive Deaggregation website for a 2% probability of exceedence in 50 years (2475 return period) hazard. The seismic parameters, peak ground acceleration of 0.802g and magnitude of 7.3, were used for the liquefaction analysis.

Based on our calculations, potential for liquefaction at the site to occur within various layers of sandy silt and silty sand occurring below 8 feet (maximum historic groundwater table); therefore, the liquefaction susceptibility of the site is very high.

3.2.2 Seismically Induced Settlement

Seismically induced settlement consists of dry dynamic settlement (above groundwater) and liquefaction induced settlement (below groundwater). These settlements occur within silty sand and sandy silt soils due to reduction in volume during and shortly after an earthquake event.

Due to the presence of loose and soft layers of silty sand and sandy silt, high seismic settlement was anticipated. For the on-site (untreated) soil the maximum potential total seismic settlement at the site has been estimated to be on the order of about 10 inches (considering the historically highest groundwater table at the depth of about 8 feet, Mw=7.3, peak ground acceleration of 0.802g and using depth reduction factor). This potential settlement is generally due to liquefaction settlement.

Due to the high seismic settlement, in the following sections we recommend soil mitigation and treatment to reduce the seismic settlement.

3.2.3 Earthquake-Induced Lateral Displacement

In general, relatively severe and shallow liquefaction could cause lateral ground displacements. Since no vertical free face or sloping ground is close to the site, the potential for lateral displacement is considered low.

3.2.4 Surface Manifestations of Liquefaction

Due to the high seismic settlement, there is a potential for surface manifestation of liquefaction of on-site soil that will be mitigated by the recommended soil treatment methods.

3.2.5 Seismically Induced Landslide

There are no significant slopes that exist near the site. As the site is relatively flat and no slopes are proposed, the possibility for earthquake-induced landslides is considered negligible.

3.2.6 Hydro-Collapsible Soils

Collapsible soils are fine sandy and silty soils that have been laid down by the action of flowing water, usually in alluvial fan deposits. Terrace deposits and fluvial deposits can also contain collapsible soil deposits. The soil particles are usually bound together with a mineral precipitate. The loose structure is maintained in the soil until a load is imposed on the soil and water is introduced. The water breaks down the inter-particle bonds, and the newly imposed loading densifies the soil.

The Geotechnical Engineering and Geologic Hazards Study Report (UHIG, 2015) for the nearby building project (Instructional Building #1) reported potential hydro-collapsible soils on site. To evaluate the potential of hydro-collapse of the soil layers versus depth laboratory collapse tests performed on the on-site soil samples collected from B-8 at a depth of about 6 feet and B-11 at a depth of about 11 feet. For the tested samples, the potential of collapse found to be negligible at an applied overburden pressure of 2,200 pounds per square foot (psf).

3.2.7 Other Hazards

Flood hazards generally consist of shallow sheet flooding caused by surface water runoff during large rain storms. According to the Federal Emergency Management Agency Flood Insurance Map (FIRM, 2008), the site is within a zone designated as “Other Flood Areas-Zone X: Areas of Reduced Flood Risk due to Levee.”

Subsidence of the land surface, as a result of the activities of man, has been occurring in California for many years. Subsidence can be divided, on the basis of causative mechanisms, into four types: groundwater withdrawal subsidence, hydrocompaction subsidence, oil and gas withdrawal subsidence, and peat oxidation subsidence (CDMG, 1973). According to CDMG (1973), the site lies either within, or near, an area potential land subsidence due to withdrawal of oil and gas from nearby oil and gas fields.

Tsunamis, often incorrectly called tidal waves, are long period waves of water usually caused by underwater seismic disturbances, volcanic eruptions, or submerged landslides. The site is not within a potential tsunamis hazard zone according to the Tsunami Inundation Maps for the Long Beach and Venice Quadrangles (California Emergency Management Agency, 2009). Tsunamis are not a potential hazard at the site.

A seiche is an oscillation of a body of water in an enclosed or semi-enclosed basin that varies in period. Seiches are often caused by tidal currents, landslides, earthquakes, and wind. There are no bodies of water adjacent or near to the site. A seiche is not a potential inundation hazard.

Earthquake-induced flooding is flooding caused by failure of dams or other water-retaining structures as a result of earthquakes. The site is mapped within an area shown as Potential Dam Inundation Areas on the Los Angeles County General Plan Dam and Reservoir Inundation Routes Map (General Plan 2035 Figure 9.4). Since the site is located in the inundation area of the Whittier Narrows Dam (11 miles upstream from Compton), the Hansen Dam (30 miles upstream from Compton), and the Sepulveda Dam (29 miles upstream from Compton), the potential of earthquake-induced flooding exists at the site, if one of these dams fails during a strong earthquake.

4. CONCLUSIONS AND RECOMMENDATIONS

Based on our geotechnical investigation findings, it is our opinion that the site is suitable for the proposed buildings and associated improvements provided the recommendations in this report are taken into account during design and construction of the project. We did not encounter any geotechnical constraints, geological hazards within the subject site that cannot be mitigated by proper planning, design, and sound construction practices.

The most significant design considerations for this project are liquefaction, seismic settlement, and seismic shaking. Presented herein are our recommendations for site grading, seismic parameters, foundation design parameters, lateral earth pressures, and construction considerations for the project.

4.1 Earthwork

All earthworks should be performed in accordance with the latest edition of the Standard Specifications for Public Works Construction (Greenbook), unless specifically revised or amended below or by future review of project plans.

All site grading operations should conform to the local building and safety codes and rules and regulations of the governing governmental agencies having jurisdiction over the subject construction.

Earthwork is expected to consist of excavation/overexcavation of loose, soft and/or disturbed soils and placement of fill soils for the proposed site improvements. Recommendations for site earthwork are provided in the following paragraphs.

4.1.1 Site Preparation

The site should be cleared of all debris and unsuitable materials. All undocumented fill soils should be removed from the site. Prior to construction, it will be necessary to demolish the existing buildings including utilities (if needed), remove all existing concrete slabs within the limits of planned grading. Structure removal should include foundations and flatwork. Concrete fragments and debris from the demolition operation should be disposed off site. The existing near surface soils that are disturbed during demolition of the existing improvements should be recompacted or removed as needed to make it firm stable subgrade soils. The need for and extent of removal of soils disturbed by site demolition should be determined by the Geotechnical Engineer at the time of grading.

Any existing vegetation and organic contaminated soil should be stripped and disposed off site. Removal of trees and shrubs should also include root balls and attendant root system.

Any existing utility lines should be removed and/or rerouted if they interfere with the proposed construction. The cavities resulting from removal of utility lines and any buried obstructions should be properly backfilled and compacted as recommended in Sections 4.1.3 and 4.11 of this report. In addition, if any uncontrolled artificial fill is encountered, it should be removed.

Excavations located along property lines and/or adjacent to existing structures (e.g., buildings, walls, fences, etc.) should not be permitted within 2 feet of existing foundations.

4.1.2 Excavation/Overexcavation in Building Pad Area and the Exterior Flatwork Area for Slab-On-Grade

Existing fill soils within the proposed buildings pads should be over-excavated to a minimum depth of 3½ feet below existing grade or to a sufficient depth to remove all of the undocumented fill materials in their entirety from within the proposed buildings pads areas. Deeper undocumented fill layers are anticipated to be present at the site and the depth and extent of the fill should be verified during the grading operation.

In order to remove the upper compressible soil and undocumented fill and to reduce the potential for adverse differential settlement of the proposed structures, the underlying subgrade soil must be prepared in such a manner that a uniform response to the applied loads is achieved. For the proposed buildings, we recommend that a minimum of 4 feet of engineered fill be provided under the buildings pads at a minimum overexcavation depth of 5 feet from existing grade, whichever provides the deeper overexcavation The fill shall be placed in loose lifts of 6 to 8 inches in thickness, moisture-conditioned to above the optimum moisture content as needed (generally about 2% above optimum) and compacted to a minimum of 92% relative compaction (per ASTM D1557).

The excavated removal bottoms shall be evaluated by a geotechnical engineer to confirm competent native soil materials are encountered. In general, native soils with at least 85% relative compaction of maximum dry density (ASTM D1557) is considered suitable. If unsuitable soil conditions are encountered deeper excavation may be recommended. The overexcavation should extend below any underground obstructions to be removed. The overexcavation and recompaction should extend a minimum of 5 feet laterally from the edges of the footings, where feasible. The soil below exterior slabs-on-grade (non-vehicular) should be overexcavated and recompacted a minimum of 24 inches below the bottom of the proposed slab or 24 inches below the existing ground surface, whichever is deeper.

Areas outside the overexcavation limits of the proposed buildings planned for asphalt or concrete pavement and flatwork and areas to receive fill should be overexcavated to a minimum depth of 24 inches below the existing ground surface or 24 inches below the proposed finish grade, whichever is deeper.

Local conditions may require that deeper overexcavation be performed. If encountered, such areas should be evaluated by the geotechnical consultant of record during grading.

In addition to the above recommendations, all uncontrolled fill, if encountered, should be removed from structural areas prior to fill placement.

After completion of the overexcavation, and prior to fill placement, the exposed surfaces should be scarified to a minimum depth of 8 inches, moisture conditioned to about 2% above optimum, and recompacted to a minimum 90% relative compaction.

4.1.3 Fill Placement and Compaction

Following subgrade approval by the Geotechnical Engineer, the bottom of the removal excavation should be scarified to a depth of 8 inches, moisture conditioned as needed and recompacted to 90% relative compaction as determined by ASTM D1557. However, if the subgrade is dense and consists of undisturbed alluvium the scarification should not be performed, and measures should be taken to prevent subgrade disturbance.

Any fill soil should be placed in loose lifts of 6 to 8 inches in thickness, moisture-conditioned to above the optimum moisture content as needed (generally about 2% above optimum) and compacted to a minimum of 92% relative compaction (per ASTM D1557).

4.1.4 Fill Materials

On-site soils that are free of organics, debris and oversize particles (e.g., cobbles, rubble, etc. that are greater than 3 inches in the largest dimension) and an expansion index less than 50 can be reused as fill as approved by the Geotechnical Engineer.

Import materials, if needed, should contain sufficient fines (binder material) so as to be resulted in a stable subgrade when compacted. The imported materials should have an expansion index less than 20 and should be free of organic materials, corrosion impacts, debris, and cobbles larger than 2 inches with no more than 35% passing the #200 sieve. A bulk sample of potential import material, weighing at least 35 pounds, should be submitted to the Geotechnical Consultant at least 72 hours before fill operations. Proposed import materials should be tested for corrosivity, should be environmentally cleared from contamination and should be approved by the Geotechnical Consultant prior to being imported on site (some more tests such as: R-Value, may be required).

If base materials are imported to be placed instead of soil backfill, these may be either crushed aggregate base or crushed miscellaneous base in conformance with the Sections 200-2.2 and 200-2.4 of the Standard Specifications for Public Works Construction (Green Book), 2018 Edition, respectively.

Soil engineer should be notified at least 72 hours prior to borrow materials in order to sample and test materials from proposed borrow sites.

4.2 CBC Seismic Design Parameters

In order to provide the preliminary seismic design parameters, based on the field data, the subsurface conditions, geology of the site and to the best of our knowledge and understanding, we have assumed that site's soil profile may be characterized within the category of 'Stiff Soil Profile' with Site Class D according to Section 1613A.2.2 of the 2019 CBC accordance with Chapter 20 of ASCE7-16.

Corresponding CBC seismic design parameters for this soil profile and the site location (Latitude: 33.876960 °N; Longitude: -118.211102 °W) are determined based on general ground motion analysis in accordance with Section 1613A.2 of the 2019 CBC. These parameters are summarized in Table 3. Proposed development at the site should be designed for the seismic parameters presented in Table 3.

Table 3 – California Building Code Seismic Design Parameters

Categorization/Coefficient	Design Value
Site Class	D
Risk Category	III
Mapped MCE_R Spectral Acceleration for Short (0.2 Second) Period, S_S	1.694
Mapped MCE_R Spectral Acceleration for a 1-Second Period, S_1	0.606
Short Period (0.2 Second) Site Coefficient, F_a	1.0
Long Period (1 Second) Site Coefficient, F_v	1.7
Adjusted Spectral Response Acceleration at 0.2-Second Period, S_{MS}	1.694
Adjusted Spectral Response Acceleration at 1-Second Period, S_{M1}	1.031
Design (5% damped) Spectral Response Acceleration for Short (0.2 Second) Period, S_{DS}	1.129
Design (5% damped) Spectral Response Acceleration for a 1- Second Period, S_{D1}	0.687
Peak ground acceleration value, PGA_M	0.802
Seismic Design Category	D

A site-specific ground motion analysis was performed as part of our investigation. As part of the site-specific analysis, base ground motions were evaluated in conjunction with both a Probabilistic Seismic Hazard Analysis (PSHA) and a Deterministic Seismic Hazard Analysis (DSHA) to characterize earthquake ground shaking that may occur at the site during future seismic events.

The PSHA is based on an assessment of the recurrence of earthquakes on potential seismic sources in the region and on ground motion prediction models of different seismic sources in the region. The United States Geological Survey (USGS) Unified Hazard Tool (USGS, 2021a) was used to develop seismic hazard curves for various periods and the USGS Risk-Targeted Ground

Motion Calculator (USGS, 2021b) was used to analyze ground motions for each corresponding period. Maximum directional scale factors were applied to the results to develop the probabilistic ground motion response spectrum specific to this site.

The DSHA is represented by the 84th percentile of the spectral accelerations for different periods. The logarithmic means and standard deviations of various periods were calculated using the USGS Response Spectra Tool (USGS, 2021c) with ground motion model(s) "Combined: WUS 2018 (5.0, deep basins)." This combined model utilizes attenuation relationships of Abrahamson-et al (2014) NGA West 2, Boore-et al (2014) NGA West 2, Campbell & Bozorgnia (2014) NGA West 2, and Chiou & Youngs (2014) NGA West 2.

ASCE 7-16 indicates that the deterministic ground motions shall be calculated for the characteristic earthquakes on all known active faults within the region. The largest such acceleration for each period shall be used to create the deterministic (84th percentile) spectrum. The input parameters for DSHA were obtained from the USGS Shakemap Scenarios.

The site-specific Risk-Targeted Maximum Considered Earthquake (MCE_R) was taken as the lesser of the spectral response accelerations determined from the PSHA and DSHA for each period. The site-specific design response spectral accelerations were compared to the design response spectrum from ASCE 7-16, Section 11.4.6 (SEAOC, 2021) to verify that the values obtained from the site-specific analysis are not less than 80% of the accelerations obtained from Section 11.4.6. The site coefficients and maximum considered earthquake spectral response acceleration parameters are presented in Table 4.

Table 4 – 2019 California Building Code / ASCE 7-16 Site-Specific Parameters

Site Coordinates	
Latitude: 33.876960	Longitude: -118.211102
Site Coefficients and Spectral Response Acceleration Parameters	**Value**
Site Class	D
Risk Category	III
Site Amplification Factor at 0.2 Second, F_a	1.000
Site Amplification Factor at 1.0 Second, F_v	2.500
Spectral Response Acceleration at Short Period, S_S	1.882g
Spectral Response Acceleration at 1-Second Period, S_1	0.656g
Spectral Response Acceleration at Short Period, Adjusted for Site Class, S_{MS}	1.882g
Spectral Response Acceleration at 1-Second Period, Adjusted for Site Class, S_{M1}	1.639g
Design Spectral Acceleration at Short Period, S_{DS}	1.255g
Design Spectral Acceleration at 1-Second Period, S_{D1}	1.093g
Site Specific Peak Ground Acceleration	0.774g

The proposed development shall be designed based on the seismic parameters provided in Tables 3 and 4, whichever is more conservative.

4.3 Soil Treatment

The proposed PE building and the associate structural elements shall be supported on foundations designed to accommodate the static and seismic total and differential settlements without undue distress occurring to the building. As discussed in previous sections, the project site is susceptible to potential static settlement due to column loads and seismic settlements (liquefaction and dry settlements) induced by the design earthquake.

The seismic and static settlements can be reduced or controlled by soil mitigation methods using deep soil mixing method under the proposed foundation systems below the columns and walls. The preliminary recommendations provided in this report shall be verified and confirmed during project construction and during the performing of the deep soil mixing columns, including proper tests in the field and Lab.

4.3.1 Deep Soil Mixing, Preliminary Recommendations

Deep soil mixing is an in-situ ground improvement technique that enhances the characteristics of weak soils by mechanically mixing them with a cementitious binder. The action of mixing materials such as cement with soil causes the properties of the soil to become more like soft rock.

Generally, the upper 37 feet of the soil can be mitigated by deep soil mixing. The diameter of each column could be about 6 feet with about 6 inches of overlap with about 27 ½ feet of square grids. A minimum replacement ratio on the order of about 30% is our preliminary recommendation.

We strongly recommend at least the foundation system (e.g., under the columns and under the structural bearing walls,…), be supported by the deep soil mixing columns.

It should be noted that in the event of a major local earthquake, some damages to the project will occur and repairs to the damaged parts and portions should be anticipated; however, the soil mitigation and treatment for the entire site of the project will be safer.

4.3.2 Settlement of the Treated soil

Based on our analyses performed on borings B-4 and B-10 (considering the historically highest groundwater table at the depth of about 8 feet, Mw= 7.3, PGA_M = 0.802 and using depth reduction factor, Cetin. et. al.), the total seismic settlement for the treated soil is estimated to be on the order of about 2½ inches or less. The differential seismic settlement can be considered to be on the order of about 1¼ inches over a horizontal distance of 40 feet.

The total static settlement of the treated soil under the structural loads has been estimated to be on the order of about ¾ inch with the differential static settlement of about ½ inch over a horizontal distance of 40 feet.

4.3.3 Continuous Foundation System Supported by Deep Soil Mixing (DSM) Columns

We recommend using a continuous foundation system supported on the treated soil: deep soil mixing columns We assumed that the continuous foundation system would be at least 2 to 2½ feet thick The continuous foundation system shall be thick enough to limit the total and differential static and seismic settlements within the required threshold indicated in this report. For the continuous foundation system supported by deep soil mixing columns, we recommend an allowable net bearing pressure of 6,000 psf for gravity loads: dead and live load. During transient loads such as wind or earthquake, this bearing pressure can be increase by 33% up to 8,000 psf.

A subgrade modulus of 125 pounds per cubic inch (pci) can be applied to the areas covered with deep soil mixing properly. No need to reduce if the area is properly covered by deep soil mixing.

4.4 Minor Footings

Minor footings may be required for low height exterior landscape walls (4 feet or less in height), or other small ancillary structures. These footings should be supported on at least 3 feet of new engineered fill and should be embedded at least 36 inches below the existing grade. A vertical bearing pressure of 2,000 psf may be used for these footings. No undocumented fill is allowed under the footings.

Adjacent utilities or foundations should be avoided within the zone of an imaginary plane extending downward at a 1½H:1V: 1V (horizontal: vertical) inclination from the bottom edge of the foundation.

4.5 Resistance to Lateral Loads

Resistance to lateral loads can be provided by friction acting at the base of the concrete and by passive earth pressure. A coefficient of friction of 0.35 may be assumed for base friction. An allowable passive lateral earth pressure of 220 psf per foot of depth up to a maximum of 2,200 psf may be used for sides of the foundation poured against properly compacted fill. This allowable passive pressure is applicable for level ground conditions only (slope equal to or flatter than 5H:1V).

The above lateral bearing values may be increased by 33% for short duration of loading, including the effects of wind or seismic forces.

4.6 Slab-On-Grade

Slabs-on-grade should be placed on properly prepared subgrade soil as described in the earthwork section of this report (Section 4.1 and the pertinent subsections). Prior to concrete placement, the exposed subgrade should be scarified to at least 8 inches, moisture-conditioned to moisture content of about 2% above optimum and compacted to a minimum of 90% relative compaction (per ASTM D1557). The subgrade should not be allowed to dry prior to concrete placement.

The structural engineer should design the actual slab thickness and reinforcement based on structural load requirements. We recommend a minimum slab thickness of 4 inches. Frequent continuous joints should be provided to help control slab cracking.

Care should be taken to avoid slab curling if slabs are poured in hot weather. Slabs should be designed and constructed as promulgated by the Portland Cement Association. Prior to the slab pour, all utility trenches should be properly backfilled and compacted.

In areas where a moisture-sensitive floor covering (such as vinyl, tile, or carpet) is used, a moisture/vapor barrier should be placed per our recommendation in Section 4.7.

4.5.1 Exterior Concrete

To reduce the potential for excessive cracking of concrete flatwork (such as walkways, etc.), concrete should be a minimum of 4 inches thick and provided with construction or weakened plane joints at frequent intervals.

4.7 Moisture/Vapor Mitigation for Concrete Floor Slab-on-Grade

In order to reduce the potential for moisture/water vapor migration up through the slab and possibly affecting floor covering, a moisture/vapor retarder is recommended under concrete floor slab-on-grade. The moisture barrier should be properly installed, lapped and sealed in accordance with the manufacturer's specifications. Punctures and rips should be repaired prior to placement of sand.

Atlas recommends a qualified waterproofing consultant be retained in order to recommend a product or method which would provide protection for the concrete slabs-on-grade for your project based on the project needs. Please refer to the latest version of the "ACI Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials" for your design.

The moisture/water vapor protection for concrete slab-on-grade should be selected based on cost and construction considerations, and considering potential future problems resulting from improper and uncontrolled landscape irrigation practices. Regardless of the moisture/water vapor retarder option selected, it should be emphasized that proper control of irrigation and landscape water adjacent to the structure is of paramount importance.

4.8 Temporary Excavations

All temporary excavations, including utility trenches, pool and retaining wall excavations and other excavations should be performed in accordance with project plans, specifications and all Occupational Safety and Health Administration (OSHA) requirements.

No surcharge loads should be permitted within a horizontal distance equal to the height of cut or 5 feet, whichever is greater from the top of the slope, unless the cut is shored appropriately. Excavations that extend below an imaginary plane inclined at 45 degrees below the edge of any adjacent existing site foundation should be properly shored to maintain support of the adjacent structures.

Excavations located along property lines and adjacent to existing structures (i.e., buildings, walls, fences, etc.) should not be permitted within 2 feet from existing foundations.

4.9 Minor Retaining Wall

Minor retaining walls in the range of about 1½ to 4 feet in height may be associated with the improvements. The pressure behind retaining walls depends primarily on the allowable wall movement, wall inclination, type of backfill materials, backfill slopes, surcharge, and drainage. Determination of whether the active or at-rest condition is appropriate for design will depend on the flexibility of the walls. Walls that are free to rotate at least 0.002 radians at the top (deflection at the top of the wall of at least 0.002 x H, where H is the unbalanced wall height) can be designed for active conditions. The recommended active and at-rest pressures for the site soil backfill are presented in Table 5.

Table 5 – Earth Pressures for Retaining Walls

Wall Movement	Backfill Condition	Equivalent Fluid Pressure (on-site soil) (pcf)
Free to Deflect	Level	40
Restrained	Level	62

The above lateral earth pressures do not include the effects of surcharge (e.g., traffic, footings), hydrostatic pressure or compaction. Any surcharge (live, including traffic, or dead load) located within a 1:1 plane drawn upward from the base of the excavation should be added to the lateral earth pressures. The lateral pressure addition of a surcharge load located immediately behind walls may be calculated by multiplying the surcharge by 0.33 for cantilevered walls and 0.5 for restrained walls. For vehicular surcharge adjacent to driveways or parking areas a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot traffic surcharge, should be used.

The equivalent fluid pressures provided in Table 5 are based on a full drainage system behind the wall. A drainage system should be provided behind the walls to reduce the potential for development of hydrostatic pressure.

Walls should be properly drained and waterproofed. Except for the upper 2 feet, the backfill immediately behind retaining walls (minimum horizontal distance of 12 inches) should consist of free-draining, ¾-inch crushed rock wrapped with filter fabric. A 4-inch diameter perforated PVC pipe with perforations placed downward at the bottom of the crushed rock backfill, leading to a suitable gravity outlet, should be installed. If a drainage system is not installed, the walls should be designed to resist the hydrostatic pressure in addition to the earth pressure.

The wall footings should be underlain by 3 feet of engineered fill. The footing embedment should be at least 3 feet below the lowest adjacent grade. The maximum allowable bearing pressure recommended is 2,000 psf.

In the event of a large earthquake, the lateral earth pressure on a cantilever wall may be higher. We suggest using a dynamic earth pressure increment of 25 psf per foot for cantilever yielding walls with level backfill, assuming the wall will not exceed 6 feet in height. The pressure should be taken as an inverted triangular distribution with the zero-pressure point at the toe of the wall and 25H (psf where H in feet) at the top of the wall, where H is the wall height in feet. The point of application of the dynamic thrust may be taken at 0.6H above the toe of the wall. When combining both static and seismic lateral earth pressures, a decreased factor of safety may be used in design of retaining walls when checking for sliding and overturning stability. The Structural Engineer should determine if a seismic increment of lateral earth pressure is applicable based on wall heights and allowable wall movements.

4.10 Surface Drainage

All pad and roof drainage should be collected and transferred to an approved area in non-erosive drainage devices. Drainage should not be allowed to descend any slope in a concentrated manner, pond on the pad or against any foundation.

The CBC recommends a minimum 5% slope away from the perpendicular face of the building wall for a minimum horizontal distance of 10 feet (where space permits). We recommend a minimum 5% slope away from the building foundations for a horizontal distance of 3 feet be established for any landscape areas immediately adjacent to the building foundations. In addition, we recommend a minimum 2% slope away from the building foundations be established for any impervious surfaces immediately adjacent to the building foundations for a minimum horizontal distance of 10 feet (where space permits). Lastly, we recommend the installation of roof gutters and downspouts which deposit water into a buried drain system be installed instead of discharging surface water into planter areas adjacent to structures.

It is the responsibility of the contractor and ultimately the developer and/or property owner to ensure that all drainage devices are installed and maintained in accordance with the approved plans, our recommendations, and the requirements of all applicable municipal agencies. This includes installation and maintenance of all subdrain outlets and surface drainage devices. It is recommended that watering be limited or stopped altogether during the rainy season when little irrigation is required. Over-saturation of the ground can cause major subsurface damage. Maintaining a proper drainage system will minimize the hydro-collapse potential of sub-soils.

Drainage swales should not be constructed within 5 feet of building structure. Irrigation adjacent to buildings should be avoided wherever possible.

As an option, sealed-bottom planter boxes and/or drought resistant vegetation may be used within 5 feet of buildings.

4.11 Trench Backfill

Utility trenches should be backfilled with compacted fill in accordance with Sections 306-12 of the Standard Specifications for Public Works Construction, (“Greenbook”), 2018 Edition.

Utility trenches can be backfilled with on-site soils free of debris, organic and oversized material (maximum size not exceeding 3 inches). However, prior to backfilling utility trenches, pipes should be bedded in and covered with import granular material that has a Sand Equivalent (SE) value greater than 30. Bedding sands may be placed by mechanical compaction (rolling sheepsfoot wheel attached to backhoe) or by jetting. Native soil backfill over the pipe bedding zone should be placed in thin lifts – loose lift thickness not exceeding 8 inches – moisture conditioned as necessary, and mechanically compacted to a minimum of 90% relative compaction (per ASTM D1557) in paved and any structural areas. For the vehicular area, the upper 12 inches of the backfill material shall be compacted to 95% based on the recommendations provided in this report.

4.12 Preliminary Pavement Section

Below sections provide preliminary design for pavements based on the results of our R-Value tests. The design can be verified during construction with more R-Value tests.

4.12.1 Asphalt Concrete (AC) Pavement

The required pavement structural sections depend on the expected wheel loads, volume of traffic, and subgrade soils. The characteristics of subgrade soils are determined by R-value testing. Based on soil classification and the results of the R-value tests, we assumed two R-values, one for sandy silt and one for silty sand. The R-values should be verified and confirmed with additional tests, if necessary, at the time of construction. The following pavement sections were calculated based on assumed traffic indices of 4, 5, 6 and 7. The project Civil Engineer should determine the traffic index to be used for different areas of the site.

Table 6 – Asphalt Pavement Sections

Traffic Index	Assumed R-Value for Sandy Silt = 13		Conservatively Assumed R-Value for Silty Sand = 35	
	Asphalt Thickness (in)	Base Course (CAB) Thickness (in)	Asphalt Thickness (in)	Base Course (CAB) Thickness (in)
4	3.0	4.5	3.0	4.5
5	4.0	6.0	3.5	4.5
6	5.5	7.0	4.5	5.0
7	6.5	8.0	5.0	6.5

Base course material should consist of Crushed Aggregate Base (CAB) as defined by Section 200-2.2 of the Standard Specifications for Public Works Construction (“Greenbook”). Base course should be compacted to at least 95% of the maximum dry density of that material. Crushed Miscellaneous Base (CMB) may be used only if the supplier can demonstrate that the aggregate does not contain contaminated material.

The subgrade underlying the pavement areas should be overexcavated 18 inches below the proposed base course layer. Prior to fill placement, the subgrade should be scarified to a minimum depth of 8 inches, moisture conditioned within 2% of optimum moisture content, and compacted to at least 90% of the maximum dry density obtained per ASTM D1557. The upper 12 inches of subgrade should be compacted to 95% relative compaction. The subgrade should be in a "non-pumping" condition at the time of compaction.

Any on-site surficial organic soils within landscaped/turf areas should not be used as subgrade materials. Where feasible, the overexcavation should be laterally extended a minimum of 2 feet beyond the perimeters and edges of parking areas, roadways and curbs. Any abandoned footing and/or underground concrete structure within the work limit should be removed entirely and the excavation should be backfilled to grade.

4.12.2 Portland Cement Concrete Pavement

The grading recommendations for vehicular Portland Cement Concrete (PCC) pavement are generally provided in Section 4.1 (and the pertinent subsections) of this report. Base course material, used in the vehicular pavement sections, should consist of Crushed Aggregate Base (CAB) as defined by Section 200-2.2 of the Standard Specifications for Public Works Construction (Greenbook 2018). The aggregate base course should be compacted to at least 95% of the maximum dry density of that material. Crushed Miscellaneous Base (CMB) may be used only if the supplier can demonstrate that the aggregate does not contain contaminated material.

The recommendations presented herein should be used for design and construction of the slabs and pertaining grading work underlying the vehicular pavement area. A minimum modulus of rupture of 550 pounds per square inch (psi) for concrete has been assumed in designing of the PCC pavement sections; this corresponds to a concrete compressive strength of approximately 4,000 psi at 28 days. A qualified design professional should specify where heavy duty and standard duty slabs are used based on the anticipated type and frequency of traffic. Fire access roads are normally considered heavy duty pavement. The preliminary recommended vehicular PCC pavement sections are provided in Table 7.

Table 7 – Vehicular PCC Pavement Sections

Pavement Type	Portland Cement Concrete Thickness (inches)	Base Course (CAB) Thickness (inches)
Light Duty	6.5	6
Heavy Duty	7.0	6

The above pavement sections can be verified during construction of the projects. These vehicular concrete pavement sections should be increased for bus and very heavy traffic where applicable. The following recommendations should also be incorporated into the design and construction of PCC pavement.

- The pavement sections should be reinforced with No. 3 rebars spaced at 18 inches on centers each way to reduce the potential for shrinkage cracking.
- Joint spacing in feet should not exceed twice the slab thickness in inches, e.g., 12 feet for a 6-inch thick slab. Regardless of slab thickness, joint spacing should not exceed 15 feet.

- Layout joints should form square panels. When this is not practical, rectangular panels can be used if the long dimension is no more than 1.5 times the short one.

- Control joints should have a depth of at least 1/4 the slab thickness, e.g., 1 inch for a 4-inch thick slab.

- Pavement section design assumes that proper maintenance such as sealing and repair of localized distress will be performed on a periodic basis.

- The recommendations for PCC provided in this section should be verified and confirmed if necessary, at the time of construction.

- The upper 12 inches of subgrade should be compacted to at least 95% relative compaction (ASTM D1557)

4.13 General Note for Concrete and Rebar Recommendation

The requirements for concrete and rebar for slabs, concrete flat works, concrete pavements,…presented in this report are preliminary recommendations. The Project Design/Civil/Structural Engineer should provide the final recommendations for structural design of concrete and rebar for foundation system, floor slab, exterior concrete, slab on grade, concrete pavements and, … in accordance with the latest version of the applicable codes and standards.

4.14 Percolation Test

We performed four percolation tests, two deep borehole tests and two shallow borehole tests to assess storm water infiltration feasibility, in general conformance with the County of Los Angeles testing guidelines.

Based on the County of Los Angeles testing guidelines the raw flow rate for the borehole percolation tests were estimated by calculating the volume of water discharged into the bore hole (cubic feet) in a given amount of time (hr). To find the raw measured infiltration rate, the stabilized flow rate was divided by surface area of the hole test (sum of all wetted areas including the bottom surface area of the boring and sidewalls). The measured stabilized flow rate and raw measured percolation rate are provided in Tables 8 and 9. The values provided in the tables do not included reduction factors for the test procedure (RFt), site variability (RFv) and long-term siltation plugging (RFs) that are considered in order to assess long-term design infiltration rate. The borehole percolation tests were performed using relatively clean water free of particulates, silt etc.

The long-term infiltration rate is the raw measured infiltration rate dividing by a series of reduction factors including test procedure (RFt), site variability (RFv) and long-term siltation plugging and maintenance (RFs). The preliminary recommended reduction factors are presented in Table 10. The reduction factors can be finalized by the designed Engineer. The long-term infiltration rate is the raw measured infiltration rate divided by the total reduction factor (RFt x RFv x RFs).

Table 8 – Deep Borehole Percolation Rate Test Results

Test Location	Test Depth (feet)	Test Head (Water Column) (feet)	Total Test Water (gallons)	Stabilized Flow Rate (cf/hr)	Raw Measured Infiltration Rate (ft/hr)
B-11/BP-2	25	19	168.3	3.2	0.08
B-13/BP-3	25	19	162.0	4.3	0.11

Table 9 – Shallow Borehole Percolation Rate Test Results

Test Location	Test Depth (feet)	Test Head (Water Column) (feet)	Total Test Water (gallons)	Stabilized Flow Rate (cf/hr)	Raw Measured Infiltration Rate (ft/hr)
B-14/BP-1	5	1	7.2	0.4	0.2
BP-4	5	1	16.2	0.9	0.4

Table 10 – Reduction Factors

Reduction Factor	Factor
Test procedure, boring percolation, RFt	2
Site variability, number of tests, etc. RFv	2
Long-term siltation plugging and maintenance, RFs	Assumed 3
Total Reduction Factor, RF = RFt x RFv x RFs	12

The results of our percolation tests indicate that the shallow silty SAND layers have more infiltration rate than the deep Silty layer. Based on the results of the percolation tests, the average raw measured infiltration rate is 0.095 ft/hr (1.1 in/hr) for the deep borehole tests and 0.3 ft/hr (3.6 in/hr) for the shallow borehole tests. Considering a reduction factor of 12, we recommended long-term infiltration rate of 0.0079 ft/hr (0.09 in/hr) for the deep borehole tests (Sandy SILT:ML) and 0.0255 ft/hr (0.30 in/hr) for the shallow boreholes (Silty SAND:SM). The recommended infiltration rates can be verified by the designed engineer.

It should be noted that the in-situ field percolation tests performed provide short-term infiltration rates, which apply mainly to the initiation of the infiltration process due to the short time of the test (hours instead of days) and the amount of water used. The small-scale percolation testing cannot model the complexity of the effect of interbedded layers of different soil composition, and our test results should be considered only as index values of infiltration rates. Please note that the results of our percolation/infiltration study are based on our field measurements at the certain depth of the tested boreholes. Other depths and locations generally may have similar, less or higher values for percolation/infiltration rates.

4.15 Construction Observation and Testing

All excavation and grading during construction should be performed under the observation and testing of the geotechnical consultant at the following stages:

- Upon removal of the upper soils to the proposed excavation/overexcavation bottoms
- During preparation of the removal bottoms, any fill placement, and grading for the proposed improvements
- During preparation of the footing subgrades
- When any unusual or unexpected geotechnical conditions are encountered

4.16 Limitations

The conclusions and recommendations in this report are based in part upon data that were obtained from a limited number of soil samples and laboratory test results. Such information is by necessity limited. Subsurface conditions may vary across the site. Therefore, the findings, conclusions, and recommendations presented in this report can be relied upon only if Atlas has the opportunity to observe the subsurface conditions during grading and construction of the project, in order to confirm that our findings are representative for the site.

This report is not authorized for use by and is not to be relied upon by any party except, Compton Community College District, their successors and assignees as the owner of the property. Use of or reliance on this report by any other party is at that party's risk. Unauthorized use of or reliance on this report constitutes an agreement to defend and indemnify Atlas from and against liability, which may arise as a result of such use or reliance.

Geotechnical investigation and relevant engineering evaluations for this project were performed in substantial conformance with the general practices of geotechnical engineering in southern California at the time of this report. No other warranty is expressed or implied.

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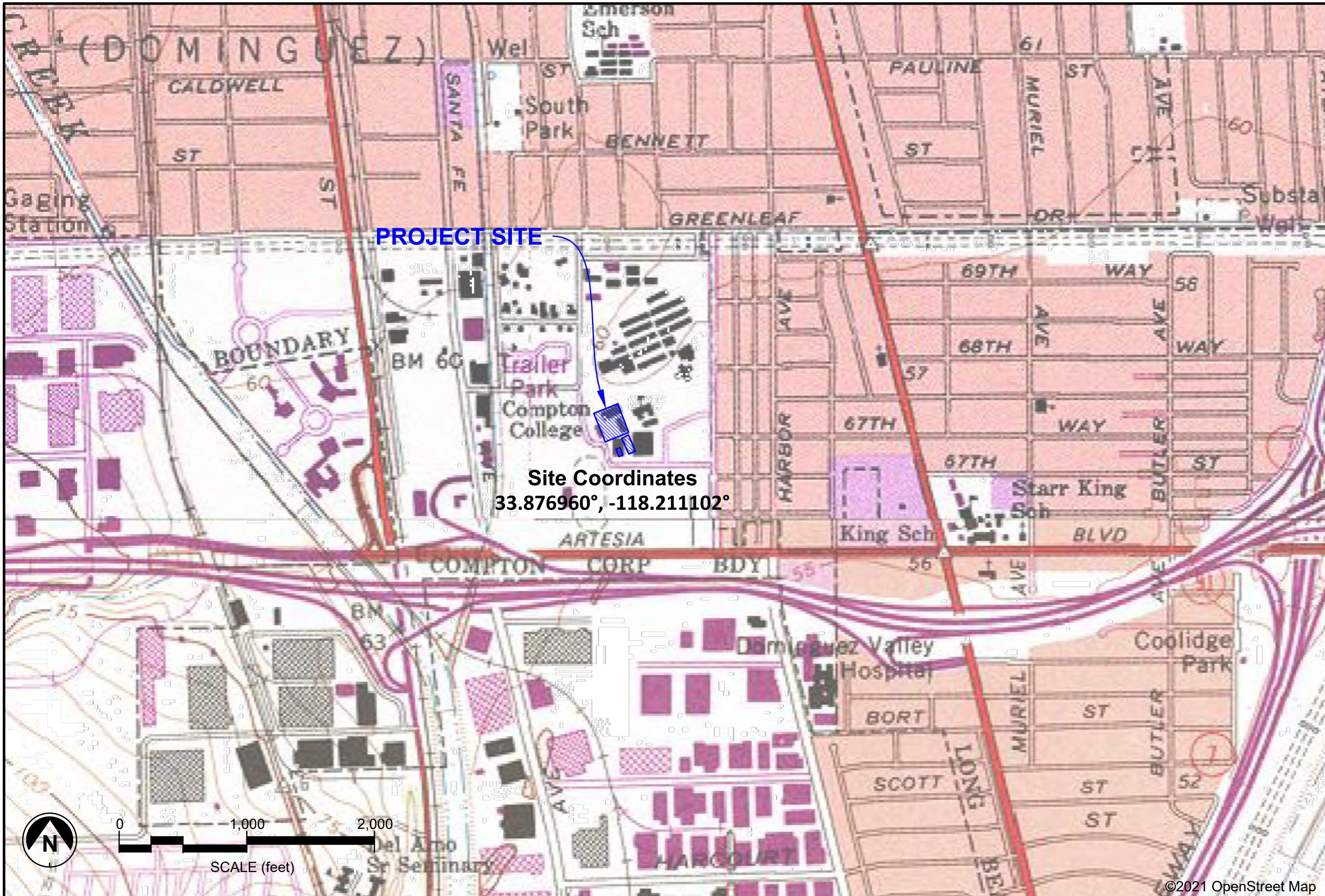
APPENDIX I
FIGURES

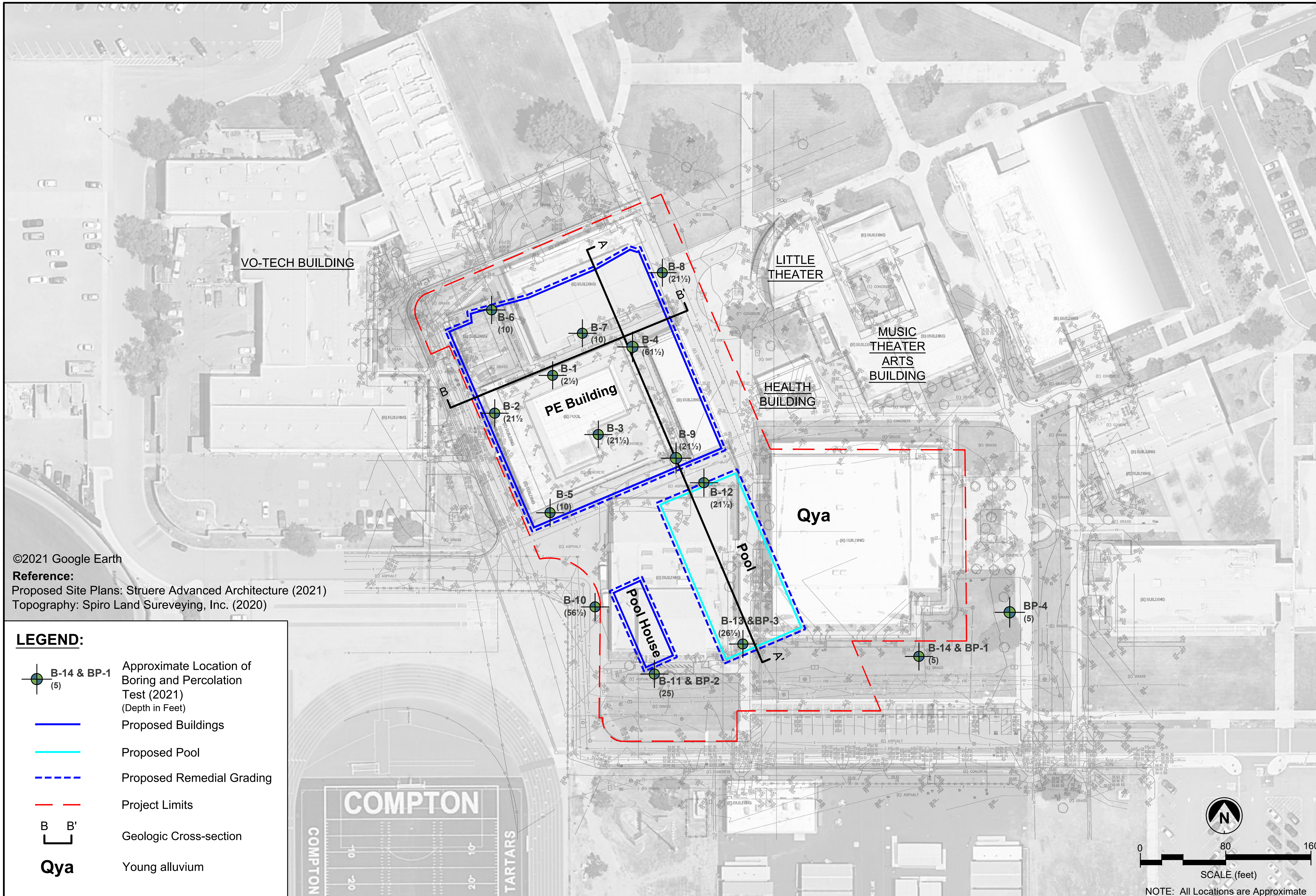


ATLAS

SITE VICINITY MAP
Physical Education Complex Replacement
Compton, California

Date:	April, 2021
By:	ACF
Job No.:	10-57575PW

Figure:
1

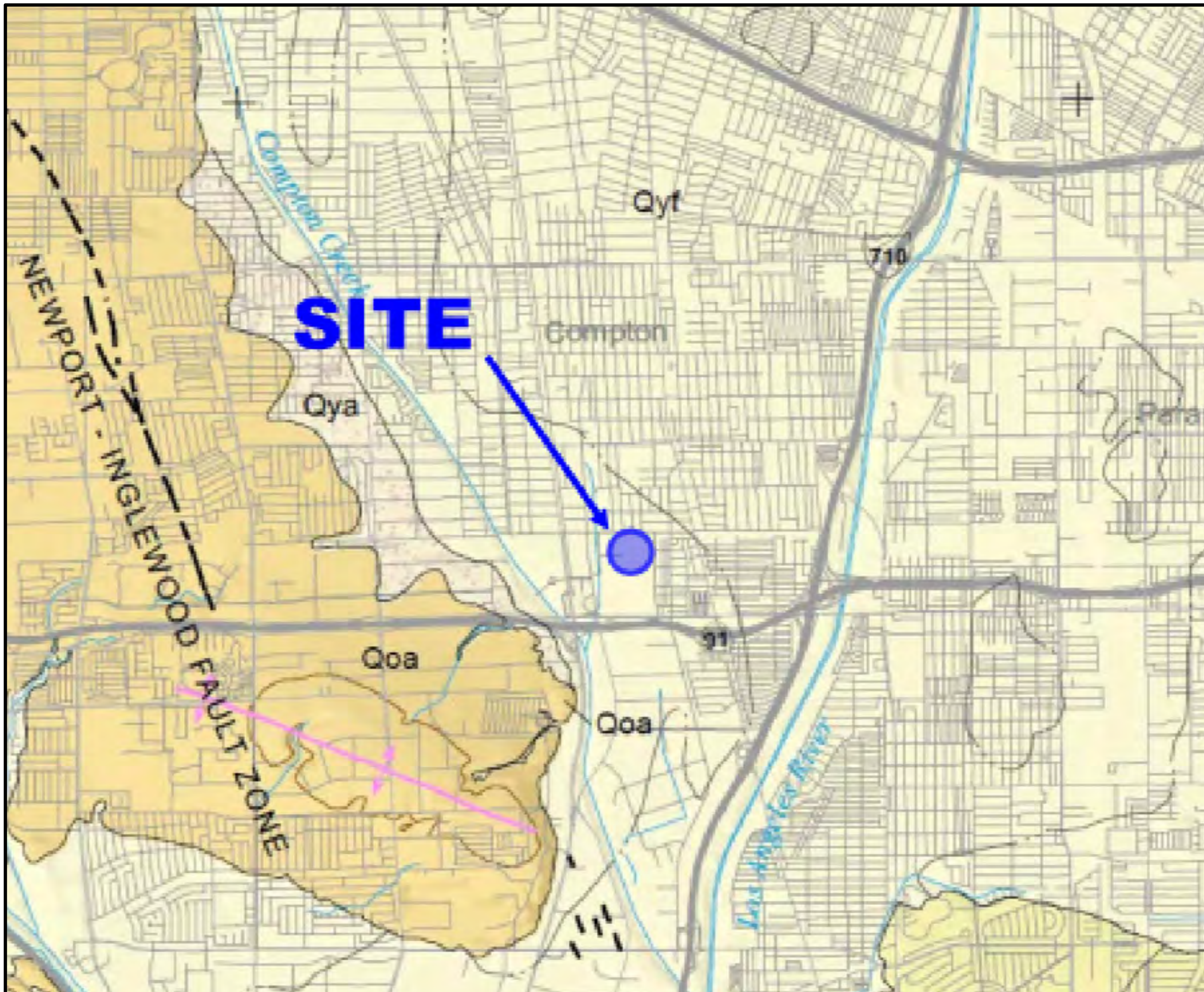


Date: April, 2021
By: ACF
Job No.: 10-57575PW

GEOTECHNICAL MAP
Physical Education Complex Replacement
Compton, California

ATLAS

Figure: 2



EXPLANATION:

Qyf Young alluvial flood-plain deposits (Holocene and late Pleistocene)

Qya Young alluvial flood-plain deposits (Holocene and late Pleistocene)

Qoa Old alluvial flood-plain deposits, undivided

Anticline Fold - Solid where well defined; short dash where inferred

Syncline Fold - Solid where well defined; short dash where inferred

70 U D **Fault** - Solid where accurately located; dashed where approximately located; dotted where concealed. U = upthrown block, D = downthrown block. Arrow and number indicate direction and angle of dip of fault plane.

N

0 6,000 12,000

SCALE (feet)

NOTE: All locations are approximate.

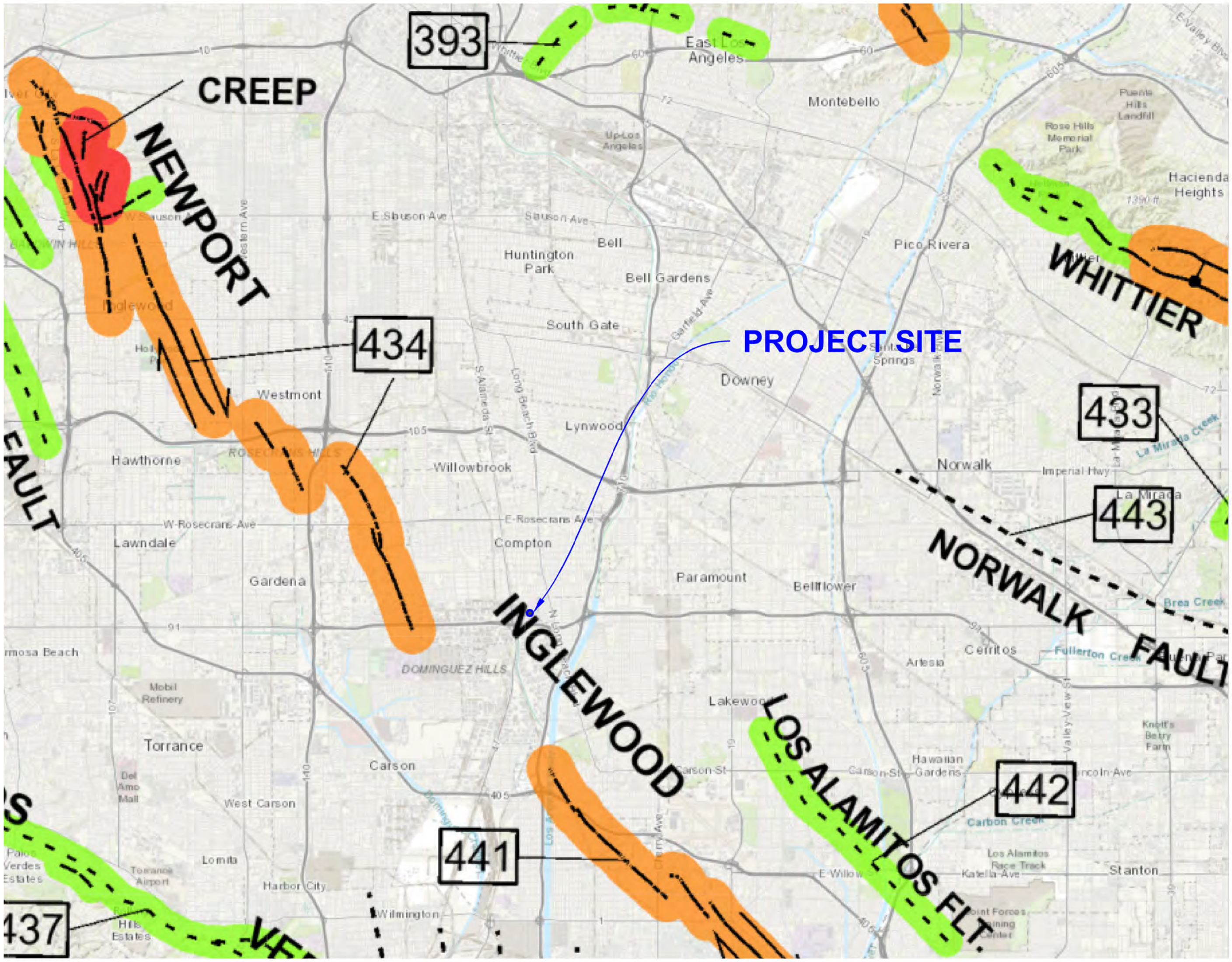
Reference:
CGS (200) Geologic Compilation of Quaternary Surficial Deposits in Southerm California Onshore Portion of the Long Beac 30'60 Quadrangle; CGS Special Report 217, Plate 8.

ATLAS

REGIONAL GEOLOGY MAP
Physical Education Complex Replacement
Compton, California

Date:	April, 2021
By:	ACF
Job No.:	10-57575PW

Figure: 3



EXPLANATION:

Fault along which historic (last 200 years) displacement has occurred

Holocene fault displacement (during past 11,700 years) without historic record.

Late Quaternary fault displacement (during past 700,000 years).

Quaternary fault (age undifferentiated).

Pre-Quaternary fault (older than 1.6 million years) or fault without recognized Quaternary displacement.

Low angle fault (barbs on upper plate).

393	Unnamed fault - Unnamed fault west of Monterey Park (conceadled)
433	Faults in W. Coyote Hills - Faults in W. Coyote Hills (certain)
434	Avalon-Compton Fault - Newport-Inglewood-Rose Canyon fault zone
441	Cherry-Hill Fault - Newport-Inglewood-Rose Canyon fault zone (concealed)
442	Los Alamitos Fault - Los Alamitos Fault fault zone (concealed)

N

0 15,000 30,000

SCALE (feet)

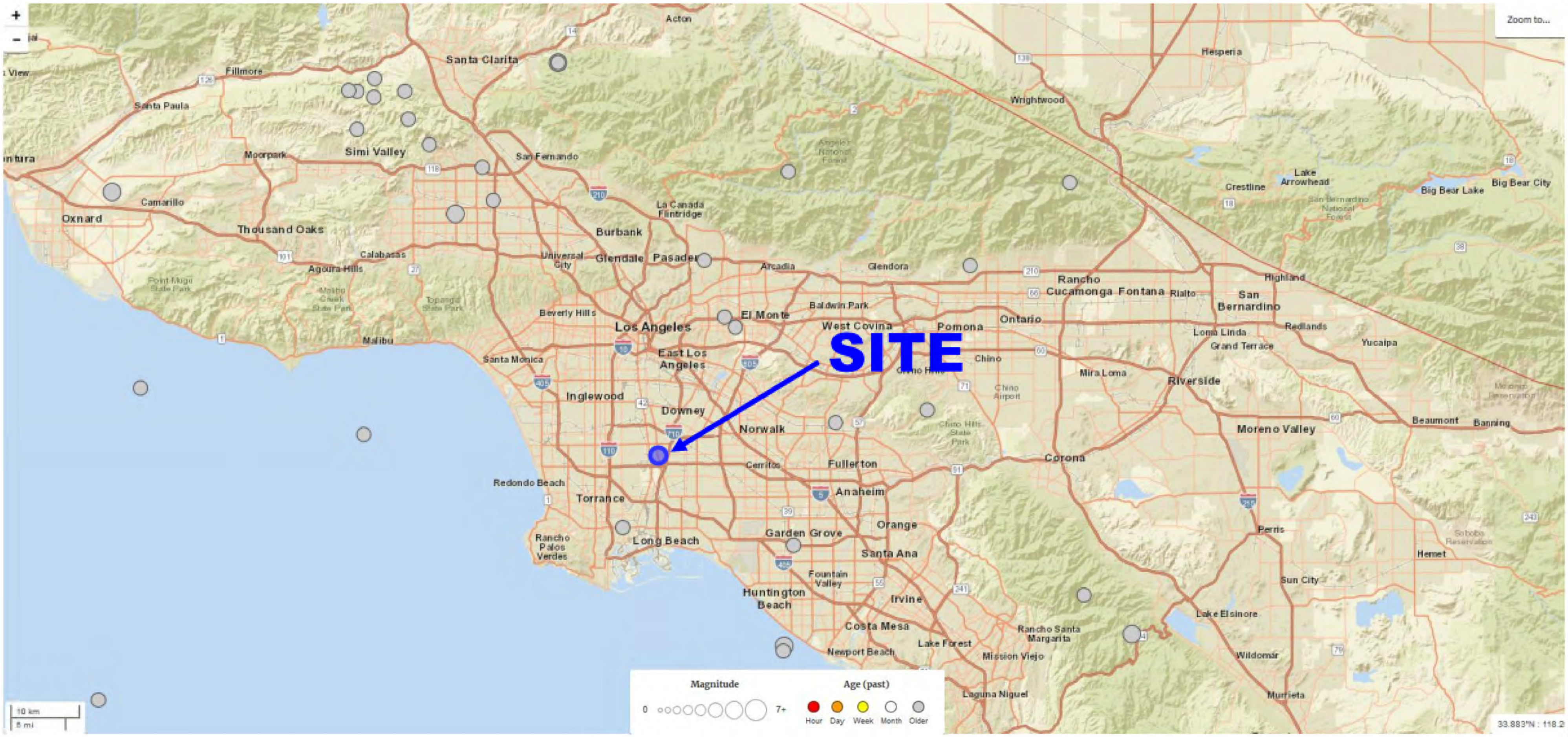
Reference:
Jennings, C.W., Bryant W.A., Esri, HERE, Garmin, FAO, NOAA, USGS, EPA, Fault Activity Map of California (2010), maps.conservation.ca.gov/cgs/fam; Accessed January, 2021

ATLAS

REGIONAL FAULT MAP
Physical Education Complex Replacement
Compton, California

Date: April, 2021
By: ACF
Job No.: 10-57575PW

Figure:
4

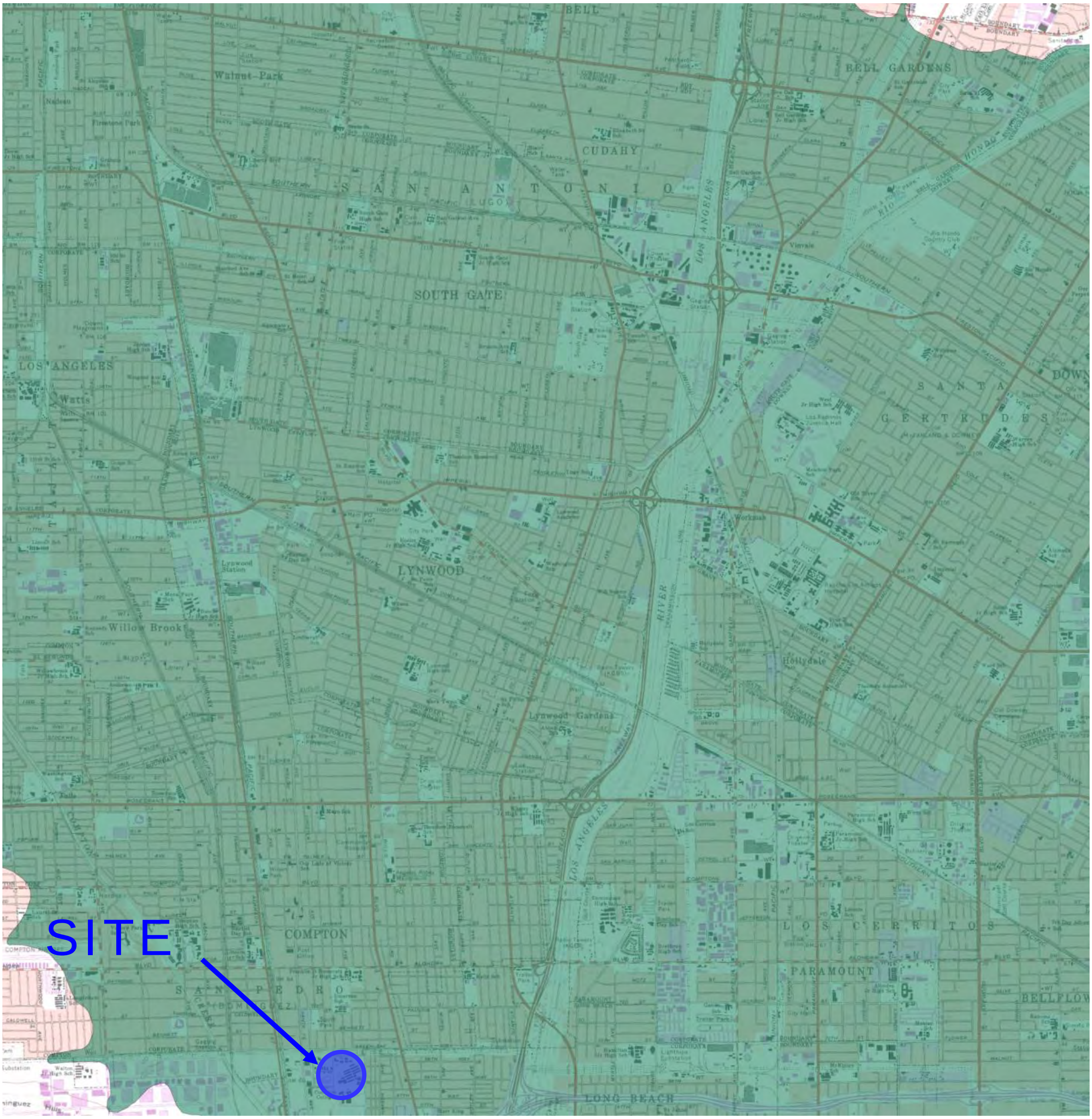


Location of Historic Earthquake Epicenter (M_w>5)

N

Reference:
https://earthquake.usgs.gov/earthquakes/map/, Accessed January, 2021

ATLAS	**REGIONAL SEISMICITY MAP** Physical Education Complex Replacement Compton, California	Date: April, 2021 By: ACF Job No.: 10-57575PW	Figure: 5



LIQUEFACTION

Liquefaction
Areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslides
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

REFERENCE: California Geologic Survey, 2016, Earthquake Zones of Required Investigation, South Gate Quadrangle, Los Angeles County, California;.

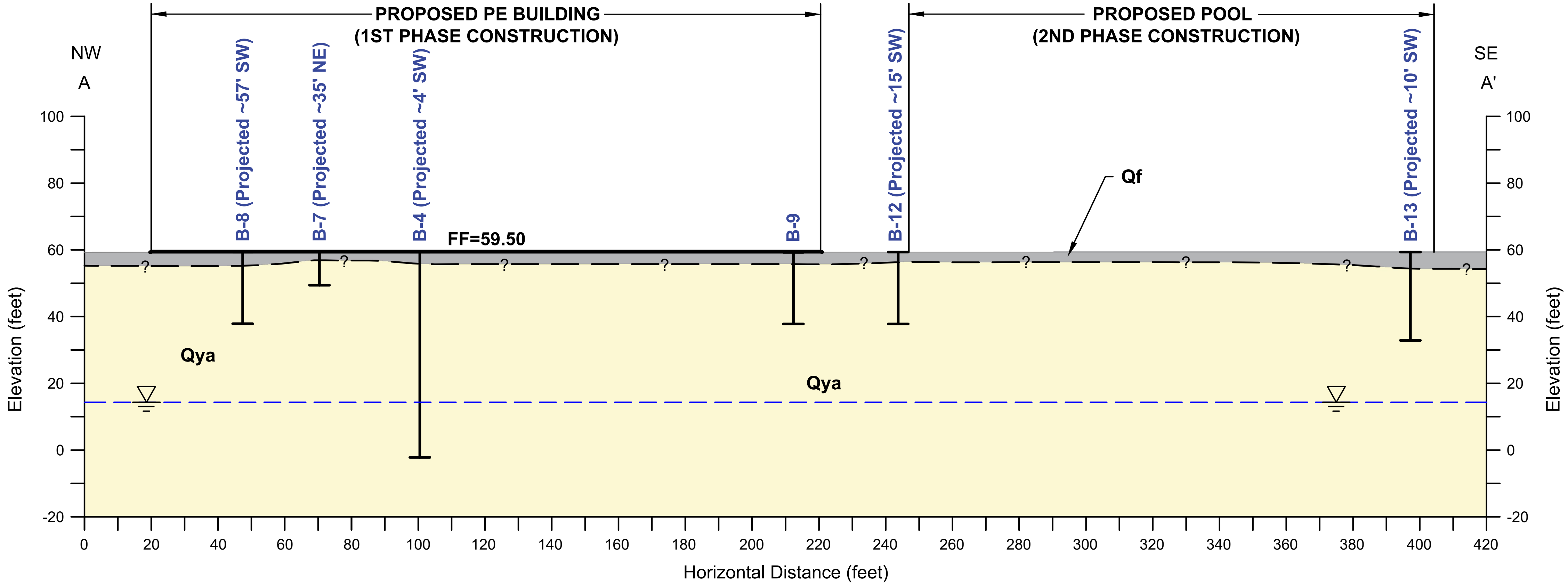
ATLAS

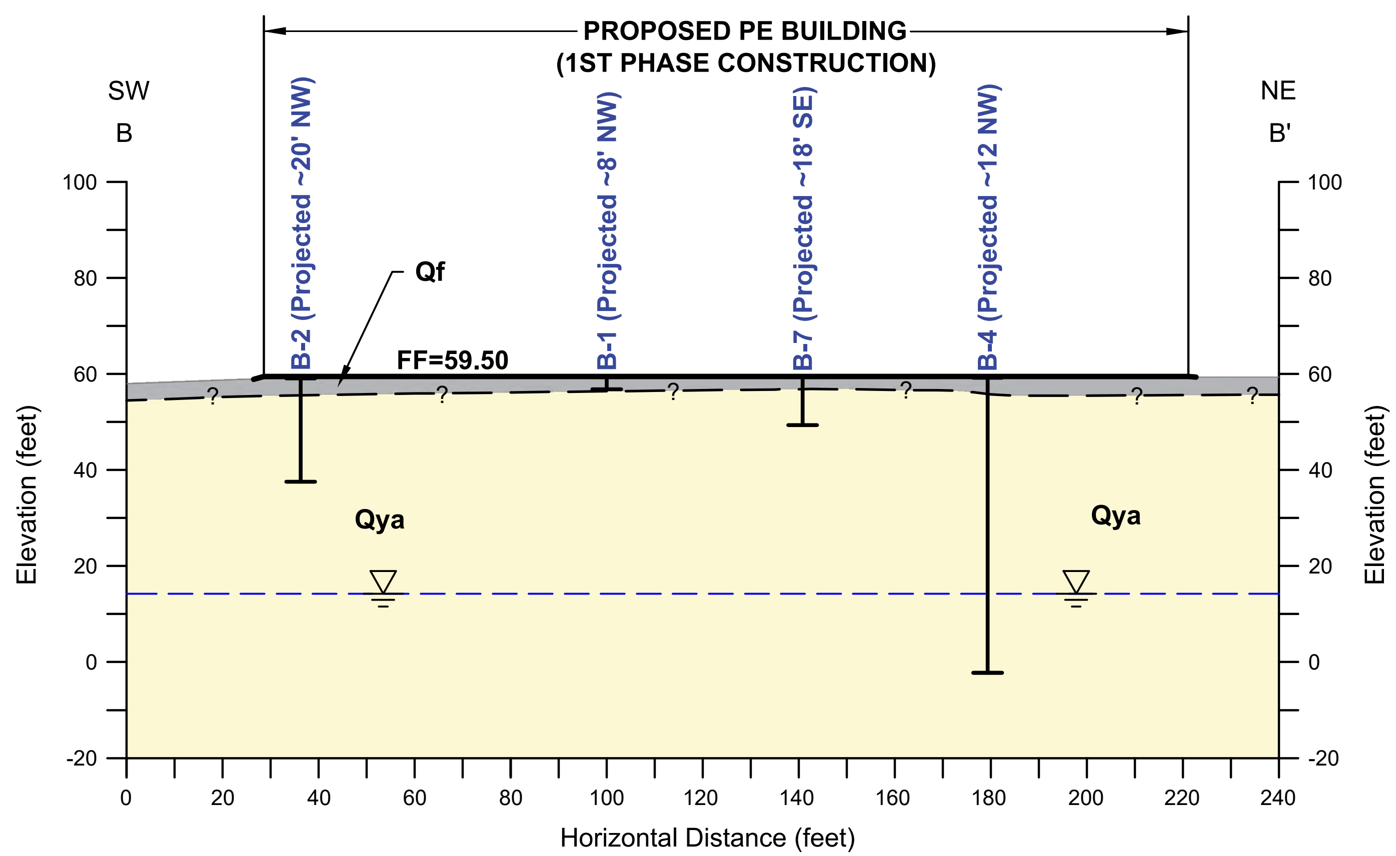
Figure 6 – Liquefaction Susceptibility Map

Physical Education Complex Replacement
Compton, California

N

Date:	April, 2021
By:	ACF
Job No.:	10-57575PW





LEGEND:

B-13 Location of Boring

Qf Fill

Qya Young Alluvium

Groundwater

—?— Geologic Contact, Queried Where Uncertain

FF = Finish Floor

0 40 80
SCALE (feet)

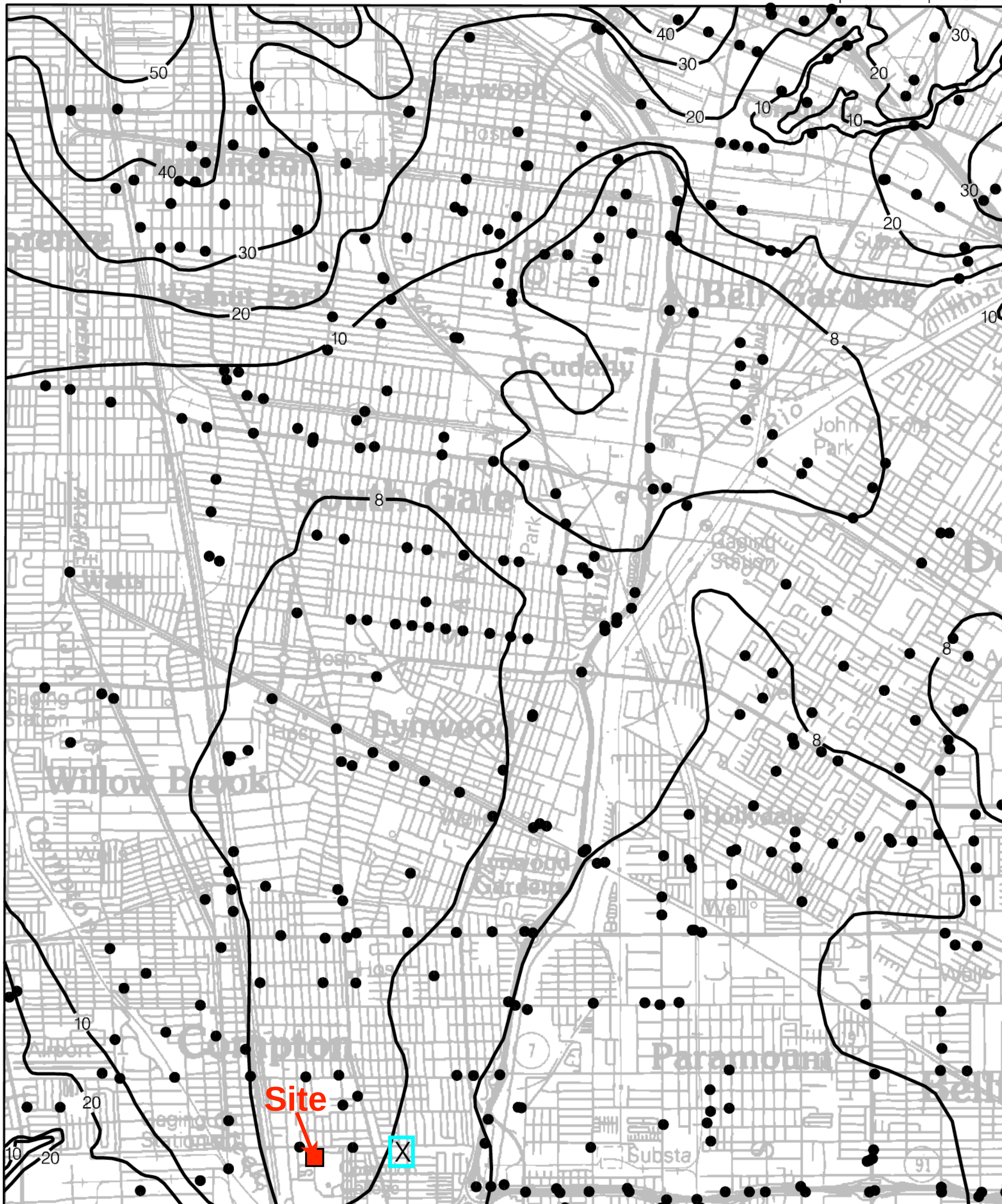
NOTE: All locations and depths are approximate.

Date: April, 2021
By: ACF
Job No.: 10-57575PW

GEOLOGIC CROSS-SECTIONS
Physical Education Complex Replacement
Compton, California

Figure:
7

Open–File Report 98–25



Base map enlarged from U.S.G.S. 30 x 60–minute series

Plate 1.2 Historically Highest Ground Water Contours and Borehole Log Data Locations, South Gate Quadrangle.

● Borehole Site

30 Depth to ground water in feet

X Site of historical earthquake–generated liquefaction. See "Areas of Past Liquefaction" discussion in text.

ONE MILE

SCALE

APPENDIX II
FIELD EXPLORATION

The field investigation was performed on March 2, 2020 under the supervision of an Atlas representative. A staff engineer performed a site reconnaissance to identify exploratory locations. The exploratory boring locations for the project were marked in the field by our staff engineer from existing site features. Atlas notified Underground Service Alert (USA) to identify the locations of subsurface utilities that may be in potential conflict with the boring locations. Geophysics test performed on site to find the approximate location of the underground utilities.

Subsurface exploration included drilling and sampling of 15 borings to depths ranging from about 5 feet to 61.5 feet below the ground surface within the project improvements. All the soil investigation borings and percolation borings were drilled with the diameter of 8 inches. The borings were drilled using a CME - 75 drilling rig (hollow stem auger) or hand auger. Relatively undisturbed soils samples and standard penetration tests samples were collected at regular intervals. The relatively undisturbed samples were obtained using California samplers. Standard penetration tests were also performed in general accordance with ASTM D1586. The sampler was driven 18 inches into the subsurface soils using a 140 pound hammer with a 30 inch drop. The number of blows (blow count) to drive the sampler into the subsurface soils were recorded at 6-inch intervals, and the blow counts required to drive the sampler the final 12 inches are recorded on the boring logs. The borings were backfilled with appropriate soils and materials. The boring records are presented in this Appendix.

LOG OF TEST BORING

ATLAS PROJECT NAME	ATLAS PROJECT NUMBER	
Compton College PE Complex Replacement	10-57575PW	B-1

SITE	START	END	SHEET NO.
Compton, California	3/2/21	3/2/21	1

DRILLING COMPANY	DRILL METHOD	LOGGED BY	REVIEWED BY
Baja Exploration	Hand Auger	KBH	MJ

DRILLING EQUIPMENT	BORING DIA. (in.)	TOTAL DEPTH (ft)	GROUND ELEV. (ft)	DEPTH/ELEV. GROUND WATER (ft)
	8	2.5	0	AT TIME OF DRILLING ---

SAMPLING METHOD	NOTES	
	Hammer Efficiency = 73.9% N_{60}~$1.23N_{SPT}$	AT END OF DRILLING --- AFTER DRILLING ---

ELEVATION (ft)	DEPTH (ft)	BULK SAMPLE	DRIVE SAMPLE	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
					6 inches of grass and topsoil.
					FILL (af):SILTY SAND (SM), loose, medium brown, dry, fine to medium grained, micaceous, rootlets.
-2.5	2.5				**BORING TERMINATED AT 2½ FEET ON GRAVEL BEDDING**
-5.0	5.0				
-7.5	7.5				

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

Figure II-1

LOG OF TEST BORING

ATLAS PROJECT NAME	ATLAS PROJECT NUMBER	
Compton College PE Complex Replacement	10-57575PW	B-2

SITE	START	END	SHEET NO.
Compton, California	3/2/21	3/2/21	2

DRILLING COMPANY	DRILL METHOD	LOGGED BY	REVIEWED BY
Baja Exploration	Hollow Stem Auger	KBH	MJ

DRILLING EQUIPMENT	BORING DIA. (in.)	TOTAL DEPTH (ft)	GROUND ELEV. (ft)	DEPTH/ELEV. GROUND WATER (ft)
CME-75	8	21.5	0	∇ AT TIME OF DRILLING ---

SAMPLING METHOD	NOTES	
140-lb Hammer, 30-in Drop	Hammer Efficiency = 73.9% N_{60}~$1.23N_{SPT}$	▼ AT END OF DRILLING --- ∇ AFTER DRILLING ---

ELEVATION (ft)	DEPTH (ft)	BULK SAMPLE	DRIVE SAMPLE	BLOWS PER FOOT	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
									6 inches of grass and topsoil.
		X							**FILL (Qf):** SILTY SAND (SM), medium dense, brown, damp, fine to medium grained, micaceous.
									YOUNG ALLUVIUM(Qya): SILTY SAND (SM), medium dense, grayish brown, moist, fine to medium grained, micaceous.
-5	5		CAL	15		9.5	93.3		
-10	10		SPT	7	9				SANDY SILT (ML), loose to medium dense, brown, moist, mostly fine grained, micaceous.
-15	15		CAL	15					
-20	20		SPT	11	14				

BORING TERMINATED AT 21.5 FEET

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

Figure

II-2

LOG OF TEST BORING

ATLAS PROJECT NAME	ATLAS PROJECT NUMBER	
Compton College PE Complex Replacement	10-57575PW	B-3

SITE	START	END	SHEET NO.
Compton, California	3/2/21	3/2/21	3

DRILLING COMPANY	DRILL METHOD	LOGGED BY	REVIEWED BY
Baja Exploration	Hollow Stem Auger	KBH	MJ

DRILLING EQUIPMENT	BORING DIA. (in.)	TOTAL DEPTH (ft)	GROUND ELEV. (ft)	DEPTH/ELEV. GROUND WATER (ft)
CME-75	8	21.5	0	▽ AT TIME OF DRILLING ---

SAMPLING METHOD	NOTES	
140-lb Hammer, 30-in Drop	Hammer Efficiency = 73.9% N_{60}~1.23N_{SPT}	▼ AT END OF DRILLING --- ▼ AFTER DRILLING ---

ELEVATION (ft)	DEPTH (ft)	BULK SAMPLE	DRIVE SAMPLE	BLOWS PER FOOT	N_{60}	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION	LAB TESTS
							2 inches of Asphalt over 4 inches of Base	
		X					**FILL (Qf):** SILTY SAND (SM), loose, brown, dry, fine to medium grained, micaceous.	MAX
							YOUNG ALLUVIUM(Qya): SILTY SAND (SM), loose, grayish brown, moist, fine to medium grained, micaceous, silt lenses.	
-5	5		SPT	9	11			
-10	10		CAL	10				
							SANDY SILT (ML), medium dense, brown, moist, mostly fine grained, micaceous.	
-15	15		SPT	11	14		SILTY SAND (SM), medium dense, grayish brown, moist, fine to medium grained, micaceous, minor oxidation.	
-20	20		CAL	21				

BORING TERMINATED AT 21.5 FEET

ATLAS

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

Figure II-3

LOG OF TEST BORING

ATLAS PROJECT NAME	ATLAS PROJECT NUMBER	
Compton College PE Complex Replacement	10-57575PW	B-4

SITE	START	END	SHEET NO.
Compton, California	3/2/21	3/2/21	4

DRILLING COMPANY	DRILL METHOD	LOGGED BY	REVIEWED BY
Baja Exploration	Hollow Stem Auger	KBH	MJ

DRILLING EQUIPMENT	BORING DIA. (in.)	TOTAL DEPTH (ft)	GROUND ELEV. (ft)	DEPTH/ELEV. GROUND WATER (ft)
CME-75	8	61.5	0	AT TIME OF DRILLING 44.50 ft / Elev -44.50 ft

SAMPLING METHOD	NOTES	
140-lb Hammer, 30-in Drop	Hammer Efficiency = 73.9% N_{60}~$1.23N_{SPT}$	AT END OF DRILLING ---
		AFTER DRILLING ---

ELEVATION (ft)	DEPTH (ft)	BULK SAMPLE	DRIVE SAMPLE	BLOWS PER FOOT	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION	LAB TESTS
									6 inches of grass and topsoil.	
		X							**FILL (af)**:SANDY SILT (ML), loose, brown, dry, fine to medium grained, micaceous, rootlets.	EI, COR
									YOUNG ALLUVIAL FAN DEPOSITS (Qyf): SILTY SAND (SM), medium dense, grayish brown, dry to damp, fine to medium grained, micaceous.	
-5	5		SPT	13	16					
-10	10		CAL	15						DS
-15	15		SPT	14	17					

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

Figure II-4

LOG OF TEST BORING	ATLAS PROJECT NAME Compton College PE Complex Replacement	ATLAS PROJECT NUMBER 10-57575PW	B-4

SITE	START	END	SHEET NO.
Compton, California	3/2/21	3/2/21	5

DRILLING COMPANY	DRILL METHOD	LOGGED BY	REVIEWED BY
Baja Exploration	Hollow Stem Auger	KBH	MJ

DRILLING EQUIPMENT	BORING DIA. (in.)	TOTAL DEPTH (ft)	GROUND ELEV. (ft)	DEPTH/ELEV. GROUND WATER (ft)
CME-75	8	61.5	0	▽ AT TIME OF DRILLING 44.50 ft / Elev -44.50 ft

SAMPLING METHOD	NOTES	
140-lb Hammer, 30-in Drop	Hammer Efficiency = 73.9% N_{60}~$1.23N_{SPT}$	▼ AT END OF DRILLING --- ▼ AFTER DRILLING ---

ELEVATION (ft)	DEPTH (ft)	BULK SAMPLE	DRIVE SAMPLE	BLOWS PER FOOT	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION	LAB TESTS
			CAL	26		19.9	104.8		SANDY SILT (ML), medium dense, gray, moist, fine to medium grained, minor oxidation, micaceous, variable silt and sand lensing.	WA 65.1%
-25	25		SPT	7	9				Loose.	AL
-30	30		CAL	23		25.1	97.4		Medium dense.	WA 89.7%
-35	35		SPT	22	27					

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

Figure

II-5

LOG OF TEST BORING

ATLAS PROJECT NAME Compton College PE Complex Replacement		**ATLAS PROJECT NUMBER** 10-57575PW		**B-4**
SITE Compton, California	**START** 3/2/21	**END** 3/2/21		**SHEET NO.** 6
DRILLING COMPANY Baja Exploration	**DRILL METHOD** Hollow Stem Auger	**LOGGED BY** KBH		**REVIEWED BY** MJ
DRILLING EQUIPMENT CME-75	**BORING DIA. (in.)** 8	**TOTAL DEPTH (ft)** 61.5	**GROUND ELEV. (ft)** 0	**DEPTH/ELEV. GROUND WATER (ft)**
SAMPLING METHOD 140-lb Hammer, 30-in Drop	**NOTES** Hammer Efficiency = 73.9% N_{60}~$1.23N_{SPT}$			∇ AT TIME OF DRILLING 44.50 ft / Elev -44.50 ft; ▼ AT END OF DRILLING ---; ▼ AFTER DRILLING ---

ELEVATION (ft)	DEPTH (ft)	BULK SAMPLE	DRIVE SAMPLE	BLOWS PER FOOT	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION	LAB TESTS
			CAL	23					SILTY SAND (SM), medium dense, gray, moist, fine grained, minor oxidation, variable silt and sand lensing.	
								∇		
-45	45		SPT	10	12				SANDY SILT (ML), medium dense, gray, moist, fine to medium grained, minor oxidation, micaceous, variable silt and sand lensing.	AL
-50	50		CAL	25		34.8	84.9		Dark gray	WA 66.4%
-55	55		SPT	36	44				SILTY SAND (SM), dense, dark gray, moist, fine grained, minor oxidation, variable silt and sand lensing, saturated.	

ATLAS

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

Figure II-6

LOG OF TEST BORING	ATLAS PROJECT NAME: Compton College PE Complex Replacement	ATLAS PROJECT NUMBER: 10-57575PW	B-4

SITE	START	END	SHEET NO.
Compton, California	3/2/21	3/2/21	7

DRILLING COMPANY	DRILL METHOD	LOGGED BY	REVIEWED BY
Baja Exploration	Hollow Stem Auger	KBH	MJ

DRILLING EQUIPMENT	BORING DIA. (in.)	TOTAL DEPTH (ft)	GROUND ELEV. (ft)	DEPTH/ELEV. GROUND WATER (ft)
CME-75	8	61.5	0	AT TIME OF DRILLING 44.50 ft / Elev -44.50 ft; AT END OF DRILLING ---; AFTER DRILLING ---

SAMPLING METHOD	NOTES
140-lb Hammer, 30-in Drop	Hammer Efficiency = 73.9% N_{60}~$1.23N_{SPT}$

ELEVATION (ft)	DEPTH (ft)	BULK SAMPLE	DRIVE SAMPLE	BLOWS PER FOOT	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION	LAB TESTS
			CAL	80					SILTY SAND (SM), dense, dark gray, moist, fine grained, minor oxidation, variable silt and sand lensing, saturated. *(continued)*	
									BORING TERMINATED AT 61½ FEET	
-65	65									
-70	70									
-75	75									

ATLAS

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

Figure II-7

LOG OF TEST BORING

ATLAS PROJECT NAME	ATLAS PROJECT NUMBER	
Compton College PE Complex Replacement	10-57575PW	B-5

SITE	START	END	SHEET NO.
Compton, California	3/2/21	3/2/21	8

DRILLING COMPANY	DRILL METHOD	LOGGED BY	REVIEWED BY
Baja Exploration	Hand Auger	KBH	MJ

DRILLING EQUIPMENT	BORING DIA. (in.)	TOTAL DEPTH (ft)	GROUND ELEV. (ft)	DEPTH/ELEV. GROUND WATER (ft)
	8	10	0	AT TIME OF DRILLING ---
				AT END OF DRILLING ---
				AFTER DRILLING ---

SAMPLING METHOD	NOTES
	Hammer Efficiency = 73.9% N_{60}~$1.23N_{SPT}$

ELEVATION (ft)	DEPTH (ft)	BULK SAMPLE	DRIVE SAMPLE	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
					6 inches of grass and topsoil.
					FILL (af):SILTY SAND (SM), loose, brown, damp, fine to medium grained, micaceous.
-2.5	2.5				
					YOUNG ALLUVIAL FAN DEPOSITS (Qyf): SILTY SAND (SM), medium dense, grayish brown, damp, fine to medium grained, micaceous.
-5.0	5.0				Variable silt and sand lensing.
-7.5	7.5				
					BORING TERMINATED AT 10 FEET

ATLAS

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

Figure

II-8

LOG OF TEST BORING

ATLAS PROJECT NAME	ATLAS PROJECT NUMBER	
Compton College PE Complex Replacement	10-57575PW	B-6

SITE	START	END	SHEET NO.
Compton, California	3/2/21	3/2/21	9

DRILLING COMPANY	DRILL METHOD	LOGGED BY	REVIEWED BY
Baja Exploration	Hand Auger	KBH	MJ

DRILLING EQUIPMENT	BORING DIA. (in.)	TOTAL DEPTH (ft)	GROUND ELEV. (ft)	DEPTH/ELEV. GROUND WATER (ft)
	8	10	0	AT TIME OF DRILLING ---

SAMPLING METHOD	NOTES	
	Hammer Efficiency = 73.9% N_{60}~$1.23N_{SPT}$	AT END OF DRILLING --- AFTER DRILLING ---

ELEVATION (ft)	DEPTH (ft)	BULK SAMPLE	DRIVE SAMPLE	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
					6 inches of grass and topsoil.
					FILL (af):SILTY SAND (SM), loose, brown, damp, fine to medium grained, micaceous, trace gravel.
-2.5	2.5				
		X			**YOUNG ALLUVIAL FAN DEPOSITS (Qyf)**: SILTY SAND (SM), medium dense, grayish brown, damp, fine to medium grained, micaceous.
-5.0	5.0				
-7.5	7.5	X			SANDY SILT (ML), loose, dark brown, moist, fine to medium grained, micaceous.
					BORING TERMINATED AT 10 FEET

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

Figure II-9

LOG OF TEST BORING

ATLAS PROJECT NAME: Compton College PE Complex Replacement		ATLAS PROJECT NUMBER: 10-57575PW		B-7
SITE: Compton, California	START: 3/2/21	END: 3/2/21		SHEET NO.: 10
DRILLING COMPANY: Baja Exploration	DRILL METHOD: Hand Auger	LOGGED BY: KBH		REVIEWED BY: MJ
DRILLING EQUIPMENT:	BORING DIA. (in.): 8	TOTAL DEPTH (ft): 10	GROUND ELEV. (ft): 0	DEPTH/ELEV. GROUND WATER (ft): AT TIME OF DRILLING --- ; AT END OF DRILLING --- ; AFTER DRILLING ---
SAMPLING METHOD:	NOTES: Hammer Efficiency = 73.9% N_{60}~$1.23N_{SPT}$			

ELEVATION (ft)	DEPTH (ft)	BULK SAMPLE	DRIVE SAMPLE	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
					6 inches of grass and topsoil.
					FILL (af):SILTY SAND (SM), loose, brown, dry, fine to medium grained, micaceous.
-2.5	2.5				**YOUNG ALLUVIAL FAN DEPOSITS (Qyf)**: SILTY SAND (SM), medium dense, grayish brown, damp, fine to medium grained, micaceous, minor mottling.
					Variable silt and sand lensing.
-5.0	5.0				
-7.5	7.5	X			
					BORING TERMINATED AT 10 FEET

ATLAS

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

Figure II-10

LOG OF TEST BORING

ATLAS PROJECT NAME	ATLAS PROJECT NUMBER	
Compton College PE Complex Replacement	10-57575PW	**B-8**

SITE	START	END	SHEET NO.
Compton, California	3/2/21	3/2/21	11

DRILLING COMPANY	DRILL METHOD	LOGGED BY	REVIEWED BY
Baja Exploration	Hollow Stem Auger	KBH	MJ

DRILLING EQUIPMENT	BORING DIA. (in.)	TOTAL DEPTH (ft)	GROUND ELEV. (ft)	DEPTH/ELEV. GROUND WATER (ft)
CME-75	8	21.5	0	AT TIME OF DRILLING ---

SAMPLING METHOD	NOTES	
140-lb Hammer, 30-in Drop	Hammer Efficiency = 73.9% N_{60}~1.23N_{SPT}	AT END OF DRILLING --- AFTER DRILLING ---

ELEVATION (ft)	DEPTH (ft)	BULK SAMPLE	DRIVE SAMPLE	BLOWS PER FOOT	N_{60}	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION	LAB TESTS
							e inches of landscaping sand.	
		X					**FILL (Qf):** SILTY SAND (SM), loose, brown, dry, fine to medium grained, micaceous, trace gravel.	
							YOUNG ALLUVIUM(Qya): SILTY SAND (SM), loose to medium dense, grayish brown, moist, fine to medium grained, micaceous, minor mottling.	
-5	5		CAL	15				CON
-10	10		SPT	6	7			
-15	15						SANDY SILT (ML), medium dense, brown, moist, mostly fine grained, micaceous.	
			CAL	21				
-20	20						SILTY SAND (SM), medium dense, brown, moist, fine to medium grained, micaceous, minor mottling.	
			SPT	12	15			

BORING TERMINATED AT 21.5 FEET

ATLAS

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

Figure

II-11

LOG OF TEST BORING

ATLAS PROJECT NAME	ATLAS PROJECT NUMBER	B-9
Compton College PE Complex Replacement	10-57575PW	

SITE	START	END	SHEET NO.
Compton, California	3/2/21	3/2/21	12

DRILLING COMPANY	DRILL METHOD	LOGGED BY	REVIEWED BY
Baja Exploration	Hollow Stem Auger	KBH	MJ

DRILLING EQUIPMENT	BORING DIA. (in.)	TOTAL DEPTH (ft)	GROUND ELEV. (ft)	DEPTH/ELEV. GROUND WATER (ft)
CME-75	8	21.5	0	∇ AT TIME OF DRILLING ---

SAMPLING METHOD	NOTES	
140-lb Hammer, 30-in Drop	Hammer Efficiency = 73.9% N_{60}~$1.23N_{SPT}$	▼ AT END OF DRILLING --- ▼ AFTER DRILLING ---

ELEVATION (ft)	DEPTH (ft)	BULK SAMPLE	DRIVE SAMPLE	BLOWS PER FOOT	N_{60}	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
							2 inches of Asphalt over 4 inches of Base
		X					**FILL (Qf):** SILTY SAND (SM), loose, brown, moist, fine to medium grained.
							YOUNG ALLUVIUM(Qya): SILTY SAND (SM), loose, grayish brown, moist, fine to medium grained.
-5	5		CAL	17			Medium dense, dry.
-10	10		SPT	7	9		
-15	15		CAL	10			SANDY SILT (ML), medium dense, brown, moist, fine to medium grained, micaceous, minor oxidation.
-20	20		SPT	9	11		

BORING TERMINATED AT 21.5 FEET

ATLAS

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

Figure

II-12

LOG OF TEST BORING

ATLAS PROJECT NAME	ATLAS PROJECT NUMBER	
Compton College PE Complex Replacement	10-57575PW	B-10

SITE	START	END	SHEET NO.
Compton, California	3/1/21	3/1/21	13

DRILLING COMPANY	DRILL METHOD	LOGGED BY	REVIEWED BY
Baja Exploration	Hollow Stem Auger	KBH	MJ

DRILLING EQUIPMENT	BORING DIA. (in.)	TOTAL DEPTH (ft)	GROUND ELEV. (ft)	DEPTH/ELEV. GROUND WATER (ft)
CME-75	8	56.5	0	∇ AT TIME OF DRILLING 52.00 ft / Elev -52.00 ft

SAMPLING METHOD	NOTES	
140-lb Hammer, 30-in Drop	Hammer Efficiency = 73.9% N_{60}~$1.23N_{SPT}$	▼ AT END OF DRILLING --- ▼ AFTER DRILLING ---

ELEVATION (ft)	DEPTH (ft)	BULK SAMPLE	DRIVE SAMPLE	BLOWS PER FOOT	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION	LAB TESTS
0	0								**6 inches of grass and topsoil.**	
									FILL (af):SILTY SAND (SM), loose, dark brown, dry, fine to medium grained, micaceous.	
	1–5	Bulk								EI, COR
-5	5								**YOUNG ALLUVIAL FAN DEPOSITS (Qyf)**: SANDY SILT (ML), loose, moderate brown, moist, fine to medium grained, micaceous.	
			CAL	12						AL
-10	10								SILTY SAND (SM), loose, moderate brown, damp, fine to medium grained, micaceous.	
			SPT	5	6					
-15	15								Medium dense, moist, mottling, silt lenses.	
			CAL	23						

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

Figure

II-13

LOG OF TEST BORING

ATLAS PROJECT NAME	ATLAS PROJECT NUMBER	B-10
Compton College PE Complex Replacement	10-57575PW	

SITE	START	END	SHEET NO.
Compton, California	3/1/21	3/1/21	14

DRILLING COMPANY	DRILL METHOD	LOGGED BY	REVIEWED BY
Baja Exploration	Hollow Stem Auger	KBH	MJ

DRILLING EQUIPMENT	BORING DIA. (in.)	TOTAL DEPTH (ft)	GROUND ELEV. (ft)	DEPTH/ELEV. GROUND WATER (ft)
CME-75	8	56.5	0	▽ AT TIME OF DRILLING 52.00 ft / Elev -52.00 ft
				▼ AT END OF DRILLING ---
				▽ AFTER DRILLING ---

SAMPLING METHOD	NOTES
140-lb Hammer, 30-in Drop	Hammer Efficiency = 73.9% N_{60}~$1.23N_{SPT}$

ELEVATION (ft)	DEPTH (ft)	BULK SAMPLE	DRIVE SAMPLE	BLOWS PER FOOT	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION	LAB TESTS
			SPT	5	6				SANDY SILT (ML), loose, dark gray, moist, fine to medium grained, micaceous, mottled.	
-25	25									
			CAL	7		23.7	94.6			WA 56.4%
-30	30								Medium dense.	
			SPT	13	16					AL
-35	35									
			CAL	18						

LOG OF TEST BORING

ATLAS PROJECT NAME	ATLAS PROJECT NUMBER	
Compton College PE Complex Replacement	10-57575PW	B-10

SITE	START	END	SHEET NO.
Compton, California	3/1/21	3/1/21	15

DRILLING COMPANY	DRILL METHOD	LOGGED BY	REVIEWED BY
Baja Exploration	Hollow Stem Auger	KBH	MJ

DRILLING EQUIPMENT	BORING DIA. (in.)	TOTAL DEPTH (ft)	GROUND ELEV. (ft)	DEPTH/ELEV. GROUND WATER (ft)
CME-75	8	56.5	0	∇ AT TIME OF DRILLING 52.00 ft / Elev -52.00 ft

SAMPLING METHOD	NOTES	
140-lb Hammer, 30-in Drop	Hammer Efficiency = 73.9% N_{60}~1.23N_{SPT}	▼ AT END OF DRILLING --- ▼ AFTER DRILLING ---

ELEVATION (ft)	DEPTH (ft)	BULK SAMPLE	DRIVE SAMPLE	BLOWS PER FOOT	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION	LAB TESTS
			SPT	11	14				SILTY SAND (SM), medium dense, gray, moist, fine grained, micaceous, mottled.	
-45	45		CAL	32		23.5	96.0			
-50	50		SPT	17	21				SANDY SILT (SM), medium dense, dark gray, moist, fine to coarse grained, micaceous. ∇	
-55	55		CAL	26					Coarse sand lense.	

BORING TERMINATED AT 56½ FEET

ATLAS

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

Figure II-15

LOG OF TEST BORING

ATLAS PROJECT NAME	ATLAS PROJECT NUMBER	
Compton College PE Complex Replacement	10-57575PW	B-11

SITE	START	END	SHEET NO.
Compton, California	3/1/21	3/1/21	16

DRILLING COMPANY	DRILL METHOD	LOGGED BY	REVIEWED BY
Baja Exploration	Hollow Stem Auger	KBH	MJ

DRILLING EQUIPMENT	BORING DIA. (in.)	TOTAL DEPTH (ft)	GROUND ELEV. (ft)	DEPTH/ELEV. GROUND WATER (ft)
CME-75	8	25	0	AT TIME OF DRILLING --- AT END OF DRILLING --- AFTER DRILLING ---

SAMPLING METHOD	NOTES
140-lb Hammer, 30-in Drop	Hammer Efficiency = 73.9% N_{60}~$1.23N_{SPT}$

ELEVATION (ft)	DEPTH (ft)	BULK SAMPLE	DRIVE SAMPLE	BLOWS PER FOOT	N_{60}	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION	LAB TESTS
							6 inches of grass and topsoil.	
		BULK					**FILL (af)**:SILTY SAND (SM), loose, dark brown, damp, fine to medium grained, micaceous.	
-5	5						**YOUNG ALLUVIAL FAN DEPOSITS (Qyf)**: SILTY SAND (SM), medium dense, brown to grayish brown, moist, fine to medium grained, micaceous.	
			SPT	11	14			
-10	10							
			CAL	16				CON
-15	15							
			SPT	11	14			

ATLAS

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

LOG OF TEST BORING

ATLAS PROJECT NAME	ATLAS PROJECT NUMBER	
Compton College PE Complex Replacement	10-57575PW	**B-11**

SITE	START	END	SHEET NO.
Compton, California	3/1/21	3/1/21	17

DRILLING COMPANY	DRILL METHOD	LOGGED BY	REVIEWED BY
Baja Exploration	Hollow Stem Auger	KBH	MJ

DRILLING EQUIPMENT	BORING DIA. (in.)	TOTAL DEPTH (ft)	GROUND ELEV. (ft)	DEPTH/ELEV. GROUND WATER (ft)
CME-75	8	25	0	AT TIME OF DRILLING ---
				AT END OF DRILLING ---
				AFTER DRILLING ---

SAMPLING METHOD	NOTES
140-lb Hammer, 30-in Drop	Hammer Efficiency = 73.9% N_{60}~$1.23N_{SPT}$

ELEVATION (ft)	DEPTH (ft)	BULK SAMPLE	DRIVE SAMPLE	BLOWS PER FOOT	N_{60}	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION	LAB TESTS
			CAL	17			**YOUNG ALLUVIAL FAN DEPOSITS (Qyf)**: SILTY SAND (SM), medium dense, brown to grayish brown, moist, fine to medium grained, micaceous. *(continued)*	
			SPT	7	9		SANDY SILT (ML), loose, dark gray, moist, fine to medium grained, micaceous.	
-25	25						**BORING TERMINATED AT 25 FEET**	
-30	30							
-35	35							

ATLAS

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

LOG OF TEST BORING

ATLAS PROJECT NAME: Compton College PE Complex Replacement		ATLAS PROJECT NUMBER: 10-57575PW		B-12
SITE: Compton, California		START: 3/2/21	END: 3/2/21	SHEET NO.: 18
DRILLING COMPANY: Baja Exploration	DRILL METHOD: Hollow Stem Auger	LOGGED BY: KBH		REVIEWED BY: MJ
DRILLING EQUIPMENT: CME-75	BORING DIA. (in.): 8	TOTAL DEPTH (ft): 21.5	GROUND ELEV. (ft): 0	DEPTH/ELEV. GROUND WATER (ft): AT TIME OF DRILLING ---
SAMPLING METHOD: 140-lb Hammer, 30-in Drop	NOTES: Hammer Efficiency = 73.9% N_{60}~$1.23N_{SPT}$			AT END OF DRILLING --- ; AFTER DRILLING ---

ELEVATION (ft)	DEPTH (ft)	BULK SAMPLE	DRIVE SAMPLE	BLOWS PER FOOT	N_{60}	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
		BULK					**2 inches of Asphalt over 4 inches of Base**
							FILL (Qf): SILTY SAND (SM), loose, brown, moist, fine to medium grained.
							YOUNG ALLUVIUM(Qya): SILTY SAND (SM), medium dense, grayish brown, damp, fine to medium grained.
-5	5		CAL	15			
-10	10		SPT	11	14		Brown, increase in fines content.
-15	15		CAL	27			Grayish brown, minor mottling.
-20	20		SPT	5	6		SANDY SILT (ML), loose, dark gray, moist, fine to medium grained, micaceous, minor oxidation.

BORING TERMINATED AT 21.5 FEET

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

Figure II-18

LOG OF TEST BORING

ATLAS PROJECT NAME	ATLAS PROJECT NUMBER	
Compton College PE Complex Replacement	10-57575PW	B-13

SITE	START	END	SHEET NO.
Compton, California	3/1/21	3/1/21	19

DRILLING COMPANY	DRILL METHOD	LOGGED BY	REVIEWED BY
Baja Exploration	Hollow Stem Auger	KBH	MJ

DRILLING EQUIPMENT	BORING DIA. (in.)	TOTAL DEPTH (ft)	GROUND ELEV. (ft)	DEPTH/ELEV. GROUND WATER (ft)
CME-75	8	26.5	0	AT TIME OF DRILLING ---
				AT END OF DRILLING ---
				AFTER DRILLING ---

SAMPLING METHOD	NOTES
140-lb Hammer, 30-in Drop	Hammer Efficiency = 73.9% N_{60}~$1.23N_{SPT}$

ELEVATION (ft)	DEPTH (ft)	BULK SAMPLE	DRIVE SAMPLE	BLOWS PER FOOT	N_{60}	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
0	0						**5 inches of Concrete**
		BULK					**FILL (Qf):** SILTY SAND (SM), loose, grayish brown, moist, fine to medium grained.
-5	5		SPT	9	11		**YOUNG ALLUVIUM(Qya):** SILTY SAND (SM), medium dense, grayish brown, moist, fine grained.
-10	10		CAL	16			
-15	15		SPT	6	7		SANDY SILT (ML), loose, brown, moist, fine to medium grained, micaceous, minor mottling.
-20	20		CAL	36			SILTY SAND (SM), medium dense, gray, moist, fine to medium grained, micaceous, minor mottling.
-25	25		SPT	20	25		SANDY SILT (ML), medium dense, gray, moist, fine to medium grained, micaceous, minor mottling.

BORING TERMINATED AT 25.5 FEET

ATLAS

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

Figure II-19

LOG OF TEST BORING

ATLAS PROJECT NAME	ATLAS PROJECT NUMBER	
Compton College PE Complex Replacement	10-57575PW	**B-14**

SITE	START	END	SHEET NO.
Compton, California	3/1/21	3/1/21	20

DRILLING COMPANY	DRILL METHOD	LOGGED BY	REVIEWED BY
Baja Exploration	Hollow Stem Auger	KBH	MJ

DRILLING EQUIPMENT	BORING DIA. (in.)	TOTAL DEPTH (ft)	GROUND ELEV. (ft)	DEPTH/ELEV. GROUND WATER (ft)
CME-75	8	5	0	∇ AT TIME OF DRILLING ---

SAMPLING METHOD	NOTES	
140-lb Hammer, 30-in Drop	Hammer Efficiency = 73.9% N_{60}~1.23N_{SPT}	▼ AT END OF DRILLING --- ∇ AFTER DRILLING ---

ELEVATION (ft)	DEPTH (ft)	BULK SAMPLE	DRIVE SAMPLE	BLOWS PER FOOT	N_{60}	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION	LAB TESTS
							12 inches of grass and topsoil.	
		X					**FILL (af)**:SANDY SILT (ML), loose, medium brown, dry, fine to medium grained, micaceous, small rootlets.	RV WA (53.7%)
-2.5	2.5						**YOUNG ALLUVIAL FAN DEPOSITS (Qyf)**: SILTY SAND (SM), medium dense, grayish brown, dry, fine to medium grained, micaceous, minor rootlets.	
			CAL	17				
-5.0	5.0						**BORING TERMINATED AT 5 FEET**	
-7.5	7.5							

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

LOG OF TEST BORING

ATLAS PROJECT NAME	ATLAS PROJECT NUMBER	
Compton College PE Complex Replacement	10-57575PW	P-4

SITE	START	END	SHEET NO.
Compton, California	3/1/21	3/1/21	21

DRILLING COMPANY	DRILL METHOD	LOGGED BY	REVIEWED BY
Baja Exploration	Hollow Stem Auger	KBH	MJ

DRILLING EQUIPMENT	BORING DIA. (in.)	TOTAL DEPTH (ft)	GROUND ELEV. (ft)	DEPTH/ELEV. GROUND WATER (ft)
CME-75	8	5	0	AT TIME OF DRILLING ---
				AT END OF DRILLING ---
				AFTER DRILLING ---

SAMPLING METHOD	NOTES
140-lb Hammer, 30-in Drop	Hammer Efficiency = 73.9% N_{60}~$1.23N_{SPT}$

ELEVATION (ft)	DEPTH (ft)	BULK SAMPLE	DRIVE SAMPLE	BLOWS PER FOOT	N_{60}	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION	LAB TESTS
							6 inches of grass and topsoil.	
							FILL (af):SILTY SAND to SANDY SILT(SM/ML) with thin lens of CLAY(CL), loose, medium brown, dry, fine to medium grained, micaceous, rootlets.	RV
								PD
-2.5	2.5							
							YOUNG ALLUVIAL FAN DEPOSITS (Qyf): SILTY SAND (SM), medium dense, grayish brown, dry, fine to medium grained, micaceous.	
			SPT	10	12			
-5.0	5.0						**BORING TERMINATED AT 5 FEET**	
-7.5	7.5							

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

Figure

II-21

APPENDIX III
LABORATORY TEST PROCEDURES AND TEST RESULTS

The laboratory testing was performed in general accordance with applicable procedures and standards of the American Society for Testing and Materials (ASTM) and California Test Methods. Unless otherwise noted, the tests were performed at Atlas laboratories in Riverside and San Diego, California. Based on our review of the laboratory data, the undersigned engineers concur with and accept the laboratory testing results. Brief descriptions of the testing are presented in the following sections.

MOISTURE CONTENT AND DRY DENSITY: The moisture content and dry unit weight were determined for selected soil samples in general accordance with ASTM D2216 and ASTM D2937, respectively. The moisture content and dry unit weight are presented on the boring logs at the corresponding sample depths.

SIEVE ANALYSIS: Selected soil samples were tested to determine the quantitative determination of the distribution of particle sizes in soils (particle sizes larger than 75 micrometers) in general accordance with ASTM D422. The results of the Sieve analyses are presented in this Appendix.

WASH SIEVE ANALYSIS: Selected soil samples were tested to determine the percent fines (the percentage of soil passing the Standard No. 200 sieve) in general accordance with ASTM D1140. The results of the wash sieve analyses are presented at the appropriate depths on the boring logs.

DIRECT SHEAR: Direct shear tests were performed on ring and remolded samples in general accordance with ASTM D3080 to evaluate the shear strength of the soils. Samples were tested in a saturated state. Both peak and ultimate shear strengths were measured and reported in the test plots. Test results are attached in this appendix.

CORROSIVITY TESTS: Corrosivity tests were performed on a selected bulk sample to evaluate minimum resistivity, pH, water-soluble sulfates and chlorides (CTMs 643, 417 and 422 respectively). Test results are attached in this appendix.

EXPANSION INDEX TEST: Expansion Index tests were performed on selected bulk samples in general accordance with ASTM D4829 to evaluate the expansion potential of the on-site soils. Test results are attached in this appendix.

MAXIMUM DENSITY TESTS: The maximum dry density and optimum moisture content of a representative bulk soil sample were determined in accordance with ASTM D1557. Test results and a graphical plot of maximum density vs. optimum moisture content are attached in this appendix.

ATTERBERG LIMITS: Liquid Limit, Plastic Limit and Plasticity Index of the tested samples were determined in accordance with ASTM D4318. Test results and a graphical plot are attached in this appendix.

R-VALUE: R-Value of the tested samples were determined in accordance with ASTM D2844. Test results are presented in this appendix.

ATTERBERG LIMITS

ASTM D4318

SAMPLE LOCATION	LL	PL	PI
B-4 at 25½ to 26½ Feet	36	24	12
B-4 at 45½ to 46½ Feet	NP	NP	NP
B-10 at 6 to 6½ Feet	NP	NP	NP
B-10 at 30½ to 31½ Feet	34	26	8

Modified Proctor

ASTM D1557

SAMPLE LOCATION	Optiumum Moisture (%)	Maximum Dry Density (pcf)
B-3 at ½ to 3½ feet	13.9	115.7

Percent Finer than No. 200 Sieve

ASTM D1140

SAMPLE LOCATION	FINES CONTENT (%)
B-4 at 21 Feet	65.1
B-4 at 31 Feet	89.7
B-4 at 51 Feet	66.4
B-10 at 56.4 Feet	56.4
P-4 at 1 to 3½ Feet	50.5

R-VALUE

ASTM D2844

SAMPLE LOCATION	R-Value
B-14 at 1 to 2½ Feet	13
P-4 at 1 to 3½ Feet	50

EXPANSION INDEX

ASTM D4829

SAMPLE LOCATION	DESCRIPTION	EXPANSION INDEX
B-4 at ½ to 3½ feet	**FILL (af)**: SANDY SILT	9
B-10 at 1 to 5 feet	**FILL (af)**: SILTY SAND	2

Classification of Expansive Soil [1]

EXPANSIVE INDEX	POTENTIAL EXPANSION
1-20	Very Low
21-50	Low
51-90	Medium
91-130	High
Above 130	Very High

1. ASTM - D4829

RESISTIVITY, pH, SOLUBLE CHLORIDE and SOLUBLE SULFATE

pH & Resistivity (Cal 643, ASTM G51)

Soluble Chlorides (Cal 422)

Soluble Sulfate (Cal 417)

SAMPLE LOCATION	RESISTIVITY (Ω-cm)	pH	CHLORIDE (%)	SULFATE (%)
B-4 at ½ to 3½ Feet	2970	8.78	0.004	0.005
B-10 at 1 to 5 Feet	2940	8.19	0.003	0.002

Water-Soluble Sulfate Exposure [2]

Water-Soluble Sulfate (SO_4) in soil (percent by weight)	**Exposure Severity**	**Exposure Class**	**Cement Type (ASTM C150)**	**Max. W/C**	**Min. f_c' (psi)**
$SO_4 < 0.10$	N/A	S0	No type restriction	N/A	2,500
$0.10 \leq SO_4 < 0.20$	Moderate	S1	II	0.50	4,000
$0.20 \leq SO_4 \leq 2.00$	Severe	S2	V	0.45	4,500
$SO_4 > 2.00$	Very Severe	S3	V plus pozzolan or slag cement	0.45	4,500

2. Modified from ACI 318-14 Table 19.3.1.1 and Table 19.3.2.1

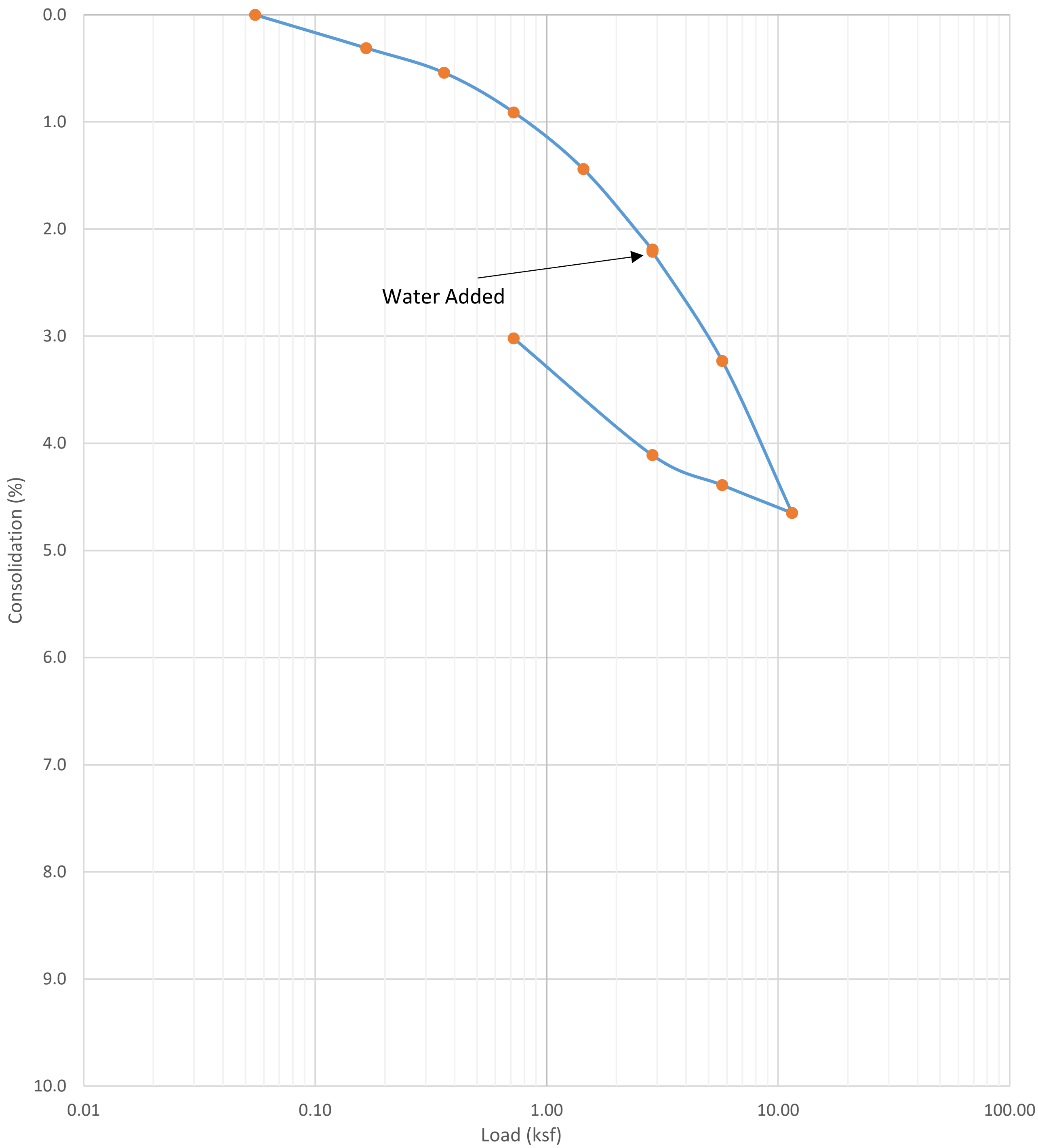
Corrosivity Ratings Based on Soil Resistivity [3]

Soil Resistivity (Ω cm)	**Corrosivity Rating**
> 20,000	Essentially noncorrosive
10,000 to 20,000	Mildly corrosive
5,000 to 10,000	Moderately corrosive
3,000 to 5,000	Corrosive
1,000 to 3,000	Highly corrosive
<1,000	Extremely corrosive

3. Roberge (2008), Corrosion Engineering, Principles and Practice

Consolidation Test Results

ASTM D2435



Sample ID: B-8 at 6 to 6½ feet

Sample Description: **YOUNG ALLUVIUM (Qya)**: SILTY SAND

γ_d	93.9 pcf
Pre-consolidation w_c	29.5 %
Post-consolidation w_c	29.8 %

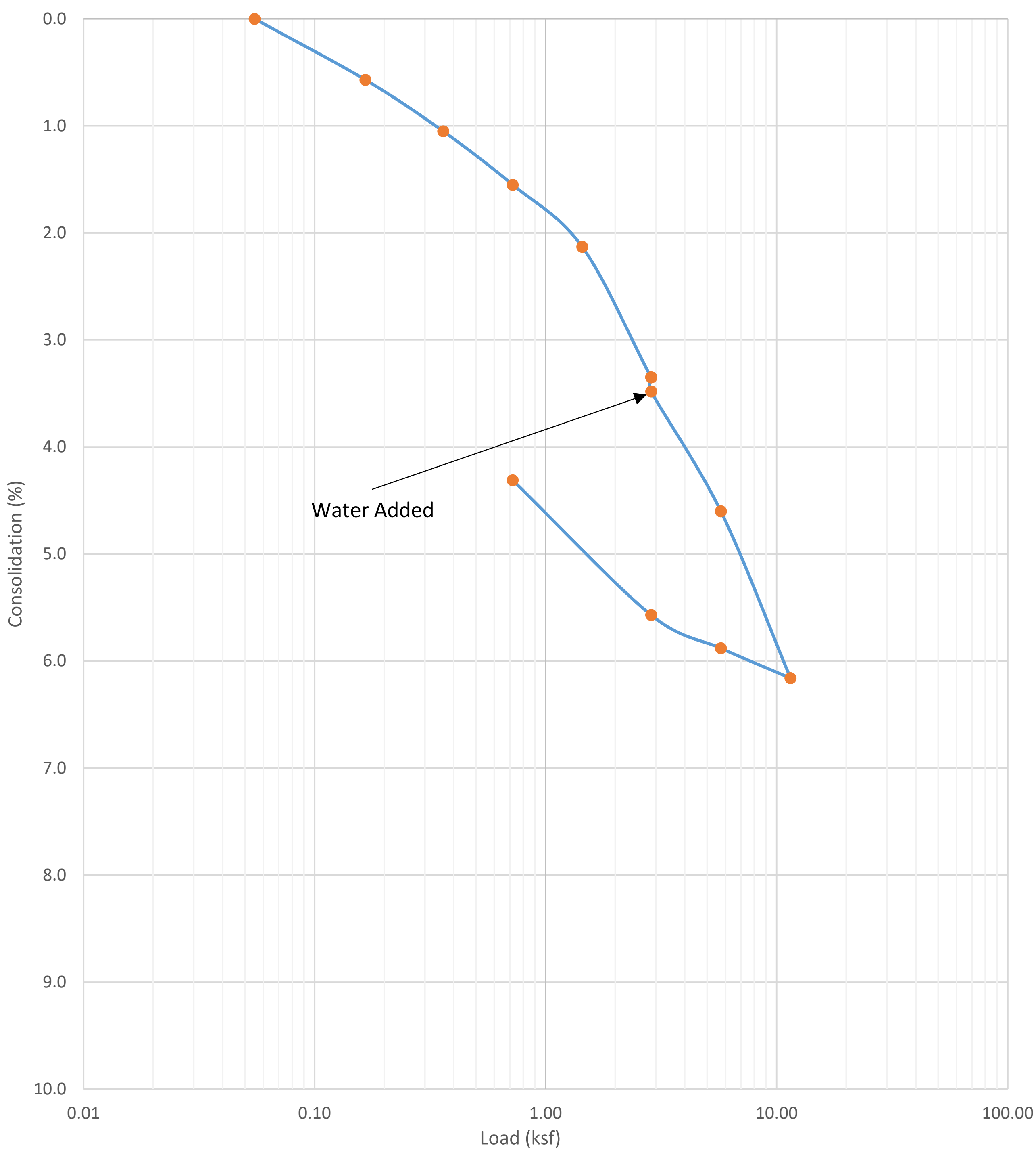
ATLAS

Compton College PE Complex Replacement
Compton, California

By:	JRD	Date:	July, 2021
Job No:	10-57575PW	Figure:	III-3

Consolidation Test Results

ASTM D2435



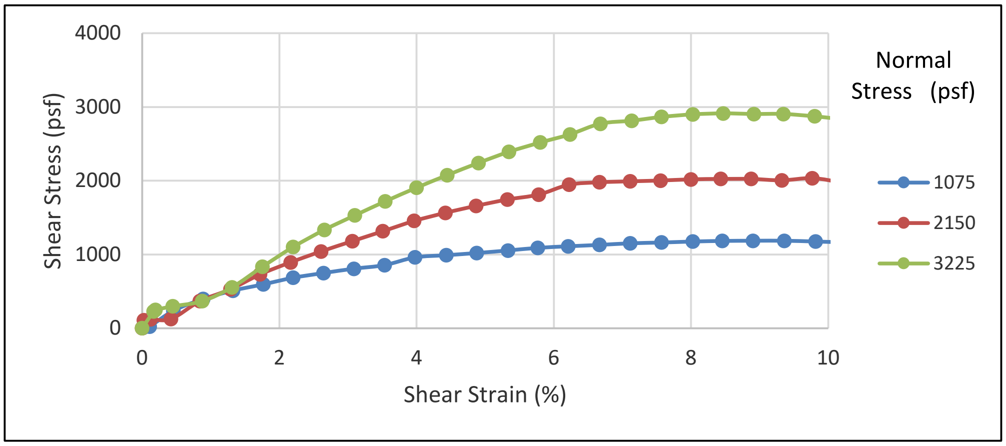
Sample ID: B-11 at 11 to 11½ Feet

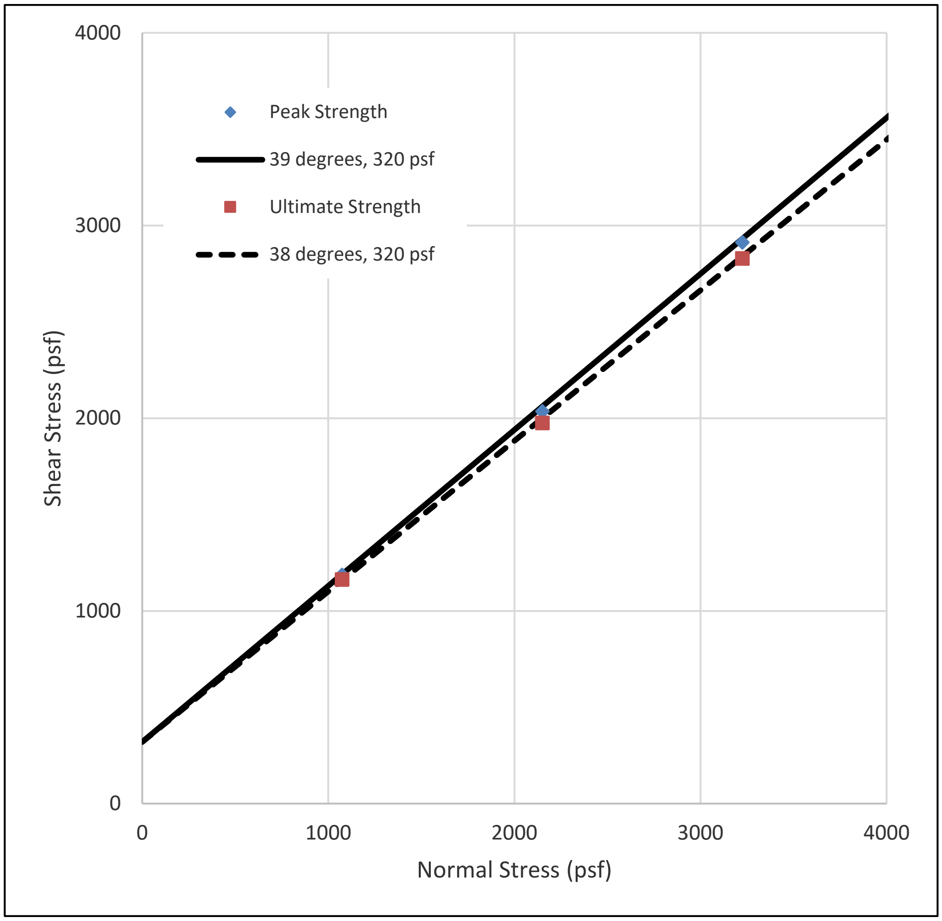
Sample Description: **YOUNG ALLUVIUM (Qya)**: SILTY SAND

γ_d	90.2 pcf
Pre-consolidation w_c	19.2 %
Post-consolidation w_c	34.7 %

ATLAS

Compton College PE Complex Replacement Compton, California			
By:	JRD	Date:	July, 2021
Job No:	10-57575PW	Figure:	III-4





SAMPLE ID: B-4 at 11 to 11½ feet

YOUNG ALLUVIUM (Qya): SILTY SAND

NOTES: Insitu
Strain Rate: 0.003 in/min
Sample was consolidated and drained

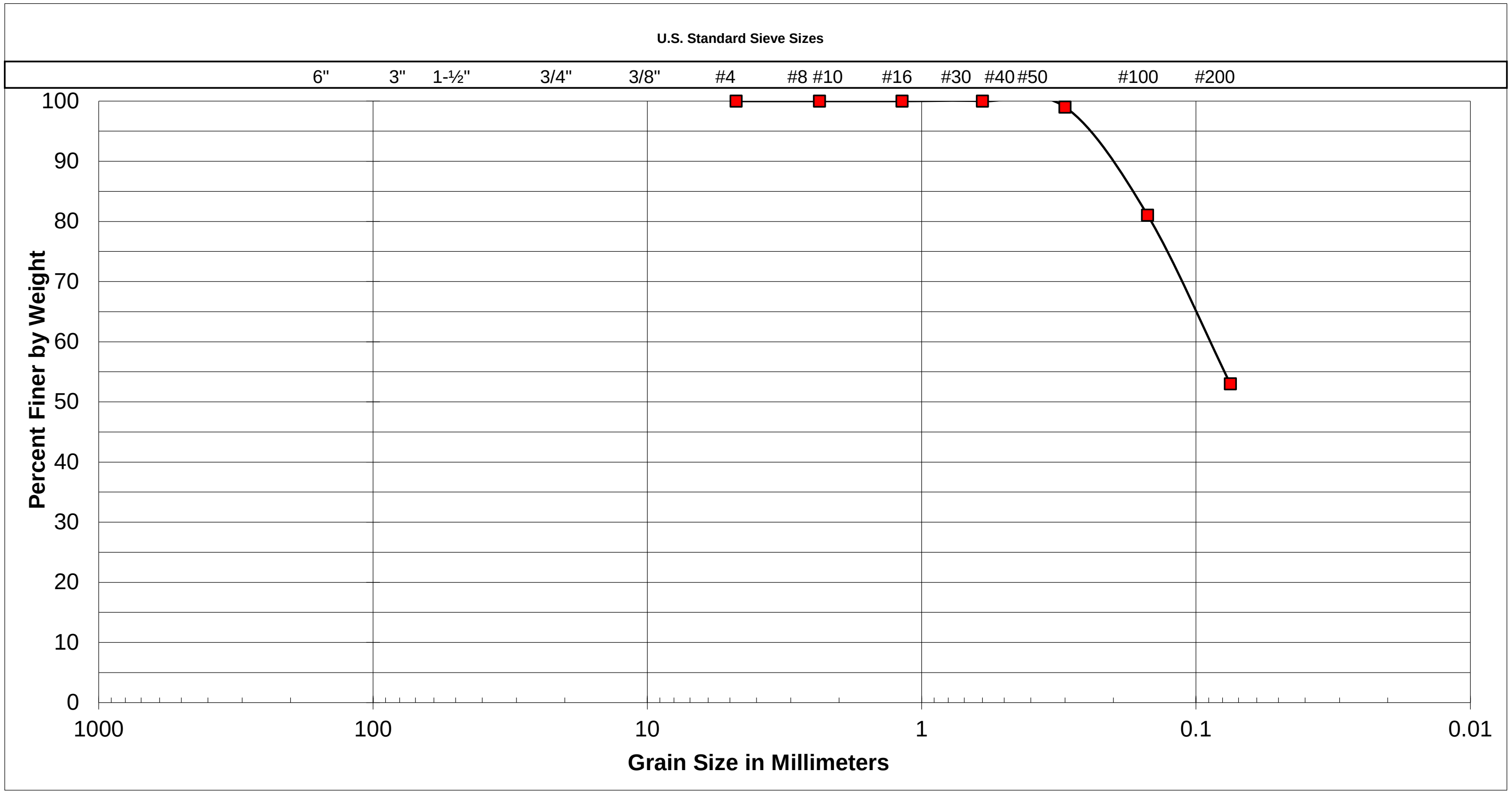
	Peak	Ultimate
Φ	39 °	38 °
c	320 psf	320 psf

	Initial	Final
γ_d	96.1 pcf	96.1 pcf
w_c	11.3 %	24.4 %
Saturation	41 %	89 %

ATLAS

Compton College PE Complex Replacement
Compton, California

By:	JRD	Date:	July, 2021
Job Number:	10-57575PW	Figure:	III-5



Cobbles	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

SAMPLE LOCATION
P-4 at 1-3.5 feet
SAMPLE NUMBER
0

UNIFIED SOIL CLASSIFICATION:	CL
DESCRIPTION	Sandy Lean Clay

ATTERBERG LIMITS	
LIQUID LIMIT	--
PLASTIC LIMIT	--
PLASTICITY INDEX	--

ATLAS

Compton College PE Complex - Compton, California			
By:	KH	Date:	July, 2021
Job Number:	10-57575PW	Figure:	III-6

14457 Meridian Parkway | Riverside, California 92518
P: 951.697.4777 | F: 951.888.3393 | www.oneatlas.com

LABORATORY COMPACTION CHARACTERISTICS OF SOIL USING MODIFIED EFFORT, ASTM D 1557

Tested For: Compton College Community District
1111 East Artessia Blvd.
Compton College, CA 90221

Project: Compton College PE Complex
1111 E. Artesia Blvd.
Compton, CA 90221

DSA File No.: NA
Dsa App No.: NA

Date: March 12, 2021

Atlas Technical Consultants Project No.: 1057575PW

Lab Sample No.:	Sample 1		
Visual Class.:	Brown Silty fine SAND	**Test Results:**	
Sample Source:	B-3 at 0.5 - 3.5 feet	**Maximum Dry Density, pcf:**	115.7
Method of Test:	ASTM D 1557 - Method A	**Optimum Moisture Content, %:**	13.9

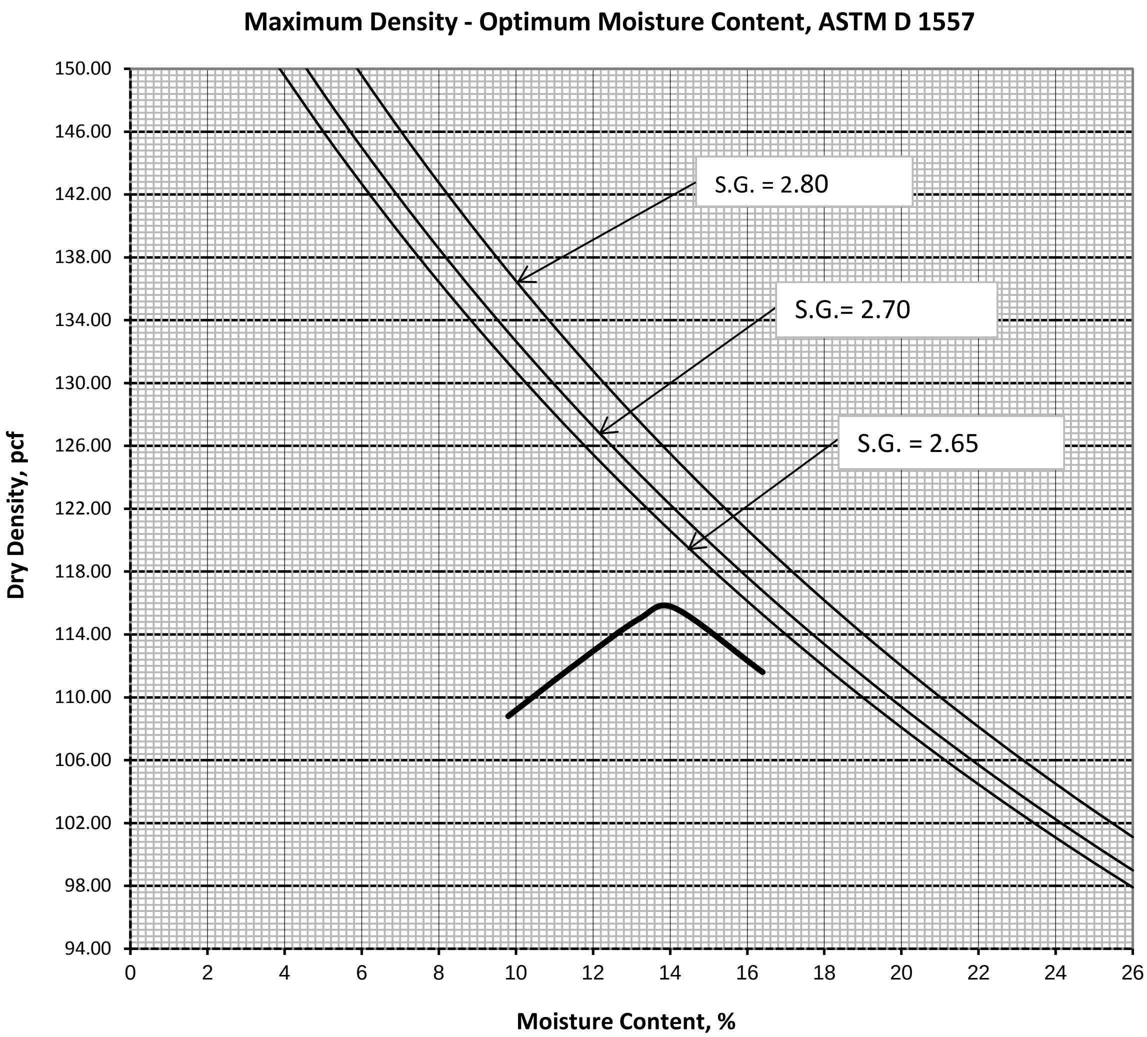
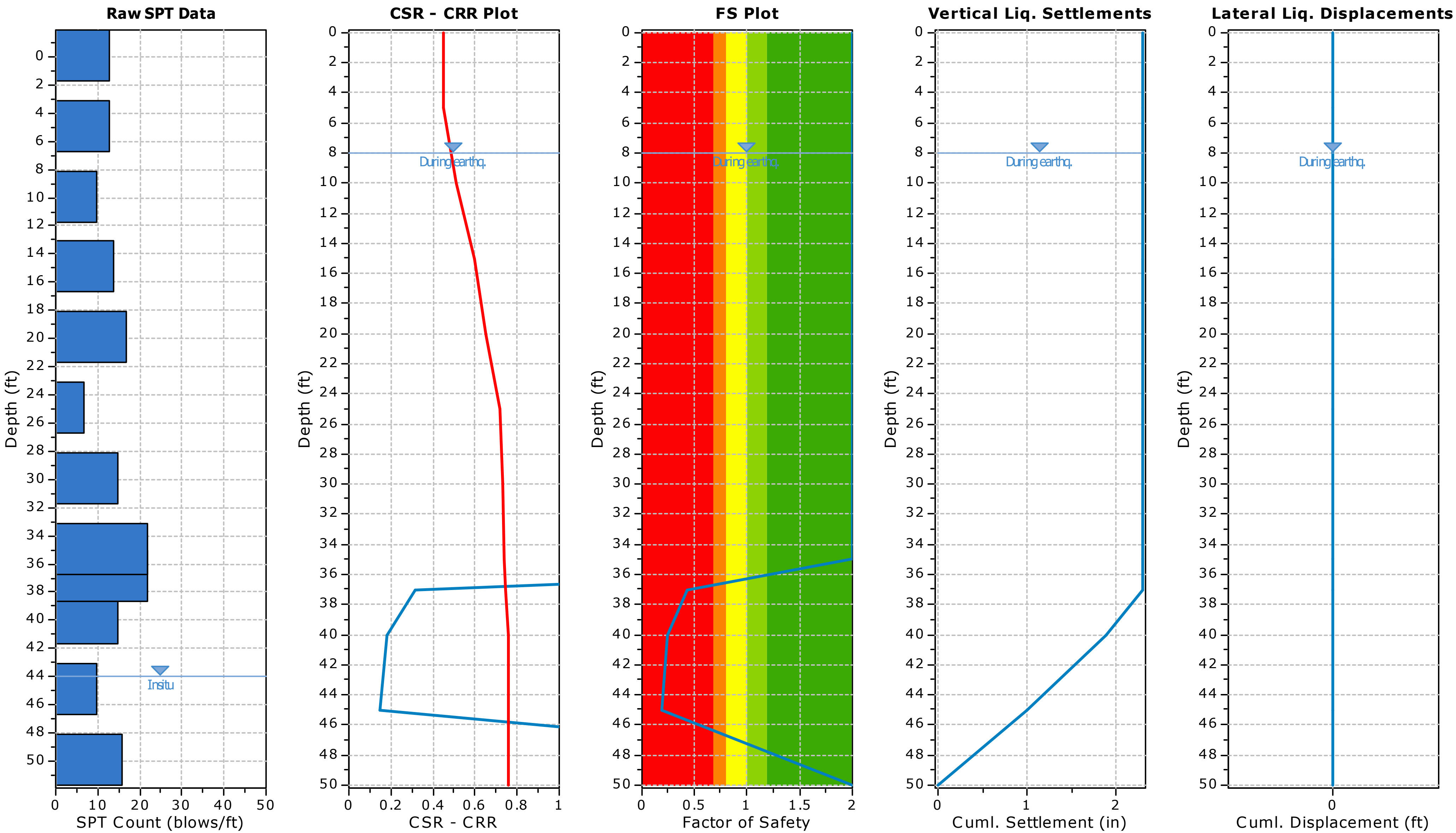


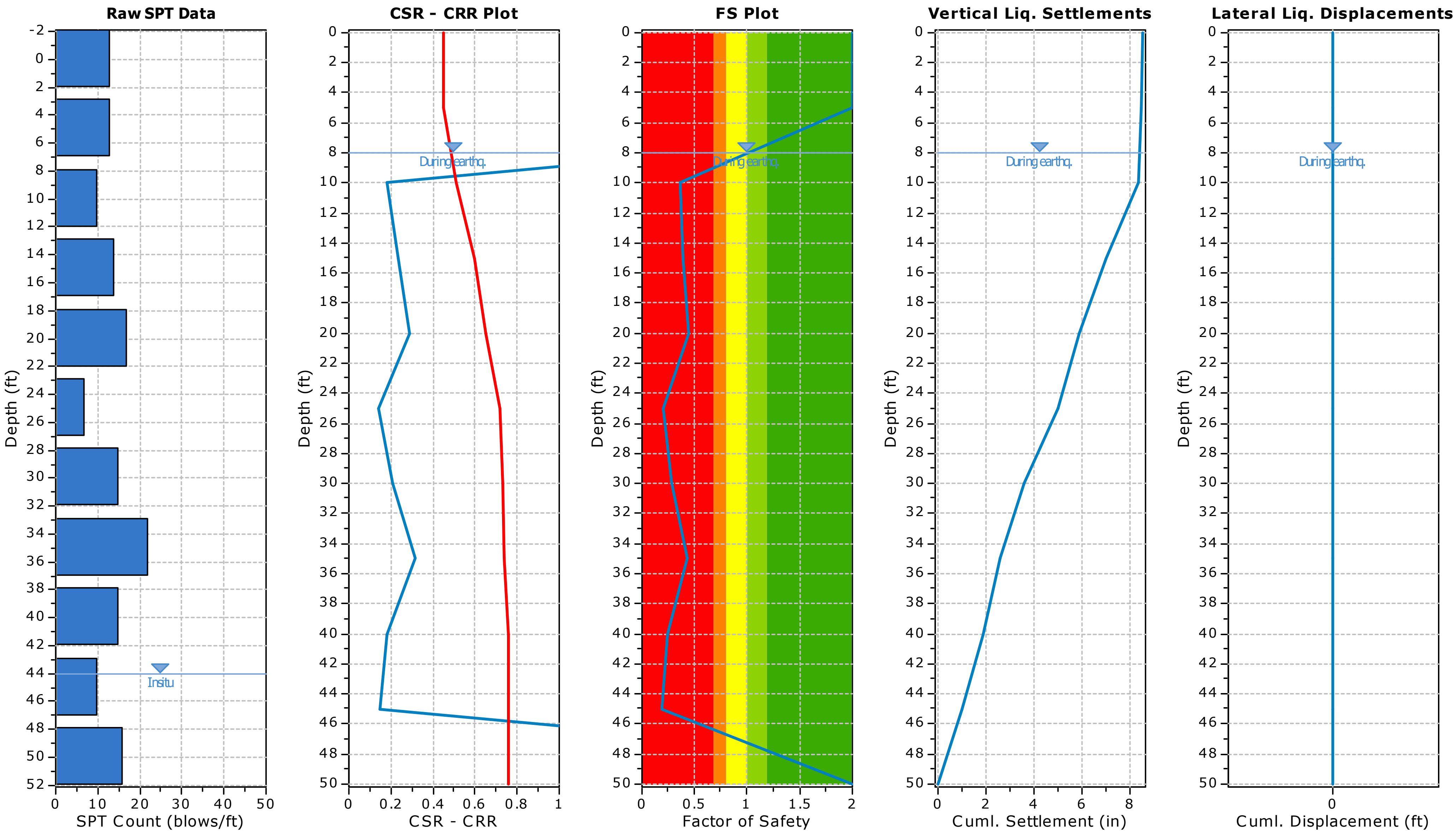
FIGURE III-7

APPENDIX IV
LIQUEFACTION RESULTS

:: Overall Liquefaction Assessment Analysis Plots ::



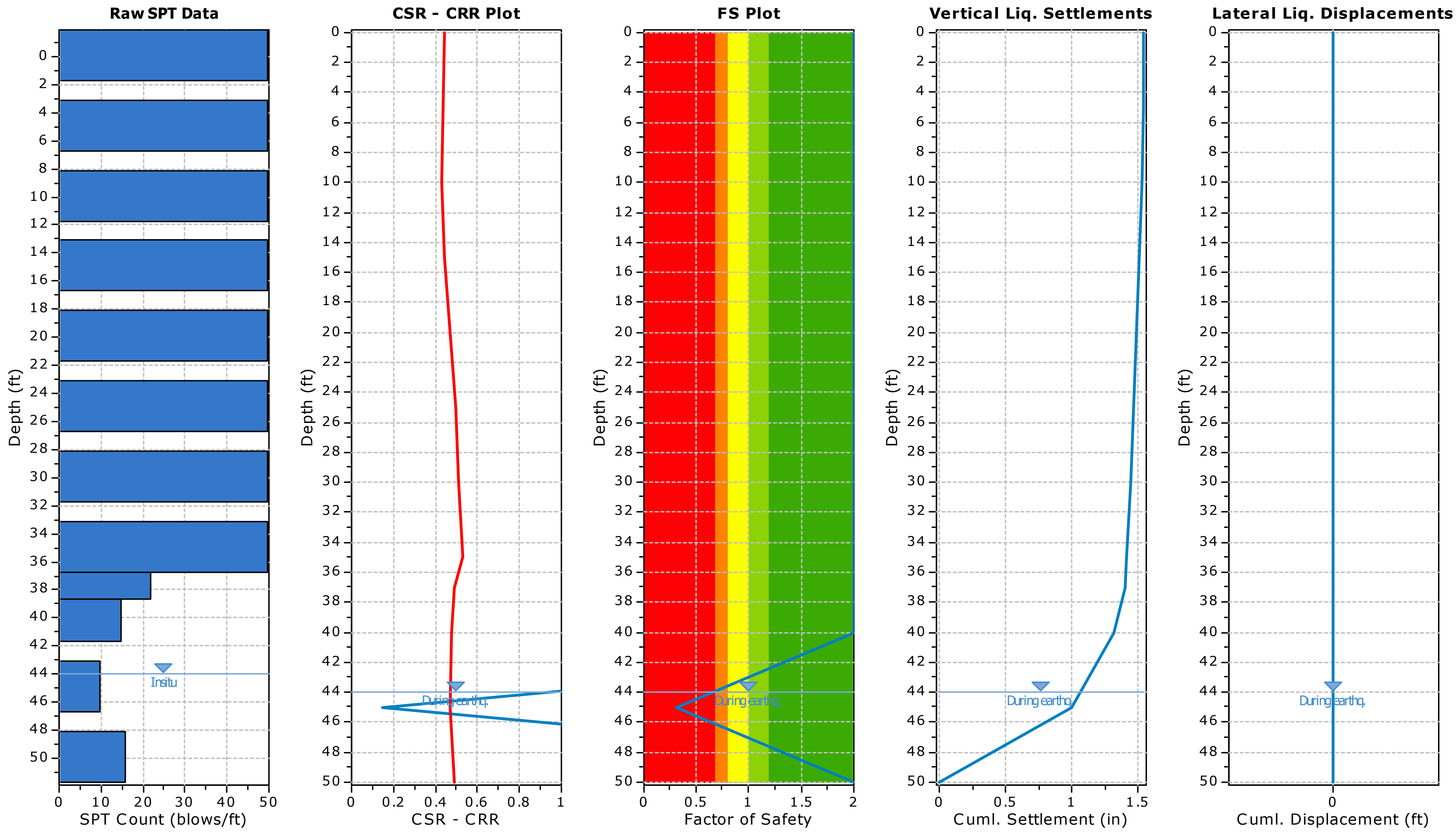
:: Overall Liquefaction Assessment Analysis Plots ::



This software is registered to: Atlas Technical Consultants (SCST)

SPT Name: B-4 (In-situ GW-After Soil Treatment)

:: Overall Liquefaction Assessment Analysis Plots ::

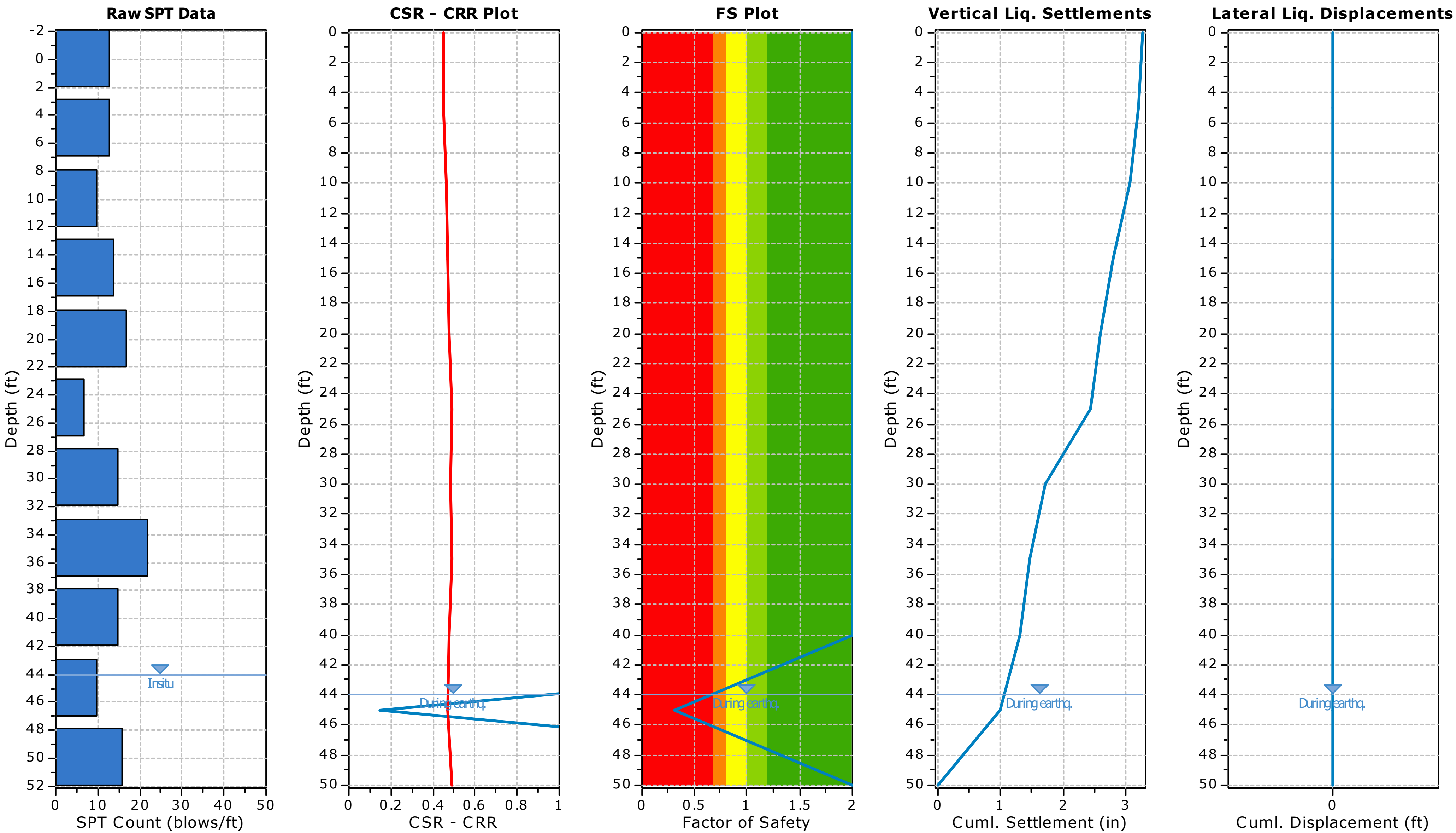


Project title : Compton College 10-57575PW

Location : Compton, California

Figure IV-3

:: Overall Liquefaction Assessment Analysis Plots ::

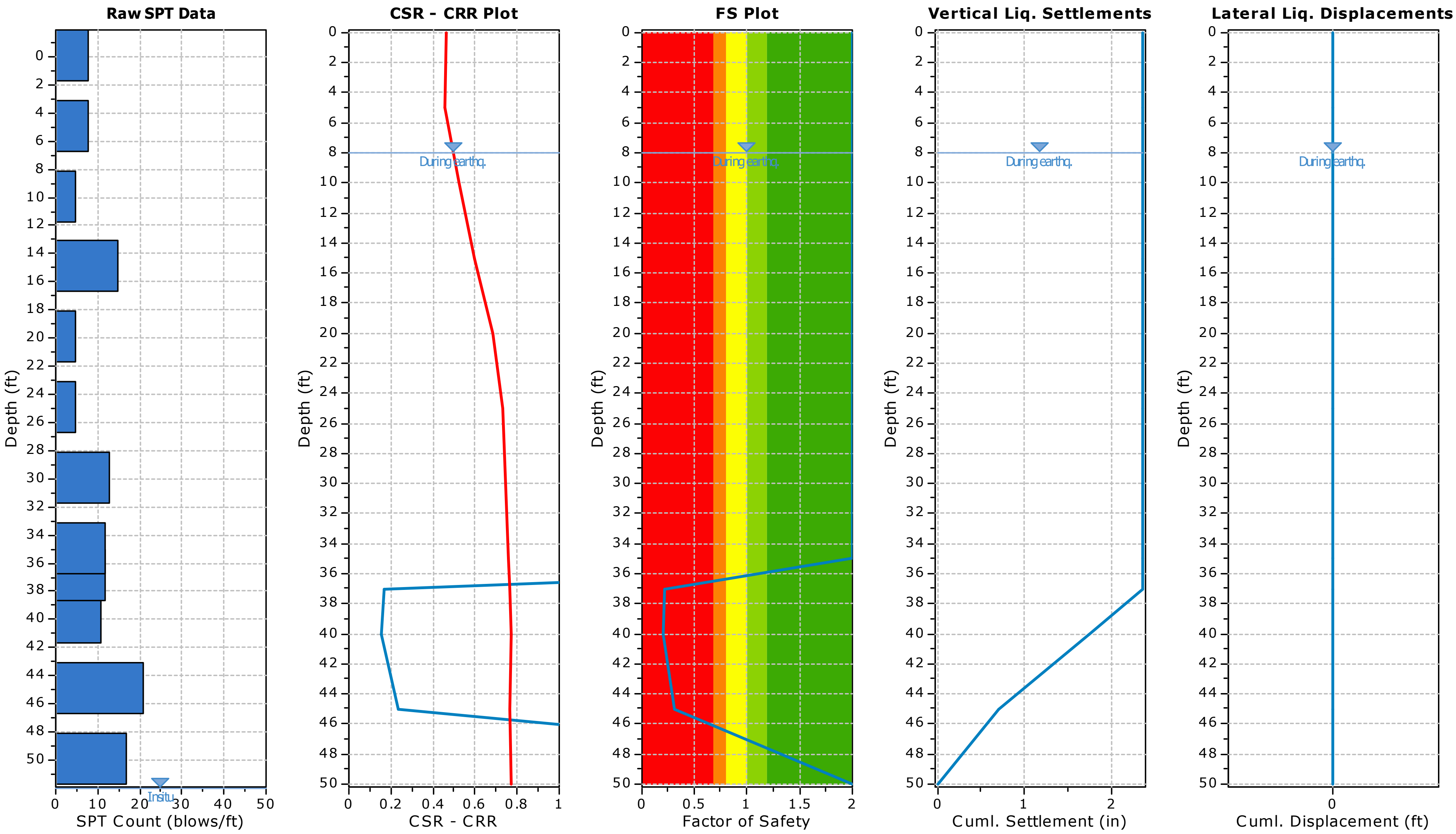


Project title : Compton College 10-57575PW

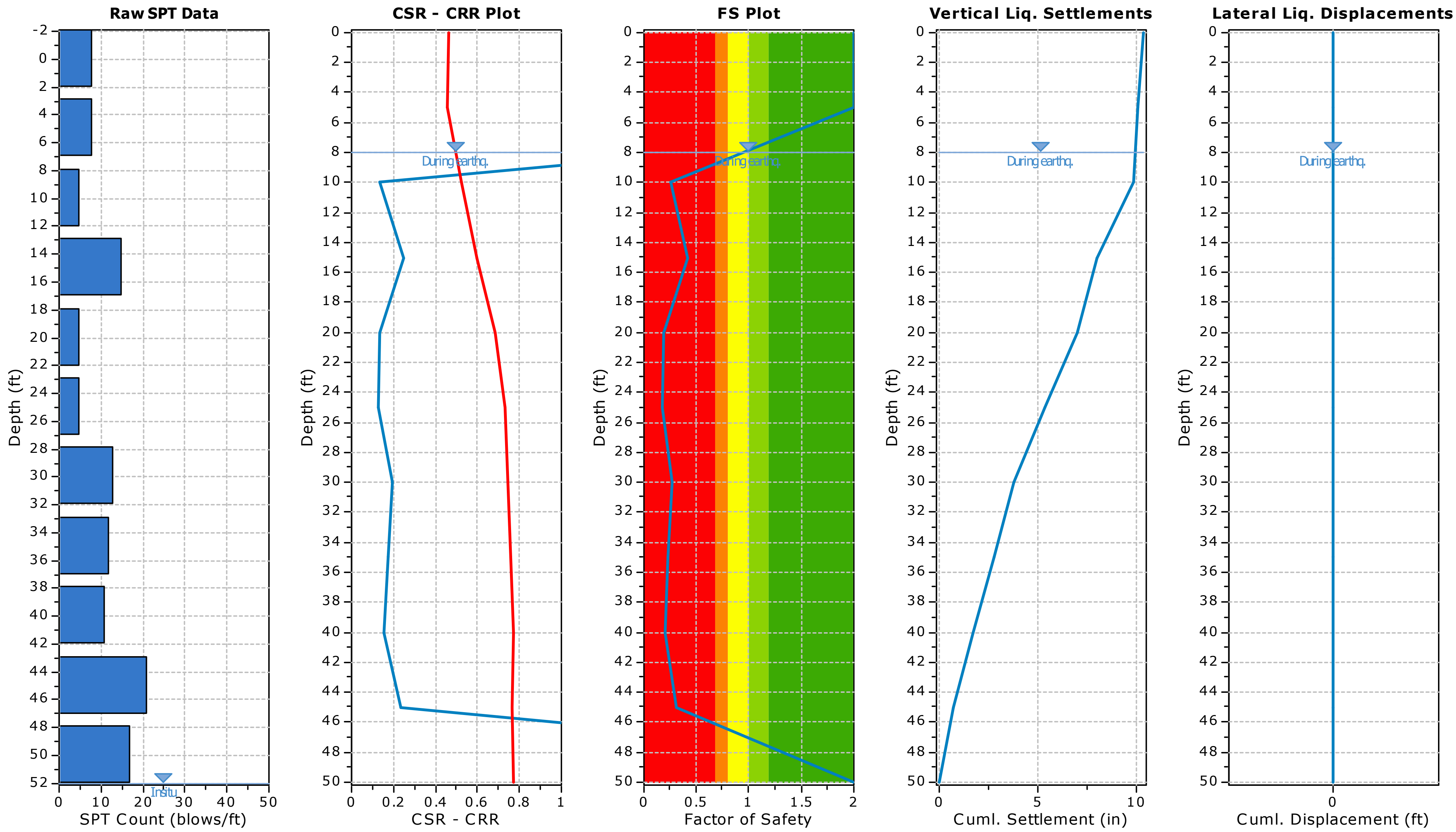
Location : Compton, California

Figure IV-4

:: Overall Liquefaction Assessment Analysis Plots ::



:: Overall Liquefaction Assessment Analysis Plots ::

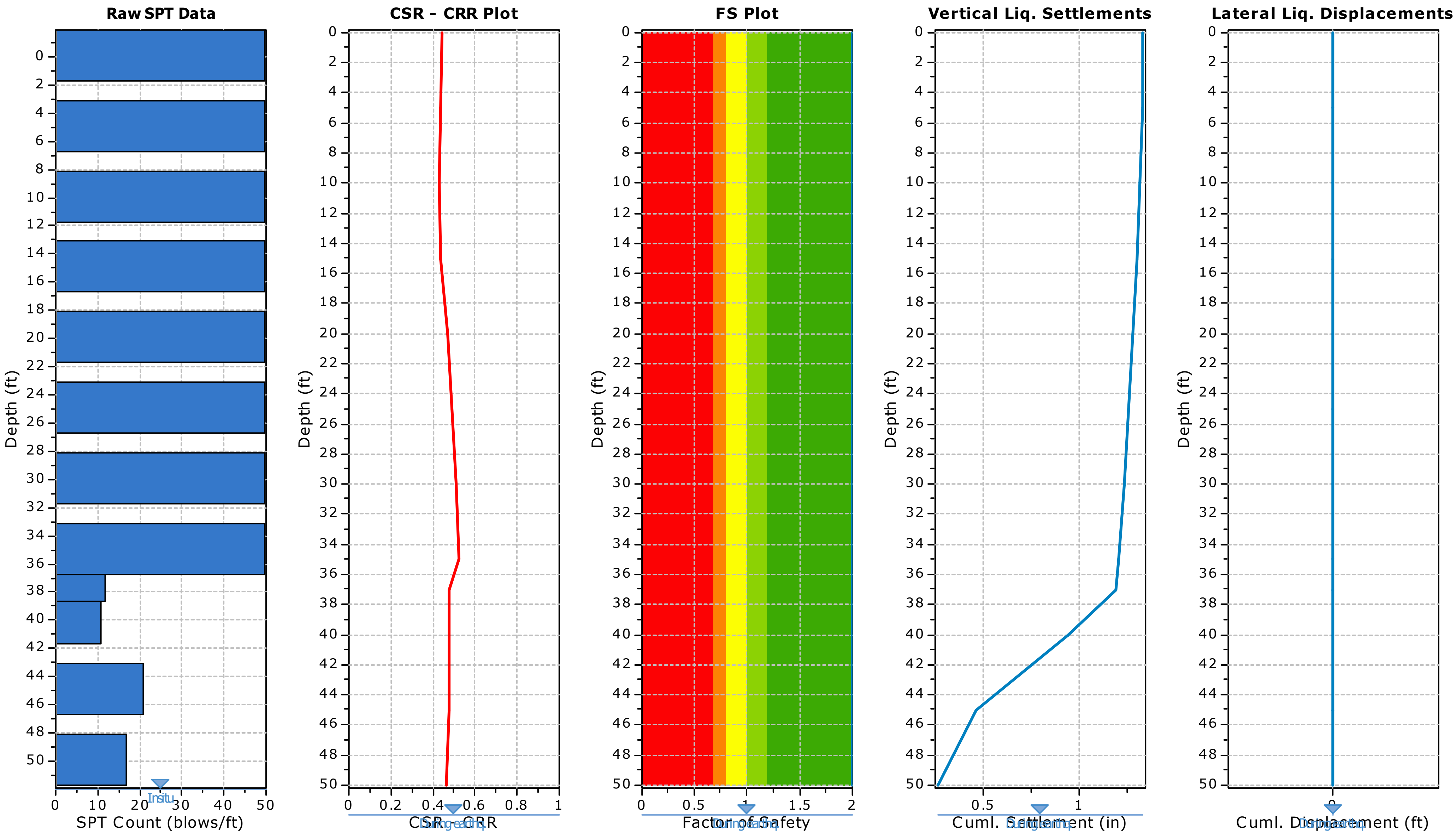


Project title : Compton College 10-57575PW

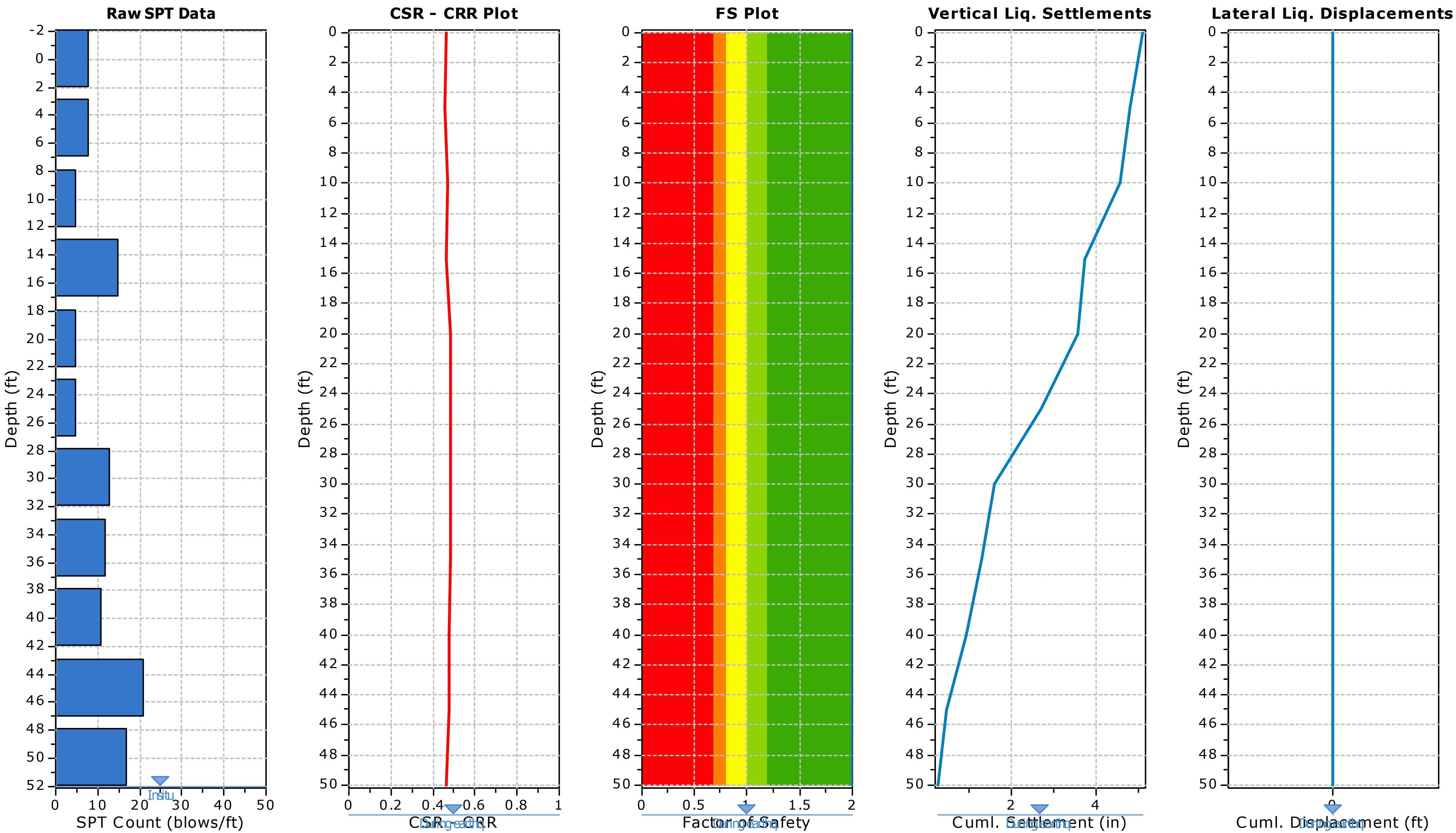
Location : Compton, California

Figure IV-6

:: Overall Liquefaction Assessment Analysis Plots ::



:: Overall Liquefaction Assessment Analysis Plots ::



APPENDIX V
SITE-SPECIFIC GROUND MOTION HAZARD ANALYSIS RESULTS

SITE-SPECIFIC GROUND MOTION ANALYSIS (ASCE 7-16)

Project:	Compton Community College PE Complex	**Latitude:**	33.87696 deg	**Calculated By:**	GLC
Client:	Compton Community College District	**Longitude:**	-118.21110 deg	**Checked By:**	RS
Job No:	10-57575PW	**Vs_{30}:**	259 m/s	**Date:**	January, 2021

Period T (sec)	PROBABILISTIC (RISK-TARGETED) GROUND MOTION ANALYSIS				DETERMINISTIC (84TH-PERCENTILE) GROUND MOTION ANALYSIS			CODE-BASED (LOWER LIMIT) ASCE 7-16 SECTION 11.4.6		SITE-SPECIFIC DESIGN RESPONSE		
	Uniform Hazard Ground Motion (g)	Risk Targeted Ground Motion (g)	Maximum Direction Scale Factor	Maximum Directional Probabilistic Sa (g)	84th Percentile Spectral Accelaration (g)	Maximum Direction Scale Factor	Maximum Directional Deterministic Sa (g)	Code Based S_a (g)	80% of Code Based S_a (g)	Design S_{aM} (g)	Design S_a (g)	T x S_a (T>1s)
PGA	0.774	0.738	1.1	0.812	0.947	1.1	1.042	0.452	0.361	0.812	0.541	---
0.10	1.302	1.265	1.1	1.392	1.366	1.1	1.503	1.008	0.807	1.392	0.928	---
0.20	1.725	1.686	1.1	1.855	1.834	1.1	2.017	1.129	0.903	1.855	1.236	---
0.30	1.952	1.859	1.125	2.091	2.249	1.125	2.530	1.129	0.903	2.091	1.394	---
0.50	1.882	1.751	1.175	2.057	2.454	1.175	2.883	1.129	0.903	2.057	1.372	---
0.75	1.536	1.407	1.2375	1.741	2.205	1.2375	2.729	0.916	0.733	1.741	1.161	---
1.00	1.268	1.157	1.3	1.504	1.952	1.3	2.538	0.687	0.549	1.504	1.003	1.003
2.00	0.672	0.607	1.35	0.819	1.094	1.35	1.477	0.343	0.275	0.819	0.546	1.093
3.00	0.424	0.381	1.4	0.533	0.632	1.4	0.885	0.229	0.183	0.533	0.356	1.067
4.00	0.290	0.260	1.45	0.377	0.410	1.45	0.595	0.172	0.137	0.377	0.251	1.005
5.00	0.213	0.191	1.5	0.287	0.291	1.5	0.437	0.137	0.110	0.287	0.191	0.955

INPUT PARAMETERS - SEAOC (https://seismicmaps.org/)

Site Class=	D	
F_a=	1.000	Short Period Site Coefficient
S_S=	1.694	Mapped MCE_R, 5% Damped at T=0.2s
S_1=	0.606	Mapped MCE_R, 5% Damped at T=1s
S_{DS}=	1.129	Design, 5% Damped at Short Periods
S_{MS}=	1.694	The MCE_R, 5% Damped at Short Periods
T_L (sec)=	8.0	Long Period Transition (Sect 11.4.6)
F_{PGA} (g)=	1.1	Site Coefficient for PGA
PGA_M (g)=	0.802	
F_v=	1.700	Used Only for Calculation of T_o and T_s
S_{M1}=	1.030	
S_{D1}=	0.687	Design, 5% Damped at T=1s
T_o (sec)=	0.122	Defined in ASCE 7-16 Sect 11.4.6
T_S (sec)=	0.608	Defined in ASCE 7-16 Sect 11.4.6

SITE-SPECIFIC DESIGN PARAMETERS

S_{DS}=	**1.255**	90% of max S_a (ASCE 7-16 Sect 21.4)	
S_{MS}=	**1.882**	MCE_R, 5% Damped, adjusted for Site Class	
S_{D1}=	**1.093**	Design, 5% Damped, at T=1s (Sect 11.4.5)	
S_{M1}=	**1.639**	MCE_R, 5% Damped, at T=1s, adjusted for Site	
F_a=	**1.000**	Short Period Site Coefficient (7-16 Sect 21.3)	
F_v=	**2.500**	Long Period Site Coefficient (7-16 Sect 21.3)	
S_S=	**1.882**	MCE_R, 5% Damped at T=0.2s	
S_1=	**0.656**	MCE_R, 5% Damped at T=1s	
$PGA_{Probabilistic}$ (g)=	**0.774**	Peak Ground Acceleration, Probabilistic	
$PGA_{Deterministic}$ (g)=	**0.947**	Peak Ground Acceleration, Deterministic	
F_{PGA} (g)=	**1.1**	Site Coefficient for PGA, when PGA = 0.5g	
$0.5*F_{PGA}$ (g)=	**0.550**	OK	(Check $PGA_{Deterministic} > 0.5 \times F_{PGA}$)
$0.8*PGA_M$ (g)=	**0.642**	PGA_M (g) (Determined from ASCE 7-16 Eq. 11.8-1)	
Site Specific PGA (g) =	**0.774**		(Check $PGA_{Site\ Specific} > 0.8 \times PGA_M$)

DETERMINISTIC (84TH-PERCENTILE) GROUND MOTION ANALYSIS											
Fault	Period, T (sec)										
	PGA	0.10	0.20	0.30	0.50	0.75	1.00	2.00	3.00	4.00	5.00
Newport-Inglewood Alt 1 (M=7.15)	0.734	1.101	1.499	1.770	1.845	1.632	1.468	0.857	0.568	0.388	0.277
Newport-Inglewood Alt 2 (M=7.15)	0.762	1.133	1.537	1.829	1.923	1.716	1.548	0.905	0.602	0.410	0.291
Compton (M=7.45)	0.947	1.366	1.834	2.249	2.454	2.205	1.952	1.094	0.632	0.396	0.274
Palos Verdes (M=7.38)	0.472	0.757	1.054	1.186	1.156	0.970	0.843	0.491	0.333	0.240	0.178
Puente Hills - Santa Fe Springs (M=6.61)	0.618	0.965	1.341	1.559	1.507	1.229	1.040	0.511	0.291	0.175	0.116
84th Percentile Spectral Accelaration	**0.947**	**1.366**	**1.834**	**2.249**	**2.454**	**2.205**	**1.952**	**1.094**	**0.632**	**0.410**	**0.291**

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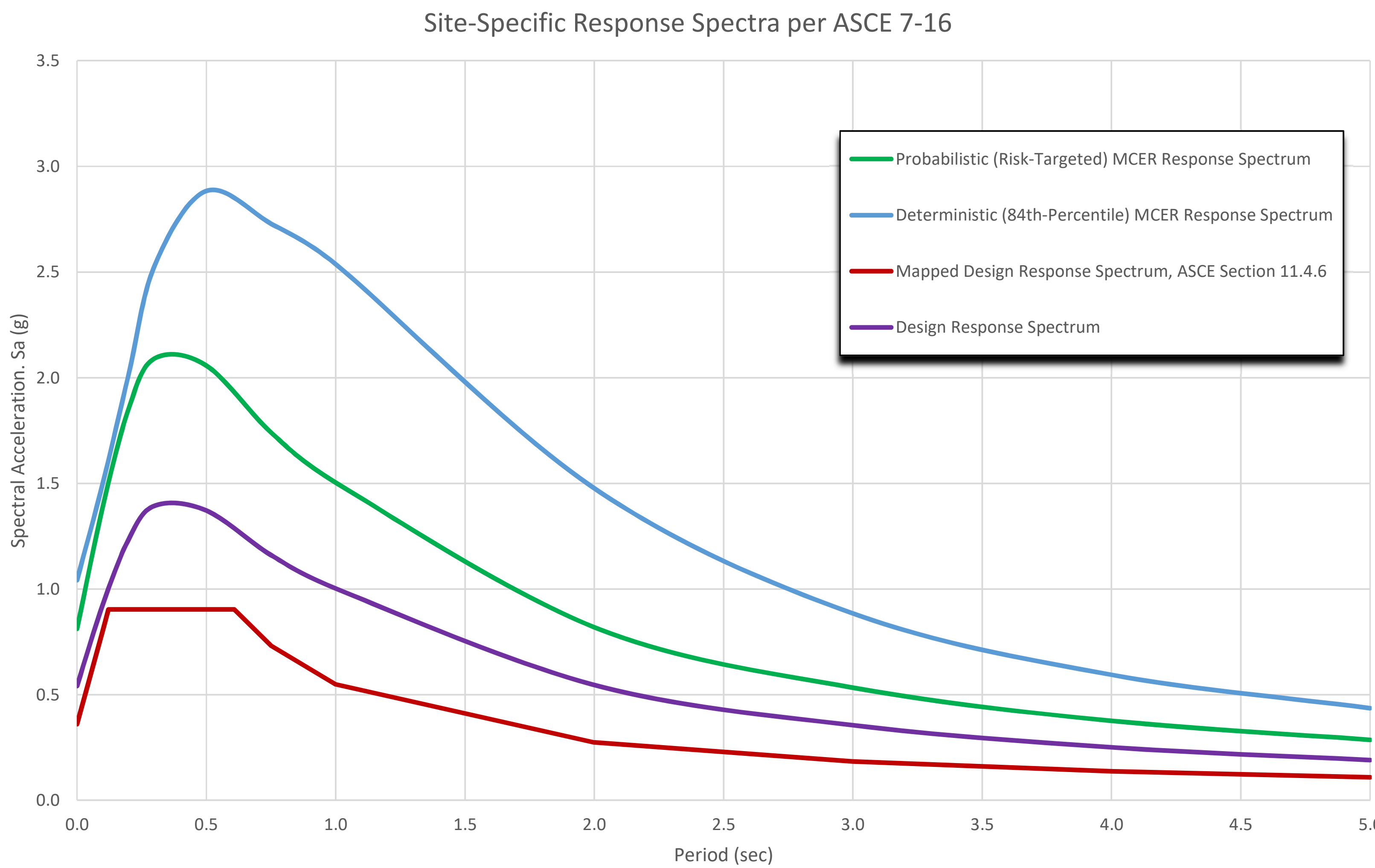
Compton College PE Complex Compton, California			
By:	GLC	Date:	July, 2021
Job Number:	10-57575PW	Figure:	V-2



Compton College PE Complex Compton, California			
By:	GLC	Date:	July, 2021
Job Number:	10-57575PW	Figure:	V-3

APPENDIX VI
INFILTRATION TEST RESULTS

We performed four borehole percolation test (BP-1 to BP-4) at different depths in general conformance with the Administrative Manual, County of Los Angeles Department of Public Works Geotechnical and Materials Engineering Division. Figures VI-1 to VI-8 present the results of the testing.

Shallow Borehole Percolation Testing Field Log

Project Name:	**PE Complex Replaccement - Compton Coll**	**Project No.:**	**10-57575PW**
Project Location:	Compton, California	**Boring Test Number:**	**BP-1**
Tested by:	KH	**Diameter of Boring (in)**	8
Liquid Description:	Water	**Depth of Boring (ft):**	5
Measurement Method:	Sounder	**Water Remaining:**	No
Depth to Test (ft):	4		

Reading Number	Time Start/End (hh:mm)	Time Interval Between Readings	Total Time Elapse (HR)	Volume of Water Needed per Reading (gal)	Cumulative Volume (gal)	Notes/Comments Head Drop
1	3:08 PM 3:18 PM	0:10	0.17	2.017	2.02	3
2	3:19 PM 3:29 PM	0:10	0.33	0.788	2.81	4 7/8
3	3:34 PM 3:44 PM	0:10	0.50	0.725	3.53	2 1/2
4	3:46 PM 3:56 PM	0:10	0.67	0.464	3.99	2 7/8
5	3:58 PM 4:08 PM	0:10	0.83	0.648	4.64	2 1/4
6	4:08 PM 4:18 PM	0:10	1.00	0.555	5.20	3/8
7	4:19 PM 4:29 PM	0:10	1.17	0.296	5.49	3 1/8
8	4:30 PM 4:40 PM	0:10	1.33	0.582	6.08	4 1/2
9	4:41 PM 4:51 PM	0:10	1.50	0.547	6.62	4 1/8
10	4:52 PM 5:03 PM	0:11	1.68	0.596	7.22	3 3/4

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FIGURE VI-1

Shallow Borehole Percolation Testing Field Log

Project Name:	**PE Complex Replacement - Compton College**	**Project No.:**	**10-57575PW**
Project Location:	Compton, California	**Boring Test Number:**	**BP-1**
Tested by:	KH	**Diameter of Boring (in)**	8
Liquid Description:	Domestic Water	**Depth of Boring (ft):**	5
Measurement Method:	Sounder	**Water Remaining:**	No

Water Depth Reading	
4	ft
Wetted Perim	
2.10	ft
Wetted Bottom	
0.35	sf
Wetted Area	
2	sf
Gravel Area	
0.21	sf
Gravel Porosity	
0.3	
Voids	
0.28	cf/ft

Raw Flow Rate	0.4	CF/HR
Raw Measured Rate	0.2	FT/HR
Reduction Factors		
Drywell Perc Test	2	
Site Variability	2	
Long-Term Siltation	3	
Total Reduction	12	
Design Infiltration Rat	0.01	FT/HR
	0.16	in/hr

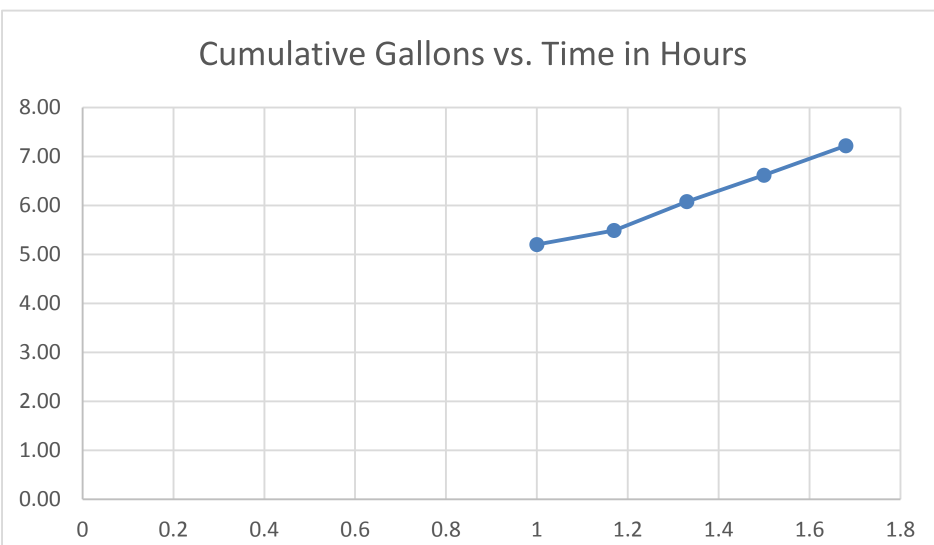

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FIGURE VI-2

Deep Borehole Percolation Testing Field Log

Project Name:	PE Complex Replacement - Compton Colle	**Project No.:**	10-57575PW
Project Location:	Compton, California	**Boring Test Number:**	BP-2
Tested by:	LM	**Diameter of Boring (in)**	8
Liquid Description:	Water	**Depth of Boring (ft):**	25
Measurement Method:	Sounder	**Water Remaining:**	No
Depth to Test (ft):	6		

Reading Number	Time Start/End (hh:mm)	Time Interval Between Readings	Total Time Elapse (HR)	Volume of Water Needed per Reading (gal)	Cumulative Volume (gal)	Notes/Comments
1	9:40 AM 9:55 AM	0:15	0.25	11.6	11.6	
2	9:55 AM 10:10 AM	0:15	0.50	10.1	21.7	
3	10:10 AM 10:25 AM	0:15	0.75	9	30.7	
4	10:25 AM 10:40 AM	0:15	1.00	8.6	39.3	
5	10:40 AM 10:55 AM	0:15	1.25	7.8	47.1	
6	10:55 AM 11:10 AM	0:15	1.50	7.8	54.9	
7	11:10 AM 11:25 AM	0:15	1.75	7.1	62	
8	11:25 AM 11:40 AM	0:15	2.00	6.8	68.8	
9	11:40 AM 11:55 AM	0:15	2.25	6.8	75.6	
10	11:55 AM 12:10 PM	0:15	2.50	6.6	82.2	
11	12:10 PM 12:25 PM	0:15	2.75	6.2	88.4	
12	12:25 PM 12:40 PM	0:15	3.00	6.2	94.6	
13	12:40 PM 12:55 PM	0:15	3.25	6.4	101	
14	12:55 PM 1:10 PM	0:15	3.50	6.3	107.3	
15	1:10 PM 1:25 PM	0:15	3.75	6.2	113.5	
16	1:25 PM 1:40 PM	0:15	4.00	6.2	119.7	
17	1:40 PM 1:55 PM	0:15	4.25	6.1	125.8	
18	1:55 PM 2:10 PM	0:15	4.50	6.2	132	
19	2:10 PM 2:25 PM	0:15	4.75	6.2	138.2	
20	2:25 PM 2:40 PM	0:15	5.00	6	144.2	
21	2:40 PM 2:55 PM	0:15	5.25	6	150.2	
22	2:55 PM 3:10 PM	0:15	5.50	6.2	156.4	
23	3:10 PM 3:25 PM	0:15	5.75	5.9	162.3	
24	3:25 PM 3:40 PM	0:15	6.00	6	168.3	

FIGURE VI-3

Deep Borehole Percolation Testing Field Log

Project Name:	**PE Complex Replacement - Compton College**	**Project No.:**	**10-57575PW**
Project Location:	Compton, California	**Boring Test Number:**	**BP-2**
Tested by:	LM	**Diameter of Boring (in)**	8
Liquid Description:	Domestic Water	**Depth of Boring (ft):**	25
Measurement Method:	Sounder	**Water Remaining:**	No

Water Depth Reading	6 ft
Wetted Perim	2.10 ft
Wetted Bottom	0.35 sf
Wetted Area	40 sf
Gravel Area	0.29 sf
Gravel Porosity	0.3
Voids	0.26 cf/ft

Raw Flow Rate	3.2	CF/HR
Raw Measured Rate	0.08	FT/HR
Reduction Factors		
Drywell Perc Test	2	
Site Variability	2	
Long-Term Siltation	3	
Total Reduction	12	
Design Infiltration Rat	0.007	FT/HR
	0.08	in/hr

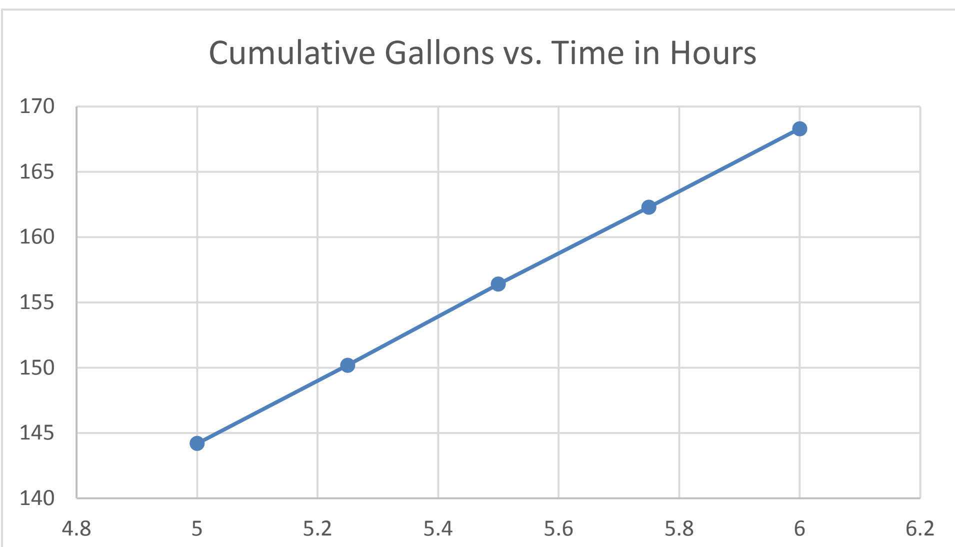

FIGURE VI-4

Deep Borehole Percolation Testing Field Log

Project Name:	**PE Complex Replacement - Compton Colle**	**Project No.:**	**10-57575PW**
Project Location:	Compton, California	**Boring Test Number:**	**BP-3**
Tested by:	KH	**Diameter of Boring (in)**	8
Liquid Description:	Water	**Depth of Boring (ft):**	25
Measurement Method:	Sounder	**Water Remaining:**	No
Depth to Test (ft):	6		

Reading Number	Time Start/End (hh:mm)	Time Interval Between Readings	Total Time Elapse (HR)	Volume of Water Needed per Reading (gal)	Cumulative Volume (gal)	Notes/Comments
1	8:15 AM 8:30 AM	0:15	0.25	72.61	72.6	
2	8:30 AM 8:45 AM	0:15	0.50	38.31	110.9	
3	8:45 AM 9:00 AM	0:15	0.75	7.666	118.6	
4	9:00 AM 9:15 AM	0:15	1.00	4.728	123.3	
5	9:15 AM 9:30 AM	0:15	1.25	3.737	127.1	
6	9:30 AM 9:45 AM	0:15	1.50	3.489	130.5	
7	9:45 AM 10:00 AM	0:15	1.75	3.09	133.6	
8	10:00 AM 10:15 AM	0:15	2.00	2.362	136.0	
9	10:15 AM 10:30 AM	0:15	2.25	1.977	138.0	
10	10:30 AM 10:45 AM	0:15	2.50	2.666	140.6	
11	10:45 AM 11:00 AM	0:15	2.75	1.768	142.4	
12	11:00 AM 11:15 AM	0:15	3.00	2.227	144.6	
13	11:15 AM 11:30 AM	0:15	3.25	1.821	146.5	
14	11:30 AM 11:45 AM	0:15	3.50	1.988	148.4	
15	11:45 AM 12:00 PM	0:15	3.75	2.613	151.1	
16	12:00 PM 12:15 PM	0:15	4.00	1.999	153.1	
17	12:15 PM 12:30 PM	0:15	4.25	1.857	154.9	
18	12:30 PM 12:45 PM	0:15	4.50	2.487	157.4	
19	12:45 PM 1:00 PM	0:15	4.75	2.248	159.6	
20	1:00 PM 1:15 PM	0:15	5.00	2.371	162.0	

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FIGURE VI-5

Deep Borehole Percolation Testing Field Log

Project Name:	**PE Complex Replacement - Compton College**	**Project No.:**	**10-57575PW**
Project Location:	Compton, California	**Boring Test Number:**	**BP-3**
Tested by:	KH	**Diameter of Boring (in)**	8
Liquid Description:	Domestic Water	**Depth of Boring (ft):**	25
Measurement Method:	Sounder	**Water Remaining:**	No

Water Depth Reading	6 ft
Wetted Perim	2.10 ft
Wetted Bottom	0.35 sf
Wetted Area	40 sf
Gravel Area	0.21 sf
Gravel Porosity	0.3
Voids	0.28 cf/ft

Raw Flow Rate	4.3	CF/HR
Raw Measured Rate	0.11	FT/HR
Reduction Factors		
Drywell Perc Test	2	
Site Variability	2	
Long-Term Siltation	3	
Total Reduction	12	
Design Infiltration Rat	0.01	FT/HR
	0.11	in/hr

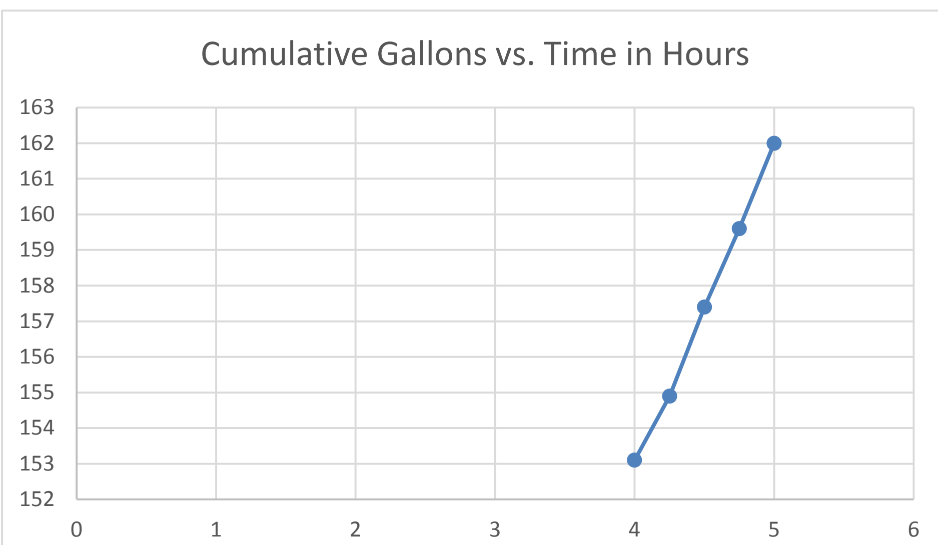


FIGURE VI-6

Shallow Borehole Percolation Testing Field Log

Project Name:	**PE Complex Replacement - Compton Colle**	**Project No.:**	**10-57575PW**
Project Location:	Compton, California	**Boring Test Number:**	**BP-4**
Tested by:	KH	**Diameter of Boring (in)**	8
Liquid Description:	Water	**Depth of Boring (ft):**	5
Measurement Method:	Sounder	**Water Remaining:**	No
Depth to Test (ft):	4		

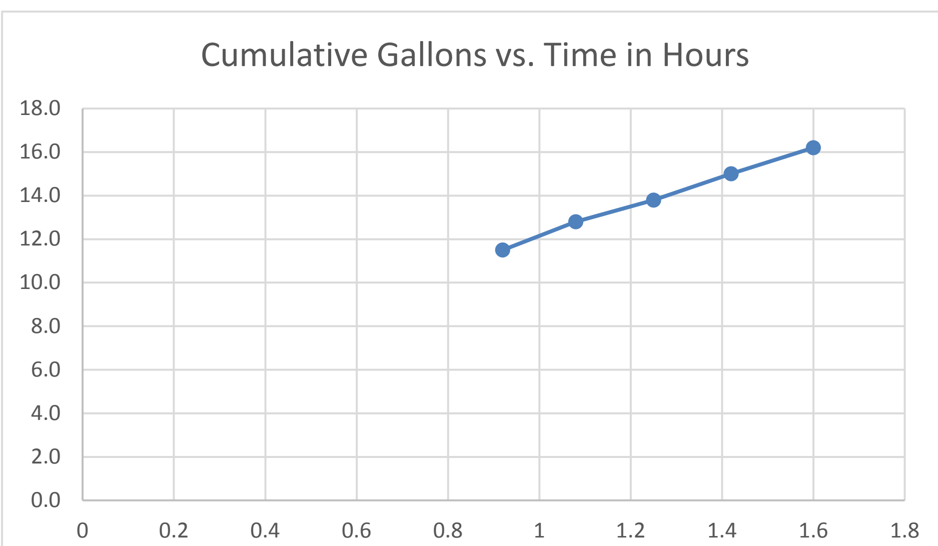
Reading Number	Time Start/End (hh:mm)	Time Interval Between Readings	Total Time Elapse (HR)	Volume of Water Needed per Reading (gal)	Cumulative Volume (gal)	Notes/Comments Head Drop
1	3:16 PM 3:31 PM	0:15	0.25	4.283	4.3	6 1/2
2	3:31 PM 3:41 PM	0:10	0.42	2.465	6.7	4 7/8
3	3:41 PM 3:51 PM	0:10	0.58	2.413	9.2	2 1/2
4	3:53 PM 4:03 PM	0:10	0.75	1.069	10.2	6 1/2
5	4:05 PM 4:15 PM	0:10	0.92	1.241	11.5	4 1/8
6	4:16 PM 4:26 PM	0:10	1.08	1.307	12.8	3 1/4
7	4:28 PM 4:38 PM	0:10	1.25	1.066	13.8	3 5/8
8	4:40 PM 4:50 PM	0:10	1.42	1.194	15.0	3 1/2
9	4:52 PM 5:03 PM	0:11	1.60	1.166	16.2	3 5/8

FIGURE VI-7

Shallow Borehole Percolation Testing Field Log

Project Name:	**PE Complex Replacement - Compton College**	**Project No.:**	**10-57575PW**
Project Location:	Compton, California	**Boring Test Number:**	**BP-4**
Tested by:	KH	**Diameter of Boring (in)**	8
Liquid Description:	Domestic Water	**Depth of Boring (ft):**	5
Measurement Method:	Sounder	**Water Remaining:**	No

Water Depth Reading		Raw Flow Rate	0.9	CF/HR
4 ft		Raw Measured Rate	0.4	FT/HR
Wetted Perim		Reduction Factors		
2.10 ft		Drywell Perc Test	2	
Wetted Bottom		Site Variability	2	
0.35 sf		Long-Term Siltation	3	
Wetted Area				
2 sf		Total Reduction	12	
Gravel Area		Design Infiltration Rat	0.03	FT/HR
0.21 sf			0.38	in/hr
Gravel Porosity				
0.3				
Voids				
0.28 cf/ft				



APPENDIX VII
HISTORIC SEISMIC EVENTS

Historic Seismicity (1900 to 2018)
Within 100 km Search Radius and M_W > 5.0
Proposed Instructional Building #2, Compton College
1111 East Artesia Blvd., Compton, CA 90221

Local System Date and Time (UTC-08:00)	Latitude	Longitude	Depth (km)	Magnitude (M_W)	Place
2014-03-29T04:09:42.170Z	33.9325	-117.9158	5.1	5.1	2km NW of Brea, CA
2008-07-29T18:42:15.670Z	33.9485	-117.7663	15.5	5.4	5km S of Chino Hills, CA
1997-04-26T10:37:30.670Z	34.3690	-118.6700	15.9	5.1	12km ESE of Piru, California
1995-06-26T08:40:28.940Z	34.3940	-118.6690	12.8	5.0	11km SW of Valencia, California
1994-03-20T21:20:12.260Z	34.2310	-118.4750	12.4	5.2	3km WNW of Panorama City, California
1994-01-29T11:20:35.970Z	34.3060	-118.5790	0.6	5.1	6km NNE of Chatsworth, California
1994-01-19T21:11:44.900Z	34.3780	-118.6190	10.8	5.1	10km SSW of Valencia, California
1994-01-19T21:09:28.610Z	34.3790	-118.7120	13.8	5.1	8km ESE of Piru, California
1994-01-18T00:43:08.890Z	34.3770	-118.6980	10.7	5.2	10km ESE of Piru, California
1994-01-17T23:33:30.690Z	34.3260	-118.6980	9.1	5.6	7km NNE of Simi Valley, California
1994-01-17T12:40:36.120Z	34.3400	-118.6140	5.4	5.2	9km N of Chatsworth, California
1994-01-17T12:31:58.120Z	34.2750	-118.4930	5.3	5.9	1km ENE of Granada Hills, California
1994-01-17T12:30:55.390Z	34.2130	-118.5370	18.2	6.7	1km NNW of Reseda, CA
1991-06-28T14:43:54.660Z	34.2700	-117.9930	8.0	5.8	13km NNE of Sierra Madre, CA
1990-02-28T23:43:36.750Z	34.1440	-117.6970	3.3	5.5	6km NNE of Claremont, CA
1988-12-03T11:38:26.450Z	34.1510	-118.1300	13.7	5.0	1km SSE of Pasadena, CA
1987-10-04T10:59:38.190Z	34.0740	-118.0980	7.7	5.3	2km WSW of Rosemead, CA
1987-10-01T14:42:20.020Z	34.0610	-118.0790	8.9	5.9	2km SSW of Rosemead, CA
1981-09-04T15:50:48.700Z	33.5575	-119.1195	5.5	5.5	11km NNW of Santa Barbara Is., CA
1979-01-01T23:14:38.620Z	33.9165	-118.6872	13.3	5.2	13km S of Malibu Beach, CA
1973-02-21T14:45:56.140Z	33.9790	-119.0502	10.0	5.3	22km W of Malibu, CA
1971-02-09T14:10:29.040Z	34.4160	-118.3700	6.0	5.3	10km SSW of Agua Dulce, CA
1971-02-09T14:02:45.740Z	34.4160	-118.3700	6.0	5.8	10km SSW of Agua Dulce, CA
1971-02-09T14:01:12.450Z	34.4160	-118.3700	6.0	5.8	10km SSW of Agua Dulce, CA
1971-02-09T14:00:41.920Z	34.4160	-118.3700	9.0	6.6	10km SSW of Agua Dulce, CA
1970-09-12T14:30:53.000Z	34.2548	-117.5343	10.8	5.2	3km W of Lytle Creek, CA
1941-11-14T08:41:38.350Z	33.7907	-118.2637	6.0	5.1	5km E of Lomita, CA
1938-05-31T08:34:56.580Z	33.6993	-117.5112	10.2	5.2	8km ENE of Trabuco Canyon, CA
1933-03-11T06:58:45.610Z	33.6238	-118.0012	6.0	5.3	7km W of Newport Beach, CA
1933-03-11T05:18:48.490Z	33.7667	-117.9850	6.0	5.0	2km ENE of Westminster, CA
1933-03-11T01:54:10.660Z	33.6308	-117.9995	6.0	6.4	7km WNW of Newport Beach, CA
1922-03-10T11:21:04.000Z	34.2430	-119.0970	10.0	6.5	Greater Los Angeles area, California
1918-04-21T22:32:29.000Z	33.6470	-117.4330	10.0	6.7	Southern California