GEOTECHNICAL AND GEOHAZARD INVESTIGATION REPORT

VISUAL AND PERFORMING ARTS BUILDING
COMPTON COMMUNITY COLLEGE DISTRICT

PREPARED FOR:

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

PREPARED BY:
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April 27, 2022

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April 27, 2022

Atlas No. 10-61187PW
Report No. 1

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

Subject: Geotechnical and Geohazard Investigation
Compton College Visual and Performing Arts Building
Compton College Campus
1111 East Artesia Boulevard, Compton, CA 90221

Dear Ms. Owens:

Atlas Technical Consultants is pleased to present this geotechnical and geohazard investigation report for the proposed Visual and Performing Arts Building, Compton College, located at 1111 East Artesia Boulevard in 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 plans provided by Struere Advanced Architecture, Inc. and our correspondence 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,
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1. INTRODUCTION

1.1 Site Location and Description

The project site is located within the half-south portion of the Compton College Campus in the city of Compton, California. The project site is surrounded by landscaped areas to the north, buildings and a landscaped areas to the south, east and west. Figure I-1 presents the site vicinity map. The project location, measured on a Google Earth map, has a latitude reading of North 33.87727° and longitude reading of West -118.21036°. 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 consist of the design and construction of a new Visual and Performing Arts (VAPA) building that will likely consists of three separate single-story buildings joined by covered breezeways with associated hardscape and utility improvements. The Project Structural Engineer provided the below information for the structural loads:

- **New Buildings**
  - Bearing wall load gravity loads. The bearing wall on Grid C is the peak case (1,900 PLF, Dead + Roof Live Load)
  - Shear wall overturning load. Peak case is 75 kip-ft at allowable level.
  - Typical column gravity and seismic loads at building front (on Grid 3, between A & B):
    - P_{Dead} = 3 kips, $P_{Roof\ Live}$ = 2 kips, P_E = 8 kips (seismic at allowable level)
- **New Exterior Canopy Structure**
  - Typical column gravity and seismic loads:
    - P_{Dead} = 5 kips, $P_{Roof\ Live}$ = 6 kips, P_E = 5 kips (seismic at allowable level)
- **Existing Building**
  - New shear wall. Vertical load is 400 PLF (Dead + Roof Live) and seismic overturning is 30 kip-ft (at allowable level). Note the shear wall to the south includes widening an existing footing.
  - New retaining wall. Vertical load is 1,700 PLF (Dead + Roof Live).
  - New built-up seating area.

We anticipate that the new buildings 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 VAPA buildings and associated improvements 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 three borings to the depths ranging from about 21 feet to 51 feet below the ground surface and three CPT tests to the maximum depth of about 75 feet 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. The borings were backfilled in accordance with regulatory requirements. Logs of the borings are presented in Appendix II. Boring locations are shown on Figure I-2 (Subsurface Exploration 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.
  - 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.
  - Hydrometer Test.
  - #200 Wash.
  - Compaction Test.
  - Soil Corrosivity.
  - Collapse/Swell potential of soil.

All laboratory tests were performed in general conformance with ASTM Standard 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 I-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 subsurface soils encountered in the borings generally consist of approximately 4 to 5 feet of undocumented fill underlain by young alluvial deposits of Holocene to late Pleistocene age (Qya_2) as shown on the geologic cross section (Figure I-4). The fill generally consists of loose and slightly moist silty sand. The young alluvial deposits encountered at the site are predominantly comprised of inter-layered sand, silt and clay.

Logs of borings is presented in Appendix I. 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 0 and 4, generally indicating very low expansion potential for onsite sandy and silty soils. To the best of our knowledge and experience with the similar soils some of the clay (CH) and silty (MH) layers on the site may have medium to high expansion potential.

2.2.2 Atterberg Limits Tests Results

The samples of the sub-surface soils collected during our investigation were tested for Atterberg Limits. Based on the lab testing results and our experience with similar type of materials, generally the on-site sand and silt are non-plastic, however, some of the tests results indicate some low plasticity silt and clay and some fat and elastic silt layers in the subsurface soil.

2.2.3 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 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 improvements were tested for water-soluble sulfate during the investigation and had a soluble sulfate content of 50 and 660 ppm that are less than 0.1% by weight (1,000 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 evaluate minimum resistivity, chloride content, and pH level. The chloride content of the samples was 60 ppm and 250 ppm. The measured resistivity of tested samples was 2,360 and 506 ohm-cm. The pH values of the samples were 8.19 and 8.02.

Based on these results, the on-site soil is generally considered to be extremely 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.

2.2.4 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 not encountered in our soil borings (B-1, B-2 and B-3). According to the California Geological Survey (CGS, 1998) seismic hazard zone report for the South Gate quadrangle, the depth of the historically shallowest groundwater level is estimated to be on the order of 8 feet. 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 variations 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 I-8 (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 1.

Table 1 – 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 site was performed to show the significant past earthquakes. Figure I-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 V.

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. According to the Ground Shaking Intensity (Isoseismal) Maps for the Magnitude 6.4, 1933 Long Beach Earthquake (from Trifunac, 2003; CGS website), the Compton College site is mapped within an area that reportedly sustained damage that ranged from Modified Mercalli Scale Intensity 7 (people run outdoors, damage to poorly build structures) to Intensity 9 (buildings shifted off foundation). In Compton, almost every building in a three-block radius on unconsolidated material and landfill was damaged; and water, electricity, gas, and phones were all turned off within minutes of the main shock (CDMG, California Geology, March 1973, p. 56). The worst of all building failures included Compton Union High School and Compton Junior College (CDMG, California Geology, March 1973, p. 57). Other buildings in Compton with reported major damage included the Young Hotel and Aranbe Hotel (Daily News with photos from Orange County Register).

Extensive damage consisted of fracturing and dislocation of streets and curbs in water-saturated, lowland sediments of the Compton basin, especially at Compton Junior College (CDMG, California Geology, March 1973, p. 58). Based on our review, it appears that most of the reported damages were due to seismic shaking/ground motion. There was no conclusive evidence of surface manifestation of liquefaction such as sand boils and/or ground cracking that was reported near El Camino College Compton Center Campus (called Compton Junior College in 1933). However, as stated in our project geotechnical report (Reference 2) the potential for liquefaction susceptibility of the site is very high, there is a potential for surface manifestations of liquefaction at the site, and the potential for seismically induced settlement is high. A summary of the major earthquakes and reported damages at the epicenter are presented 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 I-6.

A site-specific liquefaction and seismic settlement analyses were performed using LiqSVs 2.0.2.1 and CLiq v.2.3.1.15 computer programs. Seismically induced settlement analyses were performed based on the sub-surface conditions encountered in the deep boring B-3 and the three CPTs. For this analysis, we considered a historic highest groundwater level at 8 feet below ground surface as indicated on the CGS Seismic Hazards Report. 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 0.801g 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 soil layers specifically for sandy silt and silty sand occurring below 8 feet (historic highest groundwater table); therefore, the liquefaction susceptibility of the site is very high. Calculations are provided in Appendix III.

3.2.2 Seismically Induced Settlement

Seismically induced settlement consists of dry dynamic settlement (above groundwater) and liquefaction induced settlement (below groundwater). Generally, 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 soil, the maximum potential of total seismic settlement at the site: seismic dry settlement and liquefaction settlement, has been estimated generally to be on the order of about 5½ to 6 inches (considering the historically highest groundwater table at the depth of about 8 feet, Mw=7.3, peak ground acceleration of 0.801g and using depth reduction factor). The corresponding differential seismic settlement is estimated to be on the order of about 3 to 3½ inches over a horizontal distance of 40 feet. This potential settlement is generally due to liquefaction settlement. Using compacted fill for the upper 5 feet of the subgrade soil the total seismic settlement at the site can be estimated generally to be on the order of about 5 inches with the differential seismic settlement on the order of about 2½ inches over a horizontal distance of 40 feet (seismic dry settlement and liquefaction 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 the on-site soil that can be reduced by following the recommendations provided in this report.

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.

To evaluate the potential of hydro-collapse of the soil layers versus depth laboratory collapse tests performed on the on-site soil samples. For the tested samples, the potential for collapse was found to be on the order of about 0.5%.

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 (FEMA, 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. As a supplementary explanation and based on the information provided in "https://www.usgs.gov/centers/land-subsidence-in-california" the site is located within the zone of subsidence due to groundwater pumping.

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 (CEMA, 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 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

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.

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 compacted 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 debris and unsuitable materials. Prior to construction, it will be necessary to demolish the existing buildings, utilities (if needed), remove 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 firm and stable subgrade soils. The need for and extent of removal of soils disturbed by site demolition should be evaluated by the Geotechnical Engineer at the time of grading.

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.

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 buried obstructions should be properly backfilled and compacted as recommended in Sections 4.1.3, 4.12 and other pertinent sections 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 Buildings Pads 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 about 5 feet below existing grade or to a sufficient depth to remove the undocumented fill materials 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 about 5 feet of compacted 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 underground obstructions to be removed. Where feasible, the overexcavation and recompaction should extend a minimum of 5 feet laterally from the edges of the footings and/or buildings footprints whichever is greater. The soil below exterior flat work and 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 planned for asphalt or concrete pavement should be overexcavated and recompacted 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, 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 evaluated by ASTM D1557.

Fill 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, contaminant 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, contaminant 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 recommended).

If base materials are imported to be used, 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.

The Geotechnical 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: 33.87727 °N; Longitude: -118.21036 °W) are evaluated based on general ground motion analysis in accordance with Section 1613A.2 of the 2019 CBC. These parameters are summarized in Table 2.

Table 2 – California Building Code Seismic Design Parameters

Categorization/Coefficient	Design Value
Site Class	D
Risk Category	III
Spectral Response Acceleration at Short Period, S_S	1.693
Spectral Response Acceleration at 1-Second Period, S_1	0.606
Site Amplification Factor at 0.2 Second, F_a	1.0
Site Amplification Factor at 1.0 Second, F_v	1.7
Spectral Response Acceleration at Short Period, Adjusted for Site Class, S_{MS}	1.693
Spectral Response Acceleration at 1-Second Period, Adjusted for Site Class, S_{M1}	1.031
Design Spectral Acceleration at Short Period, S_{DS}	1.129
Design Spectral Acceleration at 1-Second Period, S_{D1}	0.687
Peak Ground Acceleration Value, PGA_M	0.801
Seismic Design Category	D

A site-specific ground motion analysis was performed as part of our investigation for the Compton College, PE Complex Replacement (Atlas Geotechnical Investigation Report, Project No. 10-57575PW dated July 7, 2021) and we presented the results of that study for this project as well (due to the close proximity of the two projects sites). 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 3.

Table 3 – 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 2 and 3, whichever is more conservative.

4.3 Shallow Foundation System

The following sections provide information and recommendation for shallow foundation system.

4.3.1 Mat Foundation System for: Building Structures

Due to the relatively high seismic settlements (liquefaction and seismic dry settlements), a mat foundation system on a layer of compacted fill (Section 4.1.3) is recommended for the building structures. A mat foundation can be used to distribute foundation loads to span local irregularities in the supporting capacity of the foundation soil, and to mitigate the predicted magnitude of differential settlement. The mat foundation may be designed for allowable bearing pressure up to a maximum of 1,000 pounds per square foot (psf). The total static settlement is anticipated to be on the order of about 2 inches with a differential settlement of about 1¼ inch over a horizontal distance of 40 feet.

For the design of structural mat foundation, an average modulus of subgrade reaction, Ks between 20 and 30 pounds per cubic inch (pci) may be used (including a reduction for the size of the mat). In addition, we recommend that the mat foundation be designed to tolerate static and seismically induced total and differential settlements (ASCE 7-16, Section 12.13.9).

The structural mat foundation recommended here in for building support, should be at least 2 to 2½ feet thick and the bottom of the mat foundation should be constructed at a level about 2 feet below the existing grade and should be supported on at least 3 feet of compacted fill (Section 4.1.3: undocumented fill below the foundation is not allowed). Subgrade soil should be prepared as described in the earthwork section of this report (Section 4.1).

4.3.2 Shallow Foundation System with Grade Beams/Tie Beams for Building Structures, Canopy and New Built-Up Seating Area (Platform)

Another alternative for the foundation system is using a continuous foundation system with grade/tie beams supported on a layer of compacted fill (Section 4.1.3). We assumed that the continuous foundation system would be at least 2 to 2½ feet thick and the bottom of the foundation system would be constructed at a level about 2 feet below the existing grade and should be supported on at least 3 feet of compacted fill (Section 4.1.3: undocumented fill below the foundation is not allowed). A net allowable bearing pressure of 2,000 psf may be used for these foundation systems. The total static settlement is anticipated to be on the order of 1¾ inch with a differential settlement of about 1 inch over a horizontal distance of 40 feet. This foundation system shall be designed to tolerate static and seismically induced total and differential settlements (ASCE 7-16, Section 12.13.9). The width of the footing is recommended to be at least 2 feet.

4.3.3 Continuous Foundation System (Building Wall Footing)

The bottom of continuous footing system (e.g., under building perimeter wall) should be constructed at a level about 2 feet below the existing grade and should be supported on at least 3 feet of compacted fill (Section 4.1.3: undocumented fill below the foundation is not allowed). Subgrade soil should be prepared as described in the earthwork section of this report (Section 4.1). The continuous footing may be designed for allowable bearing pressure up to a maximum of 2,000 psf. The total static settlement is anticipated to be on the order of about 1¾ inch with a differential settlement of about 1 inch over a horizontal distance of 40 feet. We recommend that this foundation system be designed to tolerate static and seismically induced total and differential settlements (ASCE 7-16, Section 12.13.9). The width of the footing is recommended to be at least 2 feet.

4.3.4 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 compacted fill and should be embedded at least 24 inches below the existing grade. A vertical allowable bearing pressure of 2,000 psf may be used for these footings. No undocumented fill is allowed under the footings. The total static settlement is anticipated to be on the order of 1¾ inch with a differential settlement of about 1 inch over a horizontal distance of 40 feet. This foundation system shall be designed to tolerate static and seismically induced total and differential settlements. The width of the footing is recommended to be at least 18 inches.

4.4 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. An allowable coefficient of friction of 0.25 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 friction resistance and passive resistance of the soils may be combined provided that the passive resistance is reduced by one third.

4.5 Important Notes for Shallow Foundation Systems

The recommendations and information provided in this section can be applied to the foundation systems indicated in Sections 4.3 and 4.10.

The subgrade soil and the fill shall be prepared as described in the earthwork section of this report (Section 4.1 and the pertinent subsections). The allowable bearing values of the foundation systems indicated in the above sections are for total dead-load and frequently applied live-loads. Adjacent utilities or foundations should be avoided within the zone of an imaginary plane extending downward at a 1½H:1V (horizontal: vertical) inclination from the bottom edge of the foundation. The foundation system shall be designed to tolerate the total and differential: static and seismic settlements as presented in this report.

In some particular cases for the foundations, 4 feet of embedment depth (depth of the bottom of the foundation) and 1 foot of compacted fill below the bottom of the foundation, the information and recommendations provided in this report (for foundation system) still are applicable. We also, generally recommend using Grade Beams/Tie Beams in two directions (perpendicular).

Footings enlarging should be based on the earthwork and the other pertinent recommendations and information provided in this report. Depends to the required depth of the excavation, underpinning to the existing footings and shoring to support the excavation wall adjacent to the existing footings may need to be designed and performed. Atlas can provide the preliminary recommendations for shoring parameters if required by the design team. The allowable bearing values (vertical and lateral) may be increased by 33% for short duration of loading, including the effects of wind or seismic forces.

In this project the total differential settlement: static and seismic (liquefaction and dry) are below the total differential settlement threshold (static and seismic: dry and liquefaction) provided by the project Structural Engineer: referenced by Table 12.13-3, ASCE 7-16 that is considered to be 0.010 L (4.8 inches).

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 5 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 4.8.

4.6.1 Exterior Concrete

To reduce the potential for excessive cracking of concrete flatwork (such as walkways, etc.), concrete should be a minimum of 5 inches thick and provided with construction or weakened plane joints at frequent intervals. Concrete should be placed on properly prepared subgrade soil.

4.7 Pole (Concrete Shaft) Foundation

The poles (concrete shaft) foundations are considered to have side friction resistance of the bearing soil as well as by lateral resistance for overturning. The allowable side friction can be assumed to be on the order of about 200 psf. The uplift capacity is considered to be half of the downward capacity, based on the side friction resistance (200 psf). The allowable passive resistance when the ground surface is level, may be assumed to be equal to the pressure developed by a fluid with a density of 200 pounds per cubic foot (pcf), to a maximum allowable value of 2,000 psf. The upper 2 feet of the soil can be neglected for side friction and passive resistance. These resistance parameters are based on the geotechnical capacity. The structural engineer of the project should evaluate the structural capacity of the poles (concrete shafts).

The preliminary recommended diameter is 3 feet and the preliminary recommended length is 8 feet. (Assumption: center to center, at least three diameters of the shaft.)

Proper construction techniques should be used to limit disturbance of the soils during shaft installation. Disturbance of the soils at the bottom of the shaft excavation may result in shaft settlement, disturbance at the top of the shaft may result in greater lateral deflection than anticipated. The disturbance of the soil should be corrected by overexcavation and/or recompaction.

Due to the type of the soil in the project site, caving, sloughing and heaving are anticipated and may happen during the shaft excavation. Precautions should be taken during the drilling operation to reduce the potential of caving, sloughing and heaving by using the proper means and methods such as using casing or specially formulated drilling fluid that may be employed by the contractor. Where excessive caving occurs during excavation in the upper 6 feet, the hole may be backfilled with sand-cement slurry and redrilled through the slurry. Experienced contractors should be retained to install drilled the shafts. We recommend that a representative of the Geotechnical Engineer perform continuous observation during drilling of holes.

After completion of drilling, the bottom of the holes should be cleaned of loose or disturbed materials. Before casting concrete, the drilled holes should be observed, and suitable condition at the bottom of the holes should be confirmed. Shafts closer than three diameters to each other should be drilled and filled with concrete alternately, and concrete should be permitted to set at least 8 hours before drilling an adjacent pile. The drilled hole should be filled with concrete as soon as possible and should not be left open overnight.

4.8 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 coverings, 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.

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.

4.9 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 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.

4.10 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 4.

Table 4 – Earth Pressures for Retaining Walls

Wall Movement	Backfill Condition	Equivalent Fluid Pressure (on-site soil) (pcf)
Free to Deflect (Cantilever)	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. Surcharges (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 compacted fill. The footing embedment should be at least 2 feet below the lowest adjacent grade. The maximum allowable bearing pressure recommended is 2,000 psf. The allowable bearing value may be increased by 33% for short duration of loading, including the effects of wind or seismic forces. The width of the footing is recommended to be at least 2 feet.

The total static settlement is anticipated to be on the order of 1¾ inch with a differential settlement of about 1 inch over a horizontal distance of 40 feet. This foundation system shall be designed to tolerate static and seismically induced total and differential settlements.

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.

4.11 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 slope in a concentrated manner, pond on the pad or against foundations.

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 landscape areas immediately adjacent to the building foundations. In addition, we recommend a minimum 2% slope away from the building foundations be established for 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 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.

4.12 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 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.

4.13 Preliminary Pavement Section

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

4.13.1 Asphalt Concrete Pavement

The pavement structural sections depend on the expected wheel loads, volume of traffic, and subgrade soils. The characteristics of subgrade soils are evaluated by R-value testing. Based on soil classification and the results of the R-value tests, we assumed an R-value on the order of about 35 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. The project Civil Engineer should determine the traffic index to be used for different areas of the site.

Table 5 – Asphalt Pavement Sections

Traffic Index	Assumed R-Value for Silty Sand = 35	
	Asphalt Thickness (in)	Base Course (CAB) Thickness (in)
4	3.0	4.5
5	3.5	4.5
6	4.5	5.0
7	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 minimum 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 about 2% above optimum 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 based on proof rolling techniques.

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. Abandoned footings and/or underground concrete structure within the work limit should be removed and the excavation should be backfilled to grade.

4.13.2 Portland Cement Concrete Pavement

Based on soil classification and the results of the R-value tests, we assumed an R-value on the order of about 35 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 load. The project Civil Engineer should determine the traffic index to be used for different areas of the site.

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 6.

Table 6 – 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).

4.14 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.

4.15 Construction Observation and Testing

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

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

4.16 Limitations

Atlas should be advised of changes in the project scope so that the recommendations contained in this report can be evaluated with respect to the revised plans. Changes in recommendations will be verified in writing. The findings in this report are valid as of the date of this report. Changes in the condition of the site can occur with the passage of time, whether they are due to natural processes or work on this or adjacent areas. In addition, changes in the standards of practice and government regulations can occur. Thus, the findings in this report may be invalidated wholly or in part by changes beyond our control. This report should not be relied upon after a period of two years without a review by us verifying the suitability of the conclusions and recommendations to site conditions at that time.

In the performance of our professional services, we comply with that level of care and skill ordinarily exercised by members of our profession currently practicing under similar conditions and in the same locality. The client recognizes that subsurface conditions may vary from those encountered at the boring locations and that our data, interpretations, and recommendations are based solely on the information obtained by us. We will be responsible for those data, interpretations, and recommendations, but shall not be responsible for interpretations by others of the information developed. Our services consist of professional consultation and observation only, and no warranty of any kind whatsoever, express or implied, is made or intended in connection with the work performed or to be performed by us, or by our proposal for consulting or other services, or by our furnishing of oral or written reports or findings.

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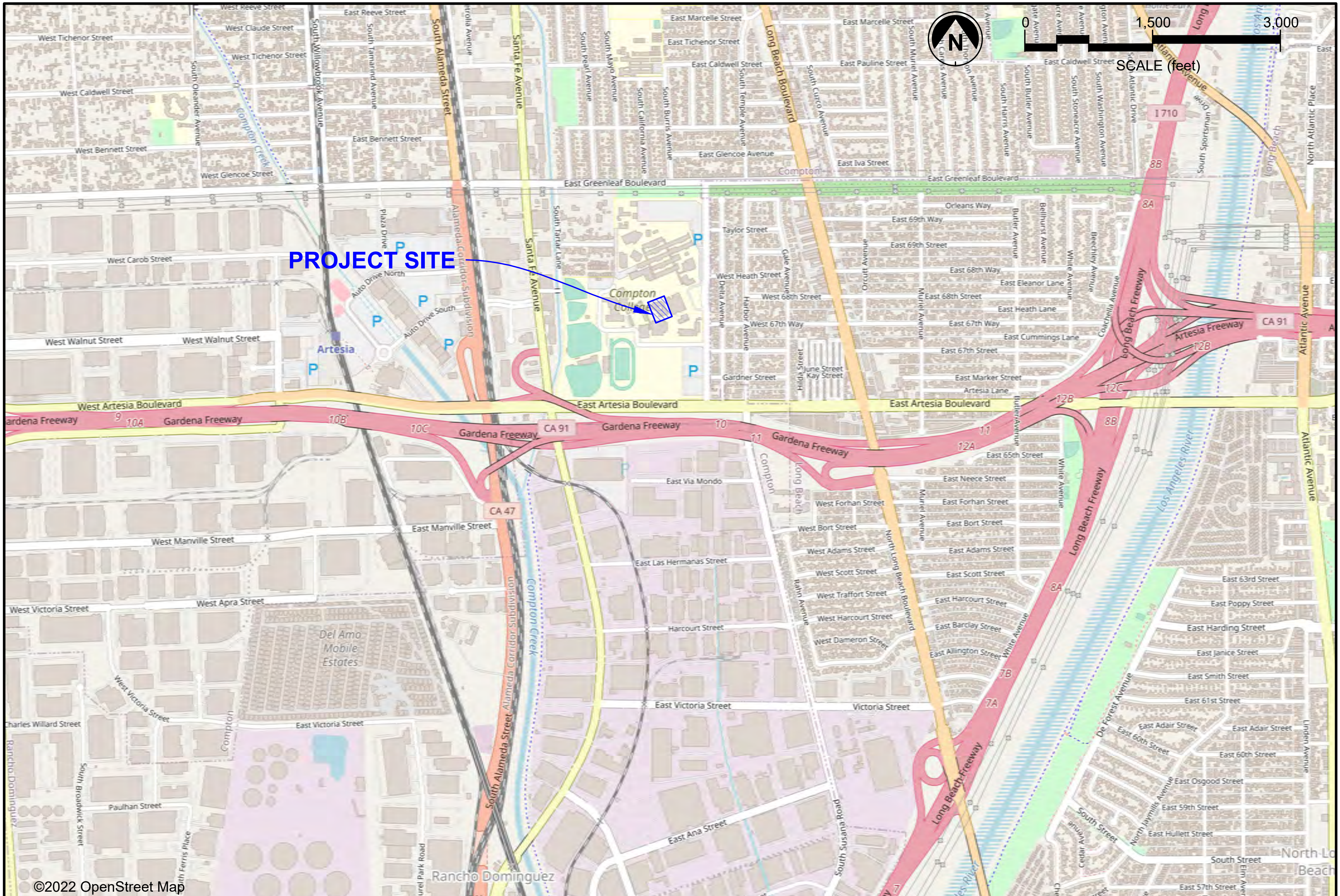
APPENDIX I
FIGURES



SITE VICINITY MAP

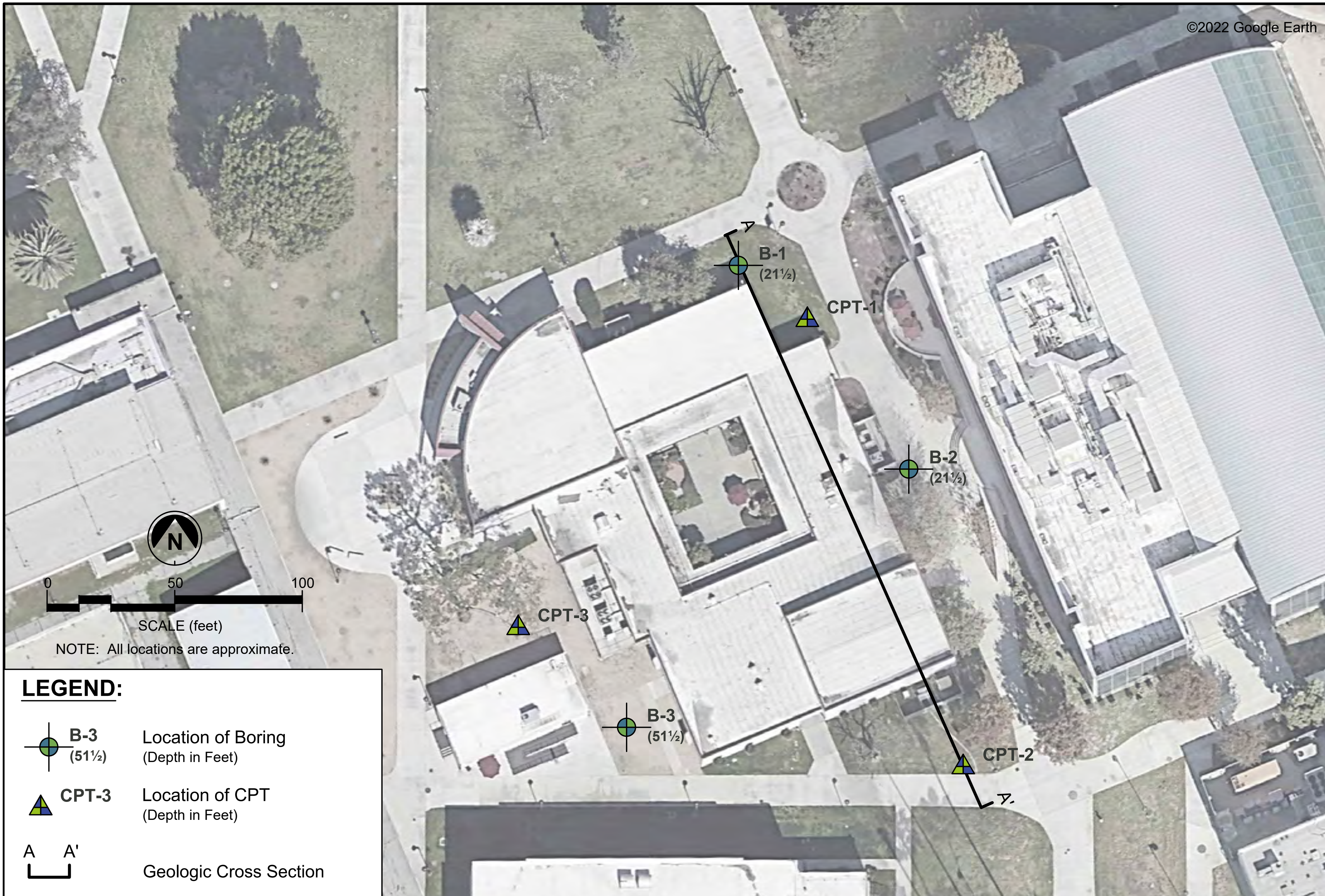
Compton Community College Visual and Performing Arts

Compton , California

Date:	March, 2022
By:	CGI
Job No.:	10-61187PW

Figure:

I-1



LEGEND:

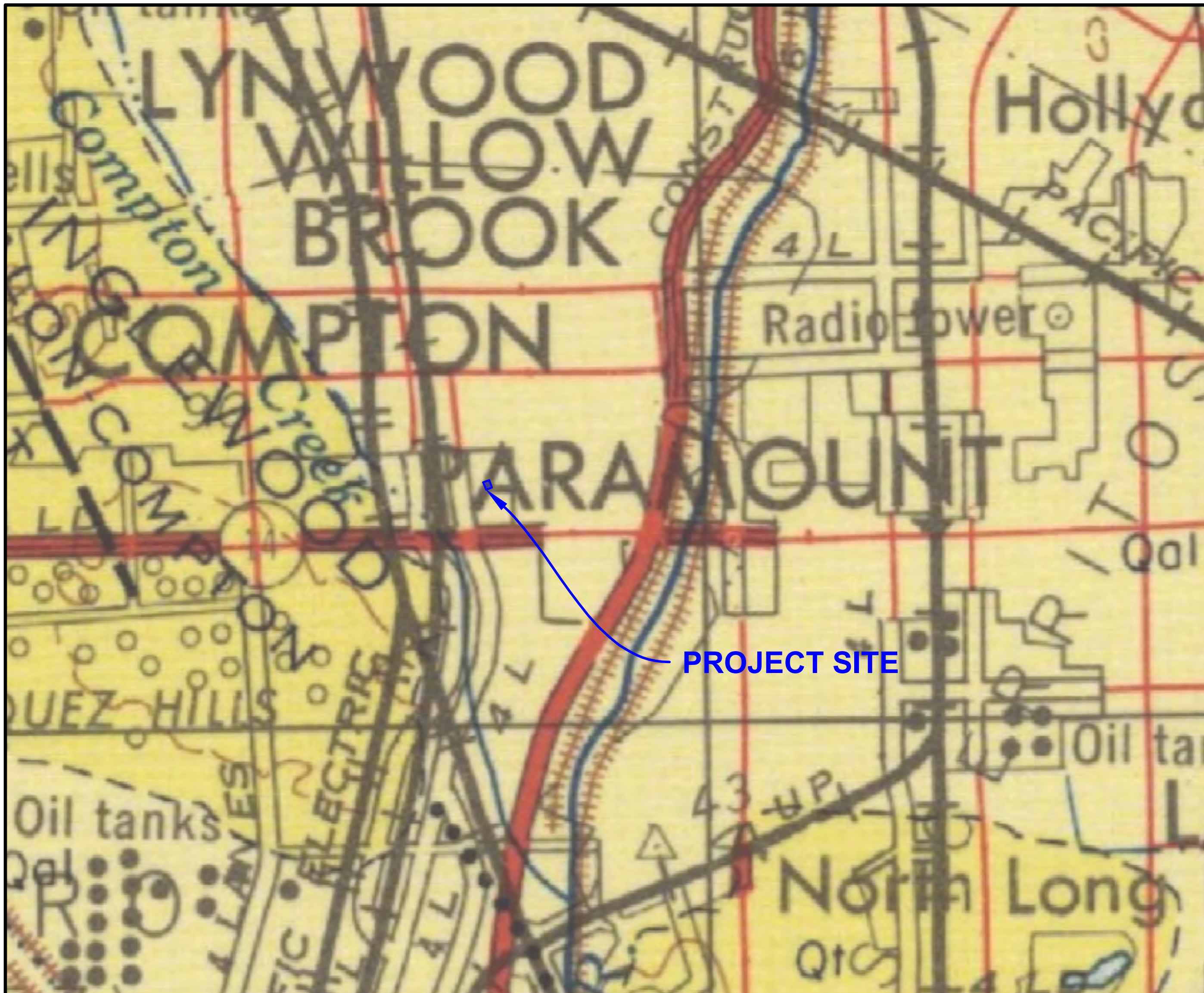
B-3 (51½)	Location of Boring (Depth in Feet)	
CPT-3	Location of CPT (Depth in Feet)	
A A'	Geologic Cross Section	

ATLAS

SUBSURFACE EXPLORATION MAP
Compton Community College Visual and Performing Arts
Compton , California

Date: March, 2022
By: CGI
Job No.: 10-61187PW

Figure:
I-2



EXPLANATION:

Qal	Alluvium
Qt	Quaternary nonmarine terrace deposits

N

0 1 2

SCALE (Miles)

NOTE: All locations are approximate.

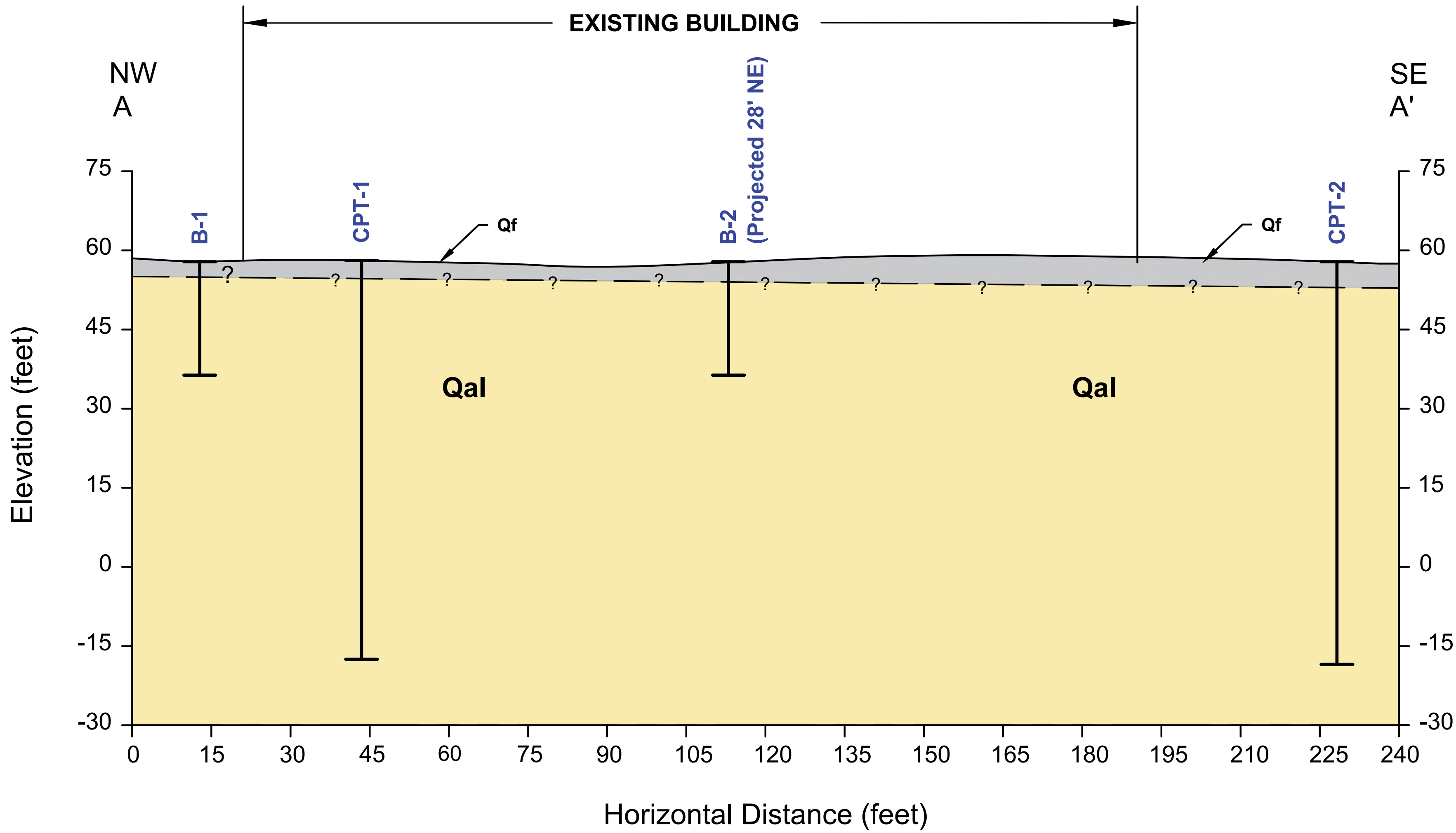
Reference:
Jennings, C.W., 1962, Geologic map of California : Long Beach sheet, California Division of Mines and Geology, series unknown, 1:250,000.

ATLAS

REGIONAL GEOLOGY MAP
Compton Community College Visual and Performing Arts
Compton , California

Date: March, 2022
By: CGI
Job No.: 10-61187PW

Figure:
I-3



LEGEND:

B-2 Location of Boring

—?— Geologic Contact, Queried Where Uncertain

Qf Fill

Qal Alluvium

0 30 60

SCALE (feet)

SCALE 1" = 30'

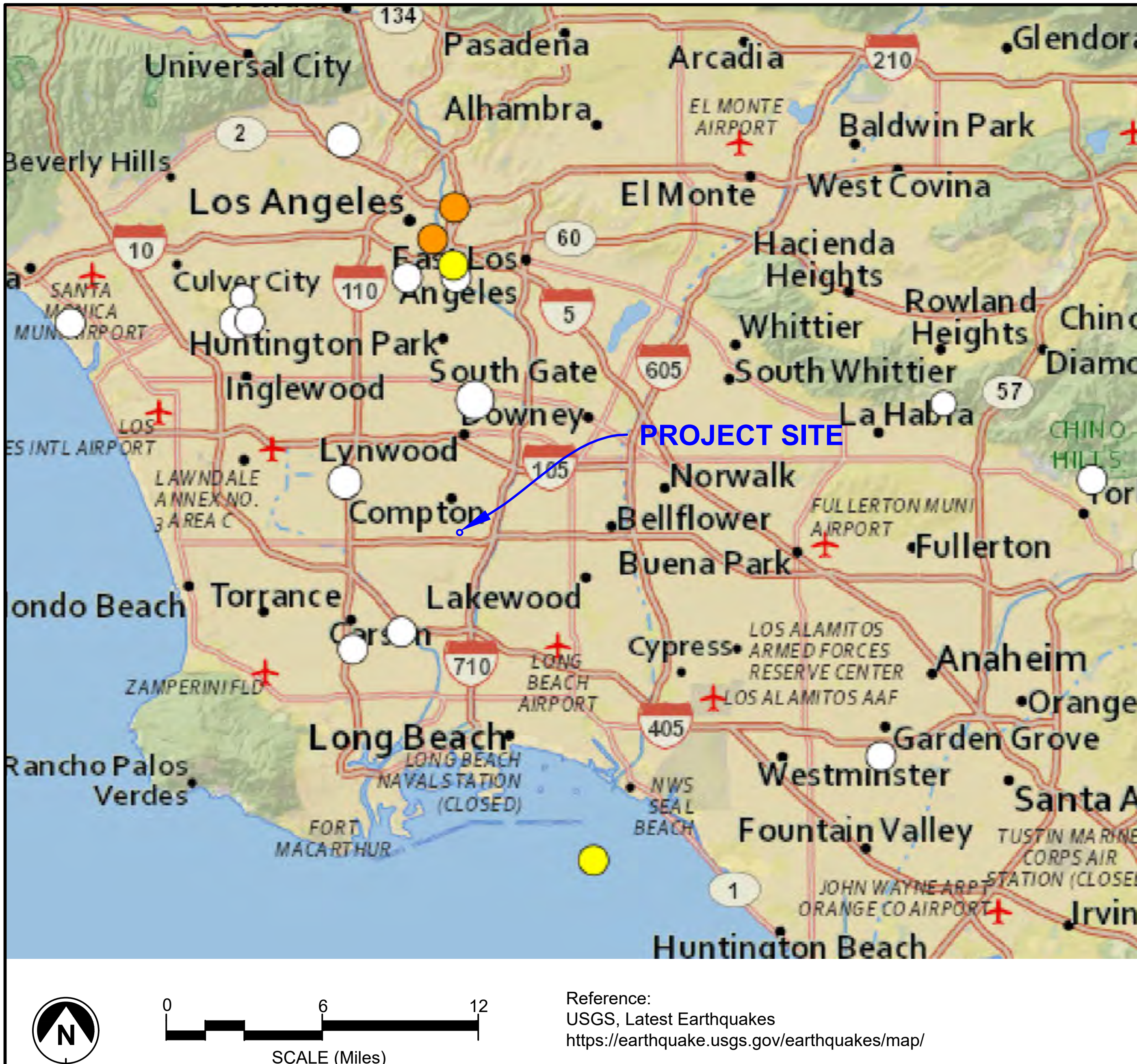
NOTE: All locations and depths are approximate.

ATLAS

GEOLOGIC CROSS-SECTION A-A'
Compton Community College Visual and Performing Arts
Compton , California

Date: March, 2022
By: CGI
Job No.: 10-61187PW

Figure:
I-4



EXPLANATION:

Magnitude

0 7+

Age (past)

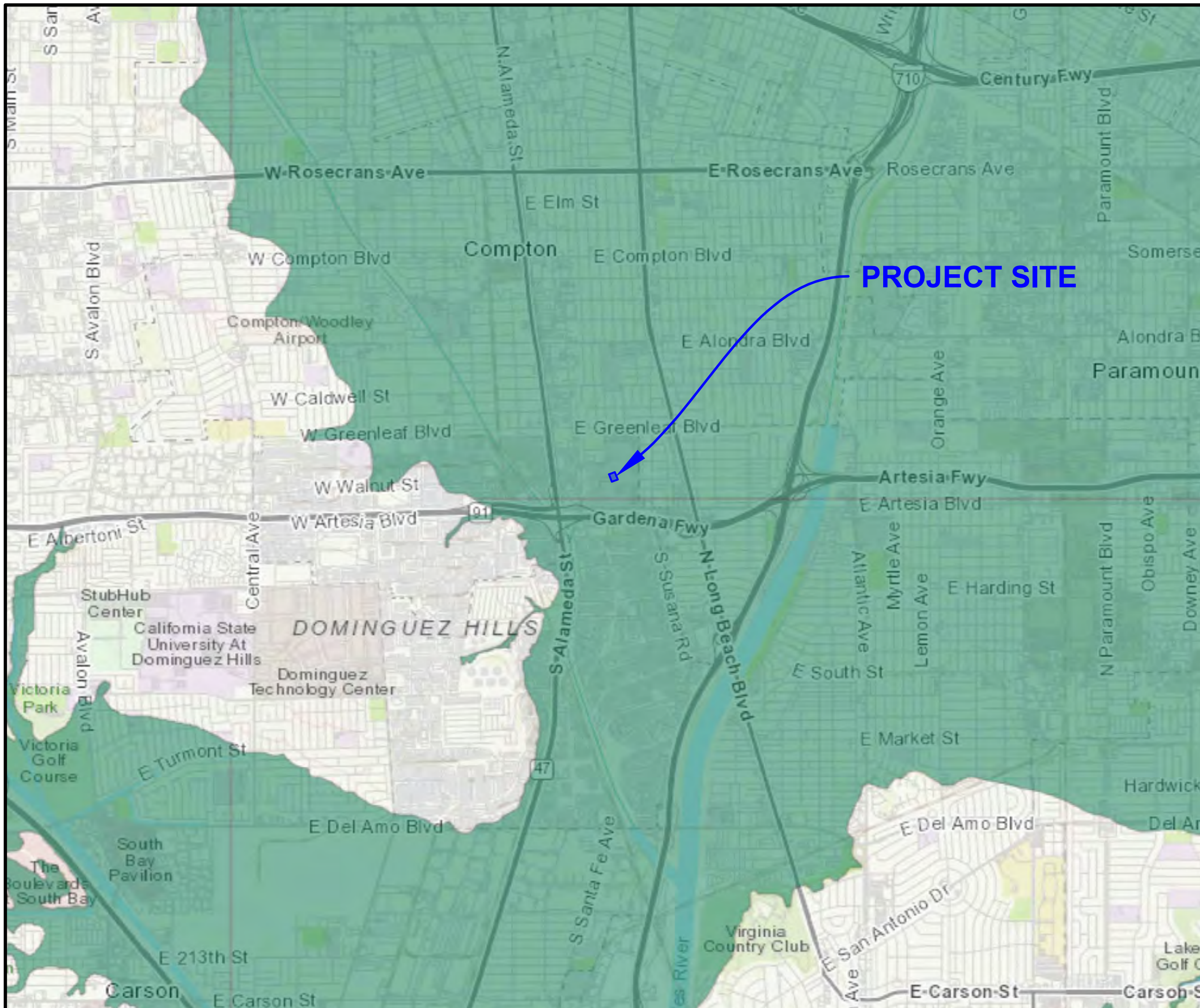
Hour Day Week Month Older

Seismic Event

Earthquake Non-Earthquake

Reference:
USGS, Latest Earthquakes
https://earthquake.usgs.gov/earthquakes/map/

ATLAS	**REGIONAL SEISMICITY MAP** Compton Community College Visual and Performing Arts Compton , California	Date: March, 2022 By: CGI Job No.: 10-61187PW	Figure: I-5



EXPLANATION:

Liquefaction Zone

N

0 1 2

SCALE (Miles)

Reference:
CGS Library, Earthquake Zones of Required Investigation,
https://maps.conservation.ca.gov/cgs/EQZApp/App; Accessed December, 2021

ATLAS

LIQUEFACTION SUSCEPTIBILITY MAP
Compton Community College Visual and Performing Arts
Compton , California

Date: March, 2022
By: CGI
Job No.: 10-61187PW

Figure: I-6

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

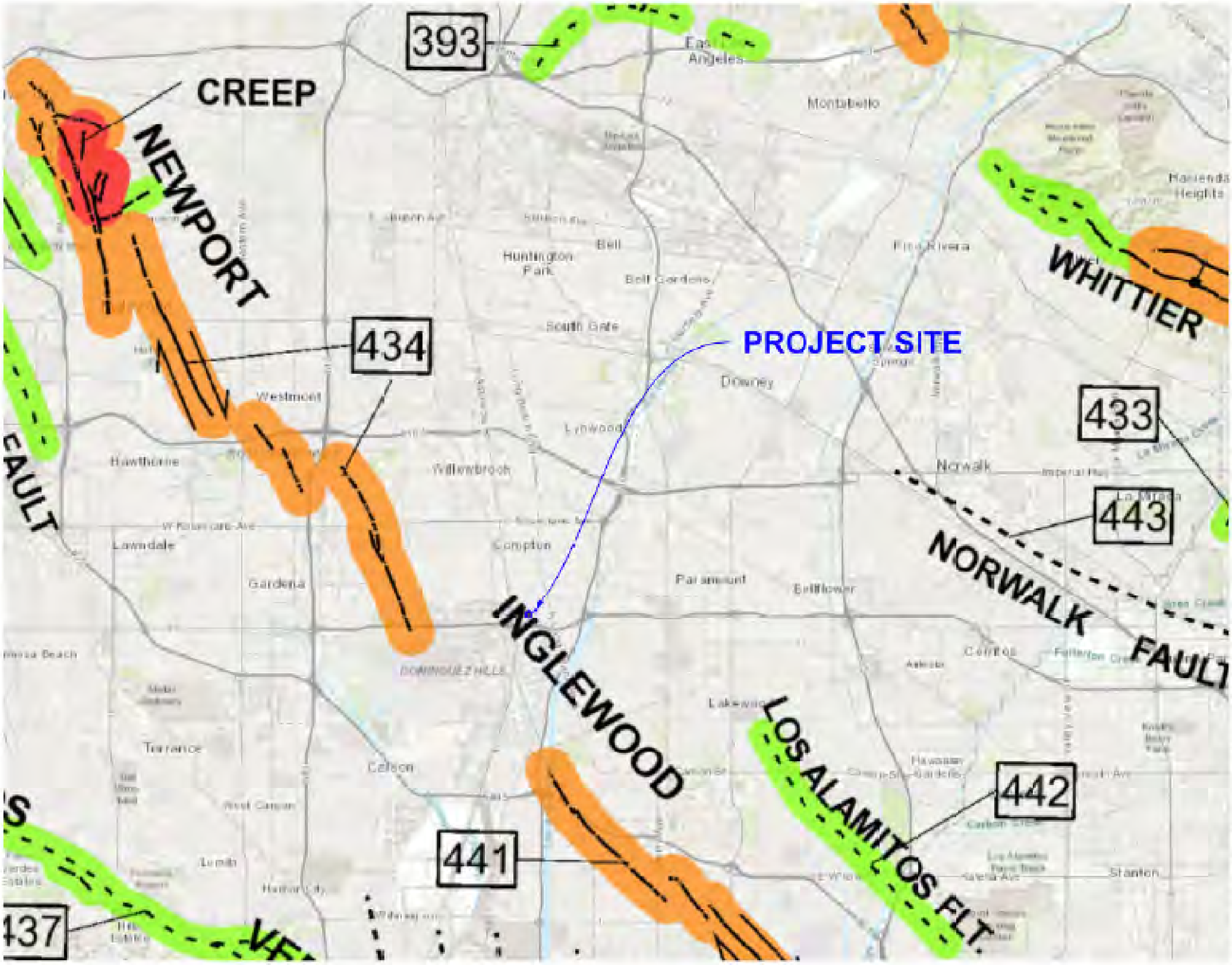
● Borehole Site

30 Depth to ground water in feet

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

ONE MILE

ATLAS	**Ground Water Contours** Compton Community College Visual and Performing Arts Compton, California	Date By: Job No:	March 2022 LM 10-61187PW	Figure : I-7



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
Compton Community College Visual and Performing Arts
Compton , California

Date: March ,2022
By: LM
Job No.: 10-61187PW

Figure:
I-8

APPENDIX II
FIELD EXPLORATION

The field investigation was performed on January 5, 2022 (B-1, B-2 and B-3) and January 13, 2022 (CPT-1, CPT-2 and CPT-3) 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 potentially conflict with boring locations. Geophysics test were performed on site to find the approximate location of the underground utilities.

Subsurface exploration included drilling and sampling of three (3) borings to depths ranging from about 20 feet to 50 feet and three (3) CPT borings to depth of about 75 feet below 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 - 85 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 - Visual and Perfoming Arts	10-61187PW	**B-1**

SITE	START	END	SHEET NO.
Compton, California	1/5/22	1/5/22	1 of 1

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

DRILLING EQUIPMENT	BORING DIA. (in.)	TOTAL DEPTH (ft)	GROUND ELEV. (ft)	DEPTH/ELEV. GROUND WATER (ft)
CME-85	8	21.5	58	∇ AT TIME OF DRILLING ---

SAMPLING METHOD	NOTES	
140-lb Hammer, 30-in Drop		▼ 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
									5.5 inches of concrete over no base	MAX
		X							FILL (Qf):**SILTY SAND (SM), loose, light brown, slightly moist, fine to medium grained**	
55										
									YOUNG ALLUVIAL FAN DEPOSITS (Qyf): SANDY SILT (ML) to SILTY SAND (SM), medium dense, light brown to tan, slightly moist, fine to medium grained	
	5		CAL	18		5.5	94.6			MD, DS
50									thin layer of Clay (CL)	
	10		SPT	12					SILTY SAND (SM) to SANDY SILT (ML), medium dense, light brown, slightly moist, fine to medium grained, micaceous	EI, COR
45									thin layer of clayey silt	
	15		CAL	27					SILTY SAND (ML), loose, light gray, slightly moist, fine tomedium garined.	CON, DS
40									thin layer of Clay (CL)	
	20		SPT	7		47.4			SANDY SILT (SM), medium dense, brown, slightly moist, fine grained, high plastic silt layers of Silt of high plasticity, elastic Silt (MH)	WA (89.7%) AL MOISTURE

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-1

LOG OF TEST BORING

ATLAS PROJECT NAME	ATLAS PROJECT NUMBER	
Compton College - Visual and Perfoming Arts	10-61187PW	**B-2**

SITE	START	END	SHEET NO.
Compton, California	1/5/22	1/5/22	1 of 1

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

DRILLING EQUIPMENT	BORING DIA. (in.)	TOTAL DEPTH (ft)	GROUND ELEV. (ft)
CME-85	8	21.5	58

SAMPLING METHOD	NOTES
140-lb Hammer, 30-in Drop	

DEPTH/ELEV. GROUND WATER (ft)
- AT TIME OF DRILLING ---
- 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								5" of Concrete over 5" of Base	
	1–5	X							FILL (Qf):**SILTY SAND (SM) to SANDY SILT (ML), loose, dark brown, slightly moist, fine to medium garined, micaceous**	MAX
55										
	5		SPT	8					**YOUNG ALLUVIAL FAN DEPOSITS (Qyf):**SANDY SILT (ML), loose, dark brown, slightly moist, fine tomedium grained, micaceous	EI
50										
									layer of silty clay	
	10		CAL	32					medium dense	CON WA (65.0%)
45										
	15		SPT	11					medium dense	
40										
	20		CAL	24		23.8	104.4		FAT CLAY (CH), very stiff, gray, moist, fine grained, trace of sand.	MD AL WA (94.7%)

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-2

LOG OF TEST BORING

ATLAS PROJECT NAME	ATLAS PROJECT NUMBER	
Compton College - Visual and Perfoming Arts	10-61187PW	B-3

SITE	START	END	SHEET NO.
Compton, California	1/5/22	1/5/22	1 of 3

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

DRILLING EQUIPMENT	BORING DIA. (in.)	TOTAL DEPTH (ft)	GROUND ELEV. (ft)	DEPTH/ELEV. GROUND WATER (ft)
CME-85	8	51.5	58	AT TIME OF DRILLING --- ; AT END OF DRILLING --- ; AFTER DRILLING ---

SAMPLING METHOD	NOTES
140-lb Hammer, 30-in Drop	

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								2 to 3" of Decompose Granite	
		X							FILL (Qf): **SILTY SAND (SM), loose, dark brown, moist, fine to medium grained**	R-VALUE
55										
	5		SPT	7					**YOUNG ALLUVIAL FAN DEPOSITS (Qyf):** :SANDY SILT (ML), loose, brown, moist, fine to medium grained	COR
50									thin layer of silty clay	
	10		CAL	24		20.3	119.5		medium dense	MD, DS
									silt lessen	
45										
	15		SPT	11					FINE SAND with trace of SILT (SP-SM), medium dense, light brown, slightly moist, fine to medium grained	
40									thin layer of silty clay	

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-3a

LOG OF TEST BORING

ATLAS PROJECT NAME	ATLAS PROJECT NUMBER	
Compton College - Visual and Perfoming Arts	10-61187PW	B-3

SITE	START	END	SHEET NO.
Compton, California	1/5/22	1/5/22	2 of 3

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

DRILLING EQUIPMENT	BORING DIA. (in.)	TOTAL DEPTH (ft)	GROUND ELEV. (ft)	DEPTH/ELEV. GROUND WATER (ft)
CME-85	8	51.5	58	AT TIME OF DRILLING ---
				AT END OF DRILLING ---
				AFTER DRILLING ---

SAMPLING METHOD	NOTES
140-lb Hammer, 30-in Drop	

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	28		37.9	80.9		CLAYEY SILT (ML), very stiff, dark brown, moist, fine grained, micaceous	AL, MD WA (95.3%)
35										
	25		SPT	12		21.5			SANDY SILT (ML), medium dense, grey, moist, fine grained, trace of clay, micaceous	AL, MOISTURE WA (65.4%)
									thin layer of silty clay	
30										
	30		CAL	16		18.6	110.5		SILTY CLAY (CL) / CLAYEY SILT (ML), stiff, gray, moist, fine grained, some fine sand	AL, MD WA (87.3%)
25										
									thin layer of sandy silt	
	35		SPT	20					SANDY SILT (ML), medium dense, gray, moist, fine grained,	AL, MD WA (72.6%)
20										
									with layers of Silty SAND (SM)	

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-3b

LOG OF TEST BORING

ATLAS PROJECT NAME	ATLAS PROJECT NUMBER	
Compton College - Visual and Perfoming Arts	10-61187PW	B-3

SITE	START	END	SHEET NO.
Compton, California	1/5/22	1/5/22	3 of 3

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

DRILLING EQUIPMENT	BORING DIA. (in.)	TOTAL DEPTH (ft)	GROUND ELEV. (ft)	DEPTH/ELEV. GROUND WATER (ft)
CME-85	8	51.5	58	AT TIME OF DRILLING ---

SAMPLING METHOD	NOTES	
140-lb Hammer, 30-in Drop		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
									SANDY SILT (ML), medium dense, gray, moist, fine grained, *(continued)*	
			CAL	51		12.7	112.4		dense, some clay	AL, MD WA (45.2%)
									with layers of Silty SAND (SM)	
									thin layer of silty clay	
15										
									thin layer of Sand (SP)	
	45								dense, trace of clay	
			SPT	31		16.5				MOISTURE
10										
									thin layer of hard silty clay	
	50								dense	
			CAL	50		31.3	91.9			AL, MD WA (89.4%)
									BORING TERMINATED AT 51.5 FEET	
5										
	55									
0										

ATLAS

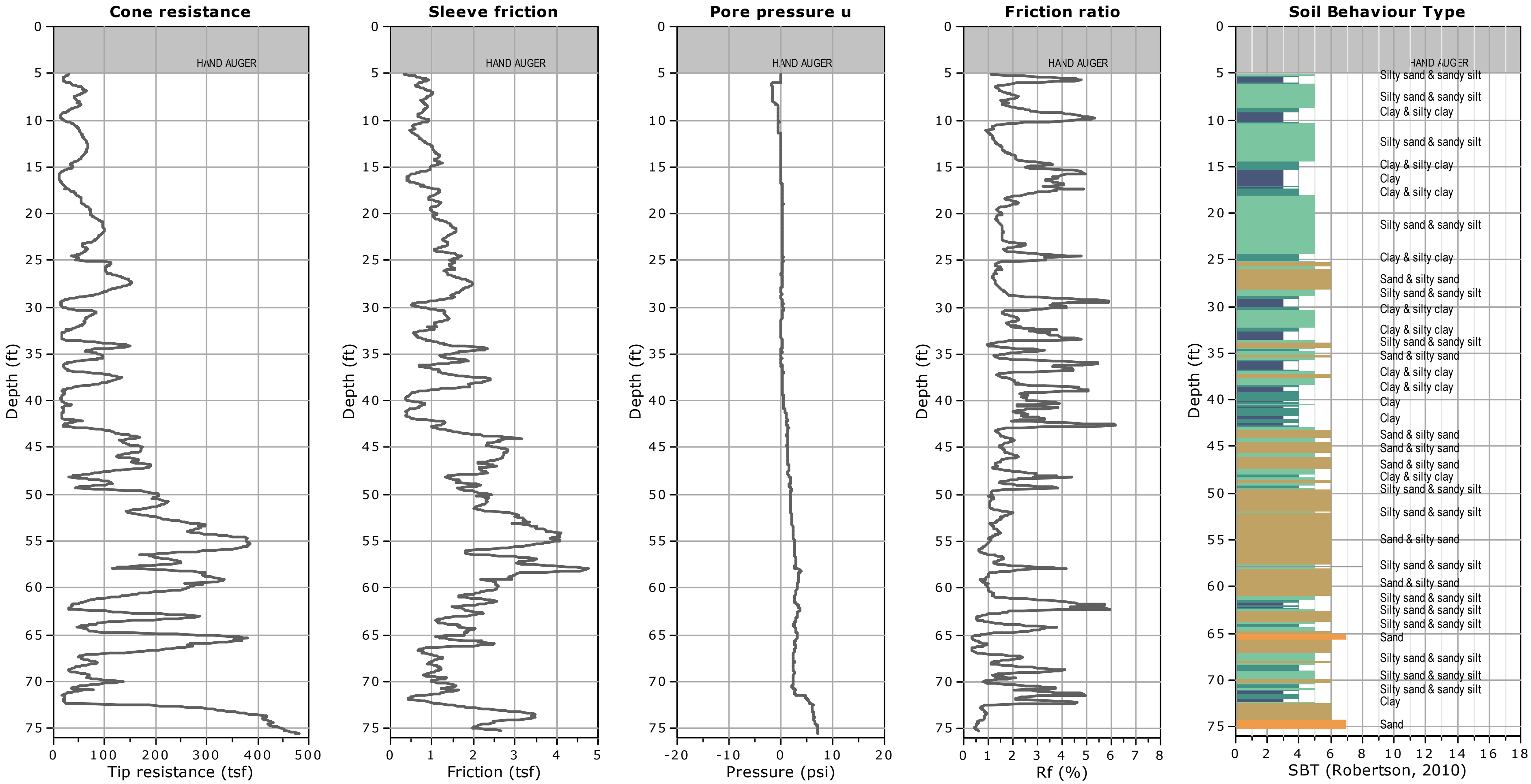
THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

Figure II-3c

Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: ATLAS / Compton Community District College **CPT-1**

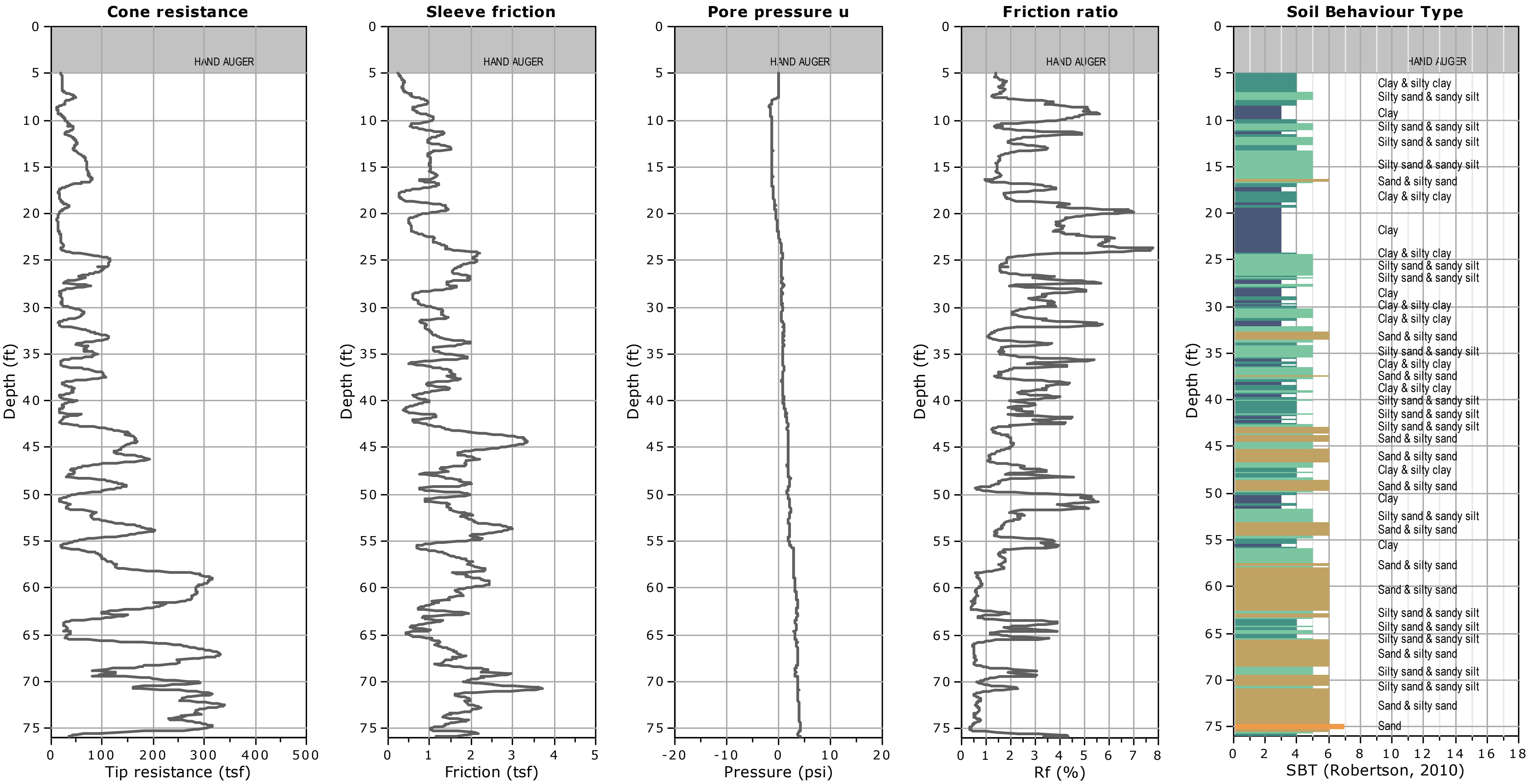
Location: Compton, CA Total depth: 75.61 ft, Date: 1/13/2022



Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: **ATLAS / Compton Community District College**
Location: **Compton, CA**

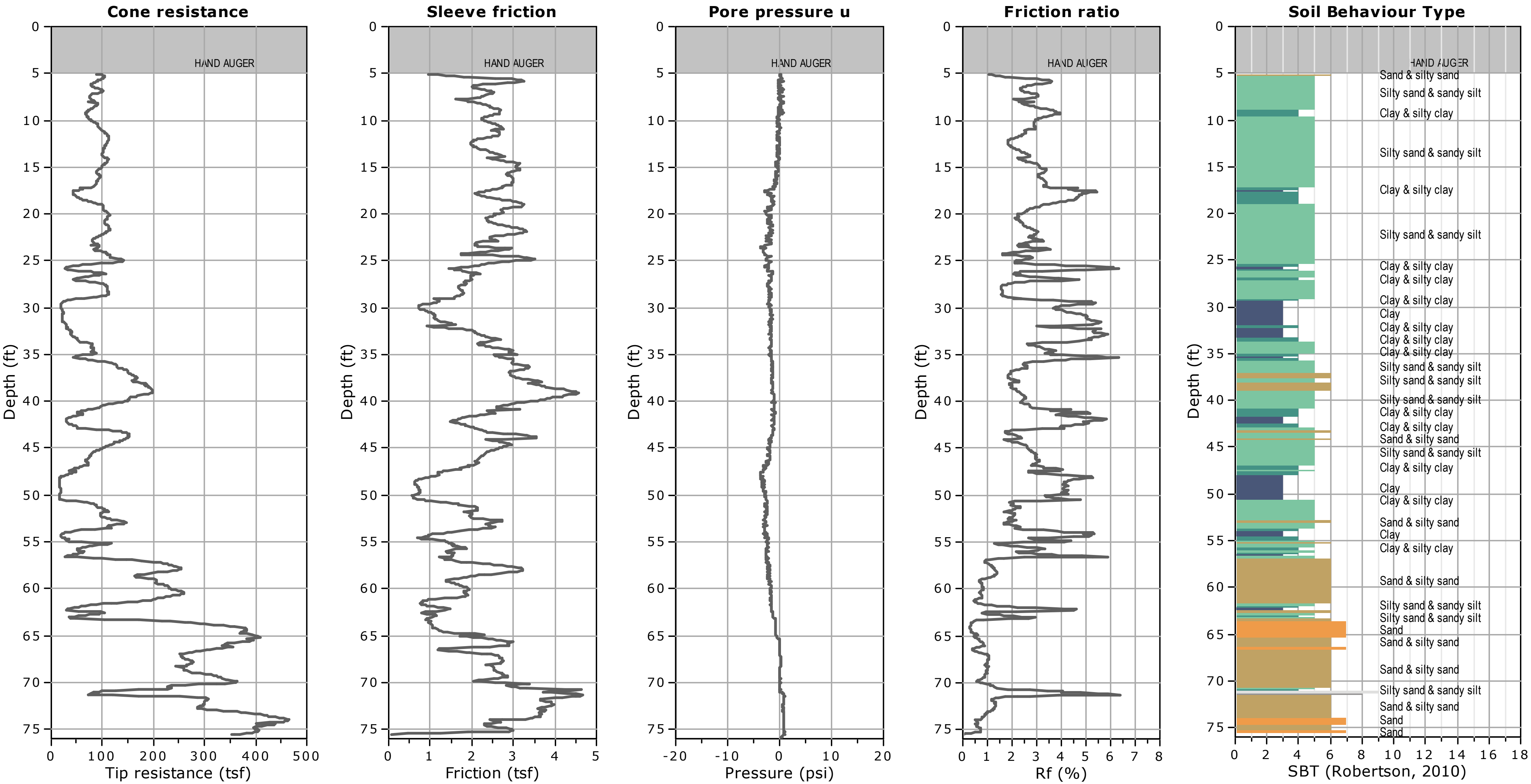
CPT-2
Total depth: 76.33 ft, Date: 1/13/2022



Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: ATLAS / Compton Community District College
Location: Compton, CA

CPT-3
Total depth: 75.89 ft, Date: 1/13/2022



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 below.

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.

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.

HYDROMETER TESTS: Hydrometer tests have been performed on the obtained samples based on ASTM D422. Test results are presented in this appendix.

Moisture-Density

ASTM D2937

SAMPLE LOCATION	Moisture Content (%)	Dry Density (pcf)
B-1 at 5 Feet	5.5	94.6
B-2 at 20 Feet	23.8	104.4
B-3 at 10 Feet	20.3	119.5
B-3 at 20 Feet	37.9	80.9
B-3 at 30 Feet	18.6	110.5
B-3 at 40 Feet	12.7	112.4
B-3 at 50 Feet	31.3	91.9

Atterberg Limits

ASTM D4318

SAMPLE LOCATION	LIQUID LIMIT	PLASTIC LIMIT	PLASTIC INDEX
B-1 at 20 Feet	53	33	20
B-2 at 20 Feet	57	28	29
B-3 at 20 Feet	NP	NP	NP
B-3 at 25 Feet	35	26	9
B-3 at 30 Feet	40	25	15
B-3 at 35 Feet	NP	NP	NP
B-3 at 40 Feet	NP	NP	NP
B-3 at 50 Feet	NP	NP	NP

Percent Finer than No. 200 Sieve

ASTM D1140

SAMPLE LOCATION	FINES CONTENT (%)
B-1 at 20 Feet	89.7
B-2 at 10 Feet	65.0
B-2 at 20 Feet	94.7
B-3 at 20 Feet	95.3
B-3 at 25 Feet	65.4
B-3 at 30 Feet	87.3
B-3 at 35 Feet	72.6
B-3 at 40 Feet	45.2
B-3 at 50 Feet	89.4

Modified Proctor
ASTM D1557

SAMPLE LOCATION	Optimum Content (%)	Maximum Dry Density (pcf)
B-2 at 0-5 Feet	12.3	122.3

R-VALUE
ASTM D2844

SAMPLE LOCATION	R-Value
B-1 at 5 Feet	64
B-3 at 5 Feet	67

EXPANSION INDEX
ASTM D4829

SAMPLE LOCATION	DESCRIPTION	EXPANSION INDEX
B-1 at 10 Feet	Silty SAND (SM)	0
B-2 at 5 Feet	Sandy SILT (ML)	4

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

Compton Community College – Visual and Performing Arts Compton, California			
By:	LM	Date:	March, 2022
Job Number:	10-61187PW	Figure:	III-2

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- 1 at 10 Feet	2360	8.19	0.006	0.005
B-3 at 5 Feet	506	8.02	0.025	0.066

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
$0.10 \le SO_4 < 0.20$	Moderate	S1	II	0.50	4,000
$0.20 \le SO_4 \le 2.00$	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

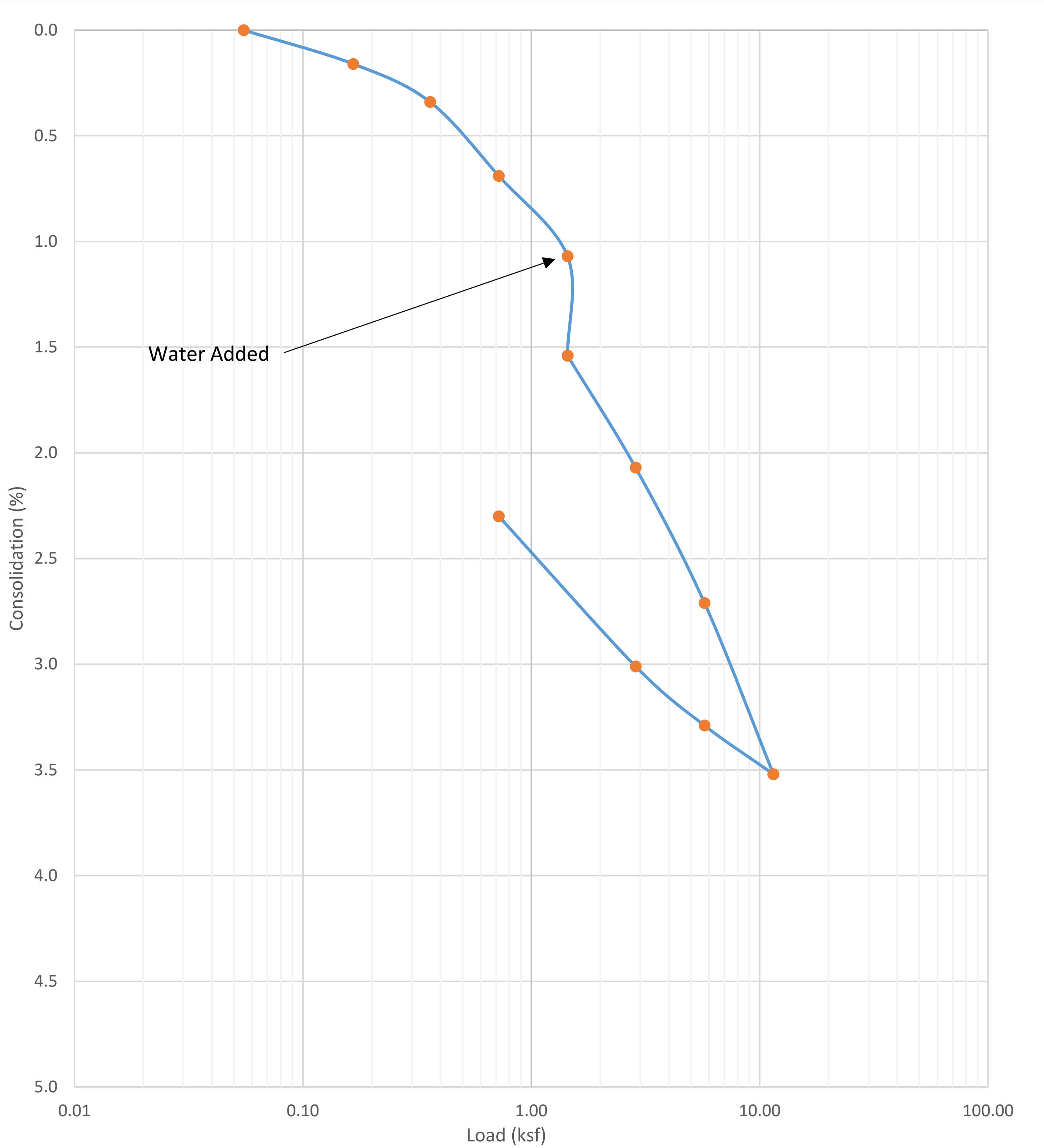
Corrosivity Ratings Based on Soil Resistivity[3]

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-1 at 15 Feet

Sample Description: **Grayish Brown Silty SAND**

γ_d 100.4 pcf

Pre-consolidation w_c 10.8 %

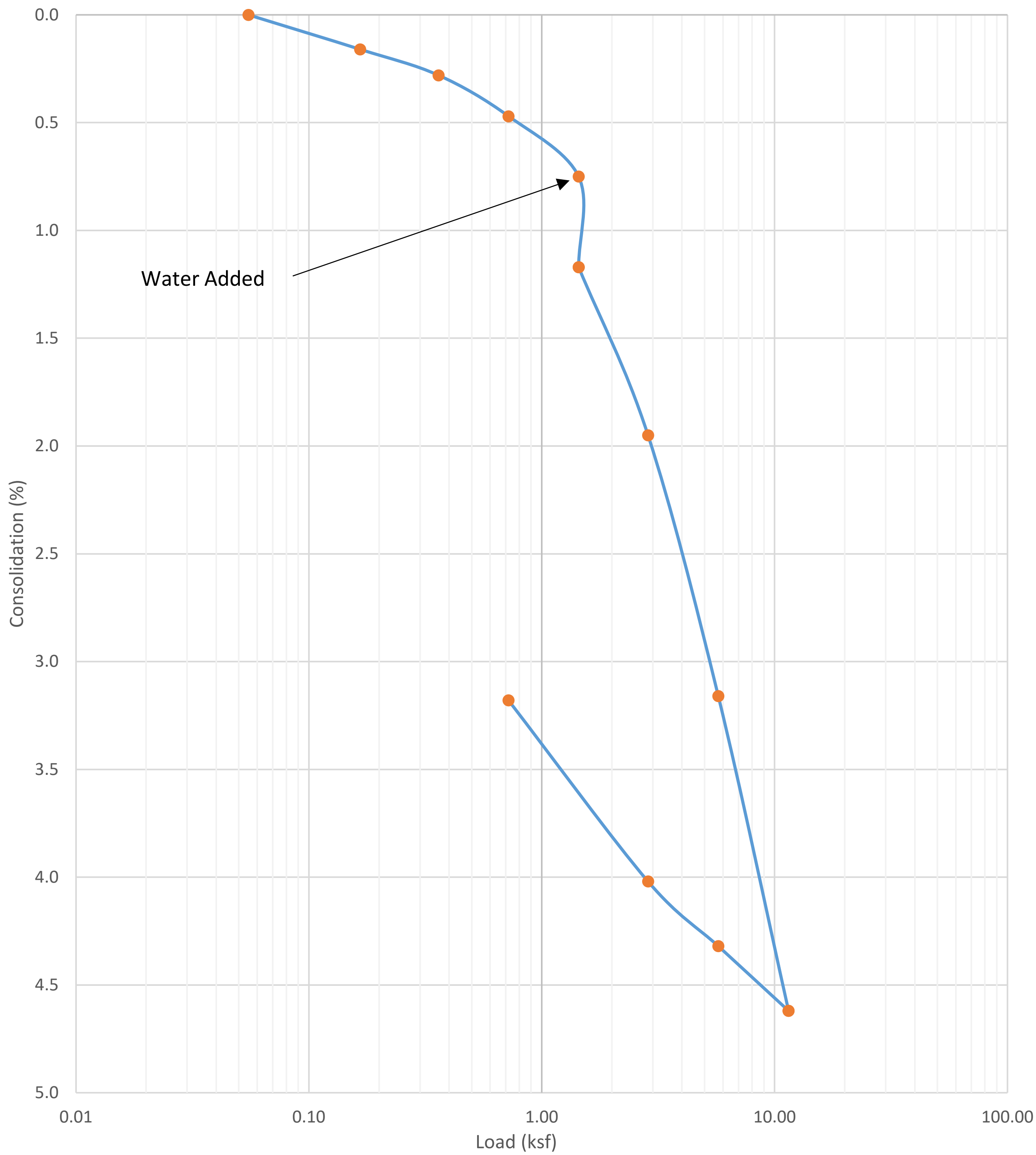
Post-consolidation w_c 27.2 %

ATLAS

Compton Community College - Visual and Perfroming Arts Compton , California			
By:	DRB	Date:	March, 2022
Job No:	10-61187PW	Figure:	III-4

Consolidation Test Results

ASTM D2435

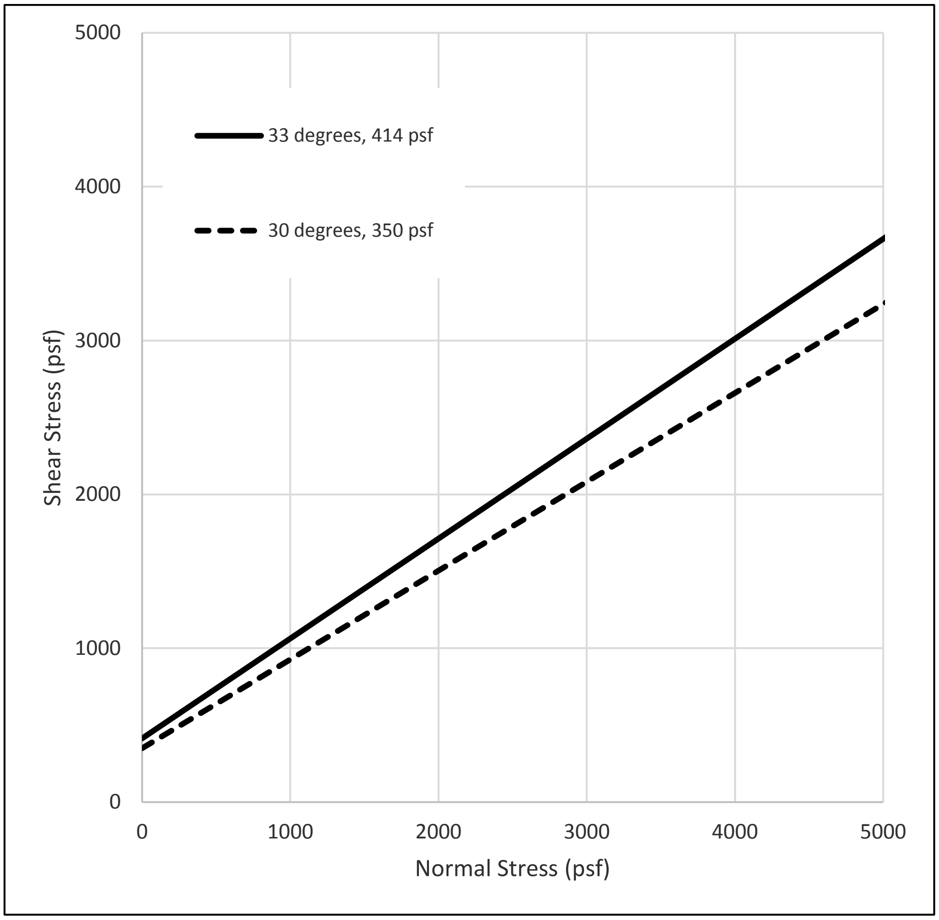


Sample ID: B-2 at 10 Feet

Sample Description: **Brown Silty SAND**

γ_d	102.9 pcf
Pre-consolidation w_c	13.3 %
Post-consolidation w_c	21.2 %

ATLAS	Compton Community College - Visual and Perfroming Arts Compton , California			
	By:	DRB	Date:	March, 2022
	Job No:	10-61187PW	Figure:	III-5



SAMPLE ID: B-1 at 5 Feet

Light Brown Fine Silty Sand

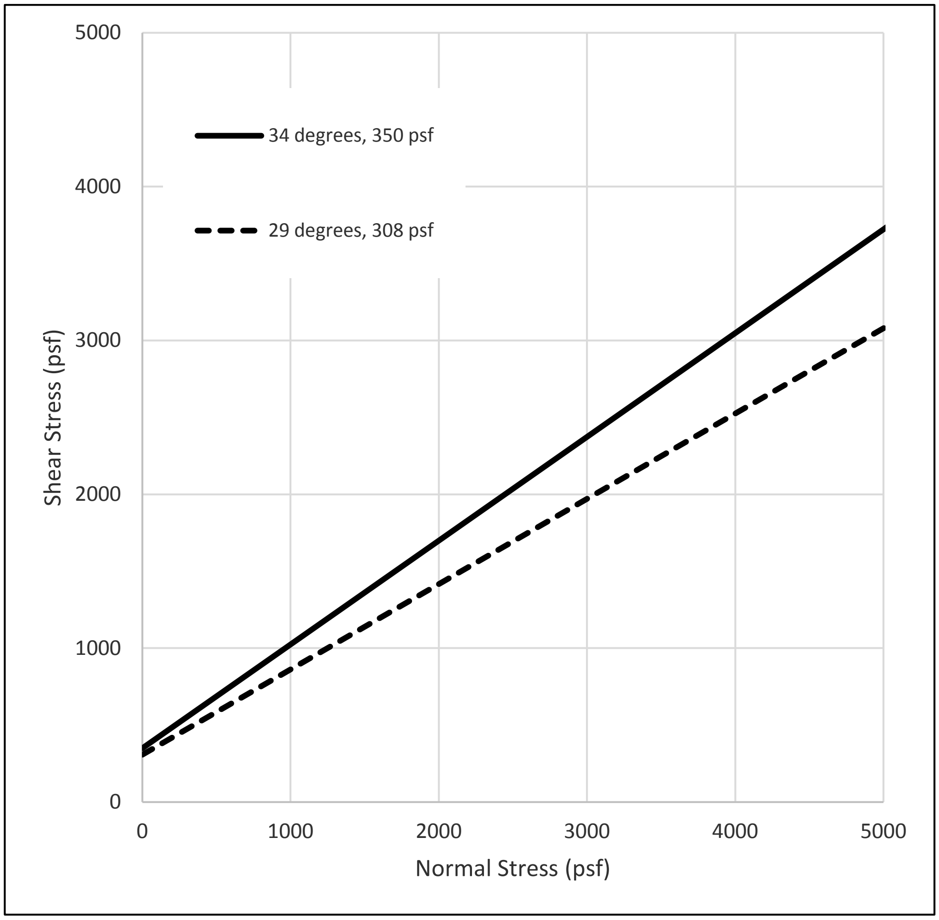
NOTES: Insitu
Strain Rate: 0.003 in/min
Sample was consolidated and drained

	Peak	Ultimate
Φ	33 °	30 °
c	414 psf	350 psf
	Initial	**Final**
γ_d	117.0 pcf	117.0 pcf
w_c	7.6 %	14.8 %
Saturation	47 %	92 %

ATLAS

Compton Community College - Visual and Performing Arts

By:	DRB	Date:	March, 2022
Job Number:	10-61187PW	Figure:	III-6



SAMPLE ID: B-1 at 15 Feet

Grey Fine Silty Sand

NOTES: Insitu

Strain Rate: 0.003 in/min

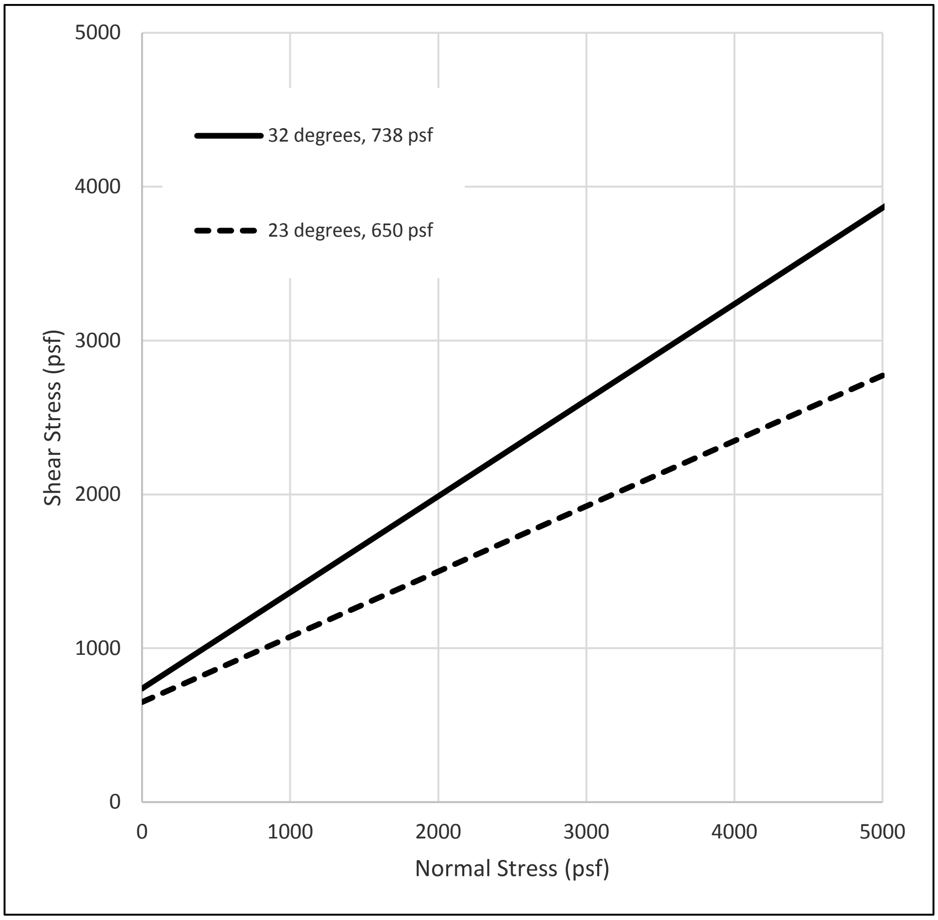
Sample was consolidated and drained

	Peak	Ultimate
Φ	34 °	29 °
c	350 psf	308 psf
	Initial	**Final**
γ_d	92.0 pcf	92.0 pcf
w_c	7.6 %	29.8 %
Saturation	25 %	98 %

ATLAS

Compton Community College - Visual and Performing Arts
Compton, California

By:	DRB	Date:	March, 2022
Job Number:	10-61187PW	Figure:	III-7



SAMPLE ID: B-3 at 10 Feet

Grey Brown Fine Silty Sand

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

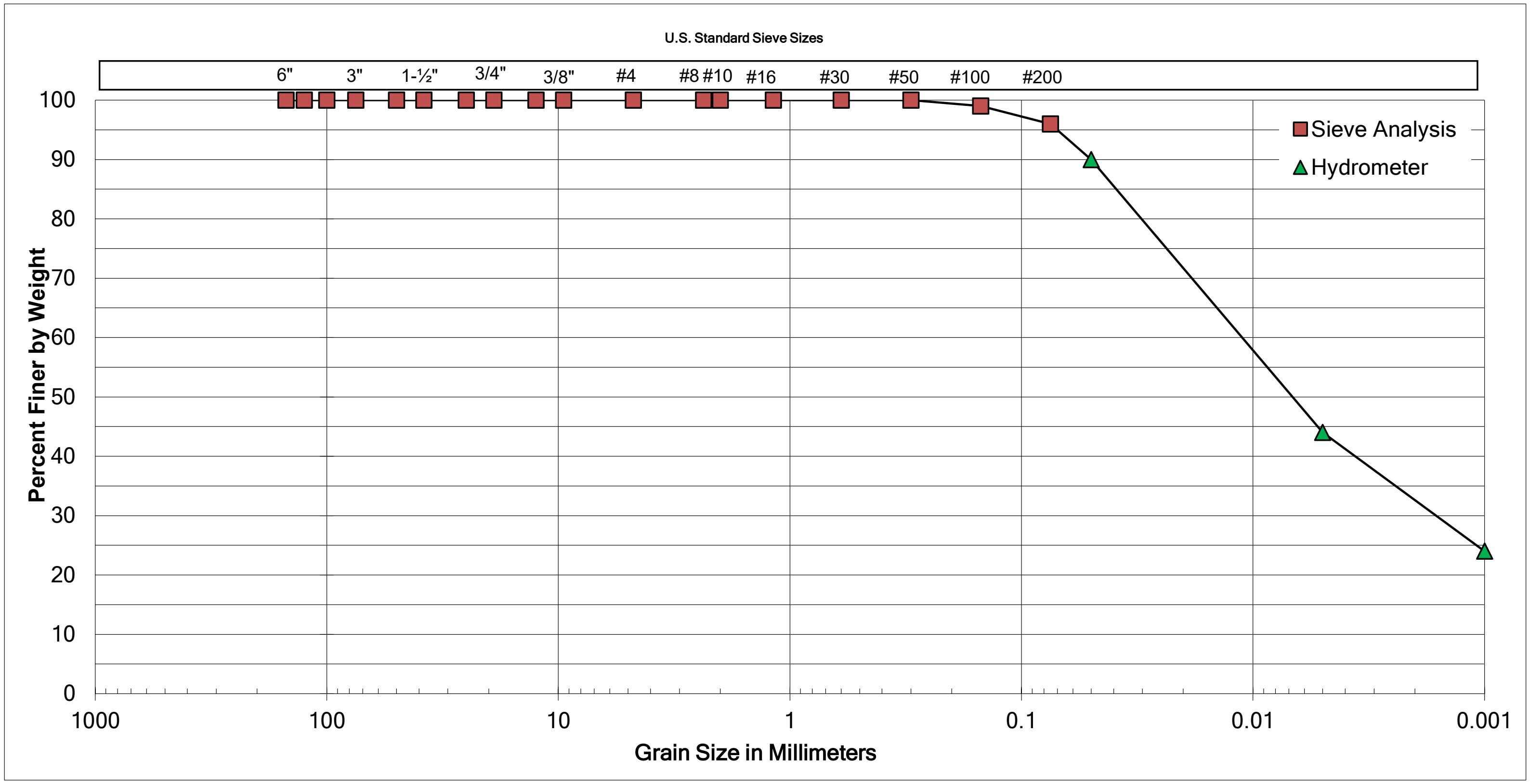
	Peak	Ultimate
Φ	32 °	23 °
c	738 psf	650 psf

	Initial	Final
γ_d	110.7 pcf	110.7 pcf
w_c	16.4 %	18.0 %
Saturation	86 %	94 %

ATLAS

Compton Community College - Visual and Performing Arts
Compton, California

By:	DRB	Date:	March, 2022
Job Number:	10-61187PW	Figure:	III-8



Cobbles	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

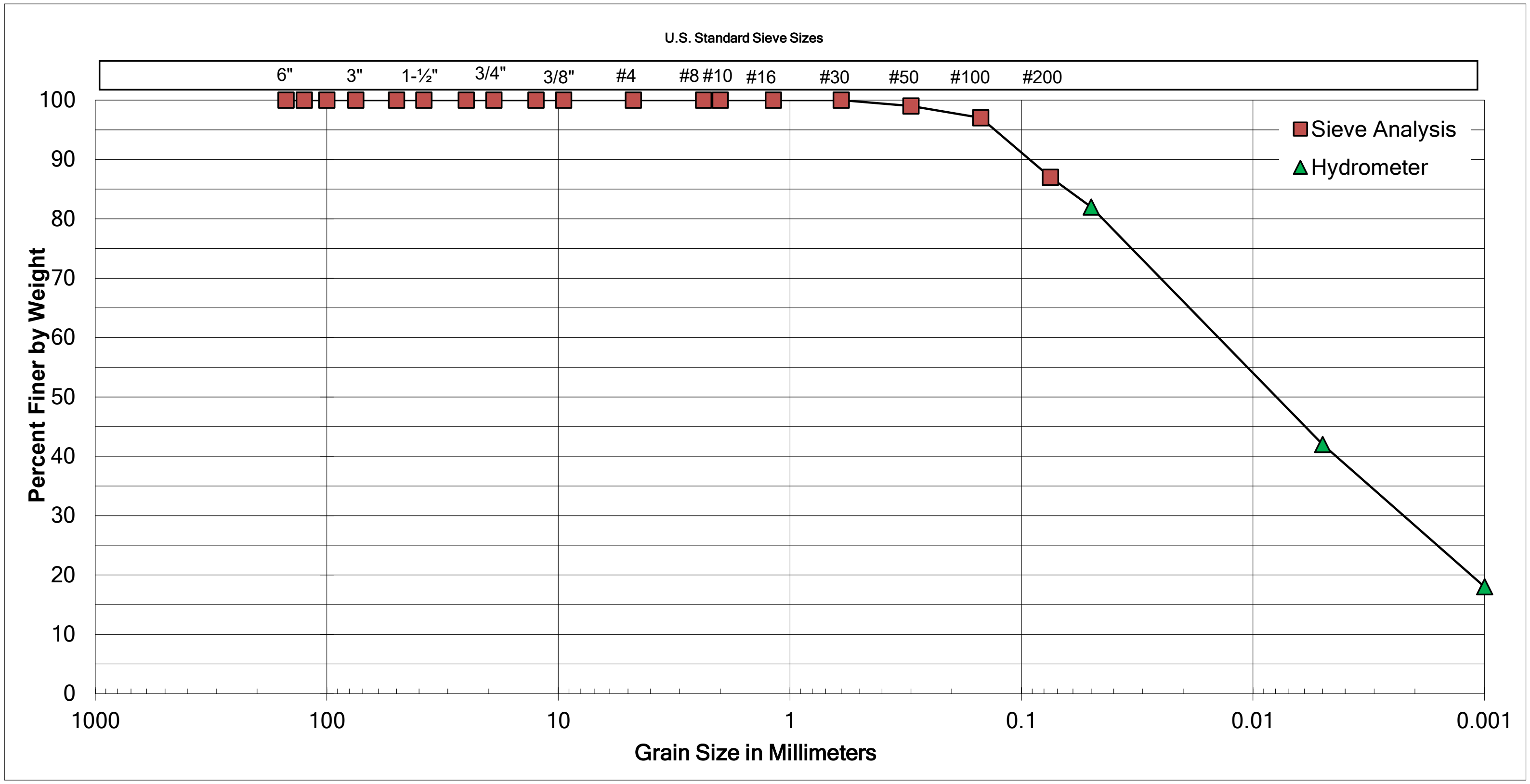
SAMPLE LOCATION
B-3 at 20 Feet
SAMPLE NUMBER
DRB

UNIFIED SOIL CLASSIFICATION:	ML
DESCRIPTION	CLAYEY SILT

ATTERBERG LIMITS	
LIQUID LIMIT	NP
PLASTIC LIMIT	NP
PLASTICITY INDEX	NP

ATLAS

Compton Community College - Visual and Perfoming Arts Compton, California			
By:	DRB	Date:	March, 2022
Job Number:	10-61187PW	Figure:	III-9



Cobbles	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

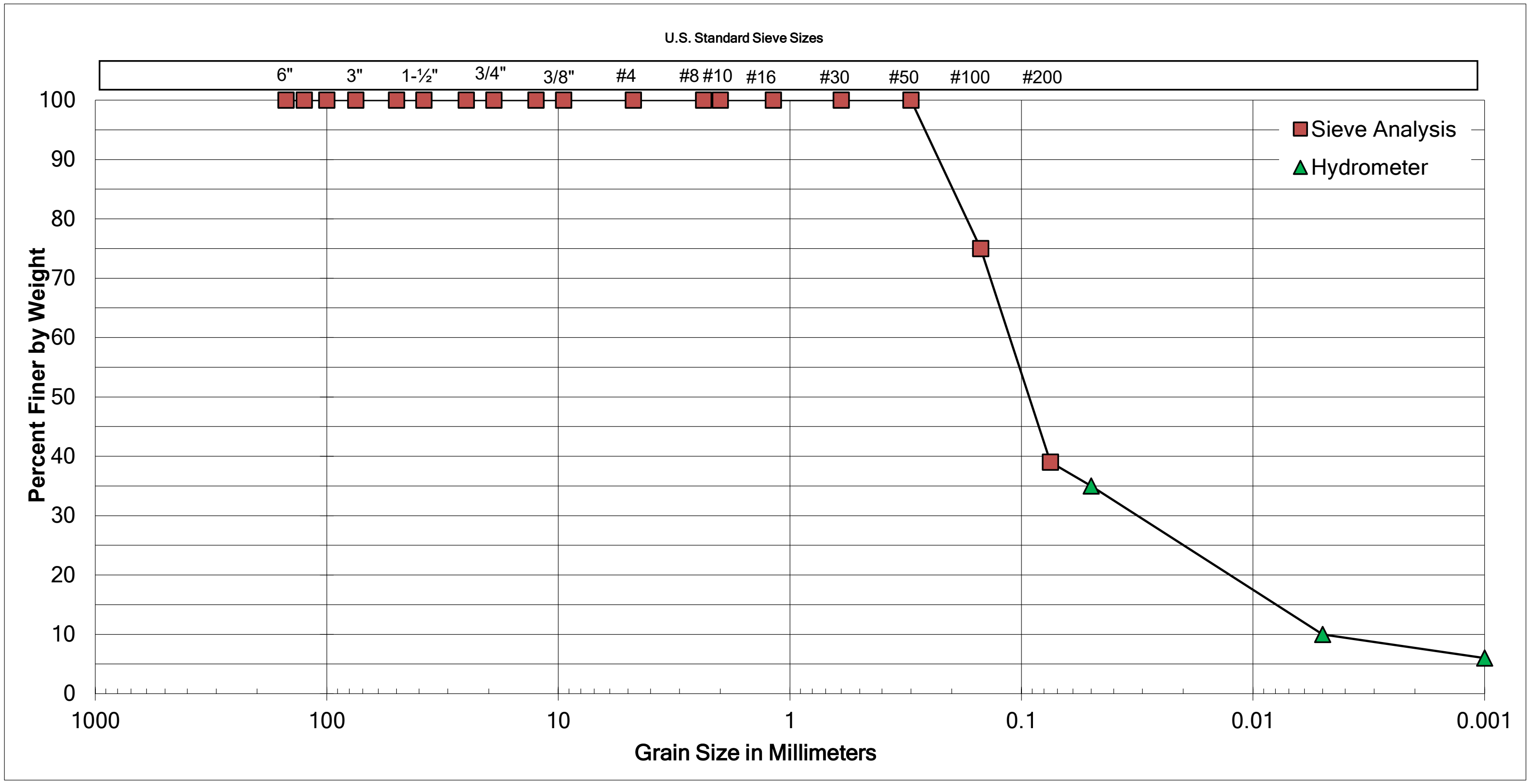
SAMPLE LOCATION
B-3 at 25 Feet
SAMPLE NUMBER
DRB

UNIFIED SOIL CLASSIFICATION:	ML
DESCRIPTION	SANDY SILT

ATTERBERG LIMITS	
LIQUID LIMIT	35
PLASTIC LIMIT	26
PLASTICITY INDEX	9

ATLAS

Compton Community College - Visual and Perfoming Arts Compton, California			
By:	DRB	Date:	March, 2022
Job Number:	10-61187PW	Figure:	III-10



Cobbles	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

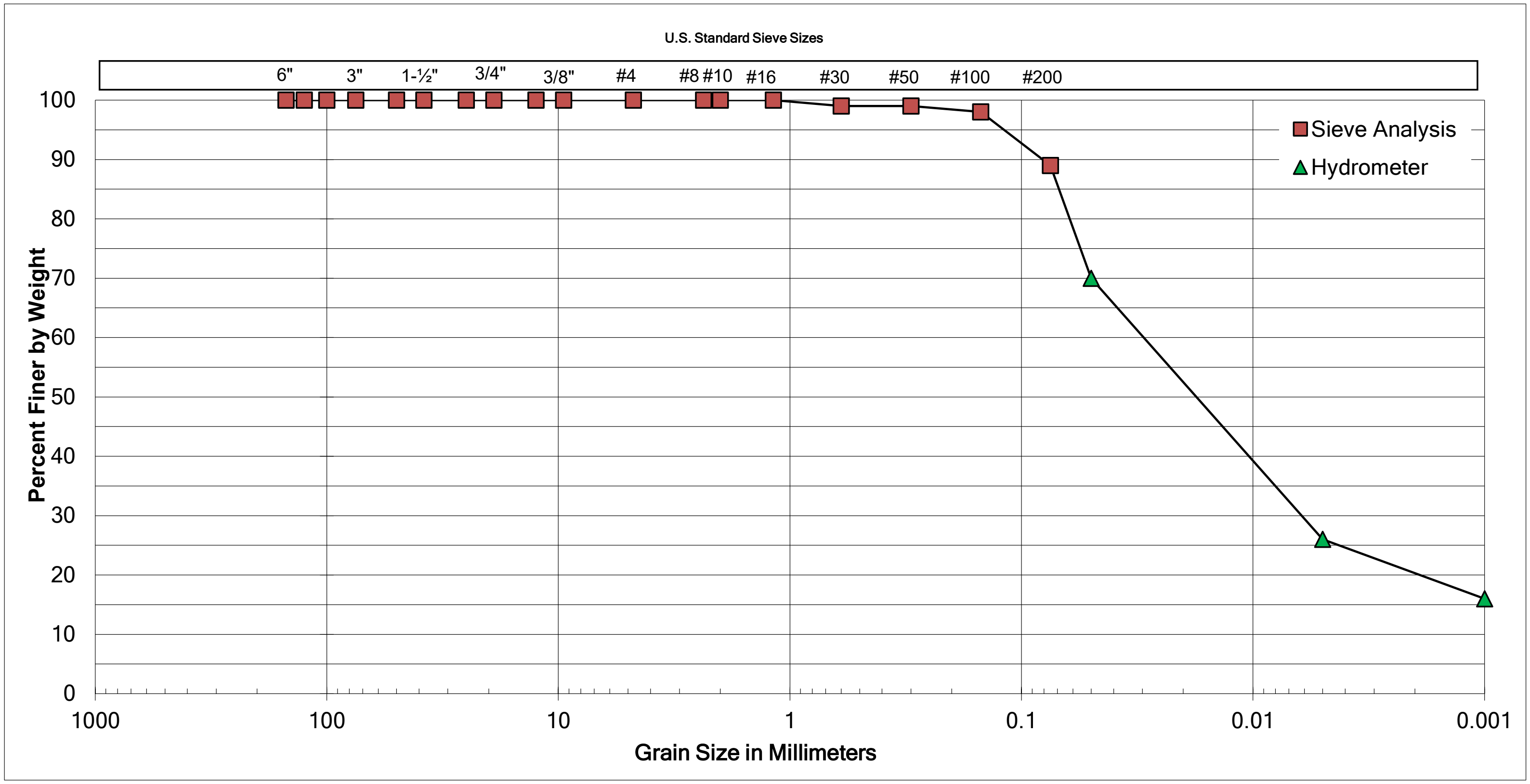
SAMPLE LOCATION
B-3 at 30 Feet
SAMPLE NUMBER
DRB

UNIFIED SOIL CLASSIFICATION:	CL/ML
DESCRIPTION	SILTY CLAY / CLAYE SILT

ATTERBERG LIMITS	
LIQUID LIMIT	40
PLASTIC LIMIT	25
PLASTICITY INDEX	15

ATLAS

Compton Community College - Visual and Perfoming Arts Compton, California			
By:	DRB	Date:	March, 2022
Job Number:	10-61187PW	Figure:	III-11



Cobbles	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

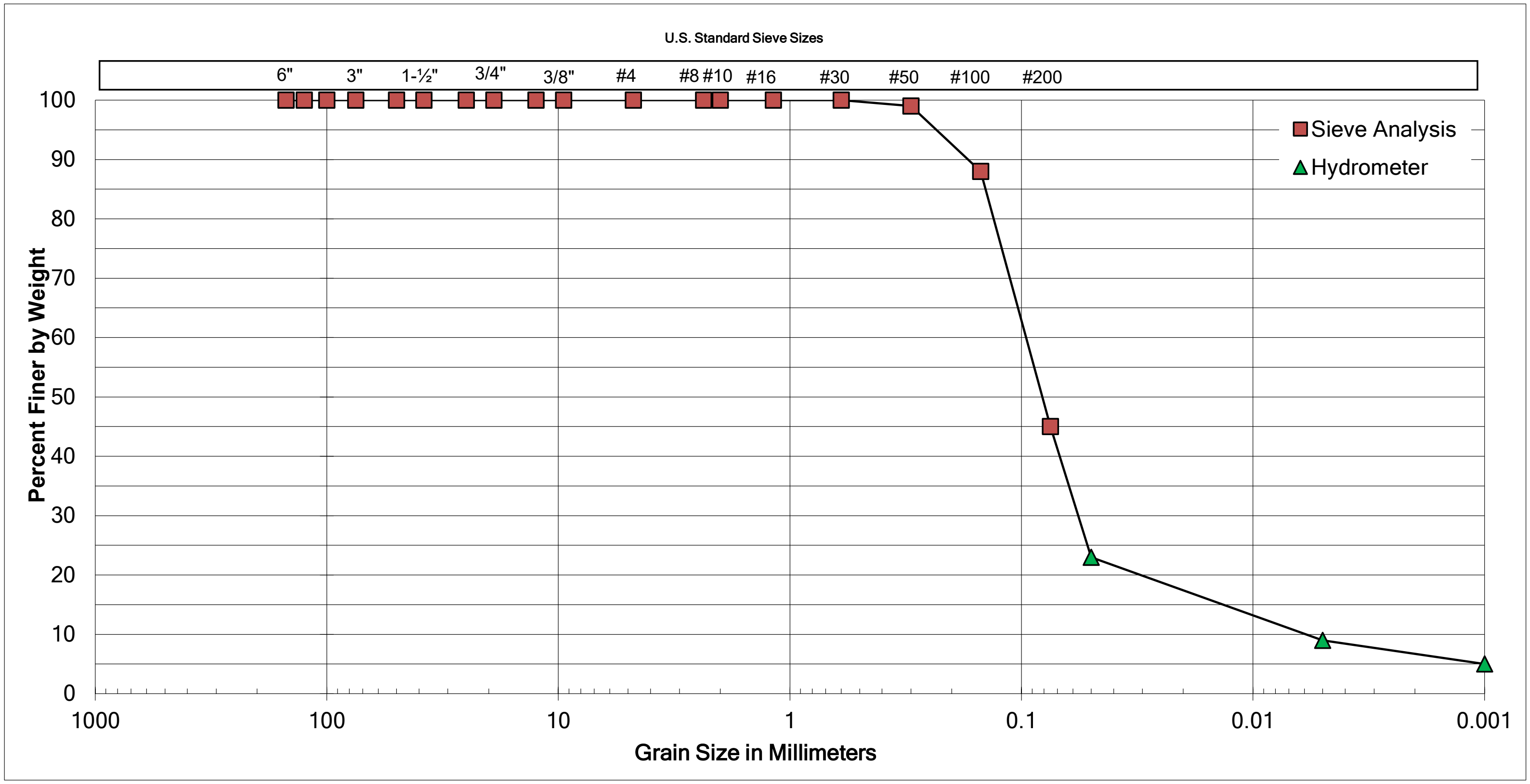
SAMPLE LOCATION
B-3 at 35 Feet
SAMPLE NUMBER
DRB

UNIFIED SOIL CLASSIFICATION:	ML
DESCRIPTION	SANDY SILT

ATTERBERG LIMITS	
LIQUID LIMIT	NP
PLASTIC LIMIT	NP
PLASTICITY INDEX	NP

ATLAS

Compton Community College - Visual and Perfoming Arts Compton, California			
By:	DRB	Date:	March, 2022
Job Number:	10-61187PW	Figure:	III-12



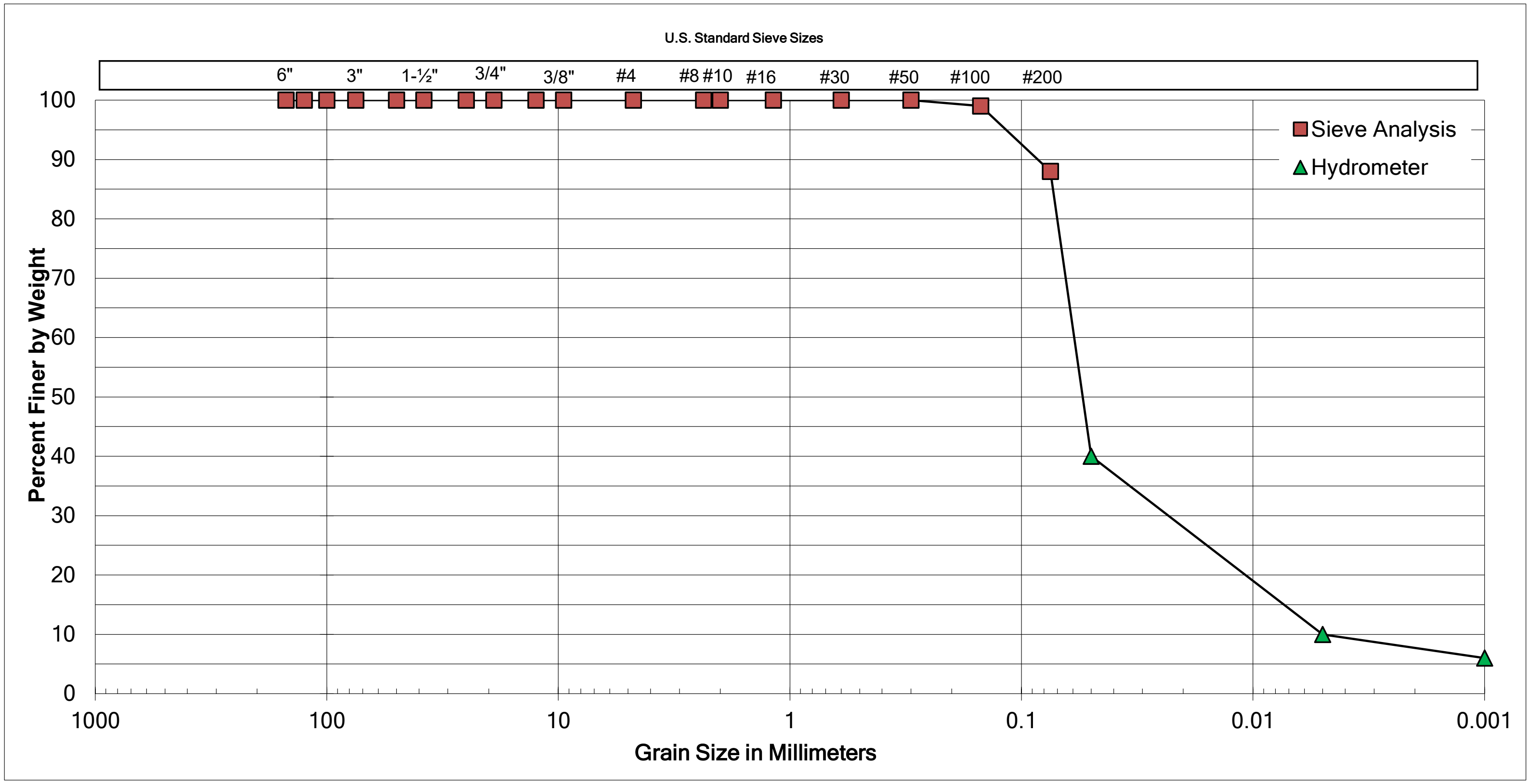
Cobbles	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

SAMPLE LOCATION
B-3 at 40 Feet
SAMPLE NUMBER
DRB

UNIFIED SOIL CLASSIFICATION:	ML
DESCRIPTION	SANDY SILT

ATTERBERG LIMITS	
LIQUID LIMIT	NP
PLASTIC LIMIT	NP
PLASTICITY INDEX	NP

Compton Community College - Visual and Perfoming Arts Compton, California			
By: DRB		Date: March, 2022	
Job Number: 10-61187PW		Figure: III-13	



Cobbles	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

SAMPLE LOCATION
B-3 at 50 Feet
SAMPLE NUMBER
DRB

UNIFIED SOIL CLASSIFICATION:	ML
DESCRIPTION	SANDY SILT

ATTERBERG LIMITS	
LIQUID LIMIT	NP
PLASTIC LIMIT	NP
PLASTICITY INDEX	NP

ATLAS

Compton Community College - Visual and Perfoming Arts Compton, California			
By:	DRB	Date:	March, 2022
Job Number:	10-61187PW	Figure:	III-14

APPENDIX IV
SITE CLASS CALCULATIONS

Project No.	10-61187PW	Project Name	Compton CC-VAPA
Boring No.	B-3		

Layer Top ft	Layer Bottom ft	Blow Count	Sampler Type Ring/SPT	Correction Factor*	Energy Correction**	Corrected Blow Count	Layer Thickness/N
0	5	10	Assumed SPT	1	1.25	12.5	0.40
5	10	7	SPT with Autohammer	1	1.25	8.8	0.57
10	15	24	Ring with Autohammer	0.65	1.25	19.5	0.26
15	20	11	SPT with Autohammer	1	1.25	13.8	0.36
20	25	28	Ring with Autohammer	0.65	1.25	22.8	0.22
25	30	12	SPT with Autohammer	1	1.25	15.0	0.33
30	35	16	Ring with Autohammer	0.65	1.25	13.0	0.38
35	40	20	SPT with Autohammer	1	1.25	25.0	0.20
40	45	51	Ring with Autohammer	0.65	1.25	41.4	0.12
45	50	31	SPT with Autohammer	1	1.25	38.8	0.13
50	55	50	Ring with Autohammer	0.65	1.25	40.6	0.12
55	60	35	Assumed SPT	1	1.25	43.8	0.11
60	75	35	Assumed SPT	1	1.25	43.8	0.34
75	100	35	Assumed SPT	1	1.25	43.8	0.57
						SUM:	4.13

*A 0.65 correction factor was used to convert ring/drive blow counts to standard (SPT) blow counts

**A correction of 1.25 was used for Autohammer

		Site Class
N average:	**24.2**	**D**

Input
Calculated

APPENDIX V
SITE-SPECIFIC GROUND MOTION HAZARD ANALYSES RESULTS

Performed for Compton College, Physical Education Complex Replacement and can be used for Compton College, Visual and Performing Arts Building.

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

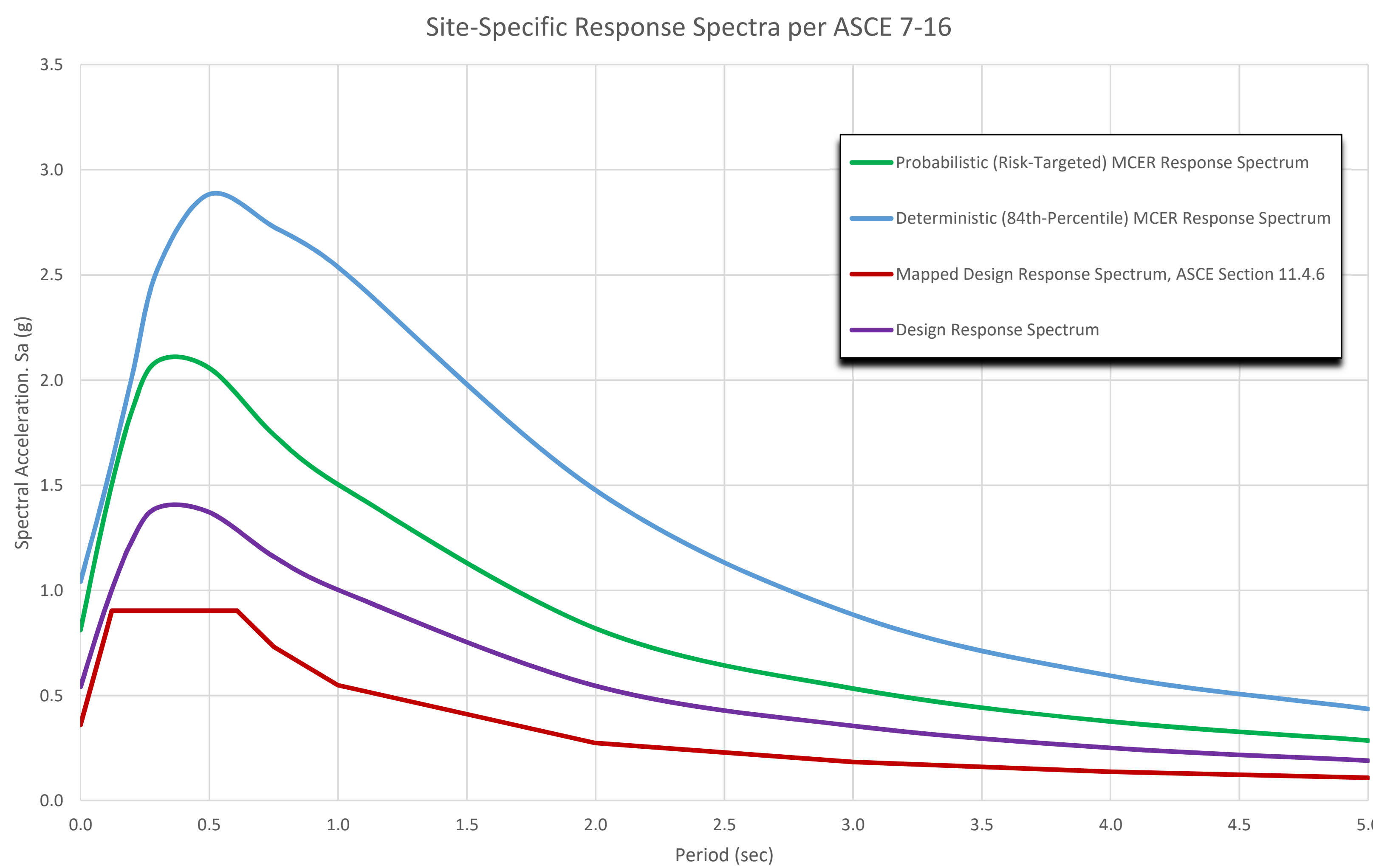
SITE-SPECIFIC DESIGN PARAMETERS

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 x 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 x PGA_M)

ATLAS

Compton College PE Complex Compton, California			
By:	GLC	Date:	July, 2021
Job Number:	10-57575PW	Figure:	V-1

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**



ATLAS

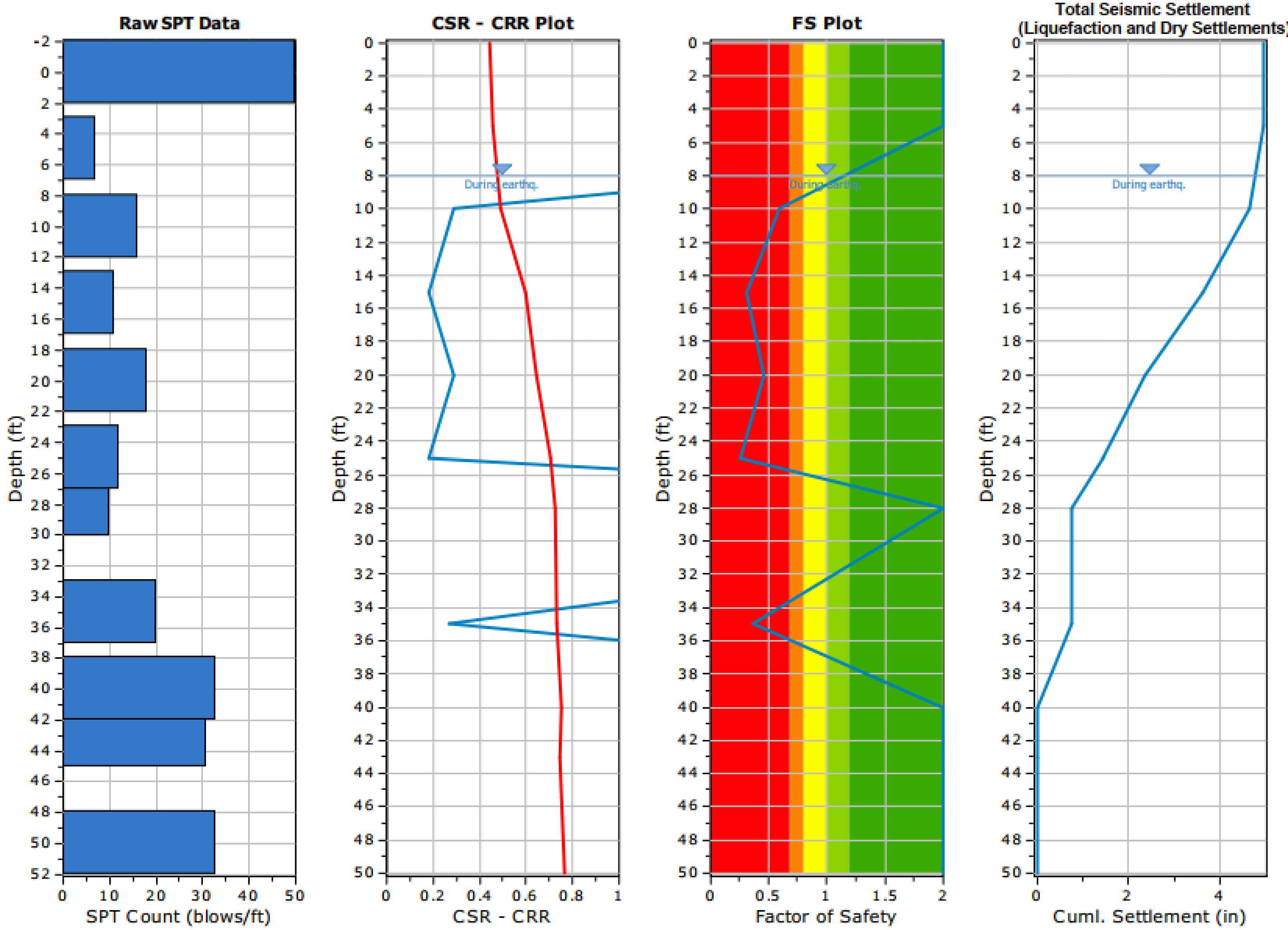
Compton College PE Complex Compton, California			
By:	GLC	Date:	July, 2021
Job Number:	10-57575PW	Figure:	V-3

APPENDIX VI
HISTORIC SEISMIC EVENTS

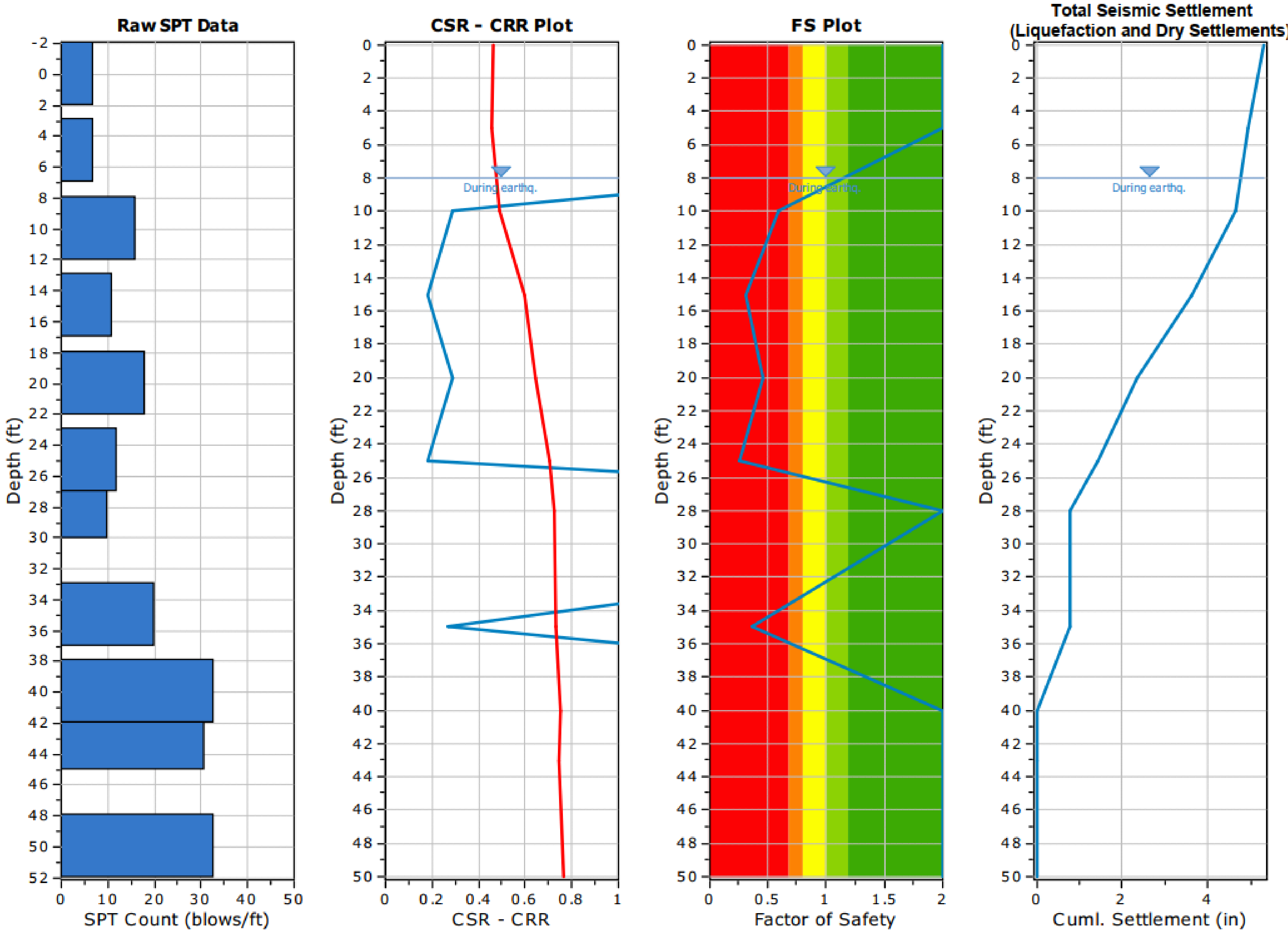
Historic Seismicity (1900 to 2018)
Within 100 km Search Radius and M_W > 5.0
Proposed Instructional Building #2, Compton College
1111 East Artesia Blvd., Compton, CA 90221

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

APPENDIX VII
LIQUEFACTION AND SEISMIC SETTLEMENTS CALCULATIONS RESULTS



ATLAS	**B&I 2014** **B-3: Historically Highest Groundwater Table, 5ft of Compacted Fill**	Date: April, 2022 By: MNK Job No.: VAPA Compton College 10-61187PW	**Figure:** VII-1

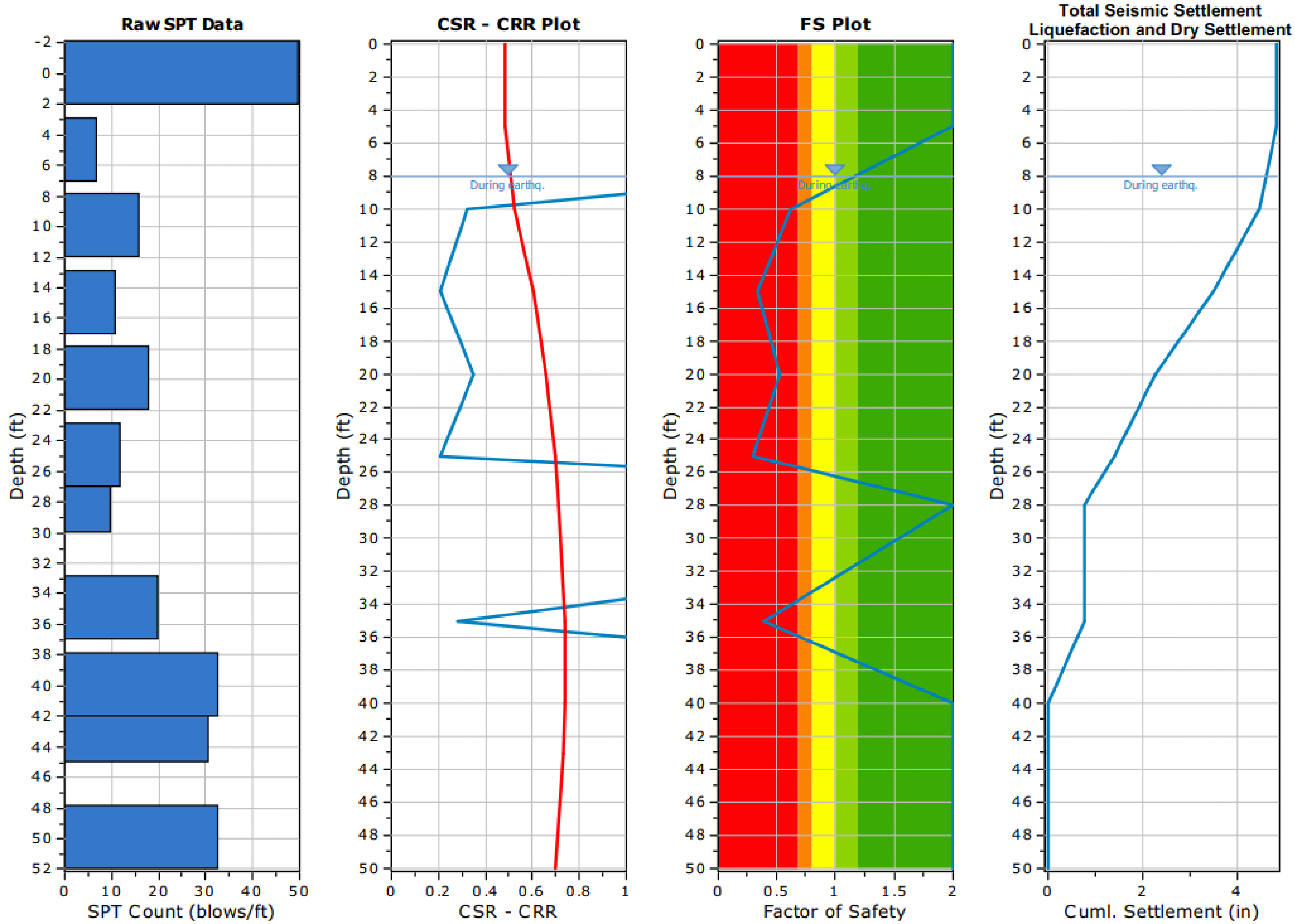


ATLAS

B&I 2014
B-3: Historically Highest Groundwater Table, No Compacted Fill

Date: April, 2022
By: MNK
Job No.: VAPA Compton College 10-61187PW

Figure:
VII-2

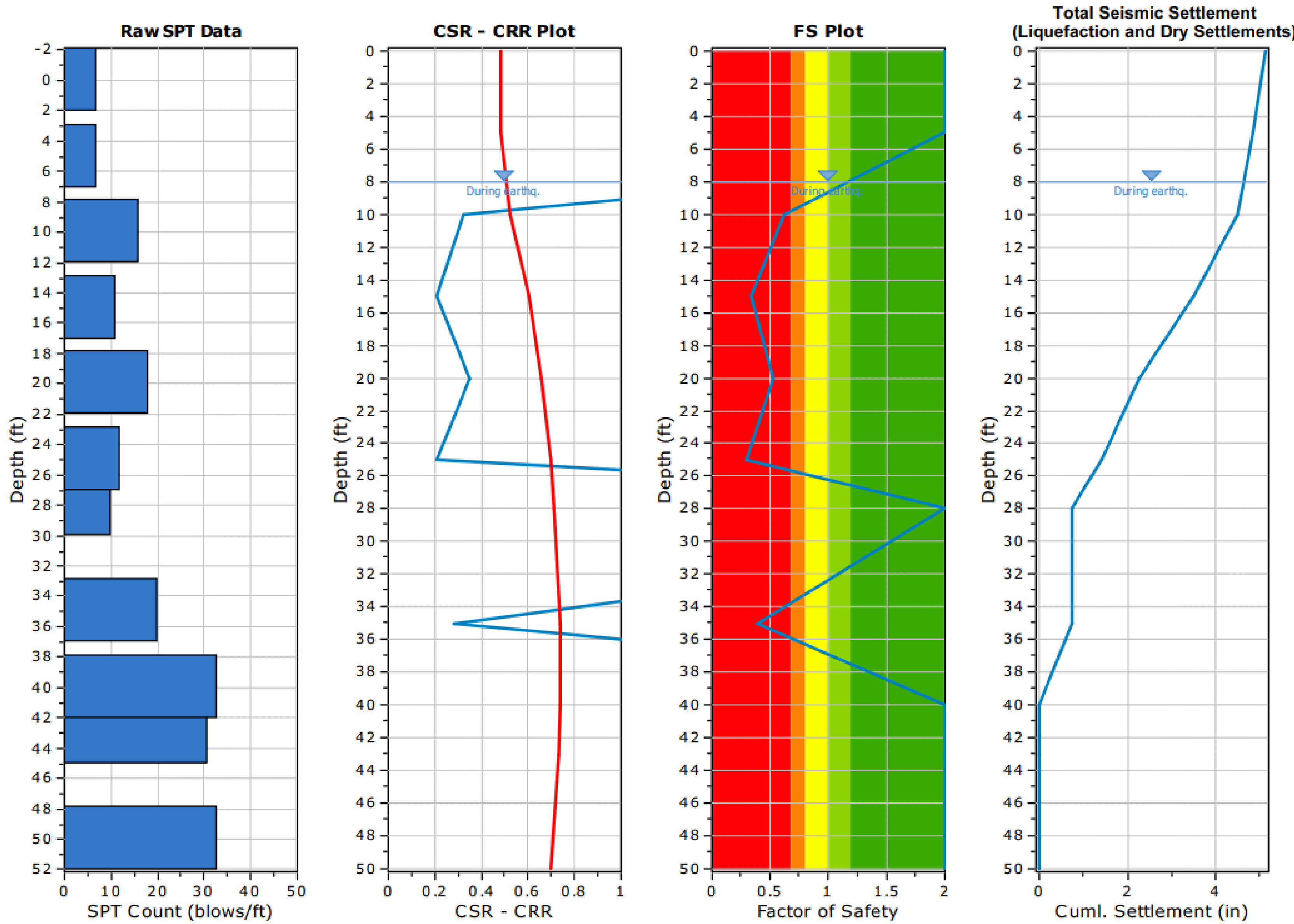


ATLAS

NCEER 1998
B-3: Historically Highest Groundwater Table, 5ft of Compacted Fill

Date: April, 2022
By: MNK
Job No.: VAPA Compton College 10-61187PW

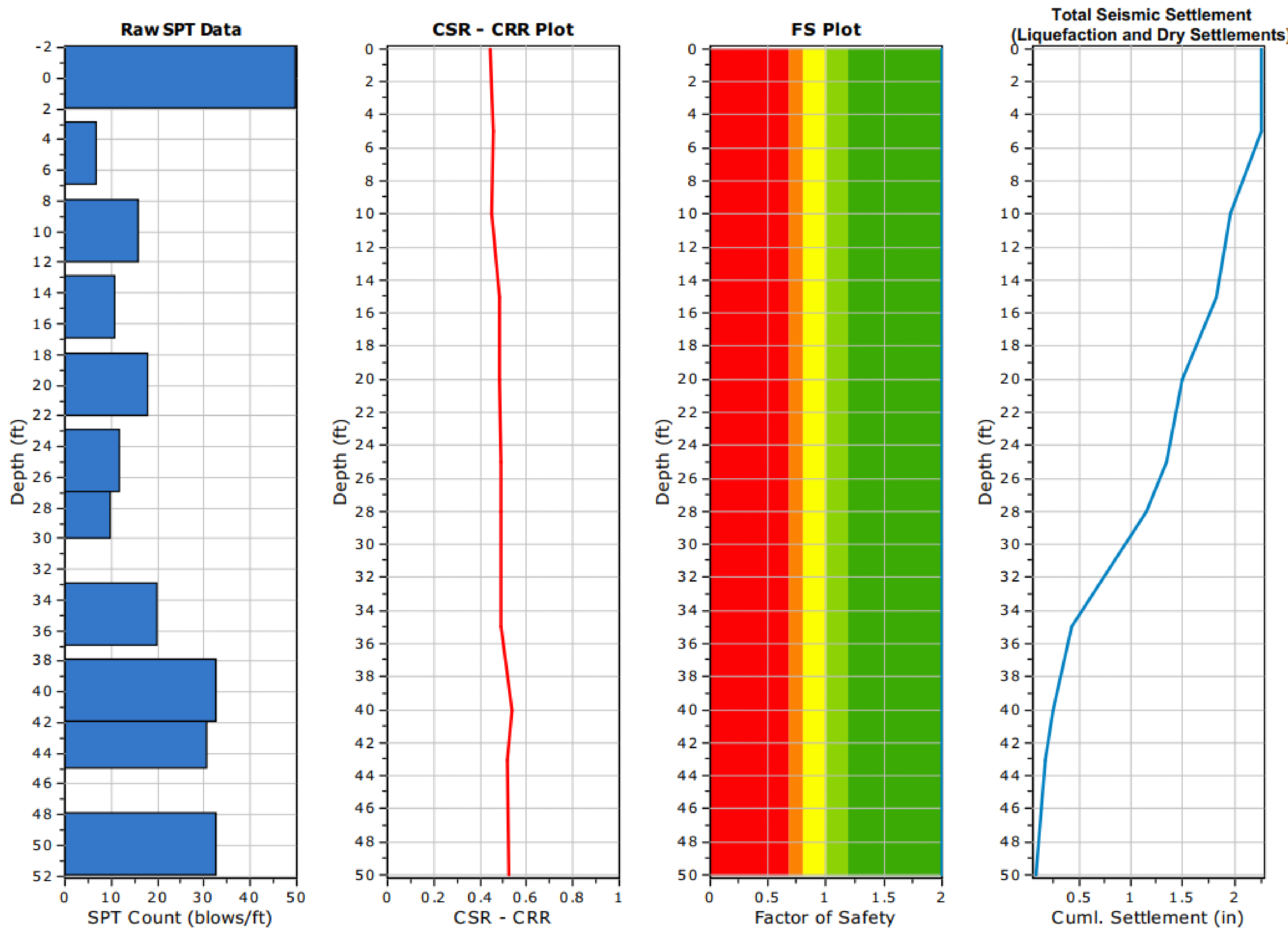
Figure:
VII-3



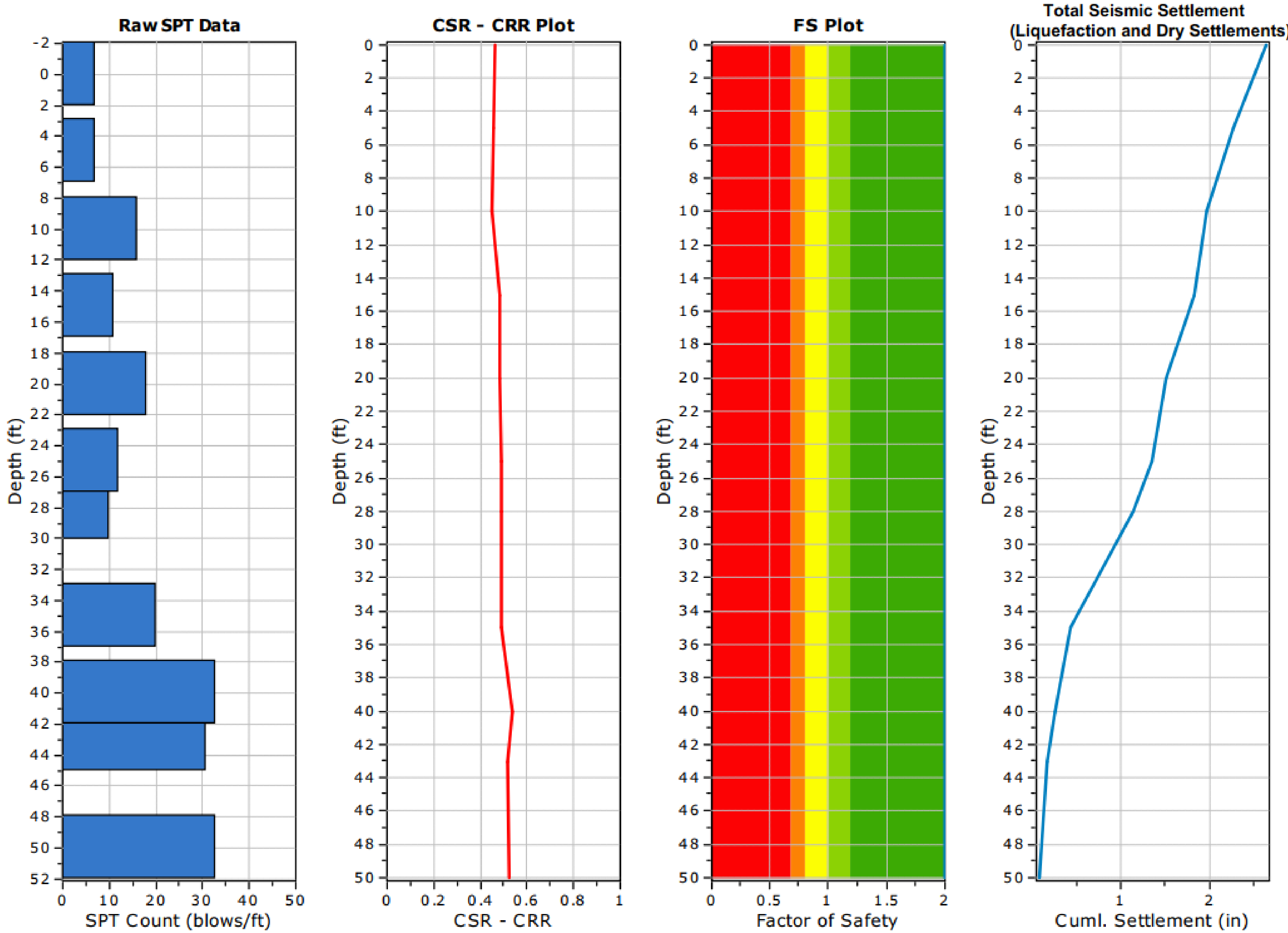
NCEER 1998
B-3: Historically Highest Groundwater Table, No Compacted Fill

Date: April, 2022
By: MNK
Job No.: VAPA Compton College 10-61187PW

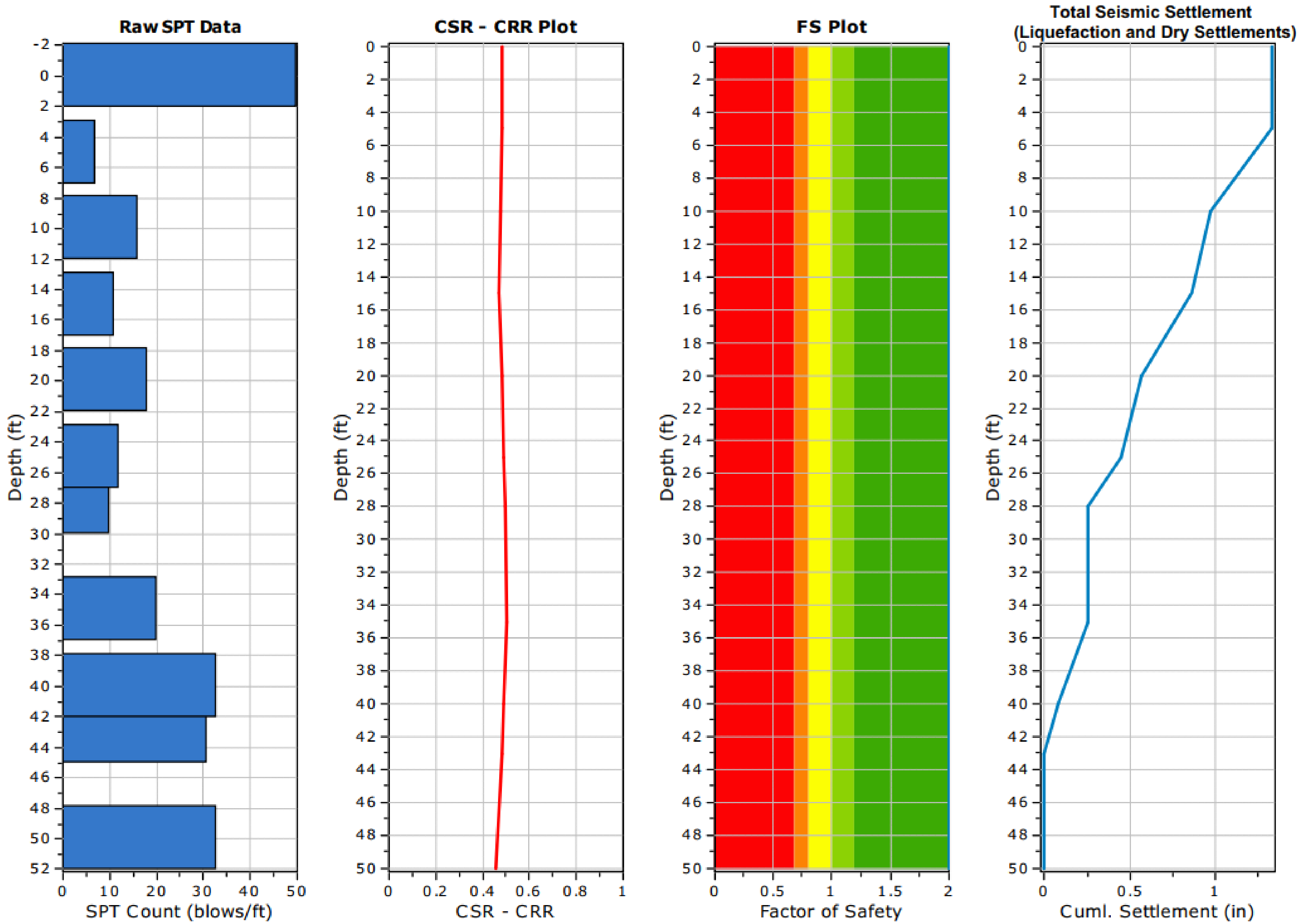
Figure:
VII-4



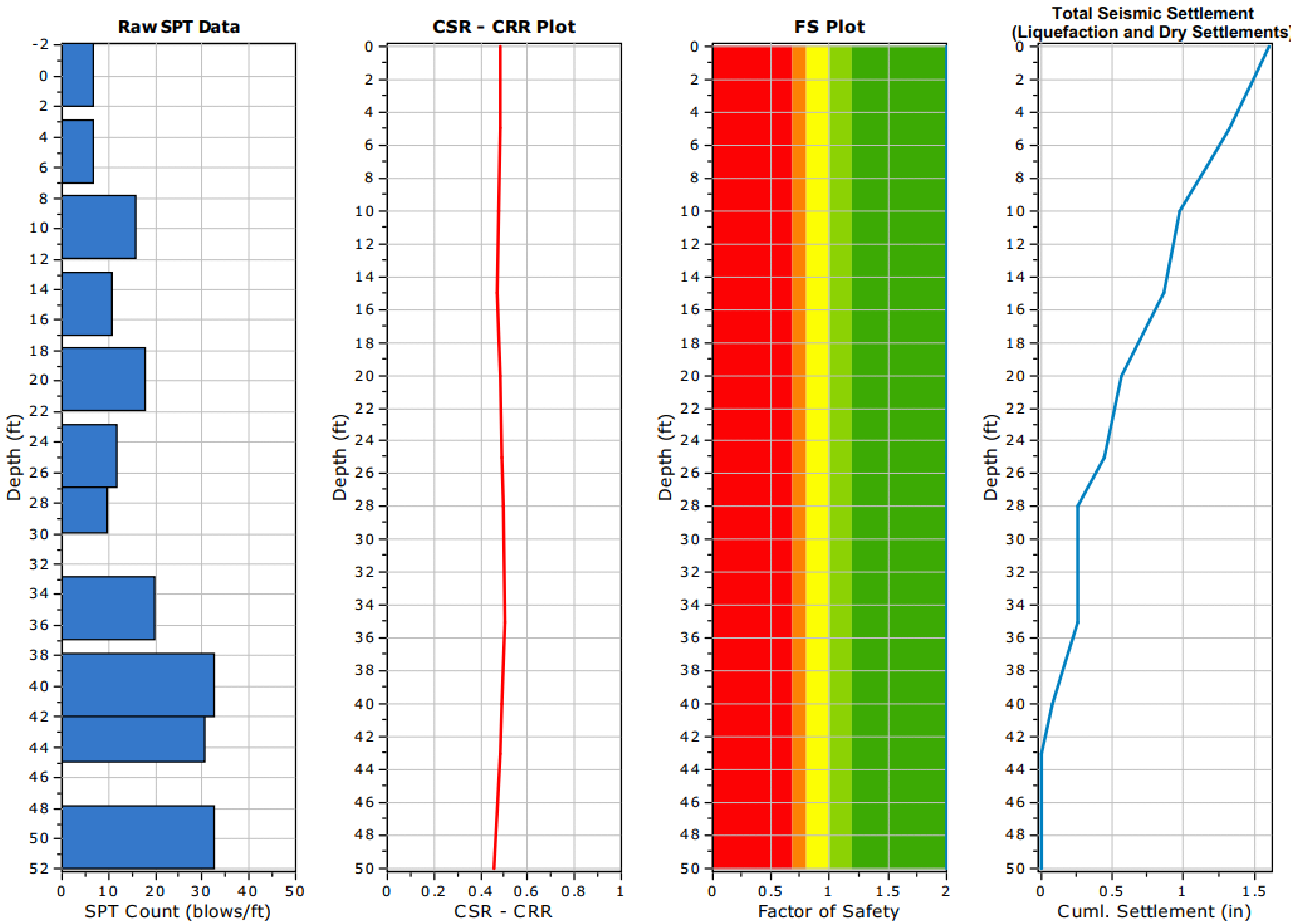
ATLAS	**B&I 2014** **B-3: Existing Groundwater Table,** **5ft of Compacted Fill**	Date: April, 2022 By: MNK Job No.: VAPA Compton College 10-61187PW	**Figure:** VII-5



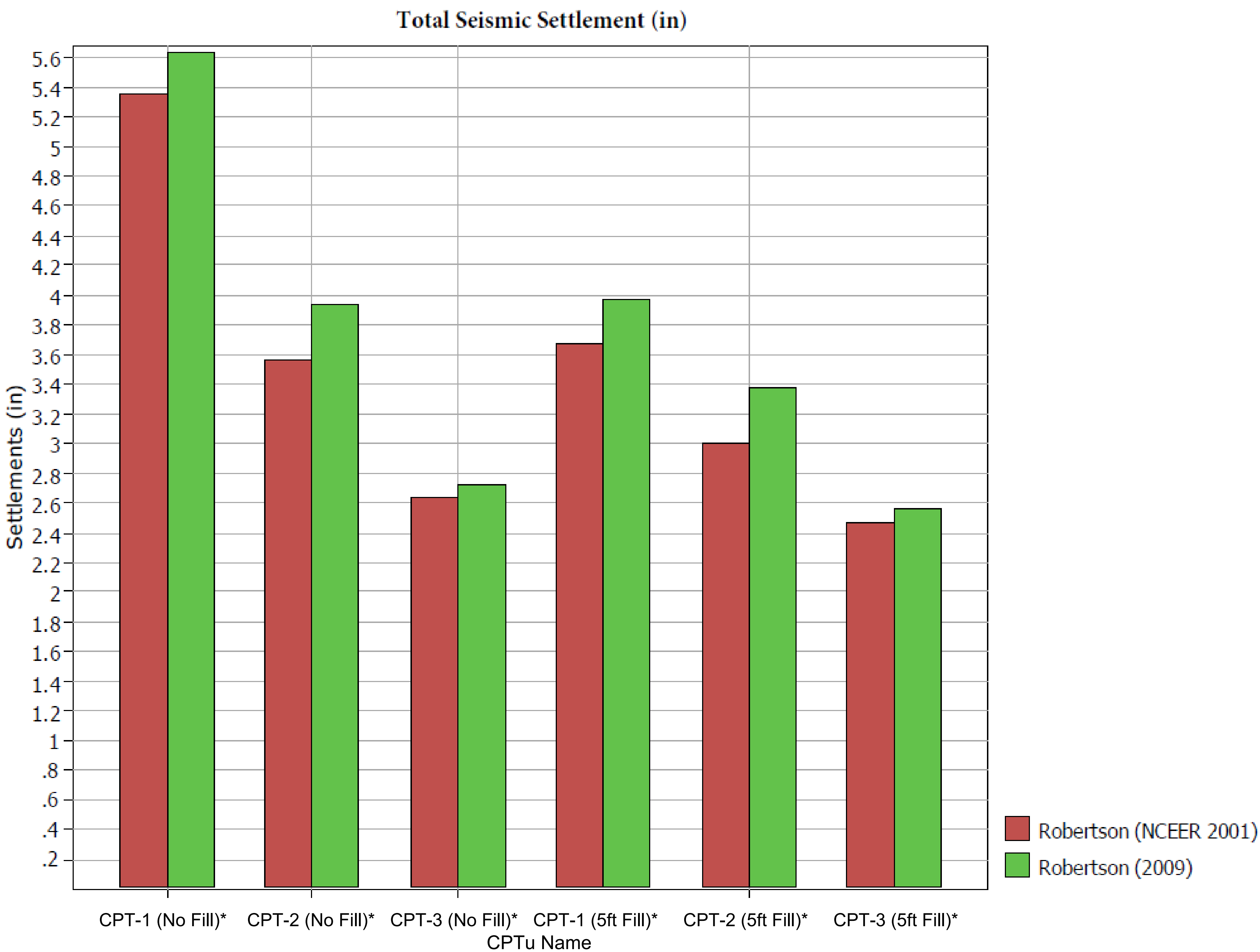
ATLAS	**B&I 2014** **B-3: Existing Groundwater Table, No Compacted Fill**	Date: April, 2022 By: MNK Job No.: VAPA Compton College 10-61187PW	**Figure:** VII-6



ATLAS	**NCEER 1998** **B-3: Existing Groundwater Table, 5ft of Compacted fill**	Date: April, 2022 By: MNK Job No.: VAPA Compton College 10-61187PW	**Figure:** VII-7



	NCEER 1998 **B-3: Existing Groundwater Table,** **No Compacted Fill**	Date: April, 2022 By: MNK Job No.: VAPA Compton College 10-61187PW	**Figure:** VII-8



ATLAS	**CPT-1,2,3: Historically High Groundwater Table** ***Fill: Compacted Fill**	Date: April, 2022 By: MNK Job No.: VAPA Compton College 10-61187PW	**Figure:** VII-9