Compton Community College District
Geotechnical and Soils Investigation Report
For
Student Services Building Project

1. **Original Report Prepared by United Heider November 13, 2018**

2. **CGS approval letter dated February 21, 2019**

3. **Supplemental Report Letter #1 for design recommendations related to the existing old building piles dated April 22, 2019**

4. **Supplemental Report Letter #2 for design recommendations related to adding cement-modified soil to excavation bottom dated September 5, 2019**

PCM3, Inc. Site Office
Compton Community College District
1111 E. Artesia Ave., Bond Trailer, Compton, CA 90221

GEOTECHNICAL INVESTIGATION REPORT
PROPOSED NEW STUDENT SERVICES BUILDING
COMPTON COLLEGE CAMPUS
1111 E. ARTESIA BLVD.
COMPTON, CA 90221

UNITED - HEIDER INSPECTION GROUP

PROJECT NO.: 10-18469PW

PREPARED For:

FACILITIES SERVICES
COMPTON COMMUNITY COLLEGE DISTRICT
1111 EAST ARTESIA BLVD.
COMPTON, CA 90221

PREPARED By:

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UNITED-HEIDER
INSPECTION GROUP
An ETS Company

November 13, 2018

To: Ms. Linda Owens
Director of Facilities
Compton Community College District
1111 East Artesia Blvd.
Compton, CA 90221

Subject: Geotechnical Investigation Report
Proposed New Student Services Building
Compton College Campus
1111 E. Artesia Boulevard
Compton, CA 90221

United - Heider Inspection Group Project No. 10-18469PW

Dear Ms. Owens:

In accordance with our proposal, United - Heider Inspection Group has prepared this geotechnical investigation report for the proposed New Student Services Building located within the Compton College Campus located at 1111 East Artesia Boulevard in the City of Compton, California.

The purpose of our investigation was to explore the subsurface conditions with respect to the planned improvements, to evaluate the general soil characteristics, and to provide geotechnical recommendations for design and construction. This investigation is based on a Site Plan provided by the tPB/Architecture and our correspondence with architects/designers.

Based upon our 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 moderately compressible and hydro-collapsible potential 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.

UNITED-HEIDER INSPECTION GROUP

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We appreciate the opportunity to work with you on this project. If you have any questions, or if we can be of further service, please call us at your convenience.

Respectfully submitted,

UNITED - HEIDER INSPECTION GROUP

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1.0 INTRODUCTION

1.1 Site Location and Description

The subject building site is located within the northern portion of the Compton College Campus in the City of Compton, California. The subject building site is surrounded by Classroom buildings on the north, Transfer/Career Center Outreach Building and parking lots on the south, various Academic Affairs office buildings on the west, and Main Campus Drive on the east. The site location, relative to existing adjacent streets, landmarks and topographic features, is shown on the Site Location Map, Figure 1. The project location, measured on a Google Earth map, has a latitude reading of North 33.878698° and longitude reading of West 118.209314°. These coordinate readings should be considered accurate only to within an approximately 50-foot radius as implied by the method used. The New Student Services Building site is currently partially occupied with the Library Building and is predominantly covered with grass and mature trees.

1.2 Proposed Development

Based on the Preliminary Site Plans by tPB/Architecture, Compton Community College District plans to build a two-story New Student Services Building at the subject site. We understand that the footprint of the building will be approximately 20,000 square feet. As this project is in the design phase, there are no foundation plans available at this time. We anticipate the building will be supported on mat foundation system/shallow footings. We anticipate that the new building will be designed and constructed under the 2016 California Building Code.

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 New Student Services Building and provide geotechnical recommendations for design and construction of the proposed project.

This investigation is based on the Site Plan provided by tPB/Architecture, showing the site boundary and proposed preliminary improvements. This plan serves as the basis for our Boring Location Map, Figure 2 (Appendix A).

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 4 hollow-stem auger borings to depths ranging from 26.5 feet to 51.5 feet below the ground surface within the building footprint. 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 (SPT) 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 loosely backfilled with soil cuttings obtained from the borings. Logs of the geotechnical borings are presented in Appendix B. Boring locations are shown on the accompanying Boring Location Map, Figure 2 (Appendix A).

- 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 onsite soils. Tests performed during this investigation include:

    - In situ moisture content and dry density of existing soils.
    - Particle Size Analysis to characterize the soil type according to USCS, and to assist in the evaluation of liquefaction susceptibility of granular soil.
    - Atterberg limit tests to classify and characterize of the engineering properties of soils.
    - Direct shear to evaluate the strength characteristics of the onsite materials.

- Expansion Index test to evaluate the expansion potential of the onsite material.
- Water-soluble sulfate concentration in the soil for sulfate exposure and cement type recommendations.
- Resistivity and pH to evaluate corrosion potential of the onsite soils.
- Maximum Density and optimum moisture content test to evaluate compaction characteristics.

All laboratory tests were performed in general conformance with ASTM Standard Methods and California Test Methods.

The results of the in-situ moisture and density tests are shown on the boring logs (Appendix B). Results of the other laboratory tests are provided in Appendix C.

- 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.

2.0 GEOLOGIC AND GEOTECHNICAL FINDINGS

2.1 Regional Geology

The site is located within the South Gate Quadrangle within the Los Angeles metropolitan region, which 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 (California Department of Water Resources, 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 fan deposits (or Young Alluvium, Unit 2), as shown on Figure 3, Regional Geology Map. As shown on the geologic map (Figure 3 - Appendix A), the project site and much of the project vicinity are underlain by Holocene to Late Pleistocene age Younger Alluvial Fan Deposits (Qyf), described by the California Geological Survey (2010) as "unconsolidated to slightly consolidated, unvisited to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon" as "Young alluvium, Unit 2" by the California Geological Survey (2016).

2.2 Subsurface Conditions

The site is underlain by about 0.5 foot of grass/top soil/surficial fill and young alluvial fan deposits of Holocene to late Pleistocene age (Qyf) as shown on the geologic cross sections (Figures 7 and 8 in Appendix A). The young alluvial fan deposits encountered at the site are predominantly comprised of inter-layered Silty SAND, Sandy SILT, SILT, and Clayey SILT. In general, the near-surface sandy soils layers are mostly loose to medium dense, and sandy soils layers at depth are medium dense to dense in relative density. The near-surface fine-grained soil layers are mostly firm to stiff and stiff to very stiff at depth in consistency.

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

2.2.1 Expansion Potential

A representative sample of the most expansive sub-surface soils within the building site that was tested for expansion index had an expansion index of 56, indicating a medium expansion potential. The Geotechnical Engineering and Geologic Hazards Study Report (Heider Inspection Group, 2018) for the adjacent building project (Instructional Building #2) reported a medium expansion potential for the site (EI = 85). Based on this finding and our experience with similar type of materials, the onsite soils are anticipated to contain a medium expansion potential (per ASTM D4829).

2.2.2 Corrosivity Potential

In general, soil environments that are detrimental to concrete have high concentrations of soluble sulfates and/or pH values of less than 5.5. Section 4.3 of ACI 318 (ACI, 2005), as referred in the CBC, provides specific guidelines for the concrete mix-design when the soluble sulfate content of the soil exceeds 0.1 percent by weight or 1,000 parts per million (ppm). The County of Los Angeles (2013) recommends implementing mitigation measures to protect any concrete structures when soluble sulfate concentrations are equal to or greater than 2,000 ppm in soil and 1,000 ppm in groundwater.

A representative sample of the subsurface soil within the building that was tested for water-soluble sulfates during the investigation had a soluble sulfate content of 48 ppm, i.e., less than 0.1 percent by weight (1000 ppm), indicating negligible sulfate exposure. Therefore, no cement type restriction/concrete class restriction is necessary per ACI Table 4.3.1 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 to foundation elements 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, a representative sample of the subsurface soil within the building site was tested to determine its minimum resistivity, chloride content, and pH level. The chloride content of the sample was 21 ppm. The minimum resistivity of the samples was 2,500 ohm-cm. The pH value of the sample was 7.7. Based on these results, the onsite soil is considered to be non-corrosive to foundation elements. 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.

The Geotechnical Engineering and Geologic Hazards Study Report (Heider Inspection Group, 2018) for the adjacent building project (Instructional Building #2) reported the following substantially conforming corrosion suite results as listed in the following table.

Table 1: Corrosion Results (Heider Inspection Group, 2015)

Boring (Heider Inspection 2018)	**Sample Depth (feet)**	**Sulfate (mg/kg)**	**Chloride (mg/kg)**	**Resistivity (ohm-cm)**	**pH**
B-2*	0-5	36	< 10	2,700	7.3

2.2.3 Excavatability

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

2.3 Groundwater

Groundwater was encountered in our soil boring B-1 at a depth of 46 feet below the existing ground surface. Groundwater was encountered at a depth of 45 ft in Borings B-1 during the United-Heider Inspection Group's previous investigation in 2018. The depths of groundwater encountered in the previous borings (2015), as well as estimated from the CPTs, ranged from 45 to 48.5 feet below existing ground surface.

According to the California Geological Survey (CGS, 1998) seismic hazard zone report for the South Gate quadrangle, historically shallowest groundwater level is estimated to be on the order of eight feet below existing grade. According to the Department of Water Resources (DWR), available groundwater level data for Well 338872N1182432W001, the nearest well located approximately two 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). The DWR groundwater level data are presented in Appendix B.

Groundwater levels generally fluctuate between different locations, years, and seasons. Therefore, variations from our observations may occur in the future; historically, these appear to be on the order of a few feet. Given the extensive use of groundwater resources and urbanization, it is unlikely groundwater levels will rise to a level that may adversely impact the design and/or during construction of this project. As such, groundwater is not expected to be a constraint to the design or construction of the proposed development.

3.0 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 2 miles west of the site. In addition to this surface fault zone, two buried thrust faults, the Lower Elysian Park and Compton, are inferred to be located about 2.5 miles north and 8 miles south, respectively, from the site. (Shaw, et al., 2002; Bilodeau, et. al., 2007)

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

Table 2: Characteristics and Estimated Earthquakes for Regional Faults

Fault Name	Approximate Distance to Site (miles)[1]	Maximum Credible Earthquake (MCE) Magnitude[2]
Newport-Inglewood	2	7.1
Lower Elysian Park Thrust	2.5[3]	6.7

[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.

Fault Name	Approximate Distance to Site (miles)[1]	Maximum Credible Earthquake (MCE) Magnitude[2]
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

[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.

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Fault Name	Approximate Distance to Site (miles)[1]	Maximum Credible Earthquake (MCE) Magnitude[2]
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

3.1.1 Regional Seismicity

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

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

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[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.

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 deaths, 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 7000 injuries

On 4/21/1918 at 22:32:30, a magnitude 6.8 (6.8 UK) earthquake occurred 45.5 miles away from the city center.

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

3.2 Secondary Seismic Hazards

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

3.2.1 Liquefaction

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

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

A site-specific liquefaction analysis was performed in accordance with the method of NCEER (Youd et al., 2001) and Boulanger and Idriss (2006) using LiquefyPro Version 5 computer program developed by Civiltech Software. Seismically-induced settlement analyses were performed based on the sub-surface conditions encountered in the deep boring B-1 and peak ground acceleration values PGA corresponding to adjusted Peak Ground Acceleration PGA_M. For this analysis, we considered a historic high groundwater level at eight feet below ground surface as indicated on the CGS Seismic Hazards Report. 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, using peak ground acceleration values PGA corresponding to adjusted Peak Ground Acceleration PGAM and modal magnitude of 7.3 Mw, were used for the liquefaction analysis. Seismically-induced settlement calculated for the soil layers has the factor of safety of less than 1.3.

Based on our calculations, potential for liquefaction at the site to occur within various loose to medium dense sandy silt/silty sand layers occurring primarily between depths of 10 and 20 feet below existing ground surface. Therefore the liquefaction susceptibility of the site is relatively high.

3.2.2 Seismically-Induced Settlement

Seismically-induced settlement consists of dry dynamic settlement (above groundwater) and liquefaction-induced settlement (below groundwater). These settlements occur primarily within loose to moderately dense sandy soil due to reduction in volume during and shortly after an earthquake event. Seismically-induced settlement analyses were performed using the methods set forth by Tokimatsu and Seed (1987).

The maximum potential liquefaction settlement at the site was estimated to be on the order of 1 inch. This potential settlement is primarily due to liquefaction settlement. The Geotechnical Engineering and Geologic Hazards Study Report (Heider Inspection Group, 2015) for the adjacent building project (Instructional Building #2) reported a revised post-earthquake settlements at the two 55-foot deep CPTs (CPT-1H and CPT-3H) approximately 2.11 and 1.84 inches, respectively.

The maximum differential settlement is estimated to be on the order of ½ of the vertical settlement, corresponding to 0.9 to 1.1 inches. A summary of our liquefaction analyses is presented in Appendix D.

The major impact of potential liquefaction would be post-earthquake settlement, which could potentially damage a structure due to excessive vertical and differential settlements. These settlements should be taken into account by the Structural Engineer during the design of the structure foundations. If the settlements are judged to be excessive, special remediation for ground improvement may be considered to reduce post liquefaction settlement.

We have performed several Atterberg limit tests on all of our fine-grained soil layers identified in our soil Boring B-1 performed as part of this investigation as well as the soil Boring B-1* performed for the adjacent Instructional Building #2. The fine-grained layers of both borings have shown PI values ranging from 10 to 27 that indicate all of the fine-grained layers at the site may exhibit a “clay-like behavior” during a seismic event as their PI values were greater than 7 (Boulanger and Idriss, 2006).

Consequences of cyclic softening of each fine-grained layer from both borings were analyzed following the procedure outlined in Idriss and Boulanger (2008) and Bray and Sancio (2006). Liquefaction potential/cyclic softening consequences of fine-grained soil layers were analyzed based on the methods referred above, and the calculation results are attached in Appendix D.

Based on our analysis, near-surface fine-grained layers exhibit a lower Liquidity Index (LI), w_c/LL below 0.8, and Sensitivity (S_t) well below 8. Therefore, these fine-grained soil layers appear to be less sensitive to remolding, and the consequences of cyclic softening of these layers are anticipated to be relatively minor. The layer at 45 feet from Boring B-

1* (Feb 2018) was classified as very stiff with an uncorrected SPT blow-count of 20 [undrained-shear strength (Su) > 1500 psf] and, therefore, anticipated to be less prone to strength loss during earthquakes.

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

Since much of the calculated liquefaction occurring relatively deep layers, the potential for surface manifestation of liquefaction is considered low to moderate.

We have reviewed the historic references (CDMG, 1998; Barrows, 1974; Hillis, 1933; Wood, 1933) that discuss the ground surface disruption due to liquefaction from the 1933 Long Beach earthquake.

The results of our review indicate that only two cracks attributed to liquefaction were reported near the Compton College campus. One of these cracks is illustrated in a photograph from Wood (1933, Plate 5a). These cracks occurred where water, sand, and mud were ejected that formed "craterlet" features and were reportedly located (CDMG, 1998) about ½ mile east of the subject proposed development on the Compton College campus. These cracks are interpreted to have formed as the result of liquefaction during earthquake ground shaking from the 1933 Long Beach earthquake. Water-soaked ground was also reported in the vicinity of City of Compton during the time of the 1933 earthquake.

However, Wood (1933, p. 52) indicated that the most severe damage associated with ground cracks due to liquefaction occurred on "ground formerly marshy in part, along Compton Creek and the former courses of the Los Angeles River, with deep deposits of loose, wet alluvium beneath." "The area most markedly affected by the extrusion of water lies west of Santa Ana and north and northwest of Newport Beach and Huntington Beach" (Wood, 1933, p. 54).

It appears that the Compton College campus site experienced much less severe ground failure due to liquefaction, because it was outside of the formerly marshy areas along the former courses of the Los Angeles

River, which experienced the most severe ground failure.

3.2.5 Seismically-Induced Landslide

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

3.2.6 Hydro-Collapsible Soils

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

The Geotechnical Engineering and Geologic Hazards Study Report (Heider Inspection Group, 2015) for the adjacent building project (Instructional Building #1) reported potential hydro-collapsible soils onsite. Based on a laboratory collapse test performed on a representative onsite soil sample collected from B-2H at a depth of 4.5 feet, a collapse potential index of about one percent was observed at an applied overburden pressure of 2,200 pounds per square foot (psf). We anticipate up to about an eight-foot thickness of the surficial onsite soils may be susceptible to collapse under saturation, corresponding to approximately one inch of collapse settlement. This calculated settlement should also be considered in designing the proposed structure foundation.

3.2.7 Other Hazards

Flood hazards generally consist of shallow sheet flooding caused by surface water runoff during large rain storms. According to the Federal Emergency Management Agency Flood Insurance Map (FIRM, 2008), the site is within a zone designated as “Other Flood Areas-Zone X: Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.”

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

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

Earthquake-Induced Flooding is a 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.

4.0 CONCLUSIONS AND RECOMMENDATIONS

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

The most significant design considerations for this project are moderately compressible and hydro-collapsible potential soil at the near surface, liquefaction and seismic settlement, and seismic shaking. Presented herein are our recommendations for site grading, seismic parameters, foundation design parameters, lateral earth pressures, and construction considerations for the project.

4.1 Earthwork

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

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

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

4.1.1 Site Preparation

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

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

Any existing utility lines should be removed and/or rerouted if they interfere with the proposed construction. The cavities resulting from removal of utility lines and any buried obstructions should be properly backfilled and compacted as recommended in Section 4.1.3 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 (i.e. buildings, walls, fences, etc.) should not be permitted within two (2) feet of existing foundations.

4.1.2 Excavation / Overexcavation

Existing fill soils within the proposed building area should be over-excavated to a depth of 1 foot below existing grade or to a sufficient depth to remove all of the undocumented fill materials in their entirety from within the proposed building area. Deeper undocumented fill layers may be present locally 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 and hydro-collapsible soil 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 building, we recommend that a minimum of 5 feet of engineered fill be provided under mat foundation/footings at a minimum overexcavation depth of 5 feet from existing grade, whichever provides the deeper overexcavation. The excavated removal bottoms of structural footings should be evaluated by a geotechnical engineer to confirm competent native soil materials are encountered. In general native soils with at least 85 percent relative compaction of maximum dry density (ASTD D1557) is considered suitable. If unsuitable soil conditions are encountered deeper excavation may be recommended. The overexcavation should extend below any underground obstructions to be removed. The overexcavation and recompaction should extend a minimum of 5 feet laterally from the edges of the footings, where feasible.

The soil below slabs-on-grade should be overexcavated and recompacted a minimum of 12 inches below the bottom of the proposed slab or 12 inches below the existing ground surface, whichever is deeper.

Areas outside the overexcavation limits of the proposed building planned for asphalt or concrete pavement and flatwork and areas to receive fill should be overexcavated to a minimum depth of 12 inches below the existing ground surface or 12 inches below the proposed finish grade, whichever is deeper.

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

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

After completion of the overexcavation, and prior to fill placement, the exposed surfaces should be scarified to a minimum depth of 6 inches, moisture-conditioned to optimum to plus 3-percent above optimum, and recompacted to a minimum 90 percent relative compaction.

4.1.3 Fill Placement and Compaction

Upon excavation/overexcavation to the recommended depths, subgrade soils at the removal bottoms should be moisture-conditioned as needed and recompacted to a minimum of 90 percent relative compaction (per ASTM D1557). No scarification at the removal bottom would be necessary.

Any fill soil should be placed in loose lifts of 6 to 8 inches in thickness, moisture-conditioned to above the optimum moisture content, and compacted to a minimum of 90 percent relative compaction (per ASTM D1557).

4.1.4 Fill Materials

Onsite soils that are free of organics, debris and oversize particles (e.g., cobbles, rubble, etc. that are greater than 3 inches in the largest dimension) and an Expansion Index less than 50 can be reused as fill as approved by the Geotechnical Engineer. Import soils, if used, should be free of organics, corrosion impacts, and should have an Expansion Index less than 21 (per ASTM D4829). Import soils should be evaluated and tested by our firm to confirm the quality of the material. If base materials are imported to be placed instead of soil backfill, these may be either crushed aggregate base or crushed miscellaneous base in conformance with the Sections 200-2.2 and 200-2.4 of the *Standard Specifications for Public Works Construction* (Green Book), 2006 Edition, respectively.

Soil engineer should be notified at least 48 hours prior to borrow materials in order to sample and test materials from proposed borrow sites.

4.2 CBC Seismic Design Parameters

A seismic cone penetration test (SCPT) at the site to measure shear wave velocities was performed by Heider Inspection Group during their 2015 investigation. Measurements were performed up to 55 feet below the existing surface. The average shear wave velocity was measured to be 779 feet per second (ft/sec; see Appendix B).

In order to provide the preliminary seismic design parameters, based on the field data 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 1613.3.2 of the 2016 California Building Code (CBC) accordance with Chapter 20 of ASCE 7. Although liquefiable soils and potential liquefaction settlement have been identified at the site, Site Class "F" was judged to not apply since, per ASCE 7-10, Section 20.3.1, the proposed building is anticipated to have a fundamental period of vibration less than 0.5 second. Therefore, based on the subsurface conditions and geology of the site, site's soil profile can be characterized within the category of 'Stiff Soil Profile' with Site Class D.

Corresponding **CBC seismic design parameters** for this soil profile and the site location (Latitude: 33.878698° N; Longitude: -118.209314° W) are determined based on general ground motion analysis in accordance with Section 1613.3 of the 2016 CBC. These parameters are summarized in Table. Proposed development at the site should be designed for the seismic parameters presented in the following Table.

Table 3 – CBC Seismic Design Parameters

Categorization/Coefficient	Design Value
Site Class	D
Mapped MCE Spectral Acceleration for Short (0.2 Second) Period, S_S	1.674
Mapped MCE Spectral Acceleration for a 1-Second Period, S_1	0.611
Short Period (0.2 Second) Site Coefficient, F_a	1.0
Long Period (1 Second) Site Coefficient, F_v	1.5
Adjusted Spectral Response Acceleration at 0.2-Second Period, S_{MS}	1.674
Adjusted Spectral Response Acceleration at 1-Second Period, S_{M1}	0.916
Design (5% damped) Spectral Response Acceleration for Short (0.2 Second) Period, S_{DS}	1.116
Design (5% damped) Spectral Response Acceleration for a 1-Second Period, S_{D1}	0.611
Peak ground acceleration value, PGA_M	0.623
Seismic Design Category	D

As the reported long period spectral response acceleration (S_1) was less than 0.75g (S_1<0.75), the project is assigned to a ***Seismic Design Category*** **"D"** based on Section 1613A.3.5 of CBC 2016.

As the site is assigned a Seismic Design Category D, a site-specific ground motion analysis is not required per CGS Note 48. As such, the above CBC Seismic Design Parameters following this USGS general procedure presented in Table 1 above should be used in design. The USGS summary reports will be included in our geotechnical report.

4.3 Foundation Design Parameters

The proposed building should be supported on foundations designed to accommodate the anticipated static and calculated seismic total and differential settlements without undue distress occurring to the building. As discussed in previous Sections, the project site is susceptible to potential settlement due to collapse settlement of the surficial silty soils, as well as liquefaction settlement induced by the design earthquake. Based on our liquefaction analyses, we calculated post-seismic liquefaction settlement on the order of 1 inch. Geotechnical Engineering and Geologic Hazards Study Report (United-Heider Inspection Group 2018) for the adjacent building project (Instructional Building #2) reported a revised seismic settlement ranging from 1.8 to 2.1 inches. Potential settlement due to collapse within the surficial silty soils was also reported to be on the order of one inch.

Hydro-collapse settlement and static settlement can be reduced or controlled by remedial grading i.e., reworking the surficial, collapse- susceptible soils as engineered fill. However, deep liquefiable layers will not be mitigated by shallow remedial grading. Therefore, due to high settlement, shallow pad and strip footing system is not recommended for this project.

We recommend using either a structural mat foundation supported on a layer of engineered fill or a conventional shallow spread footing foundation system in combination with a ground improvement method such as Geopiers or drilled displacement columns to transfer structural building loads to deeper, dense supporting strata below the bulk of the collapse and liquefaction-susceptible layers onsite.

4.3.1 Structural Mat Foundation

A mat foundation can be used to distribute foundation loads to span local irregularities in the supporting capacity of the foundation soil, and to reduce the magnitude of differential settlement. The mat foundation may be designed for any practical bearing pressure up to a maximum of 1,200 psf. Total settlement of mat foundations designed to the maximum bearing pressure are estimated to be on the order of 2½ inches or less (including seismic settlement) and differential settlement between adjacent columns should not exceed ¾ inch provided that the mat extends to a minimum two feet below lowest adjacent grade.

For the design of structural mat foundation, an average modulus of subgrade reaction, K_S of 150 pci (pounds per cubic inch) may be used. In addition, we recommend that the mat foundation be designed to tolerate a static and seismically-induced differential settlement. The magnitude of total and differential static settlement of the mat foundation will be a function of the structural design and stiffness of the mat.

Resistance to lateral loads can be provided by friction acting at the base of the foundation and by passive earth pressure. A coefficient of friction of 0.3 may be assumed with dead-load forces. An allowable passive lateral earth pressure of 200 pounds per square foot (psf) per foot of depth up to a maximum of 2,000 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 bearing values indicated above are for total dead-load and frequently applied live-loads. The above vertical and lateral bearing values may be increased by 33 percent for short durations of loading, including the effects of wind or seismic forces. 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 mat foundation.

If a structural mat foundation is selected for building support, the soils underlying the building pad should be over-excavated to construct the recommended five-foot thick engineered fill layer, and backfilled with engineered fill in order to remove the upper compressible & hydro-collapsible soil. Subgrade soil should be prepared as described in the earthwork section of this report (Section 4.1)

4.3.2 Shallow Foundations with Ground Improvement

Shallow spread footing foundations supported by a ground improvement method such as Drilled Displacement Columns (DDC), a ground improvement technique, can be used as an alternate for building foundation support. DDC is a method where a large diameter auger is advanced to the design depth, and as the auger is withdrawn, low strength concrete (CLSM) is injected under pressure as the auger is slowly withdrawn, providing soil compaction in loose and soft soil zones as well as providing a column. The method is similar to the installation of auger-cast piles except that minimal spoils are generated, and the columns serve to also transfer load of shallow Proposed Instructional Building # 2 foundations to deeper, more competent supporting strata rather than serving as a deep foundation with internal steel reinforcement.

If used, drilled displacement columns should be extended to a bearing depth 45 feet below the existing ground surface. We estimate that columns extended to a depth 30 feet will reduce potential liquefaction settlement to less than approximately ½ inch. Multiple columns may be needed at footing locations based on footing loads and dimensions, and additional columns may be required and spaced at wider intervals below slab-on-grade floors in order to minimize the potential for differential settlement of floor slabs and adjacent building columns.

DDC sizing and spacing would be determined by the design-build contractor once structural loading and foundation plans become available. The DDC work should be designed and installed by a qualified specialty contractor. The DDC work scope should include a DDC design-build submittal stamped by a California Registered Engineer, equipment and personnel mobilization, DDC load testing, and construction of DDCs. The design package should be submitted to United-Heider Inspection Group for review at least two weeks prior to mobilization for construction. Installation of DDC elements should be observed by United-Heider Inspection Group on a full-time basis.

Conventional continuous and/or isolated spread footings bearing on the improved onsite soils should be founded a minimum of 24 inches below lowest adjacent finished grade. Continuous footings should have a minimum width of at least 24 inches, and isolated column footings should have a minimum width of at least 30 inches. In addition, footings located adjacent to other footings or utility trenches should bear below an imaginary 1.5:1 (horizontal to vertical) plane projected upward from the bottom edge of the adjacent footings or utility trench. Footing reinforcement should be determined by the project Structural Engineer. Footings supported on DDC-reinforced soils can be initially designed for an allowable bearing capacity of 5,000 pounds per square foot (psf). The bearing capacity should be verified by a full-scale load test. An aggregate “cushion” layer at least eight inches thick should be placed between the DDC elements and footing. The aggregate “cushion” is typically placed and constructed by the grading contractor and is not a part of the DDC work.

Footings can be designed to resist lateral loads using an allowable coefficient of friction of 0.35. Lateral sliding resistance is derived at the concrete/aggregate interface below the footing. In addition, an ultimate passive resistance equal to an equivalent fluid weighing 200 pounds per cubic foot (pcf) acting against the foundation may be used for lateral load resistance against the sides of footings perpendicular to the direction of loading where the footing is poured neat against undisturbed material (i.e., native soils or engineered fills). The top foot of passive resistance at foundations not adjacent to and confined by pavement, interior floor slab, or hardscape should be neglected. In order to fully mobilize this passive resistance, a lateral footing deflection on the order of one to two percent of the embedment of the footing is required. If it is desired to limit the amount of lateral deflection to mobilize the passive resistance, a proportional safety factor should be applied. A one-third increase to the allowable bearing capacity and frictional resistance is permitted for short-term seismic and wind loads. The estimated long-term total and differential settlements of the DDC-supported footings should be less than one inch and ½ inch, respectively. Heider personnel should be retained to observe and confirm that foundation excavations prior to backfill or formwork and reinforcing steel placement bear in the anticipated soils suitable for the recommended maximum design bearing pressure.

4.4 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). Prior to concrete placement, the exposed subgrade should be scarified to at least 6 inches, moisture-conditioned to moisture content of optimum moisture to plus 3% over optimum. The subgrade should not be allowed to dry prior to concrete placement.

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

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

In areas where a moisture-sensitive floor covering (such as vinyl, tile, or carpet) is used, a moisture/vapor barrier should be placed per our recommendation in Section 4.9.

4.4.1 Exterior Concrete

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

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

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

United-Heider Inspection Group does not specialize in the field of slab design, concrete mix design and/or moisture vapor transmission. A qualified waterproofing consultant should be retained in order to recommend a product or method which would provide protection for the concrete slabs-on-grade in 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.

4.6 Temporary Excavations

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

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

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

4.7 Surface Drainage

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

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

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

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

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

4.8 Trench Backfill

Utility trenches should be backfilled with compacted fill in accordance with Sections 306-1.2 and 306-1.3 of the *Standard Specifications for Public Works Construction,* ("Greenbook"), 2006 Edition.

Utility trenches can be backfilled with onsite 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 percent relative compaction (per ASTM D1557) in paved and any structural areas.

4.9 Construction Observation and Testing

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

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

4.10 Limitations

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

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

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

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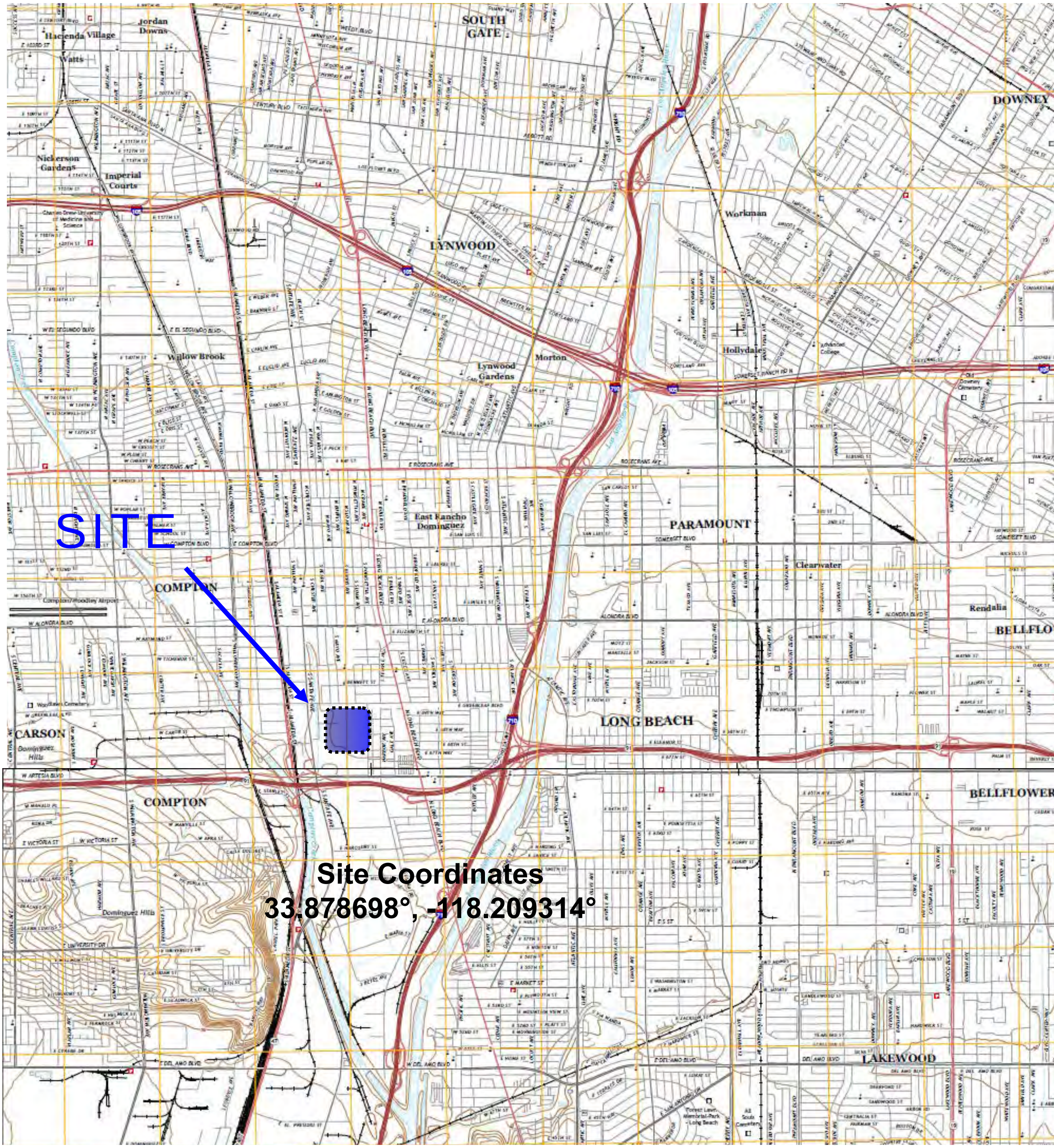
APPENDIX A

Figures



Scale

1000 0 1000 2000 3000 4000 5000 6000 7000 8000 9000 10000

FEET

REFERENCE: USGS 7.5 Minute Topographic Maps, South Gate and Long Beach Quadrangles, Los Angeles County, California (2015).

UNITED-HEIDER
INSPECTION GROUP

Figure 1 – Site Location Map

Proposed New Student Services Building EI
Compton College Campus
1111 East Artesia Blvd.
Compton, CA 90221

N

Project No. 10-18469PW

Date: Nov. 2018

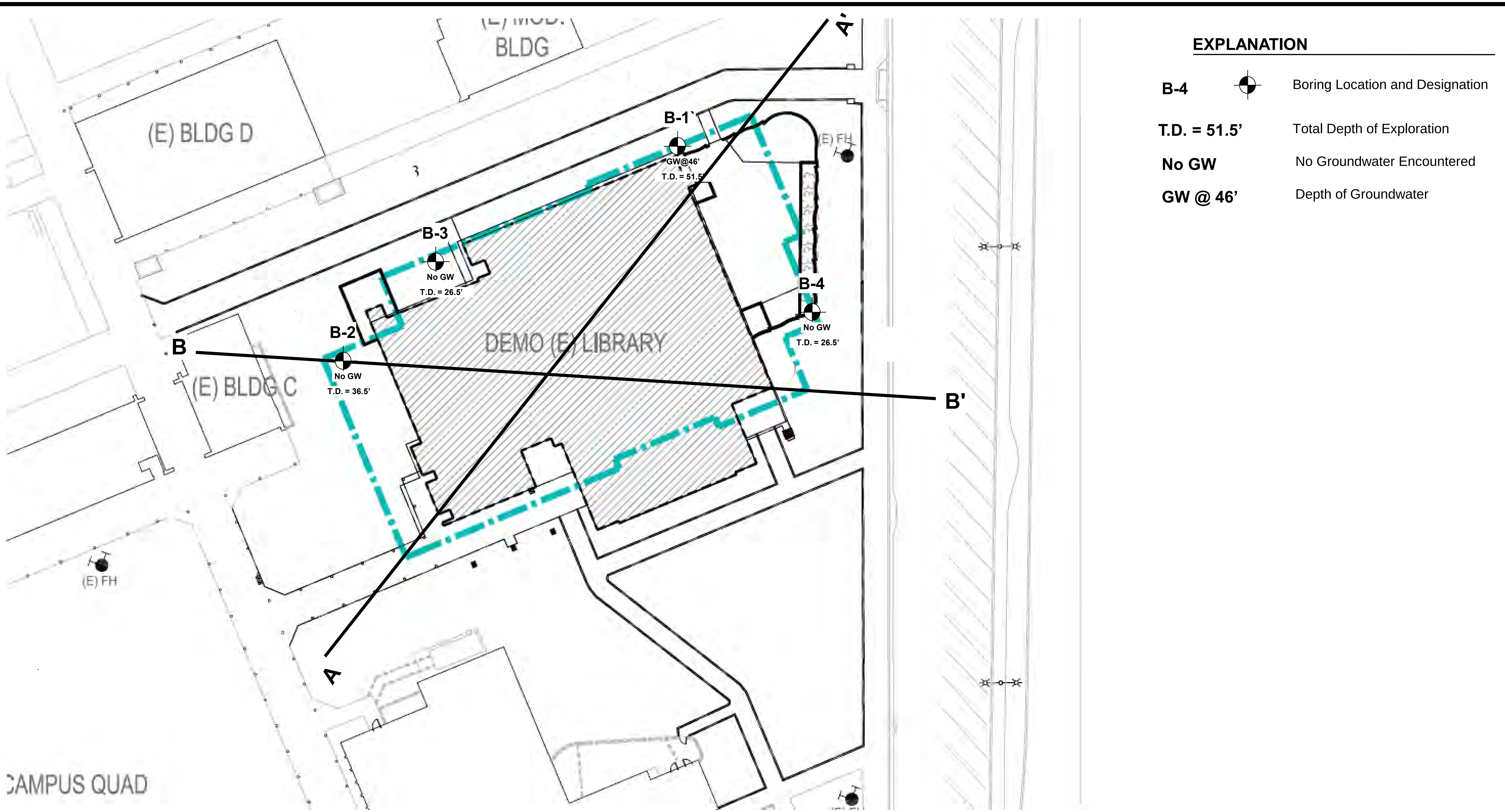


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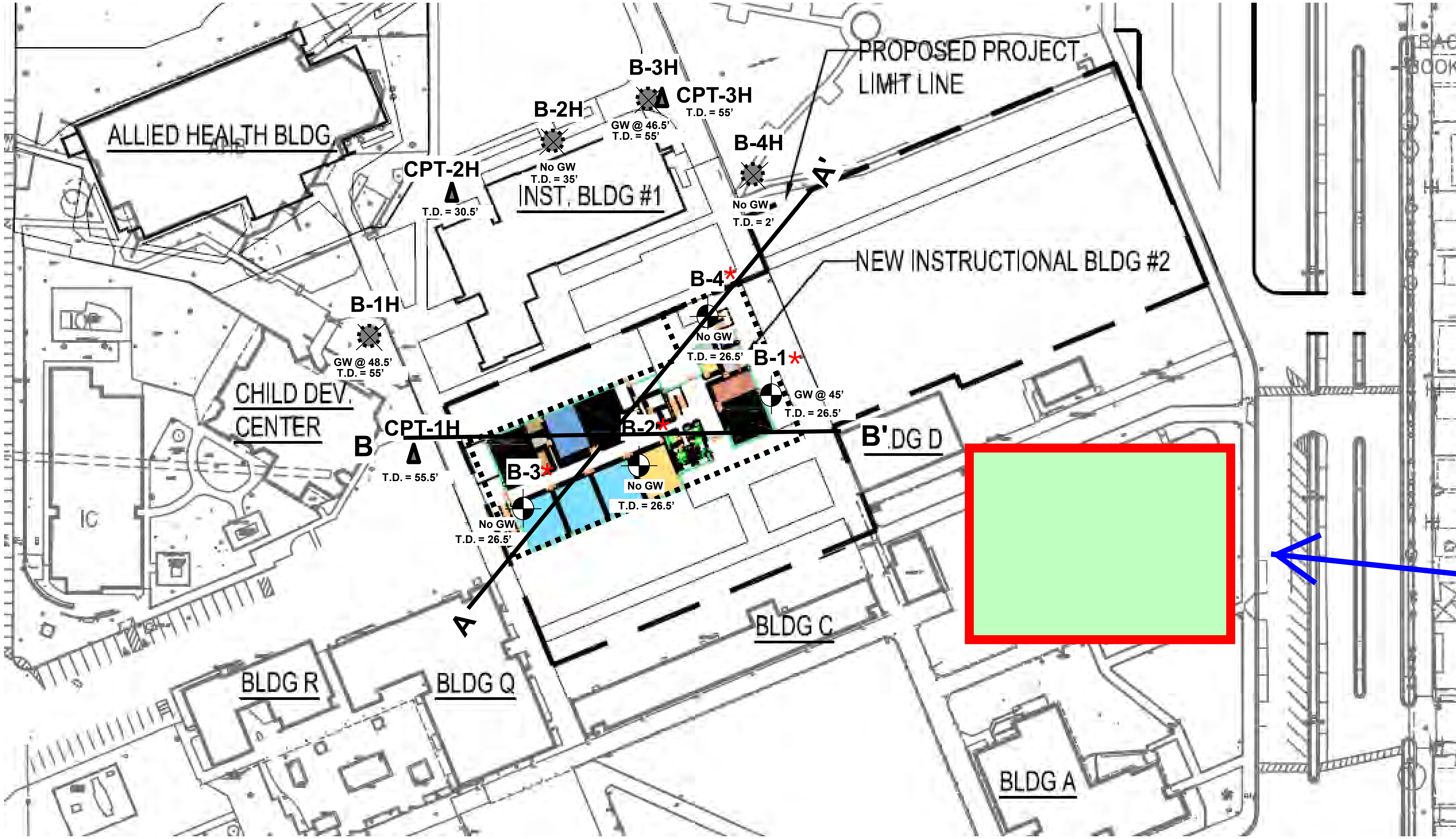
Figure 2 – Boring Location Map

Proposed New Student Services Building
Compton College Campus
1111 East Artesia Blvd.
Compton, CA 90221

Project No. 10-18469PW
Date: Nov. 2018



EXPLANATION

B-4*		Boring Location and Designation
B-4H		Boring Location and Designation by Heider Inspection Group (2015).
CPT-3H		Cone Penetration Test (CPT) Location and Designation by Heider Inspection Group (2015).
T.D. = 51.5'		Total Depth of Exploration
No GW		No Groundwater Encountered
GW @ 45'		Depth of Groundwater

Soil Boring Location Map for the Adjacent Instructional Building #2 by United-Heider Inspection Group (Feb. 2018)

SCALE: 1/80" = 1'-0'

0 40' 80' 160' 320'

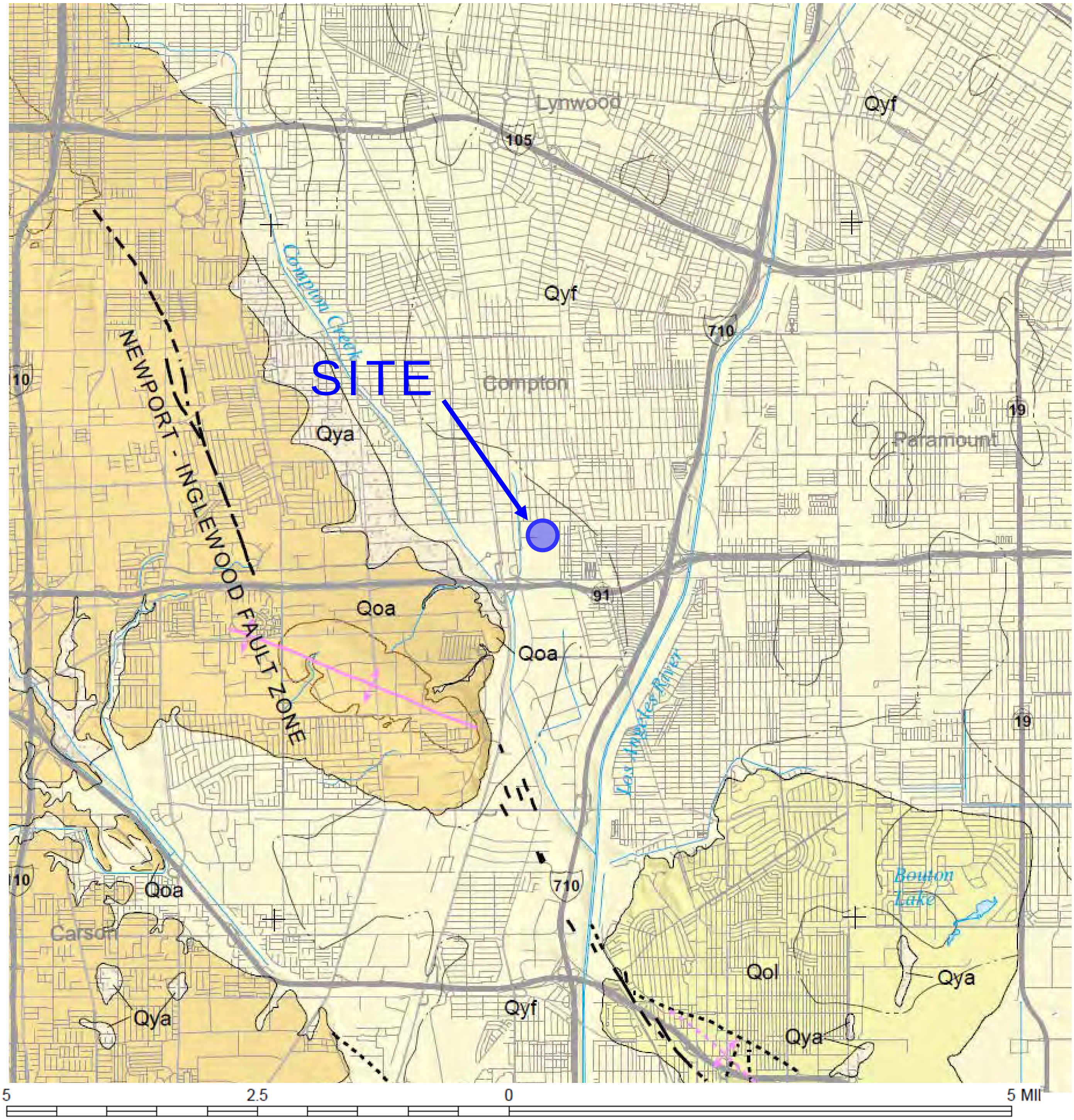
north

UNITED-HEIDER INSPECTION GROUP

Figure 2* – Boring Location Map

Proposed Instructional Building #2
Compton College Campus
1111 East Artesia Blvd.
Compton, CA 90221

Project No. 10-18020PW
Date: Feb. 2018



Qyf	**Young Alluvial Fan Deposits** - unconsolidated to slightly consolidated, undissected to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon

REFERENCE: CGS (2010) Geologic Compilation of Quaternary Surficial Deposits In Southern California Onshore Portion Of The Long Beach 30' X 60' Quadrangle; CGS Special Report 217, Plate 8.

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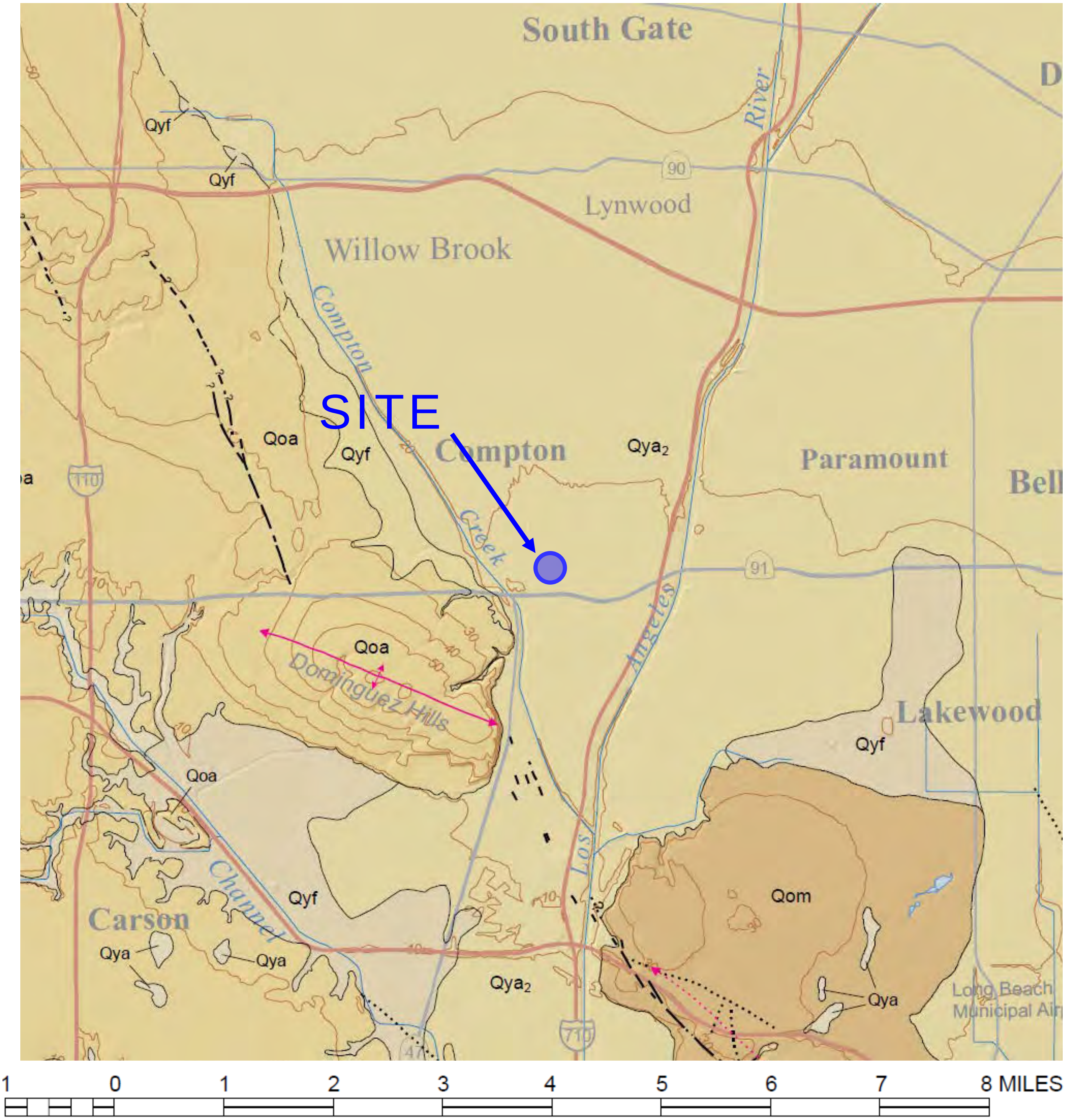
Figure 3A - Regional Geology Map 1

Proposed New Student Services Building
Compton College Campus
1111 East Artesia Blvd.
Compton, CA 90221

N

Project No. 10-18469PW

Date: Nov. 2018



Qya_2	Young alluvium, Unit 2

REFERENCE: CGS (2016) Geologic Map of the Long Beach 30'x60' Quadrangle, California.

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Figure 3B - Regional Geology Map 2

Proposed New Student Services Building
Compton College Campus
1111 East Artesia Blvd.
Compton, CA 90221

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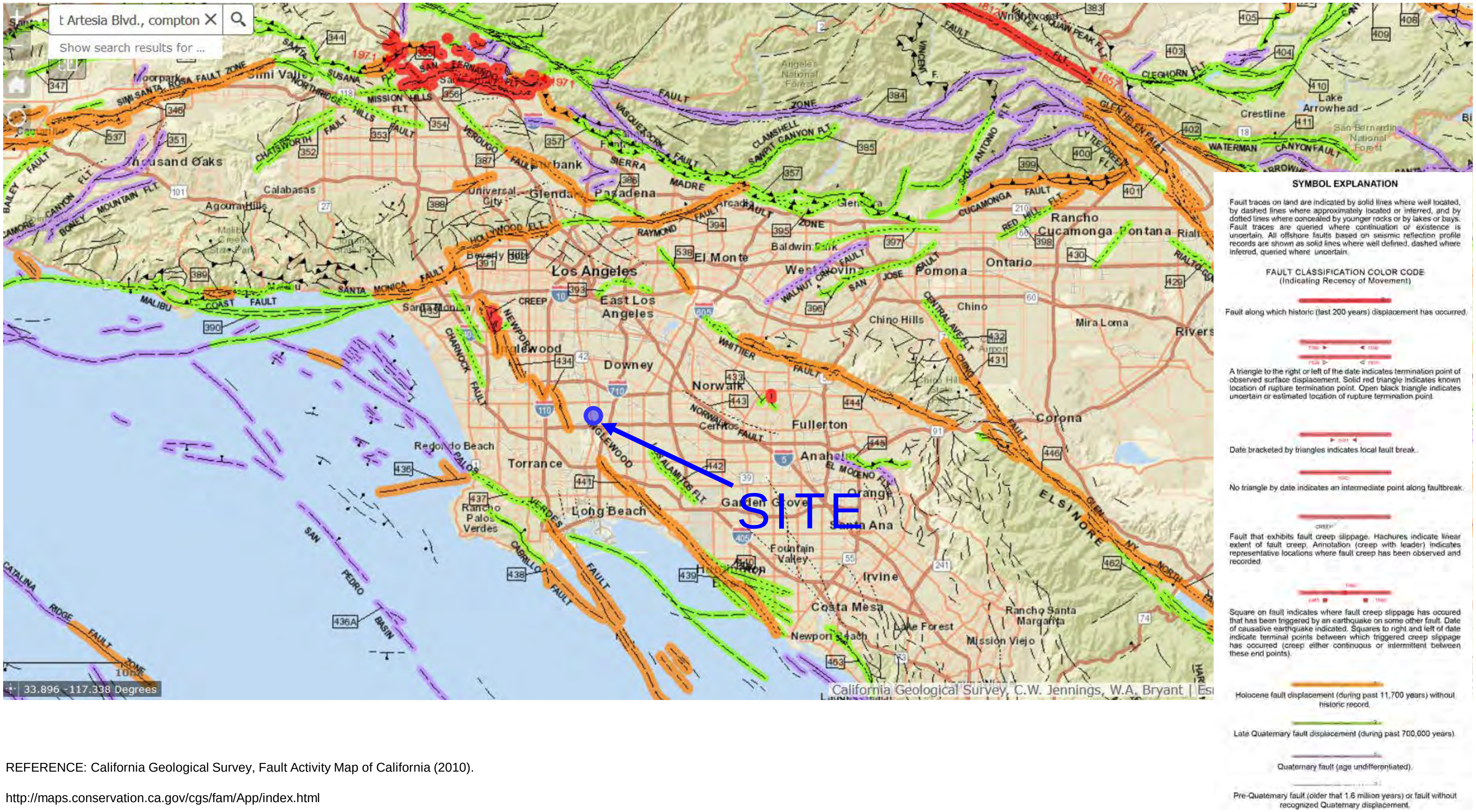
Project No. 10-18469PW

Date: Nov. 2018



REFERENCE: California Geological Survey, Fault Activity Map of California (2010).

http://maps.conservation.ca.gov/cgs/fam/App/index.html

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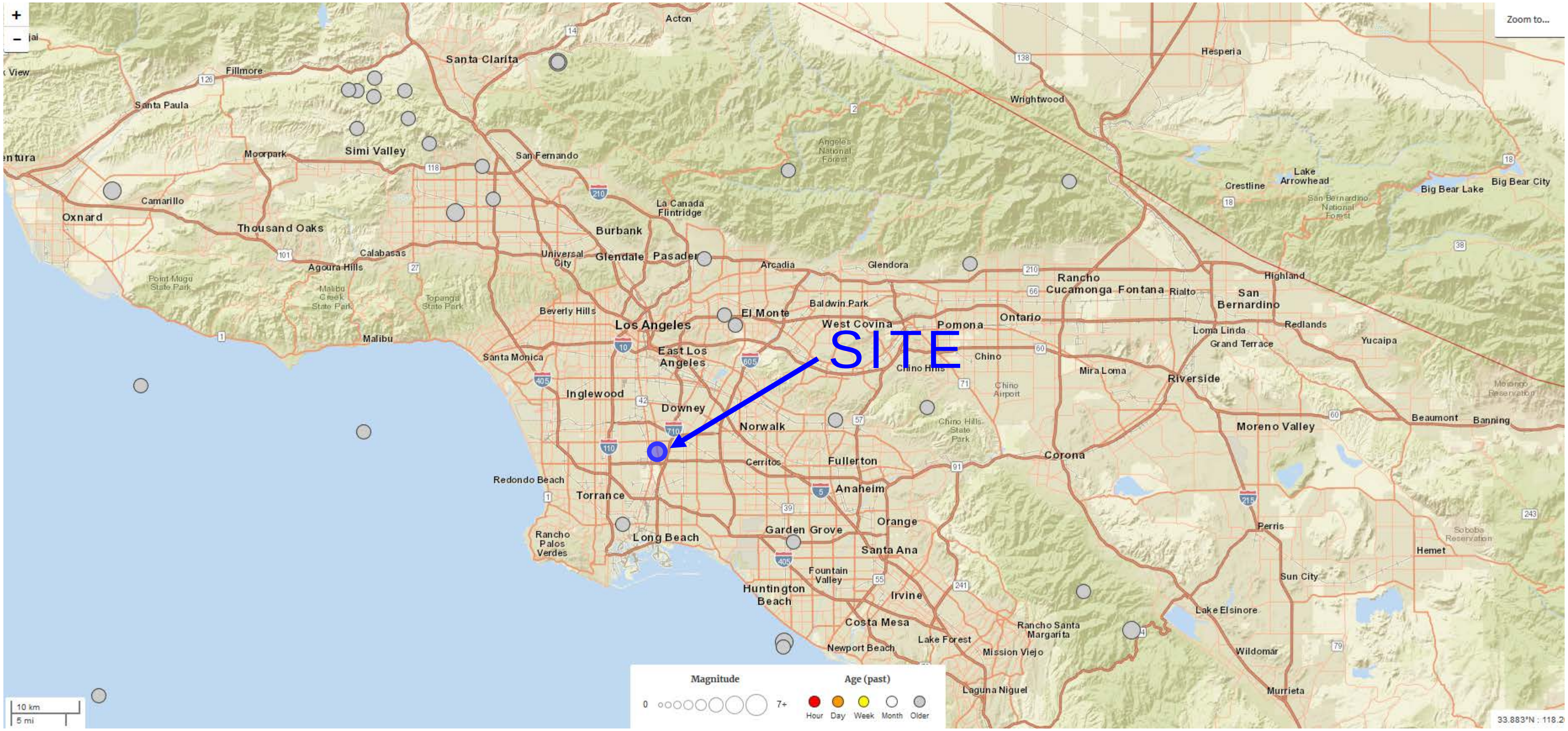
Figure 4 - Regional Fault Map

Proposed New Student Services Building
Compton College Campus
1111 East Artesia Blvd.
Compton, CA 90221

N

Project No. 10-18469PW

Date: Nov. 2018



REFERENCE: http://earthquake.usgs.gov/earthquakes

Location of Historic Earthquake Epicenter ($M_W > 5$)

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Figure 5 - Regional Seismicity Map

Proposed New Student Services Building
Compton College Campus
1111 East Artesia Blvd.
Compton, CA 90221

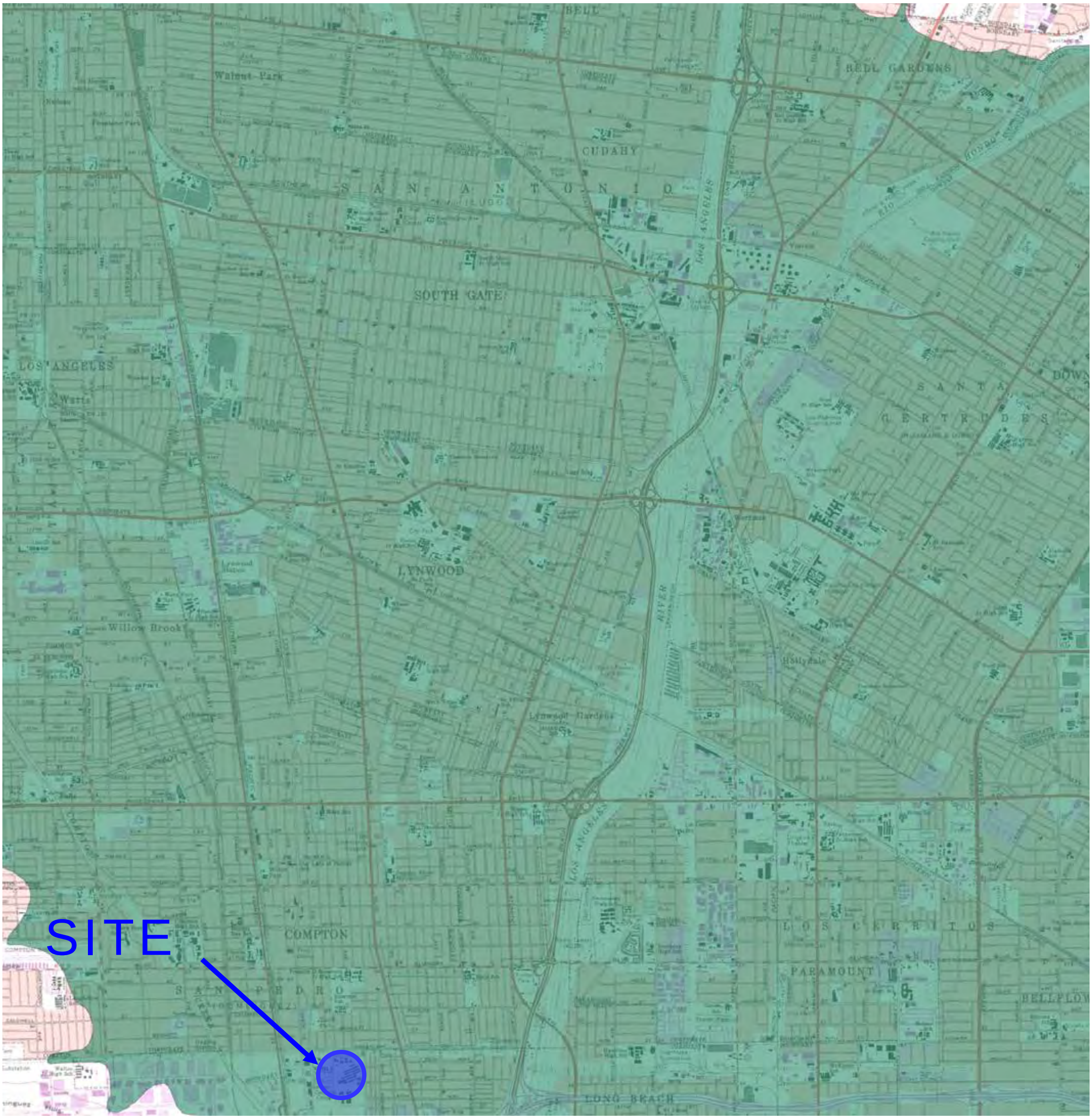
N

Project No. 10-18469PW

Date: Nov. 2018

Historic Seismicity (1900 to 2018)
Within 100 km Search Radius and M_W > 5.0
Proposed New Student Services Building, Compton College Campus
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



LIQUEFACTION

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

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

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

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Figure 6 – Liquefaction Susceptibility Map

Proposed New Student Services Building
Compton College Campus
1111 East Artesia Blvd.
Compton, CA 90221

N

Project No. 10-18469PW

Date: Nov. 2018

APPENDIX B

Field Exploration

FIELD EXPLORATION

The field investigation was performed on October 8, 2018 under the supervision of a United - Heider Inspection Groups' technical 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. United - Heider Inspection Group notified Underground Service Alert (USA) to identify the locations of subsurface utilities that may be in potential conflict with the boring locations.

Subsurface exploration included drilling and sampling of four hollow-stem auger borings (B-1 to B-4) to depths ranging from 26.5 feet to 51.5 feet. The borings were drilled using a CME - 75 drilling rig. Relatively undisturbed soils samples and Standard Penetration Tests (SPTs) 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 D 1586. The sampler was driven 18 inches into the subsurface soils using a 140-lb 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 loosely backfilled with soil cuttings. The boring records are presented in this Appendix.

UNITED-HEIDER INSPECTION GROUP
An ETS Company

United-Heider Inspection Group
22620 Goldencrest Drive, Suite 114, Moreno Valley, CA 92553
Main: (951) 697-4777 | Fax: (951) 697-4770

DATE OF DRILLING: 10/08/18 METHOD OF DRILLING: CME-75, Auto hammer; 8" Dia. Hollow Stem Auger

LOGGED BY: LM GROUND ELEVATION: NA LOCATION: See Fig. 2, Boring Location Map

Proposed Student Services Building
Compton College Campus
1111 East Artesia Blvd., Compton, CA 90221
BORING NO. **B-1**

DEPTH (FEET)	SAMPLE NUMBER	BLOWS/FOOT	RING SAMPLE	SPT SAMPLE	BULK SAMPLE	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	- # 200 (%)	SOIL DESCRIPTION	SOIL TEST
									Surficial Fill - 4" Grass and Top soil	
1									**Young Alluvial Fan Deposits (Qyf)**	
2										
3	S-1	9		X	X	10.4		67	@ 2.5': Sandy SILT (ML), loose, moist, tan brown fine sand, trace clay	
4										
5										
6	S-2	18		X		7.9		86	@ 5': Sandy SILT (ML), medium dense, moist, brown mostly fine sand	
7										
8	S-3	12		X		6.7		50	@ 5': Silty SAND (SM), medium dense, moist, light brown mostly fine sand	
10	S-4	13		X		15.4		91	@ 10': Silt (ML), stiff, moist, dark brown some fine sand	
11										
12										
13										
14										
15	S-4	18		X		9.5		46	@ 15': Silty SAND (SM), medium dense, moist, dark brown, mostly fine sand, trace clay	
16										
17										
18										
19										
20										

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Inspection Group
An ETS Company

United-Heider Inspection Group
22620 Goldencrest Drive, Suite 114, Moreno Valley, CA 92553
Main: (951) 697-4777 | Fax: (951) 697-4770

DATE OF DRILLING: 10/08/18 METHOD OF DRILLING: CME-75, Auto hammer; 8" Dia. Hollow Stem Auger

LOGGED BY: LM GROUND ELEVATION: NA LOCATION: See Fig. 2, Boring Location Map

Proposed Student Services Building
Compton College Campus
1111 East Artesia Blvd., Compton, CA 90221
BORING NO. **B-1**

DEPTH (FEET)	SAMPLE NUMBER	BLOWS/FOOT	RING SAMPLE	SPT SAMPLE	BULK SAMPLE	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	- # 200 (%)	SOIL DESCRIPTION	SOIL TEST
21	S-5	14		X		28.9		80	@ 20': Silt (ML), stiff, most, brown, trace fine sand trace clay	LL=38, PL=28 PI=10
25	S-6	15		X		36.7		76	@ 25': grades dark brown, some fine sand	
30	S-7	11		X		35.5		95	@ 30': grades dark gray	
35	S-8	14		X		18.7		94	@ 35': grades very stiff	LL=38, PL=28 PI=10
40										

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DATE OF DRILLING: 10/08/18 METHOD OF DRILLING: CME-75, Auto hammer; 8" Dia. Hollow Stem Auger

LOGGED BY: LM GROUND ELEVATION: NA LOCATION: See Fig. 2, Boring Location Map

Proposed Student Services Building
Compton College Campus
1111 East Artesia Blvd., Compton, CA 90221
BORING NO. **B-1**

DEPTH (FEET)	SAMPLE NUMBER	BLOWS/FOOT	RING SAMPLE	SPT SAMPLE	BULK SAMPLE	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	- # 200 (%)	SOIL DESCRIPTION	SOIL TEST
41	S-9	22		X		12		39	@ 40': Silty SAND (SM), medium dense, moist, light brown mostly fine to medium sand	
45	S-10	29	▼	X		36.9		79	@ 45': Sandy SILT (ML), medium dense, very moist, light brown	LL=NP, PL=NP PI=NP
50	S-11	37		X		22.2		11	@ 50': Poorly graded SAND with Silt (SP-SM), dense, moist, gray, mostly fine to medium sand	
55									- Total Depth of boring approx. 51.5 feet. - Groundwater encountered at 46 ft bgs. - Borehole was loosely backfilled with the soil cuttings.	
60										

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DATE OF DRILLING: 10/08/18 METHOD OF DRILLING: CME-75, Auto hammer; 8" Dia. Hollow Stem Auger

LOGGED BY: LM GROUND ELEVATION: NA LOCATION: See Fig. 2, Boring Location Map

Proposed Student Services Building
Compton College Campus
1111 East Artesia Blvd., Compton, CA 90221
BORING NO. **B-2**

DEPTH (FEET)	SAMPLE NUMBER	BLOWS/FOOT	RING SAMPLE	SPT SAMPLE	BULK SAMPLE	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	- # 200 (%)	SOIL DESCRIPTION	SOIL TEST
0					X				**Surficial Fill - 6" Grass and Top soil**	
1					X				**Young Alluvial Fan Deposits (Qyf)**	
3	R-1	34	X		X	15.2	95.7		@ 2.5': Sandy SILT (ML), stiff, moist, tan brown fine sand, trace clay	
5	S-2	9		X		6.4			@ 5': Silty SAND (SM), medium dense, moist, brown mostly fine sand, trace clay	
8	R-3	30	X			7.5	98.3		@ 7.5': grades same	
10	S-4	9		X		13.0			@ 10': grades same	
15	S-5	68	X			11.1	115.8		@ 15': Sandy SILT (ML), dense, moist, tan brown mostly fine sand, trace clay	
20										

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Main: (951) 697-4777 | Fax: (951) 697-4770

DATE OF DRILLING: 10/08/18 METHOD OF DRILLING: CME-75, Auto hammer; 8" Dia. Hollow Stem Auger

LOGGED BY: LM GROUND ELEVATION: NA LOCATION: See Fig. 2, Boring Location Map

Proposed Student Services Building
Compton College Campus
1111 East Artesia Blvd., Compton, CA 90221
BORING NO. **B-2**

DEPTH (FEET)	SAMPLE NUMBER	BLOWS/FOOT	RING SAMPLE	SPT SAMPLE	BULK SAMPLE	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	- # 200 (%)	SOIL DESCRIPTION	SOIL TEST
21	S-6	14		X		25			@ 20': SILT (ML), stiff, moist, brown, some fine sand, trace clay	
25	S-7	22	X			19.7	96.5		@ 25': grades very stiff	
30	S-8	11		X		33.5			@ 30': grades stiff, gray to light brown	
35	S-9	13		X		29.1			@ 35': Silty SAND (SM), medium dense, moist, gray mostly fine sand, trace clay	
40									- Total Depth of boring approx. 36.5 feet. - Groundwater was not encountered. - Borehole was loosely backfilled with the soil cuttings.	

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DATE OF DRILLING: 10/08/18 METHOD OF DRILLING: CME-75, Auto hammer; 8" Dia. Hollow Stem Auger

LOGGED BY: LM GROUND ELEVATION: NA LOCATION: See Fig. 2, Boring Location Map

Proposed Student Services Building
Compton College Campus
1111 East Artesia Blvd., Compton, CA 90221
BORING NO. **B-3**

DEPTH (FEET)	SAMPLE NUMBER	BLOWS/FOOT	RING SAMPLE	SPT SAMPLE	BULK SAMPLE	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	- # 200 (%)	SOIL DESCRIPTION	SOIL TEST
					X				**Surficial Fill - 4" Grass and Top soil**	
1					X				**Young Alluvial Fan Deposits (Qyf)**	
2					X					
3	S-1	9		X	X	15			@ 2.5': Sandy SILT (ML), loose, moist, light brown fine sand	
4					X					
5	R-2	26	X			8.1	92.7		@ 5': grades medium dense, brown	
6										
7										
8	S-3	11		X		5.7	91.2		@ 10': Silty SAND (SM), medium dense, moist, brown mostly fine sand	
9										
10	R-4	21	X			9.4			@ 10': Sandy SILT (ML), medium dense, moist, brown fine sand, trace clay	
11										
12										
13										
14										
15	S-5	17		X		12			@ 15': grades same	
16										
17										
18										
19										
20										

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DATE OF DRILLING: 10/08/18 METHOD OF DRILLING: CME-75, Auto hammer; 8" Dia. Hollow Stem Auger

LOGGED BY: LM GROUND ELEVATION: NA LOCATION: See Fig. 2, Boring Location Map

Proposed Student Services Building
Compton College Campus
1111 East Artesia Blvd., Compton, CA 90221
BORING NO. **B-3**

DEPTH (FEET)	SAMPLE NUMBER	BLOWS/FOOT	RING SAMPLE	SPT SAMPLE	BULK SAMPLE	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	- # 200 (%)	SOIL DESCRIPTION	SOIL TEST
20	R-6	20		X		28.7	94.6		@ 20': grades dark brown	
21										
25	S-7	13		X		15.0			@ 25': grades same	
									- Total Depth of boring approx. 26.5 feet. - Groundwater was not encountered. - Borehole was loosely backfilled with the soil cuttings.	
30										
35										
40										

UNITED-HEIDER INSPECTION GROUP
An ETS Company

United-Heider Inspection Group
22620 Goldencrest Drive, Suite 114, Moreno Valley, CA 92553
Main: (951) 697-4777 | Fax: (951) 697-4770

DATE OF DRILLING: 10/08/18 METHOD OF DRILLING: CME-75, Auto hammer; 8" Dia. Hollow Stem Auger

LOGGED BY: LM GROUND ELEVATION: NA LOCATION: See Fig. 2, Boring Location Map

Proposed Student Services Building
Compton College Campus
1111 East Artesia Blvd., Compton, CA 90221
BORING NO. **B-4**

DEPTH (FEET)	SAMPLE NUMBER	BLOWS/FOOT	RING SAMPLE	SPT SAMPLE	BULK SAMPLE	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	- # 200 (%)	SOIL DESCRIPTION	SOIL TEST
0					X				**Surficial Fill - 4" Grass and Top soil**	
1					X				**Young Alluvial Fan Deposits (Qyf)**	
2					X					
3	R-1	29	X		X	11	90.0		@ 2.5': Sandy SILT (ML), medium dense, moist, light brown fine sand	
4					X					
5	S-2	16		X		7.7			@ 5': grades same	
6										
7										
8	R-3	37	X			6.9	94.0		@ 10': Silty SAND (SM), medium dense, moist, light brown mostly fine sand	
9										
10	S-4	8		X		8.6			@ 10': grades loose	
11										
12										
13										
14										
15	R-5	61	X			9.5			@ 15': grades dense, some clay	
16										
17										
18										
19										
20										

United-Heider Inspection Group
22620 Goldencrest Drive, Suite 114, Moreno Valley, CA 92553
Main: (951) 697-4777 | Fax: (951) 697-4770

DATE OF DRILLING: 10/08/18 METHOD OF DRILLING: CME-75, Auto hammer; 8" Dia. Hollow Stem Auger

LOGGED BY: LM GROUND ELEVATION: NA LOCATION: See Fig. 2, Boring Location Map

Proposed Student Services Building
Compton College Campus
1111 East Artesia Blvd., Compton, CA 90221
BORING NO. **B-4**

DEPTH (FEET)	SAMPLE NUMBER	BLOWS/FOOT	RING SAMPLE	SPT SAMPLE	BULK SAMPLE	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	- # 200 (%)	SOIL DESCRIPTION	SOIL TEST
	S-6	15		X		25.4			@ 20': Clayey SILT (ML), stiff, moist, brown	
21										
25	R-7	35	X			26.0	94.4		@ 25': Sandy SILT (ML), medium dense, moist, light brown fine sand, trace clay	
30									- Total Depth of boring approx. 26.5 feet. - Groundwater was not encountered. - Borehole was loosely backfilled with the soil cuttings.	
35										
40										

Groundwater Level Data Report

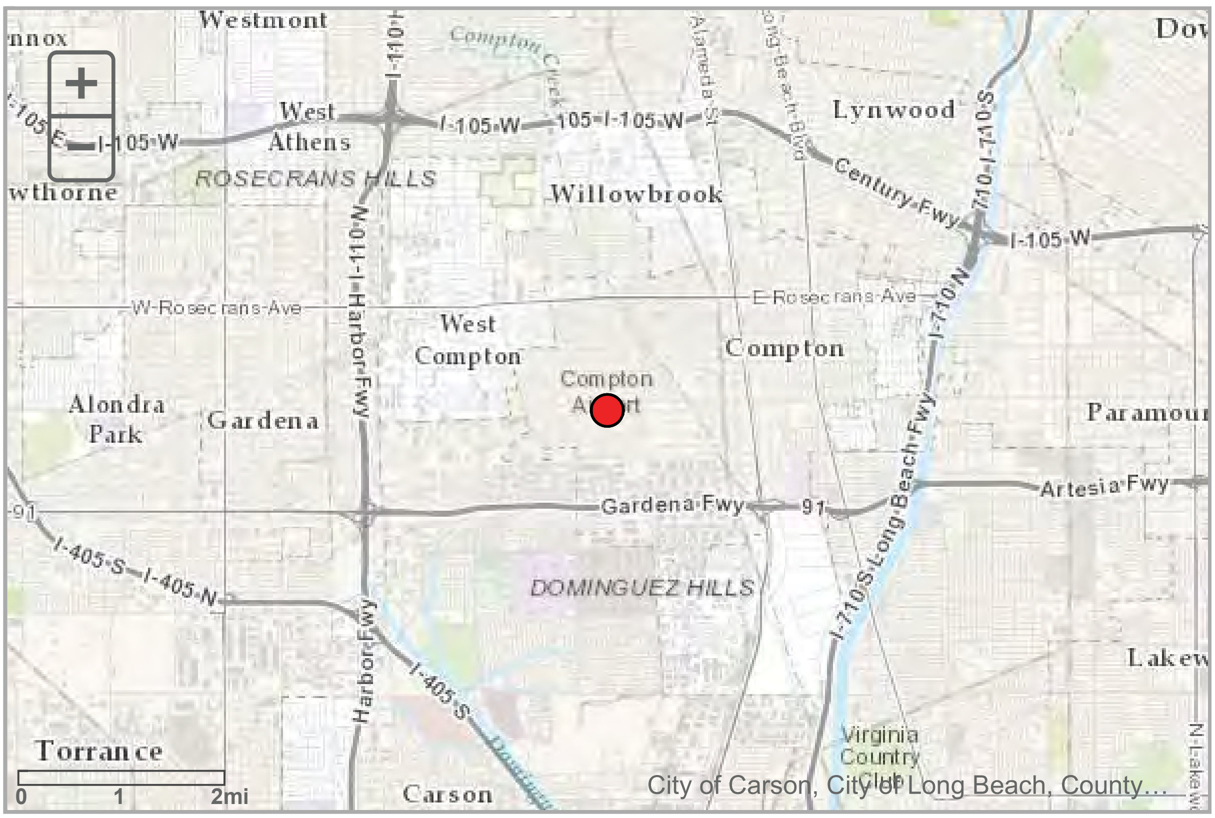
Groundwater Levels for Station 338872N1182432W001

Data for your selected well is shown in the tabbed interface below. To view data managed in the updated WDL tables, including data collected under the CASGEM program, click the "Recent Groundwater Level Data" tab. To view data stored in the former WDL tables, click the "Historical Groundwater Level Data" tab. To download the data in CSV format, click the "Download CSV File" button on the respective tab. Please note that the vertical datum for "recent" measurements is NAVD88, while the vertical datum for "historical" measurements is NGVD29. To change your well selection criteria, click the "Perform a New Well Search" button.

Station Data Recent Groundwater Level Data Historical Groundwater Level Data

State Well Number: 03S13W21R003S
Local Well ID:
Site Code: 338872N1182432W001
Latitude (NAD83): 33.887200
Longitude (NAD83): -118.2432
Groundwater Basin (code): Central (4-11.04)

Well Use: Unknown
Well Status: Active
Well Completion Report Number:
Reference Point Elevation (NAVD88 ft): 92.46
Ground Surface Elevation (NAVD88 ft): 92.46
Total Depth (ft): Confidential
Perforated Interval Depths (ft): Confidential

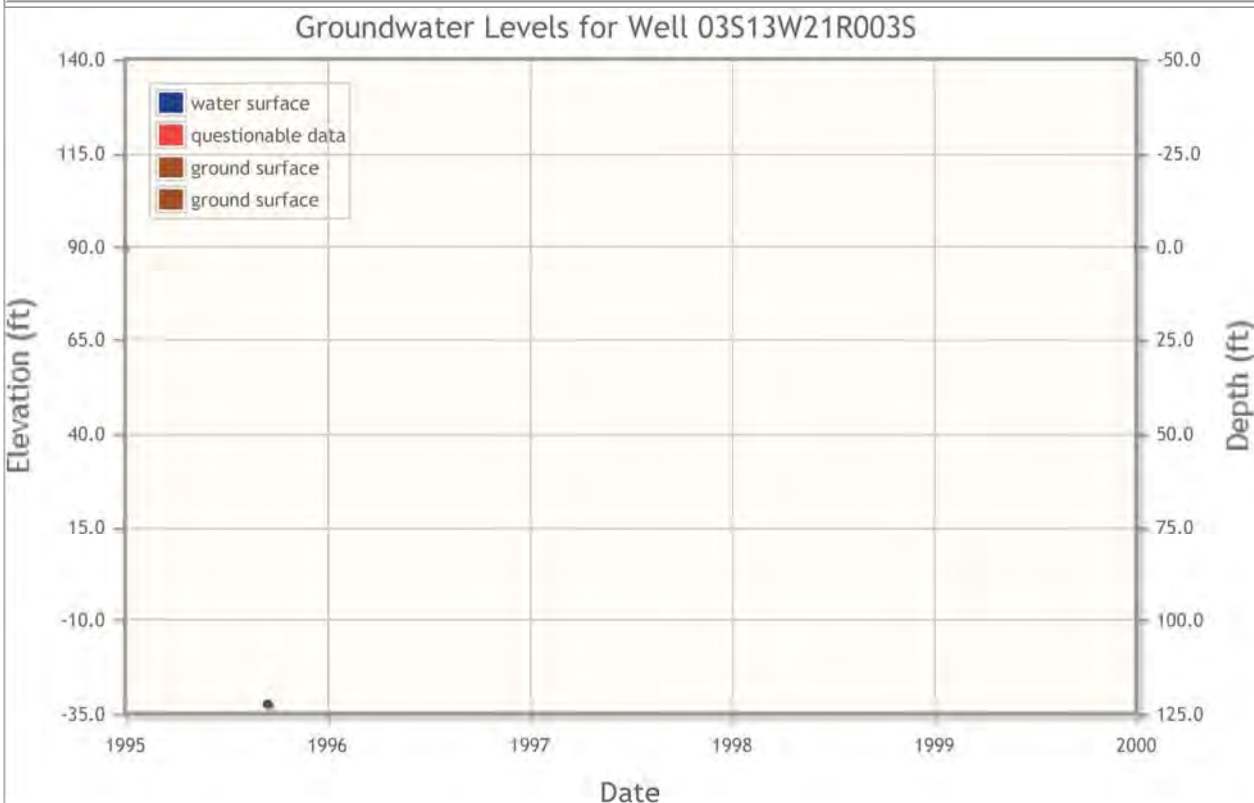


Perform a New Well Search

Groundwater Levels for Station 338872N1182432W001

Data for your selected well is shown in the tabbed interface below. To view data managed in the updated WDL tables, including data collected under the CASGEM program, click the "Recent Groundwater Level Data" tab. To view data stored in the former WDL tables, click the "Historical Groundwater Level Data" tab. To download the data in CSV format, click the "Download CSV File" button on the respective tab. Please note that the vertical datum for "recent" measurements is NAVD88, while the vertical datum for "historical" measurements is NGVD29. To change your well selection criteria, click the "Perform a New Well Search" button.

Station Data Recent Groundwater Level Data **Historical Groundwater Level Data**



Download CSV File

Date	RPE	GSE	RPWS	WSE	GS to...	NM Code	QM Code	Agency
09/14/1995 00:00	90	90	122.45	-32.45	122.45			5000

All elevation and depth measurements are in feet. The vertical datum for historical measurements is NGVD29.

Perform a New Well Search

APPENDIX C

Laboratory Test Procedures and Test Results

LABORATORY TESTING - GENERAL

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 in the United - Heider Inspection Group, Inc. laboratories in Moreno Valley, LOR Geotechnical in Riverside, and Hilltop Geotechnical, Inc. in San Bernardino, California. Based on our review of the laboratory data, the undersigned engineers concur with and accept the laboratory testing results.

Brief descriptions of the testing are presented in the following sections.

MOISTURE CONTENT AND DRY DENSITY

The moisture content and dry unit weight were determined for selected soil samples in general accordance with ASTM D 2216 and ASTM D 2937, respectively. The moisture content and dry unit weight are presented on the boring logs at the corresponding sample depths.

SIEVE ANALYSIS

Selected soil samples were tested to determine the quantitative determination of the distribution of particle sizes in soils (particle sizes larger than 75 micrometers) in general accordance with ASTM D 422. The results of the Sieve analyses are presented in this Appendix.

WASH SIEVE ANALYSIS

Selected soil samples were tested to determine the percent fines (the percentage of soil passing the Standard No. 200 sieve) in general accordance with ASTM D 1140. 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 D 3080 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

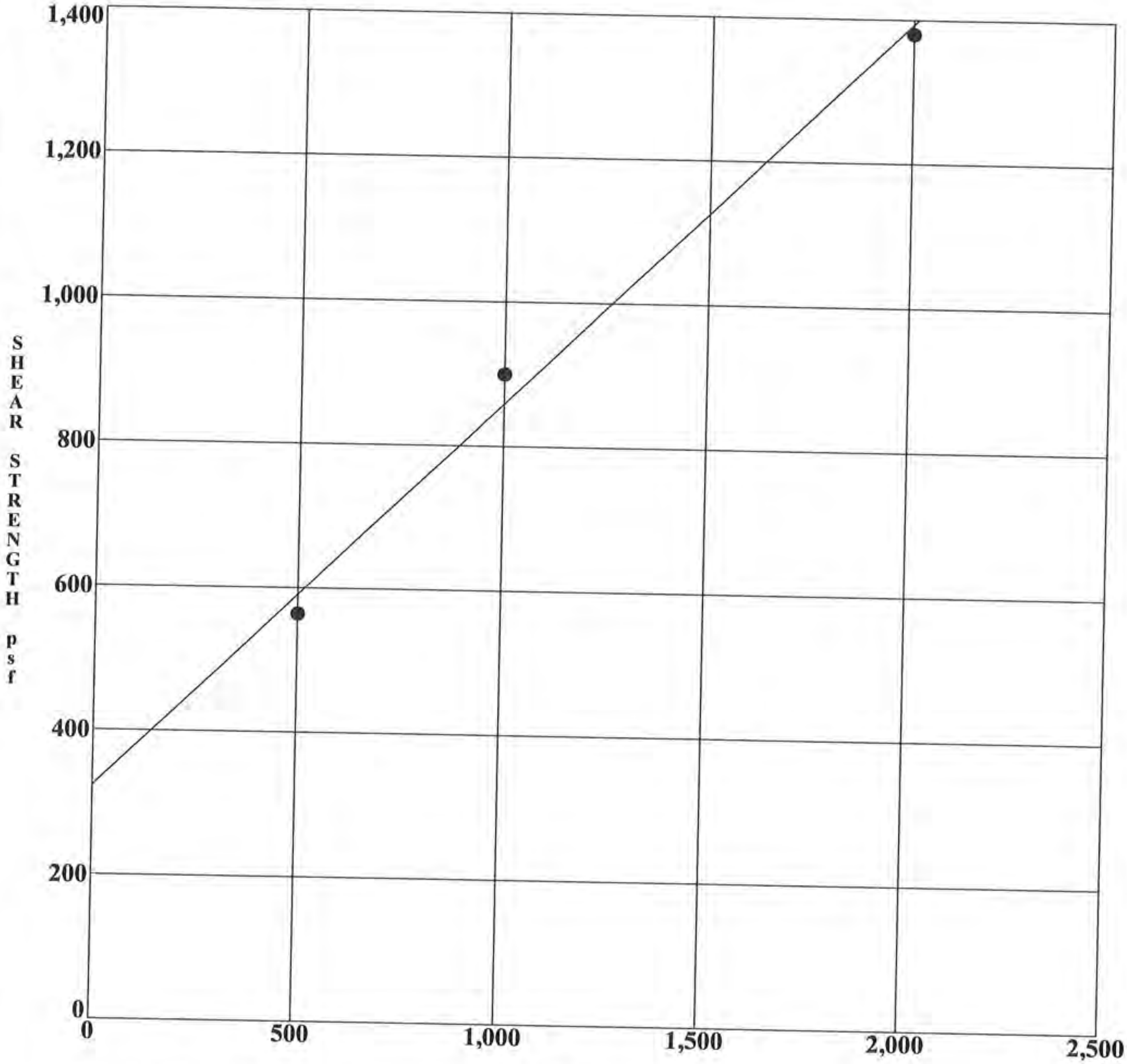
Corrosivity tests were performed on a selected bulk sample to evaluate minimum resistivity, pH, water-soluble sulfates and chlorides (CTMs 643, 417 and 422 respectively). Test results are attached in this appendix.

EXPANSION INDEX TEST

Expansion Index tests were performed on selected bulk samples in general accordance with ASTM D 4829 to evaluate the expansion potential of the onsite 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 Test Method D1557. Test results and a graphical plot of maximum density vs. optimum moisture content are attached in this appendix.



Cohesion Calc. (psf): 324
Friction Angle Calc. (°): 28

	Specimen Identification	Classification	DD	MC%
●	10-18469 B4 2.5	tan sandy SILT	106	13

PROJECT *Misc Lab Testing*

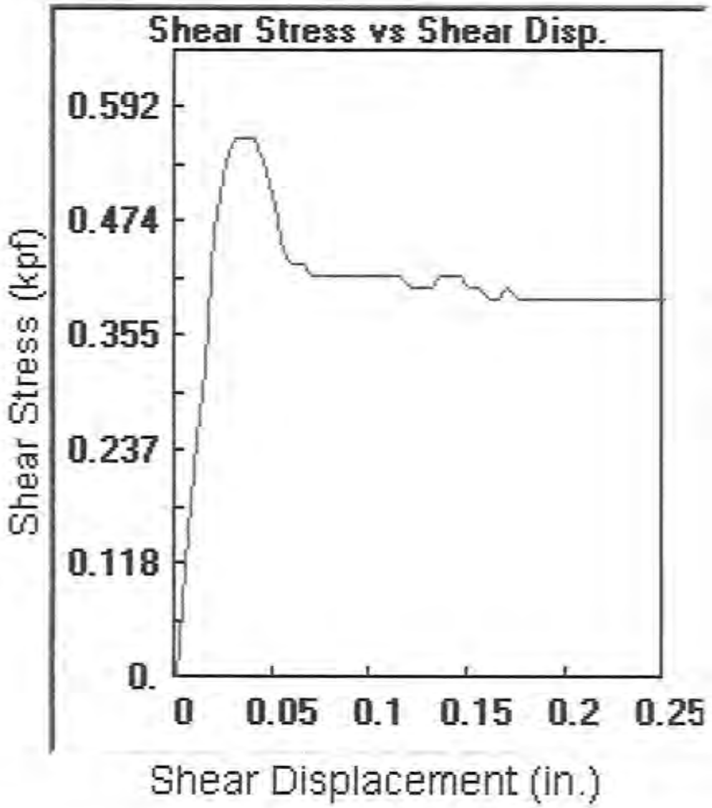
PROJECT NO. 60750.9

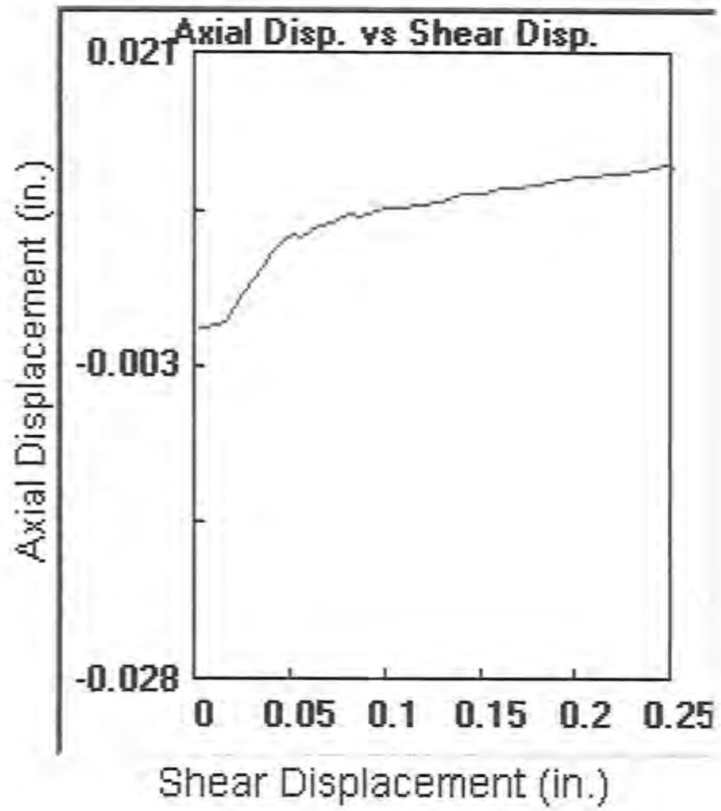
DATE 10/19/18

SHEAR TEST DIAGRAM

LOR Geotechnical Group, Inc.

Riverside, CA 92507





Parameters

Client: UNITED-HEIDER INSPECTION GROUP

Location: COMPTON COLLEGE NEW STUDENT SERVICE

Job # 10-18469	Soil Type:
Sample: B-4@2.5'	Technician: MARK
Boring: B-4	Axial Load: 500 psf
Depth: 2.5 ft.	Shear Rate: 0.01 in./min.
File:	Distance: 0.25 in.
Stress at Max Def 564 0.03	Stress at Max Disp 0.245 396

Maximum Load

564 psf

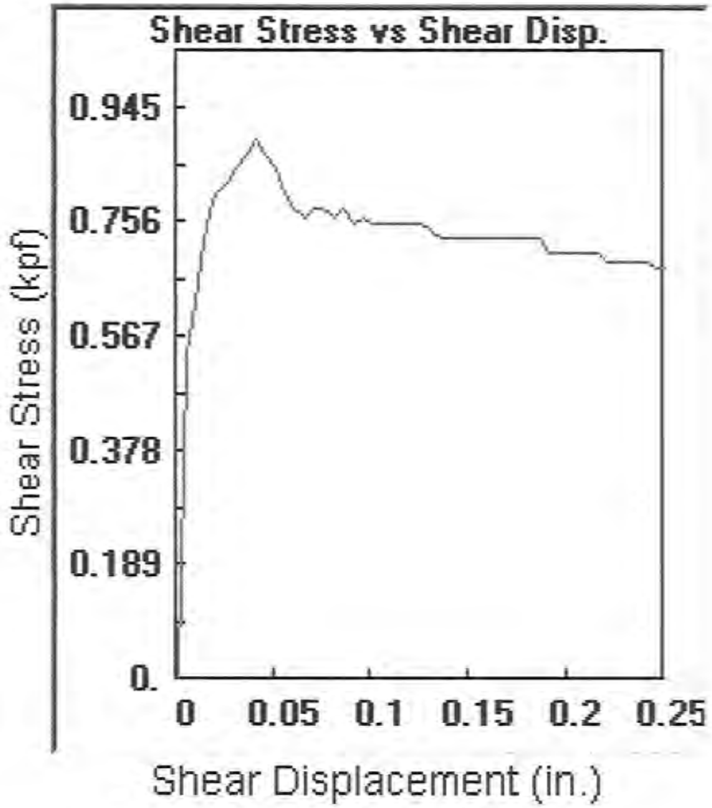
Shear Displacement at maximum Load

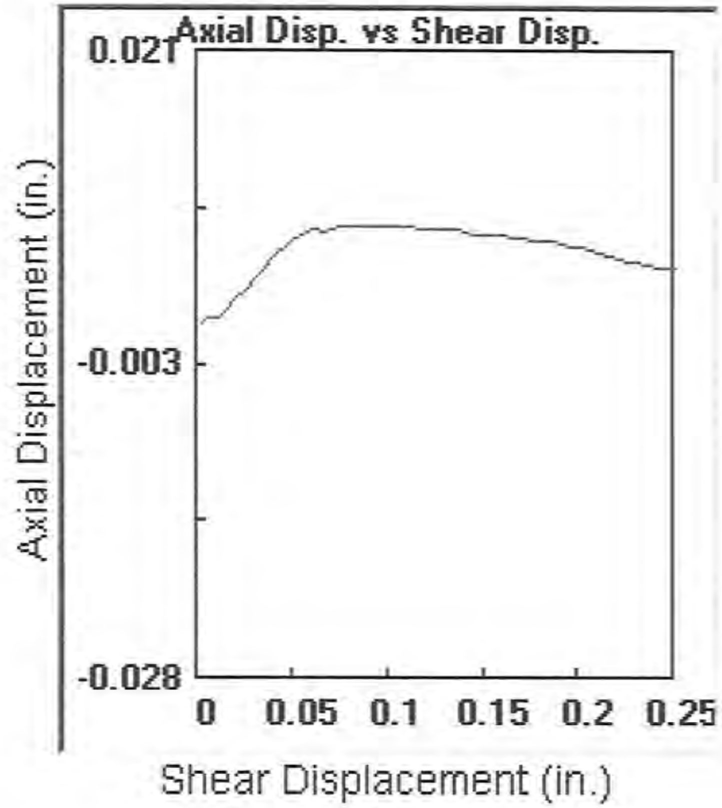
0.0300 in.

Date

10/17/2018

LOR Geotechnical





Parameters

Client: UNITED-HEIDER INSPECTION GROUP

Location: COMPTON COLLEGE NEW STUDENT SERVICE

Job # 10-18469

Soil Type:

Sample: B-4@2.5'

Technician: MARK

Boring: B-4

Axial Load: 1000 psf

Depth: 2.5 ft.

Shear Rate: 0.01 in./min.

File: 60750-10-18469-B4-1K.dat

Distance: 0.25 in.

Stress at Max Def
900 0.04

Stress at Max Disp
0.245 684

Maximum Load

900 psf

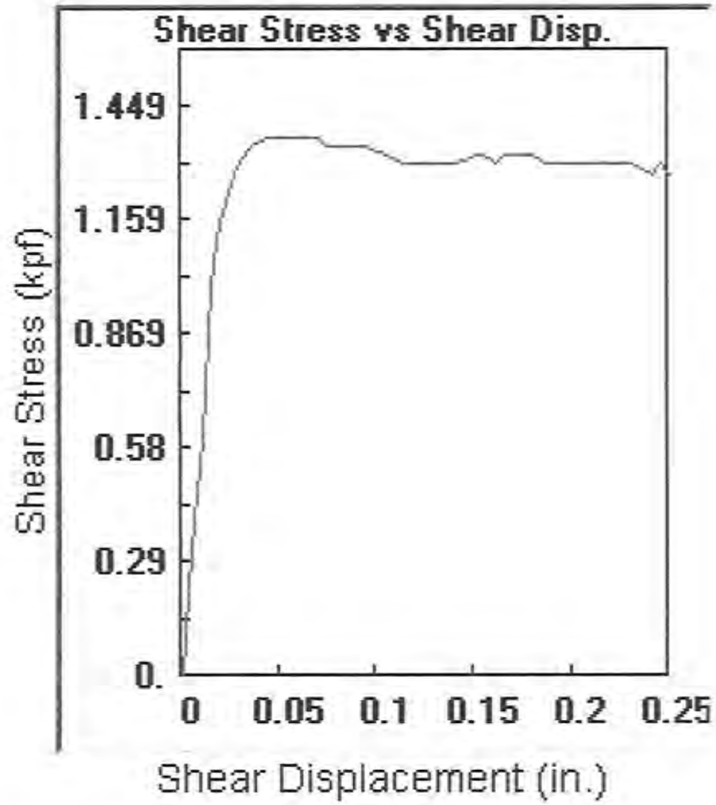
Shear Displacement at maximum Load

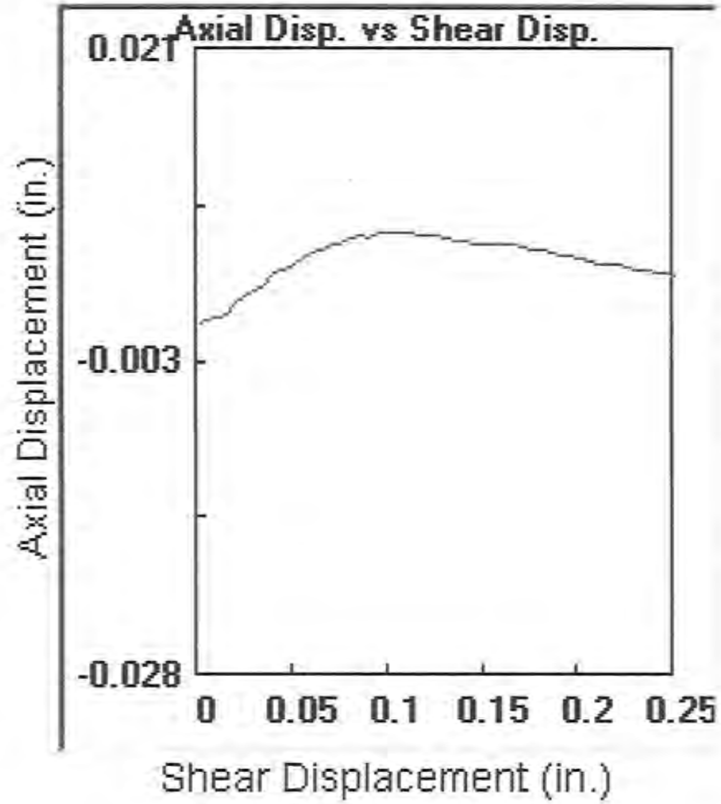
0.0400 in.

Date
10/17/2018

LOR Geotechnical

V71





Parameters

Client: UNITED-HEIDER INSPECTION GROUP

Location: COMPTON COLLEGE NEW STUDENT SERVICE

Job # 10-18469	Soil Type:
Sample: B-4@2.5'	Technician: MARK
Boring: B-4	Axial Load: 2000 psf
Depth: 2.5 ft.	Shear Rate: 0.01 in./min.
File: 60750-10-18469-B4-2K.dat	Distance: 0.25 in.
Stress at Max Def 1380 0.045	Stress at Max Disp 0.245 1308

Maximum Load

1380 psf

Shear Displacement at maximum Load

0.0450 in.

Date

10/17/2018

LOR Geotechnical

UNITED-HEIDER
INSPECTION GROUP
An ATLAS Company
22620 Goldencrest Drive, Suite 114 | Moreno Valley, California 92553
P: 951.697.4777 | F: 951.653.1143 | www.united-heider.com

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

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

Project: New Student Services Building
1111 E. Artesia Boulevard
Compton, CA 90221

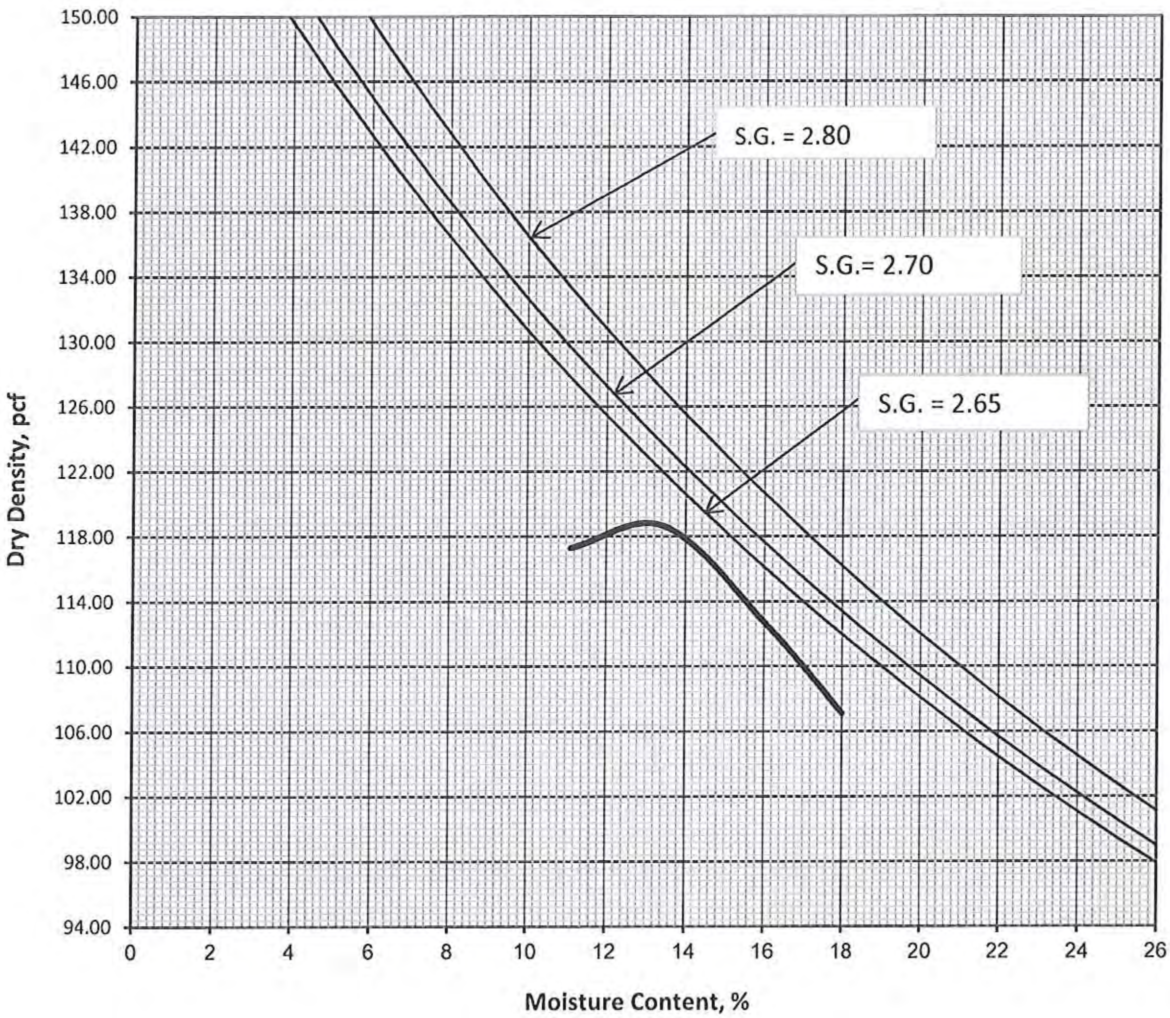
DSA File No.: N/A
Dsa App No.: N/A

Date: October 10, 2018

United-Heider Inspection Group File No.: 10-18469

Lab Sample No.:	10S181012-1		
Visual Class.:	Tan Slightly Sandy SILT with little Pea gravel	**Test Results:**	
Sample Source:	B-4 @ 2.5'	**Maximum Dry Density, pcf:**	118.4
Method of Test:	ASTM D 1557	**Optimum Moisture Content, %:**	13.5

Maximum Density - Optimum Moisture Content, ASTM D 1557



UNITED-HEIDER INSPECTION GROUP
An ATLAS Company
22620 Goldencrest Drive, Suite 114 | Moreno Valley, California 92553
P: 951.697.4777 | F: 951.653.1143 | www.united-heider.com

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

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

Project: New Student Services Building
1111 E. Artesia Boulevard
Compton, CA 90221

DSA File No.: N/A
Dsa App No.: N/A

Date: October 10, 2018

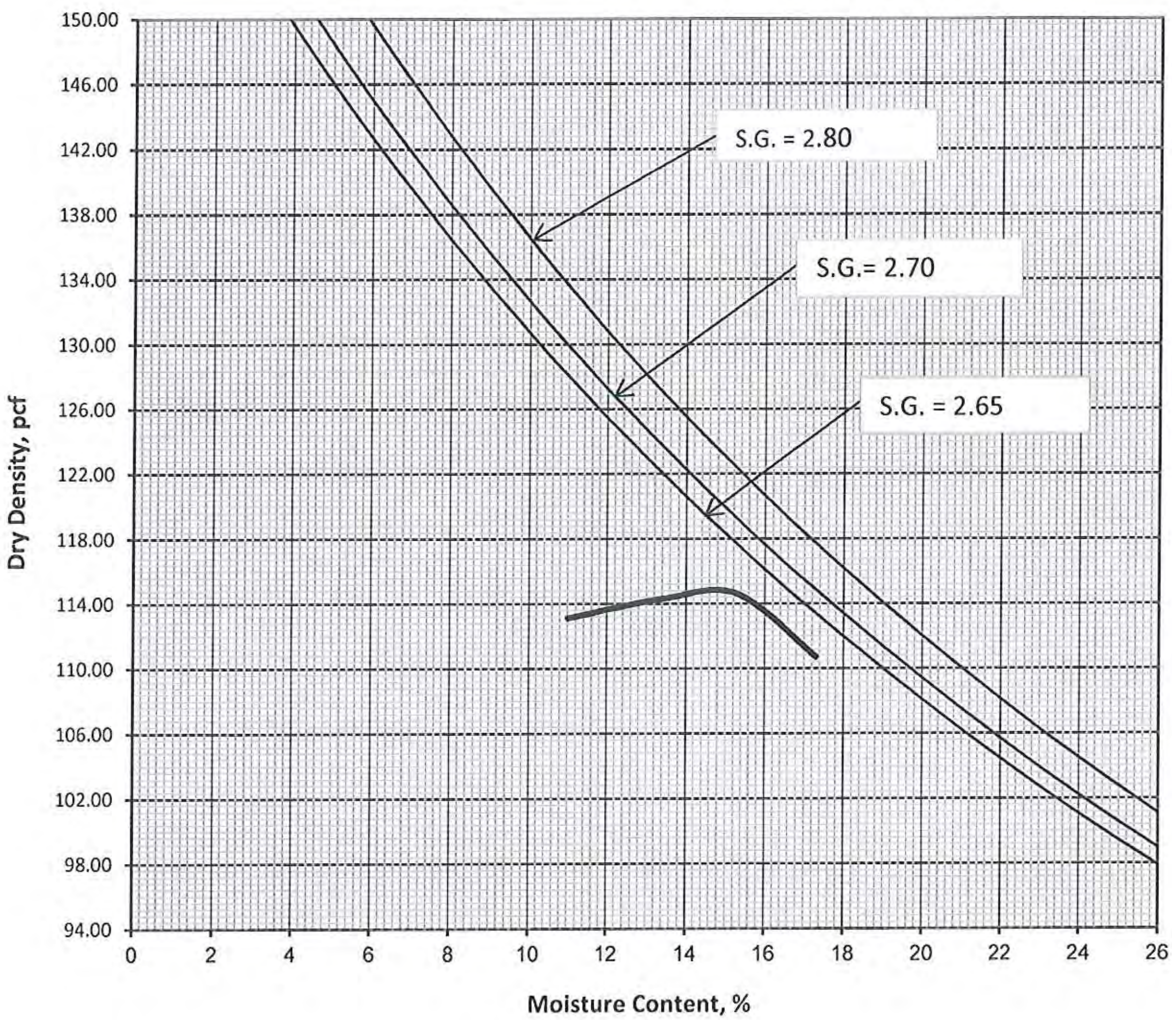
United-Heider Inspection Group File No.: 10-18469

Lab Sample No.:	10S181012-1
Visual Class.:	Tan Slightly Sandy SILT
Sample Source:	B-4 @ 10'
Method of Test:	ASTM D 1557

Test Results:

Maximum Dry Density, pcf:	114.6
Optimum Moisture Content, %:	15.3

Maximum Density - Optimum Moisture Content, ASTM D 1557



November 7, 2018
Project#: 10-18469
UHIG Lab #: 10S181106-1

COMPTON COMMUNITY COLLEGE DIST.
1111 E. Artesia Blvd.
Compton, CA. 90221

Subject: **ATTERBERG LIMIT DETERMINATION – ASTM D4318**
Compton Community College – New Student Services - Geotechnical
1111 E. Artesia Blvd.
Compton, CA. 90221

Presented below are the results of laboratory testing performed by United - Heider Inspection Group for the subject project to determine the Atterberg Limit Determinations of the soil samples submitted.

B-4 5'	**B-4 10'**	**B-1 20'**	**B-1 35'**	**B-1 45'**
LL = NP	LL = NP	LL = 38	LL = 38	LL = NP
PL = NP	PL = NP	PL = 28	PL = 28	PL = NP
PI = NP	PI = NP	PI = 10	PI = 10	PI = NP

Respectfully submitted,
United - Heider Inspection Group

REGISTERED PROFESSIONAL ENGINEER
DENNIS W. HEIDER
C 47379
EXP. 12/31/2019
CIVIL
STATE OF CALIFORNIA

Dennis W. Heider, RCE
Registered Civil Engineer

UNITED-HEIDER
INSPECTION GROUP
An ATLAS Company

October 19, 2018
Project#: 10-18469
UHIG Lab #: 10S181018-1

COMPTON COMMUNITY COLLEGE DIST.
1111 E. Artesia Blvd.
Compton, CA. 90221

Subject: **EXPANSION INDEX TEST RESULTS**
Compton Community College – Public Safety Bldg.
1111 E. Artesia Blvd.
Compton, CA. 90221

Presented below are the results of laboratory testing performed by United - Heider Inspection Group for the subject project to determine the expansion index of a soil sample submitted.

Sample Description:	Brn Clayey Silt w/Trace of Sand – (ML)	Sample Location:	B-4 @ 2.5
Date Sampled:	10/8/18	Date Tested:	10/17/18
Sampled By:	Luis Mondragon	Tested By:	Mark Tabb

EXPANSION INDEX TEST RESULTS
ASTM Test Method D4829

Expansion Index B – 4 @ 2.5	56 = Medium

Respectfully submitted,
United - Heider Inspection Group

Dennis W. Heider, RCE
Registered Civil Engineer

22620 Goldencrest Drive, Suite 114 • Moreno Valley, CA 92553 • Ph: 951.697.4777 • Fax: 951.653-1144

BABCOCK Laboratories, Inc.
The Standard of Excellence for Over 100 Years

Client Name:	United-Heider Inspection Group	Analytical Report:	Page 1 of 3
Contact:	Bob Russell	Project Name:	United-Heider Inspection - Soils
Address:	22620 Goldencrest Drive, Ste. 114 Moreno Valley, CA 92553	Project Number:	New Student Services
Report Date:	26-Oct-2018	**Work Order Number:**	**B8J2093**

Received on Ice (Y/N): No Temp: 28 °C

Attached is the analytical report for the sample(s) received for your project. Below is a list of the individual sample descriptions with the corresponding laboratory number(s). Also, enclosed is a copy of the Chain of Custody document (if received with your sample(s)). Please note any unused portion of the sample(s) may be responsibly discarded after 30 days from the above report date, unless you have requested otherwise.

Thank you for the opportunity to serve your analytical needs. If you have any questions or concerns regarding this report please contact our client service department.

Sample Identification

Lab Sample #	Client Sample ID	Matrix	Date Sampled	By	Date Submitted	By
B8J2093-01	B-4@ 2.5'	Soil	10/08/18 12:00	Luis Mondragon	10/12/18 15:43	Luis Mondragon

BABCOCK Laboratories, Inc.
The Standard of Excellence for Over 100 Years

Client Name:	United-Heider Inspection Group	Analytical Report:	Page 2 of 3
Contact:	Bob Russell	Project Name:	United-Heider Inspection - Soils
Address:	22620 Goldencrest Drive, Ste. 114 Moreno Valley, CA 92553	Project Number:	New Student Services
Report Date:	26-Oct-2018	**Work Order Number:**	**B8J2093**

Received on Ice (Y/N): No Temp: 28 °C

Laboratory Reference Number

B8J2093-01

Sample Description	Matrix	Sampled Date/Time	Received Date/Time
B-4@ 2.5'	Soil	10/08/18 12:00	10/12/18 15:43

Analyte(s)	Result	RDL	Units	Method	Analysis Date	Analyst	Flag
Saturated Paste							
pH	7.7	0.1	pH Units	S-1.10 W.S.	10/18/18 14:55	TML	
Saturated Extract							
Saturated Resistivity	2500	5	ohm-cm	SM 2520B	10/18/18 14:55	TML	
Water Extract							
Chloride	21	10	ppm	Ion Chromat.	10/15/18 23:44	KBS	N_WEX
Sulfate	48	10	ppm	Ion Chromat.	10/15/18 23:44	KBS	N_WEX

BABCOCK Laboratories, Inc.
The Standard of Excellence for Over 100 Years

Client Name:	United-Heider Inspection Group	Analytical Report:	Page 3 of 3
Contact:	Bob Russell	Project Name:	United-Heider Inspection - Soils
Address:	22620 Goldencrest Drive, Ste. 114 Moreno Valley, CA 92553	Project Number:	New Student Services
Report Date:	26-Oct-2018	**Work Order Number:**	**B8J2093**
		Received on Ice (Y/N):	No Temp: 28 °C

Notes and Definitions

N_WEX	Analyte determined on a 1:10 water extract from the sample.
ND:	Analyte NOT DETECTED at or above the Method Detection Limit **(if MDL is reported)**, otherwise at or above the Reportable Detection Limit (RDL)
NR:	Not Reported
RDL:	Reportable Detection Limit
MDL:	Method Detection Limit
* / "' :	NELAP does not offer accreditation for this analyte/method/matrix combination

Approval

Enclosed are the analytical results for the submitted sample(s). Babcock Laboratories certify the data presented as part of this report meet the minimum quality standards in the referenced analytical methods. Any exceptions have been noted.

K. Marshall **KayeLani A. Marshall**

cc:

e-Short_No Alias.rpt

mailing
P.O Box 432
Riverside, CA 92502-0432

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Riverside, CA 92507-0704

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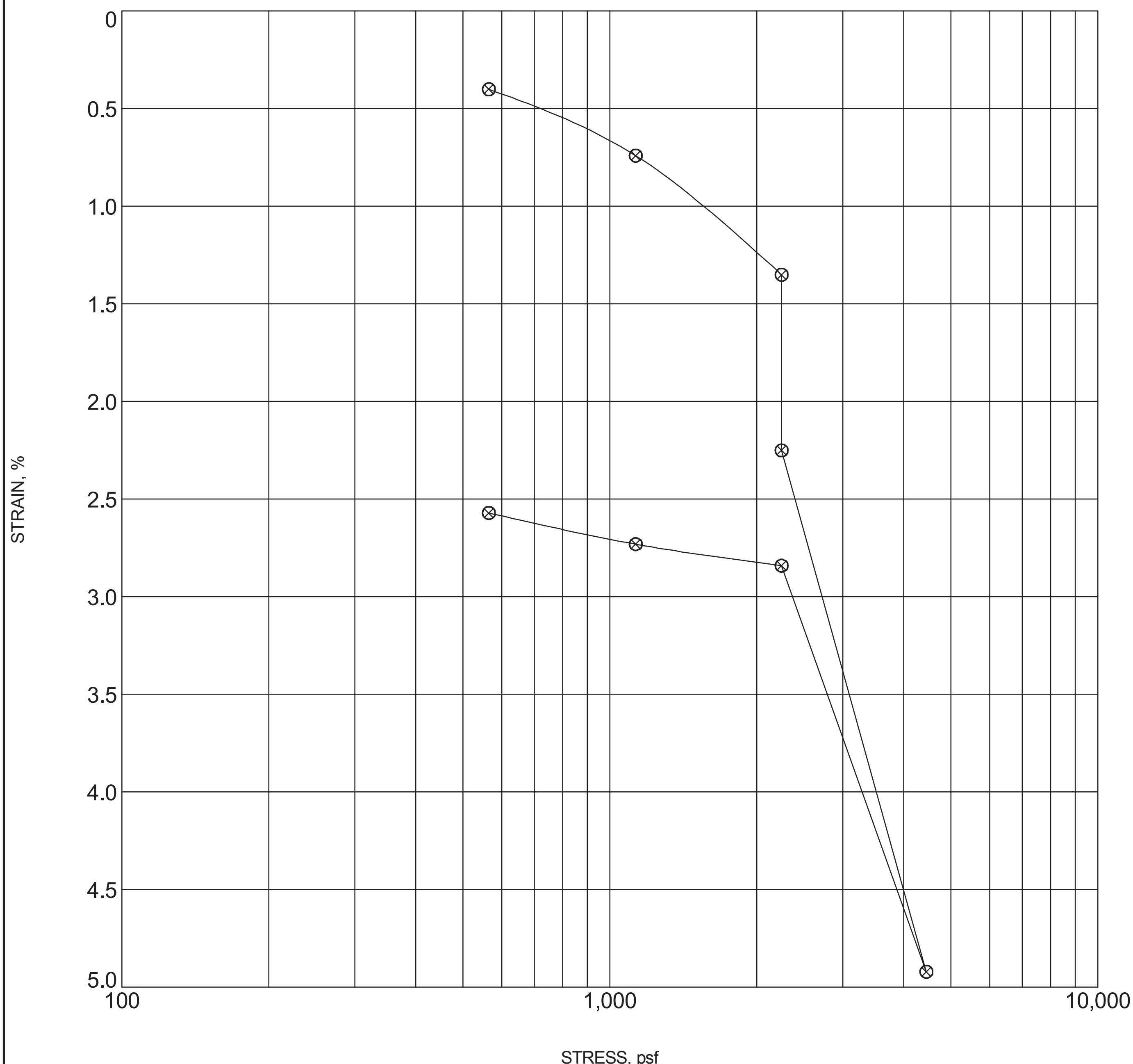
www.babcocklabs.com

CA ELAP No. 2698
EPA No. CA00102
NELAP No. OR4035
LACSD No. 10119

Heider Inspection Group - An ETS Company
800 S Rochester Ave, Ste A
Ontario, CA 91761
Office: 909-673-0292; Fax: 909-673-0272

À !" #$%&' $!Ā' ("'
For Collapse Potential

À#$(!'	Compton College Campus	)* +(À'Ā !&-(	Proposed Instructional Building I
)* +(À'Ā!,-.(*	HE15281-2	) * +(À'Ā# À&'$!	1111 E Artesia Blvd, Compton, CA 90221



Hydro-Collapse Test from Heider Inspection Group (2015)

	Specimen Identification	Classification	γ_d	MC%
⊗	./ 0 H 123	5' - 6Ā'$#' 7Ā'&! %8Ā9:;< =Ā?@Ā@AB?2	43	CD

APPENDIX D
Calculations

USGS Design Maps Detailed Report

ASCE 7-10 Standard (33.8787°N, 118.20931°W)

Site Class D – "Stiff Soil", Risk Category IV (e.g. essential facilities)

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1]	S_S = 1.674 g
From Figure 22-2 [2]	S_1 = 0.611 g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index *PI* > 20, • Moisture content *w* ≥ 40%, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk–Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4–1: Site Coefficient F_a

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = D and S_S = 1.674 g, F_a = 1.000

Table 11.4–2: Site Coefficient F_v

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at 1–s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.611 g, F_v = 1.500

Equation (11.4–1): $S_{MS} = F_a S_S = 1.000 \times 1.674 = 1.674$ g

Equation (11.4–2): $S_{M1} = F_v S_1 = 1.500 \times 0.611 = 0.916$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

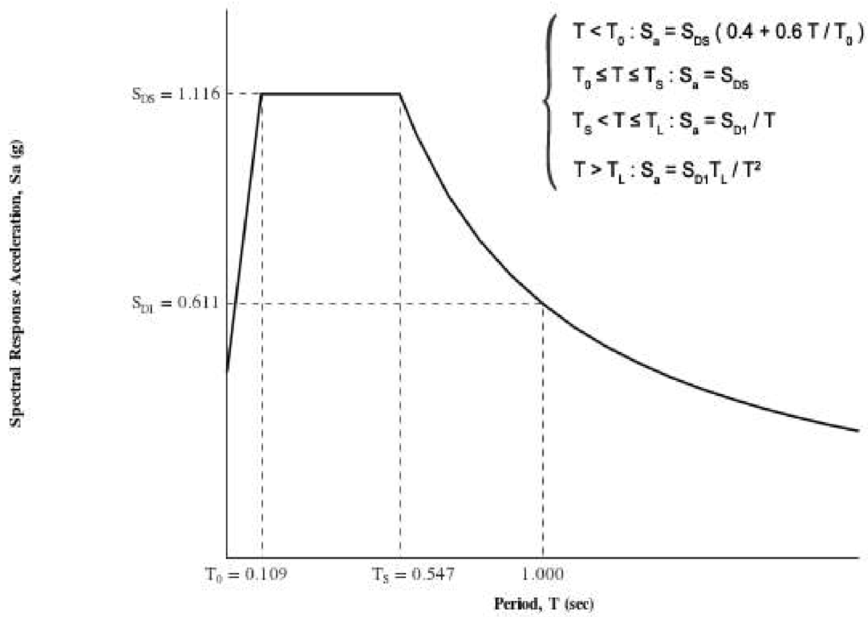
Equation (11.4–3): $S_{DS} = ⅔ S_{MS} = ⅔ \times 1.674 = 1.116$ g

Equation (11.4–4): $S_{D1} = ⅔ S_{M1} = ⅔ \times 0.916 = 0.611$ g

Section 11.4.5 — Design Response Spectrum

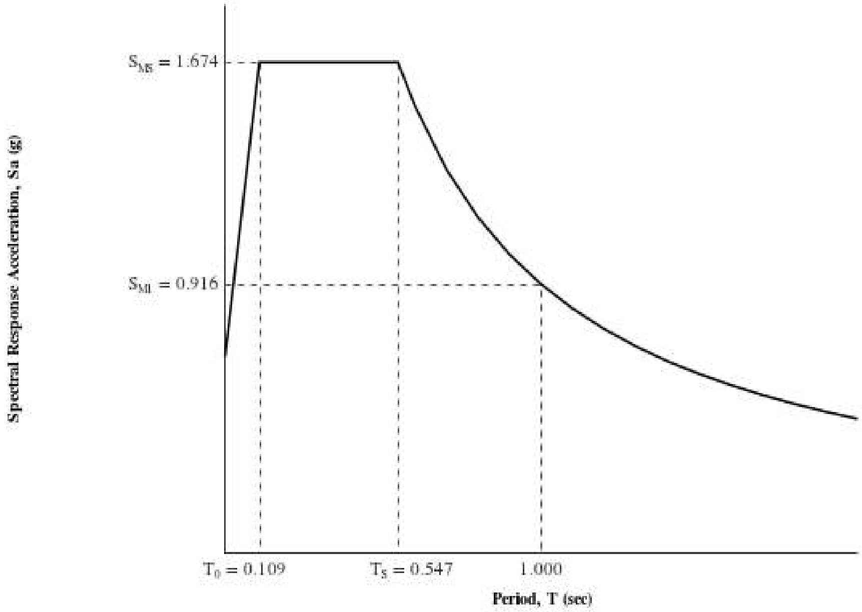
From Figure 22-12 [3] $T_L = 8$ seconds

Figure 11.4–1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4]	PGA = 0.623
Equation (11.8–1):	$PGA_M = F_{PGA}PGA = 1.000 \times 0.623 = 0.623$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	$\boxed{1.0}$
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.623 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5]	C_{RS} = 0.981
From Figure 22-18 [6]	C_{R1} = 1.000

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = IV and S_{DS} = 1.116 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = IV and S_{D1} = 0.611 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = D

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

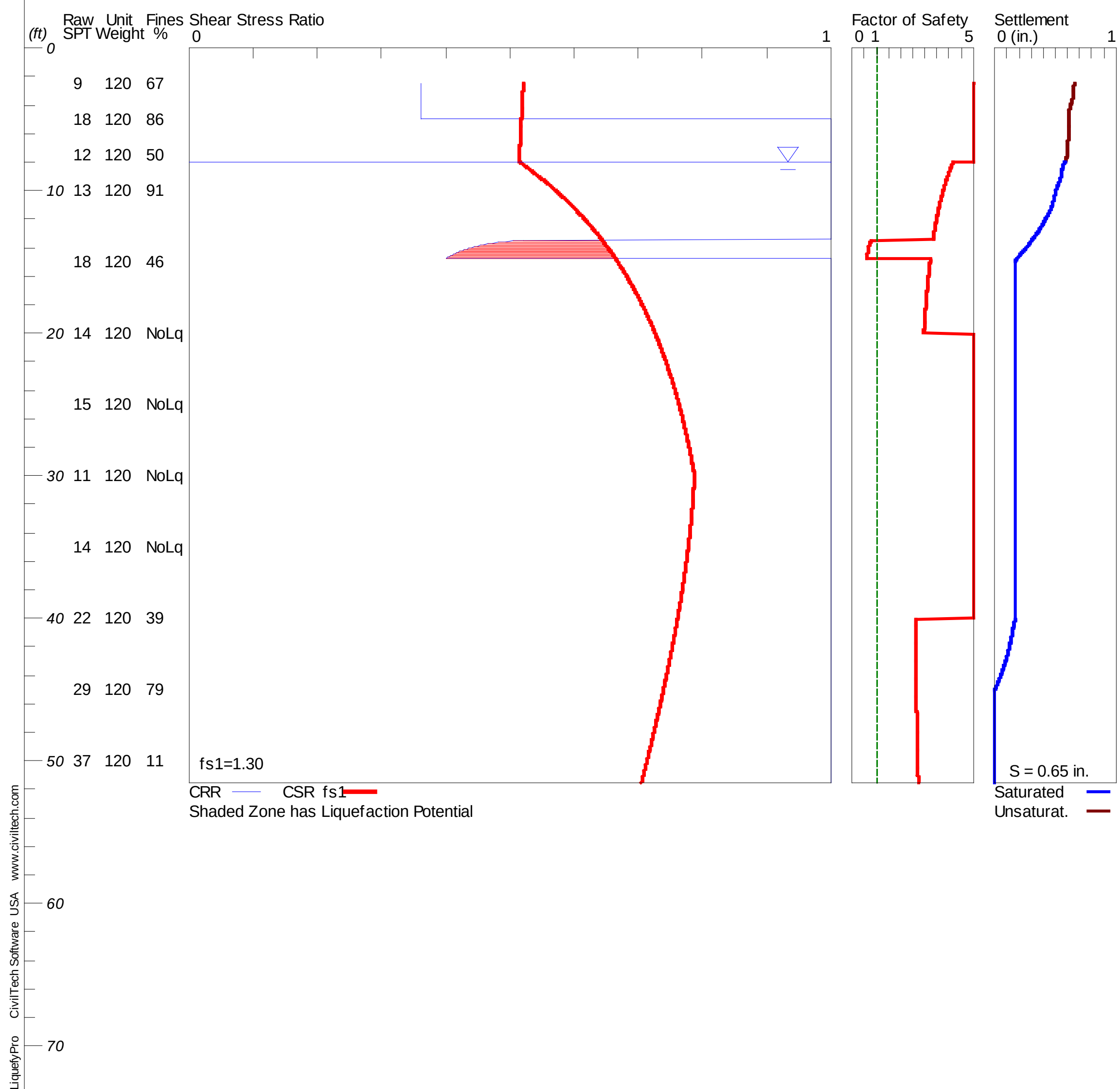
1. *Figure 22-1*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

LIQUEFACTION ANALYSIS

Compton CC- New Student Service Building

Hole No.=B-1   Water Depth=8 ft   Surface Elev.=1000

Magnitude=7.3
Acceleration=0.62g



UNITED-HEIDER INSPECTION GROUP
An ETS Company

Cyclic Softening in Clays and Plastic Silts

Project : Proposed New Student Services Building, Compton College Campus
Project # : 10-18469PW
Soil Boring : B-1

Layer	Sample Depth (ft)	Natural Water Content - w_c (%)	Liquid Limit - LL (%)	Plastic Limit - PL (%)	Bray and Sancio (2006)			Idriss and Boulanger (2008)			
					Plasticity Index -PI (%)	wc/LL	Liquefaction Susceptible	Liquidity Index (LI)	Vertical Effective Stress (atm)	Estimated Soil Sensitivity S_t	Potential Ground Deformation from Cyclic Softening
#1	20	20.7	38	28	10	0.54	Unlikely	-0.73	1.1	< 1	Minor
#2	35	29.5	38	28	10	0.78	Unlikely	0.15	2.0	< 1	Minor

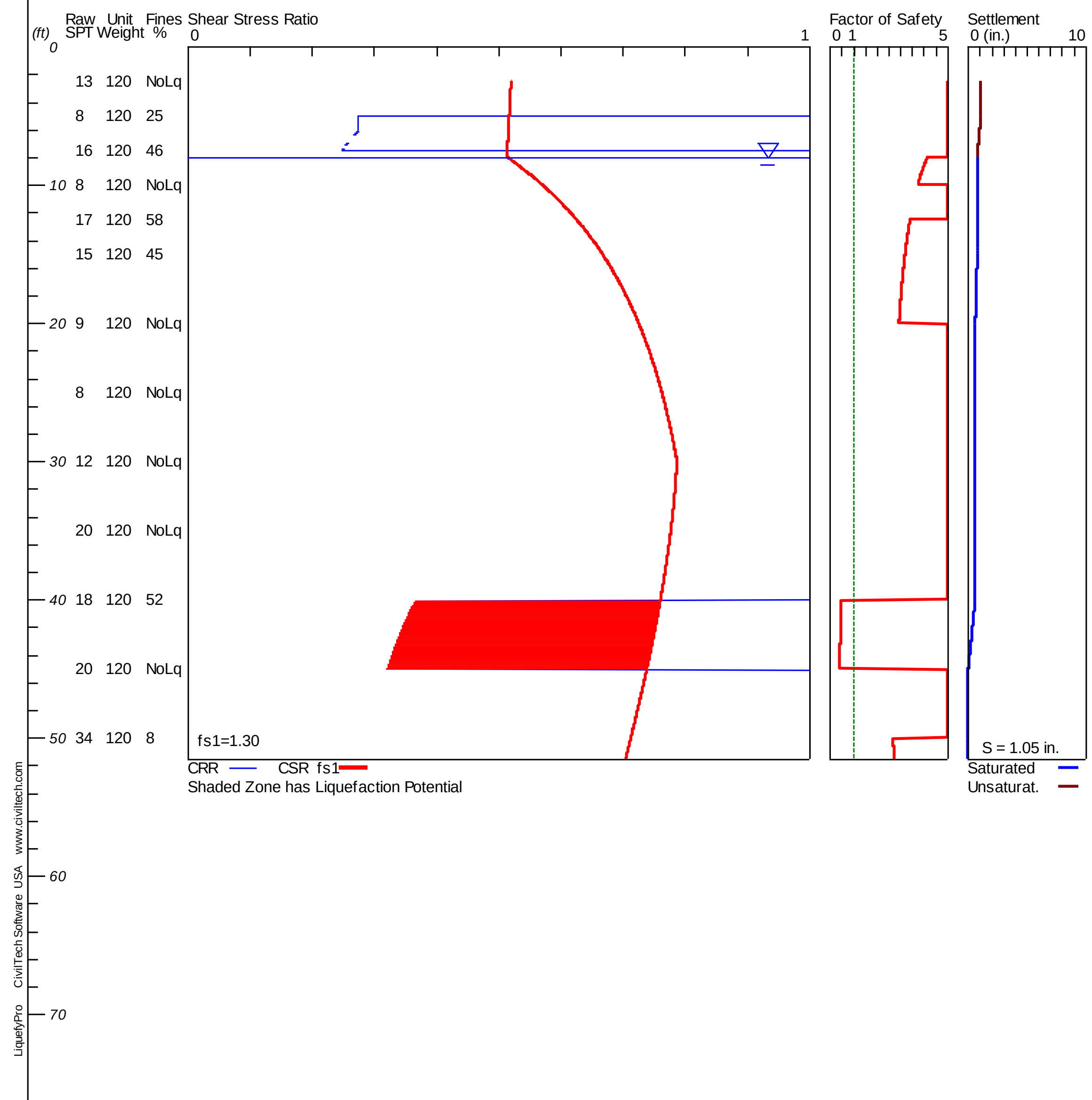
Liquefaction Analysis from United-Heider Inspection Group (Feb. 2018)
For Adjacent New Instructional Building #2

LIQUEFACTION ANALYSIS

Compton CC- Instructional Building #2

Hole No.=B-1 Water Depth=8 ft Surface Elev.=1000

Magnitude=7.3
Acceleration=0.62g



Civiltech Corporation

Plate A-1

Liquefaction Analysis from United-Heider Inspection Group (Feb. 2018)
For Adjacent New Instructional Building #2

UNITED-HEIDER INSPECTION GROUP
An ETS Company

Cyclic Softening in Clays and Plastic Silts

Project : Proposed New Instructional Building #2, Compton College Campus
Project # : 10-18020PW
Soil Boring : B-1*

Layer	Sample Depth (ft)	Natural Water Content - w_c (%)	Liquid Limit - LL (%)	Plastic Limit - PL (%)	Bray and Sancio (2006)			Idriss and Boulanger (2008)			
					Plasticity Index -PI (%)	wc/LL	Liquefaction Susceptible	Liquidity Index (LI)	Vertical Effective Stress (atm)	Estimated Soil Sensitivity S_t	Potential Ground Deformation from Cyclic Softening
#1	10	20.3	34	23	11	0.60	Unlikely	-0.25	0.6	< 1	Minor
#2	20	28.9	60	33	27	0.48	Unlikely	-0.15	1.1	< 1	Minor
#3	25	36.7	60	33	27	0.61	Unlikely	0.14	1.4	< 2	Minor
#4	30	35.5	60	33	27	0.59	Unlikely	0.09	1.7	< 2	Minor
#5	35	18.7	60	33	27	0.31	Unlikely	-0.53	2.0	< 1	Minor
#6	45	36.9	39	27	12	0.95	Moderate	0.83	2.6	~ 8.5	Moderate

Liquefaction Analyses from Heider Inspection Group (2015) Revised (Feb. 2018) for New Instructional Building #2

APPENDIX D

Seismic Design Maps and Liquefaction Analysis Results

Geosphere Consultants Inc.
2001 Crow Canyon Road, Suite # 210
San Ramon, California 94583

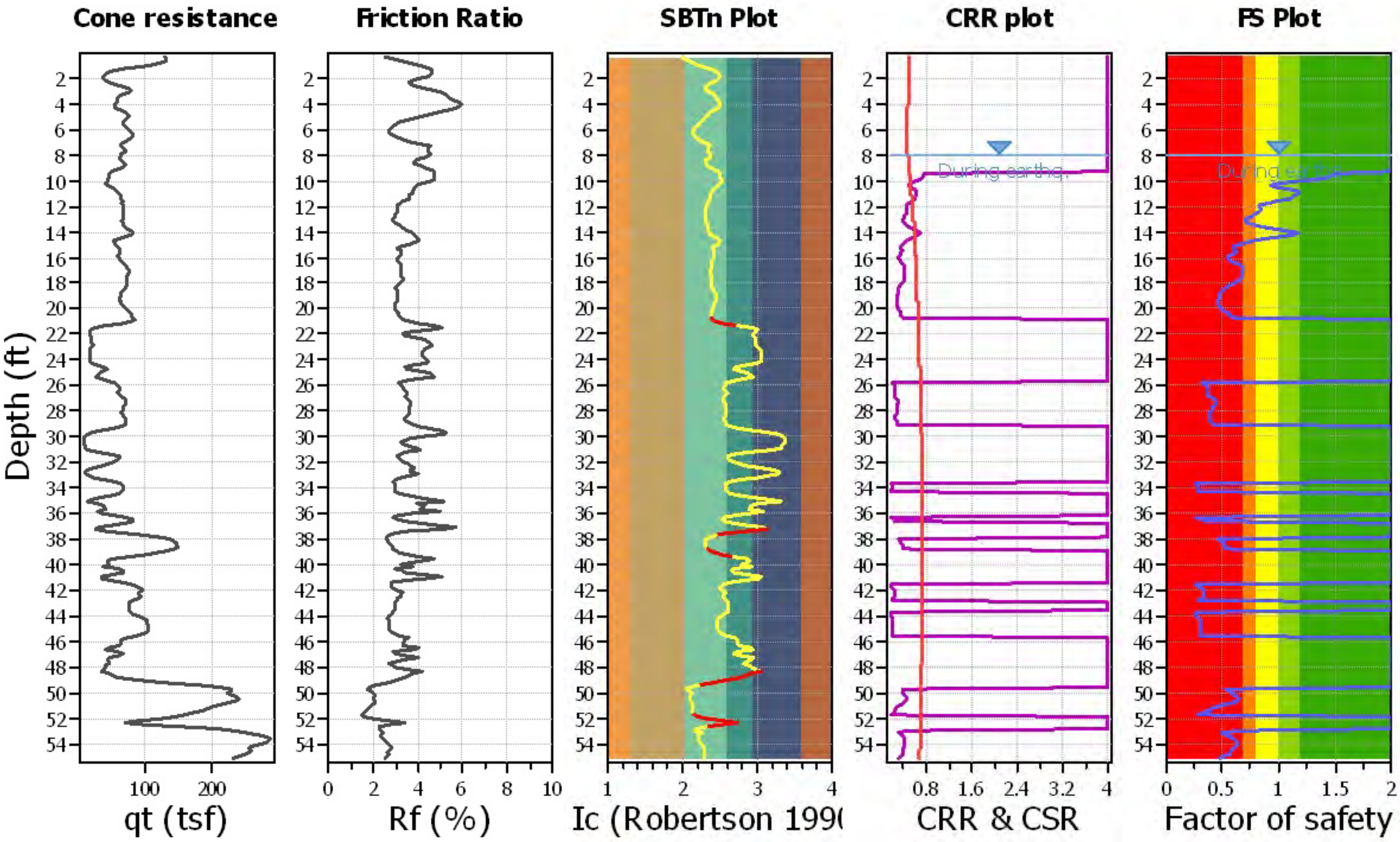
LIQUEFACTION ANALYSIS REPORT

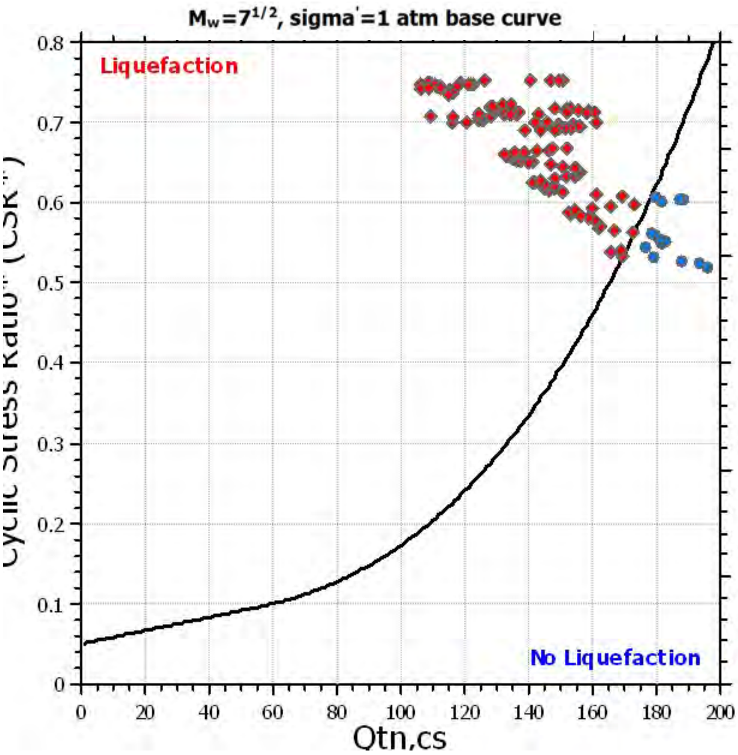
Project title : Compton College Campus **Location : Compton California**

CPT file : CPT-01

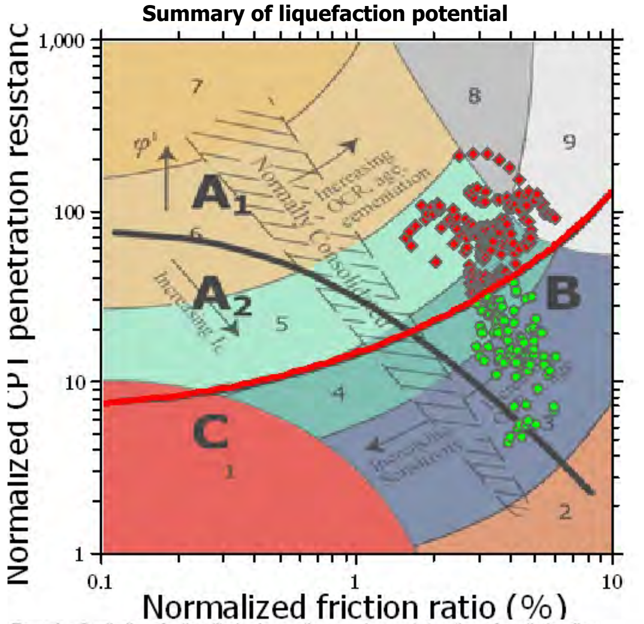
Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	46.40 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	8.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.30	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.62	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



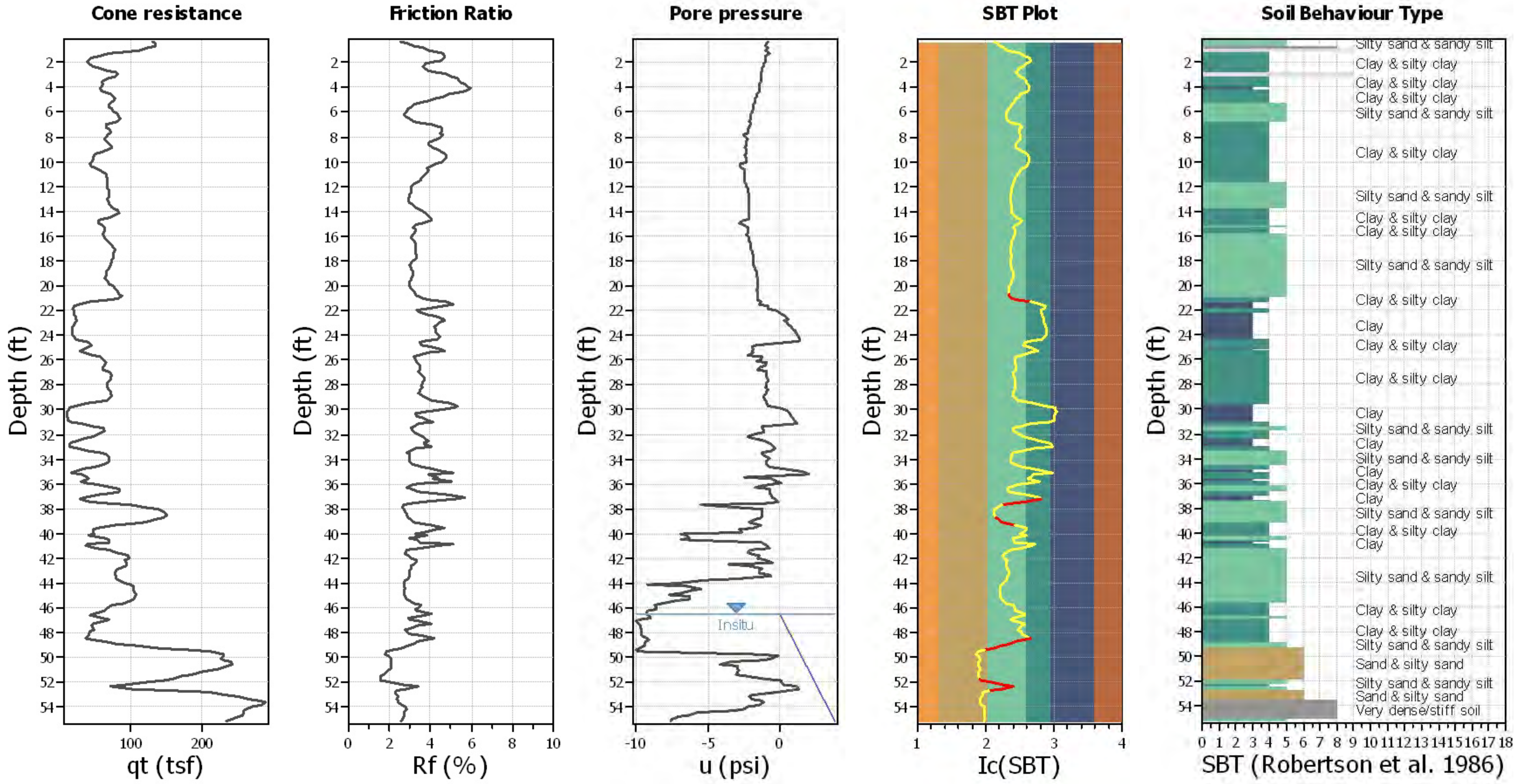


Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plc



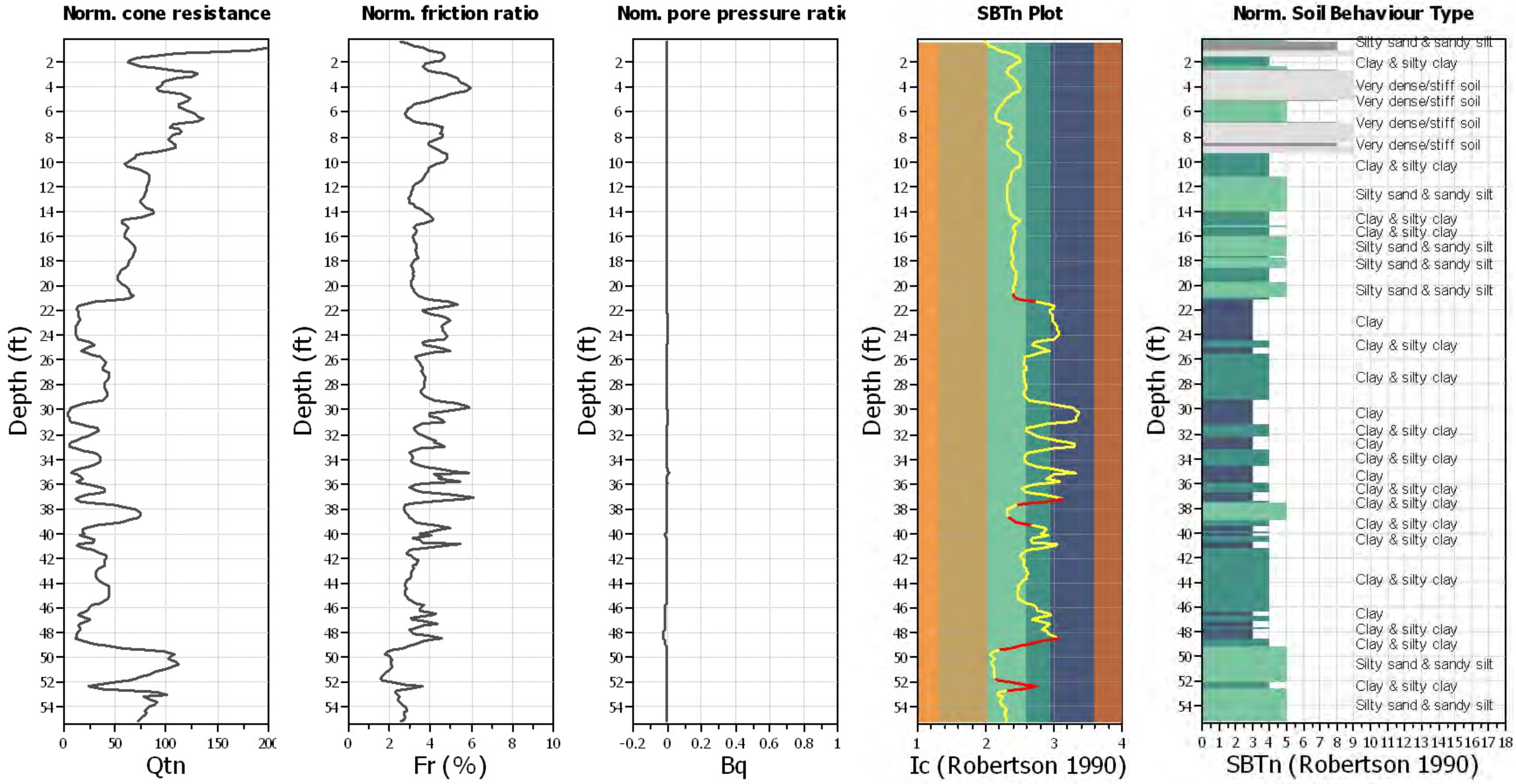
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.62	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	46.40 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to
- 9. Very stiff fine grained

CPT basic interpretation plots (normaliz



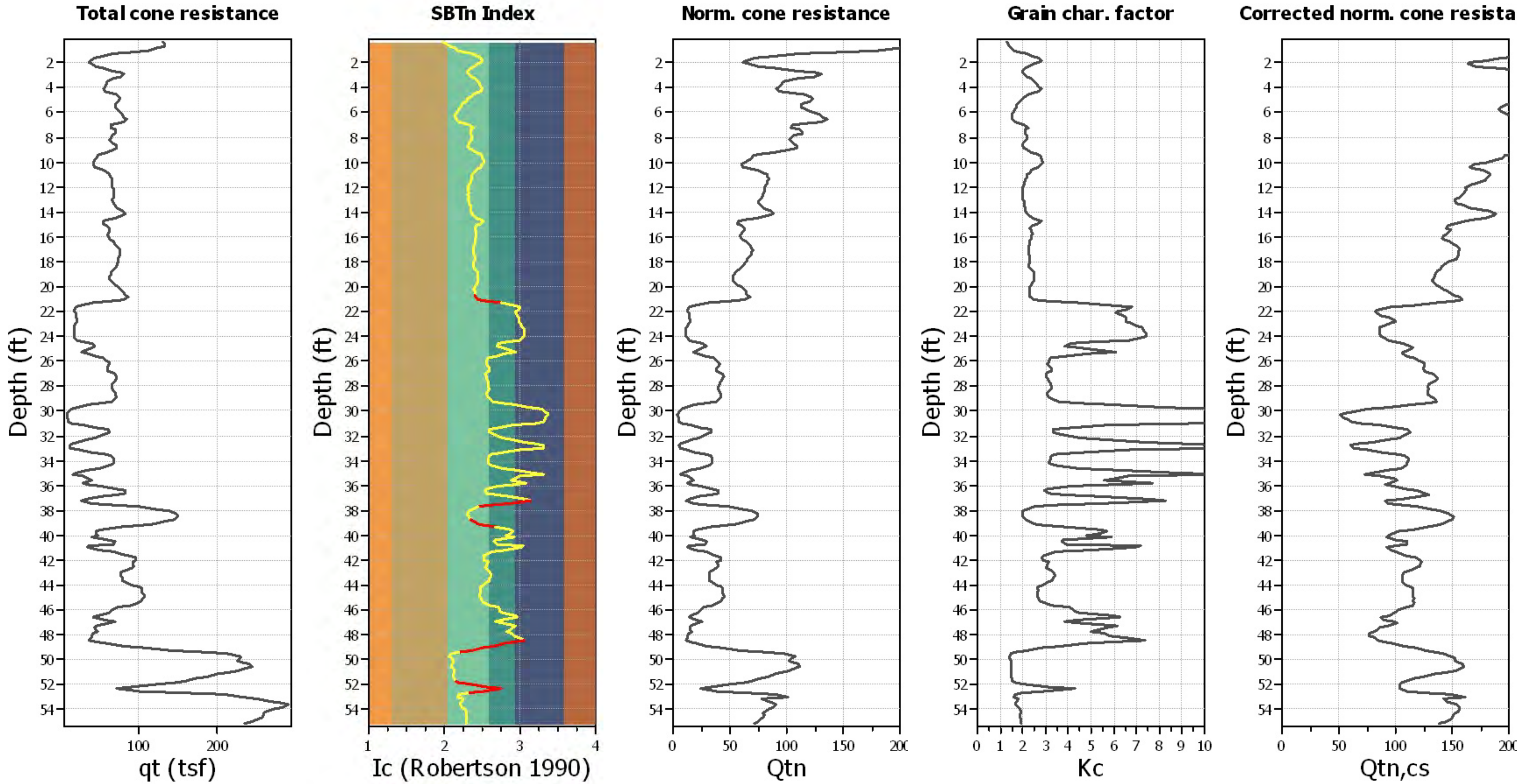
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.62	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	46.40 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

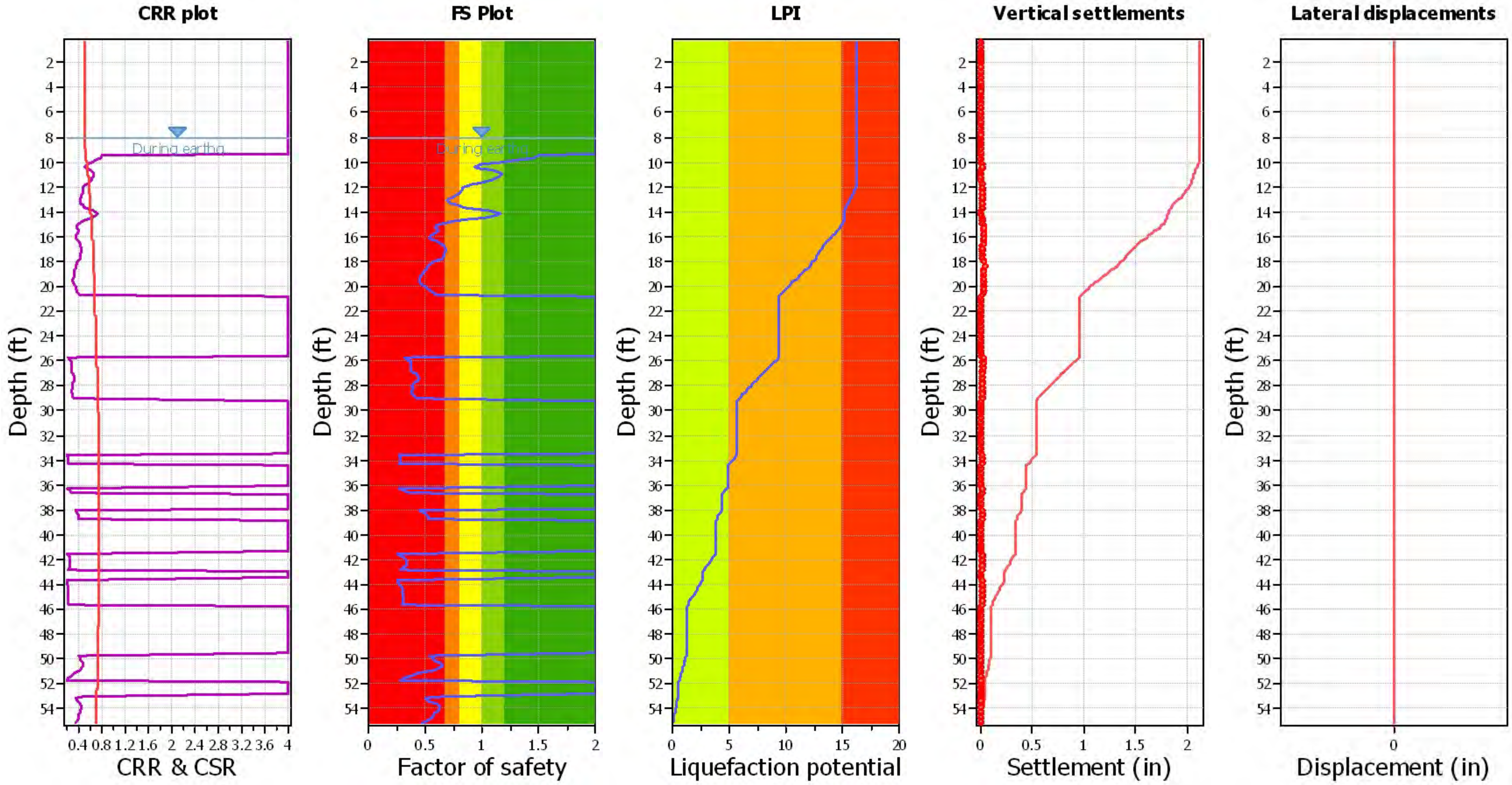
Liquefaction analysis overall plots (intermediate resu



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.62	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	46.40 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.62	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	46.40 ft	Fill height:	N/A	Limit depth:	N/A

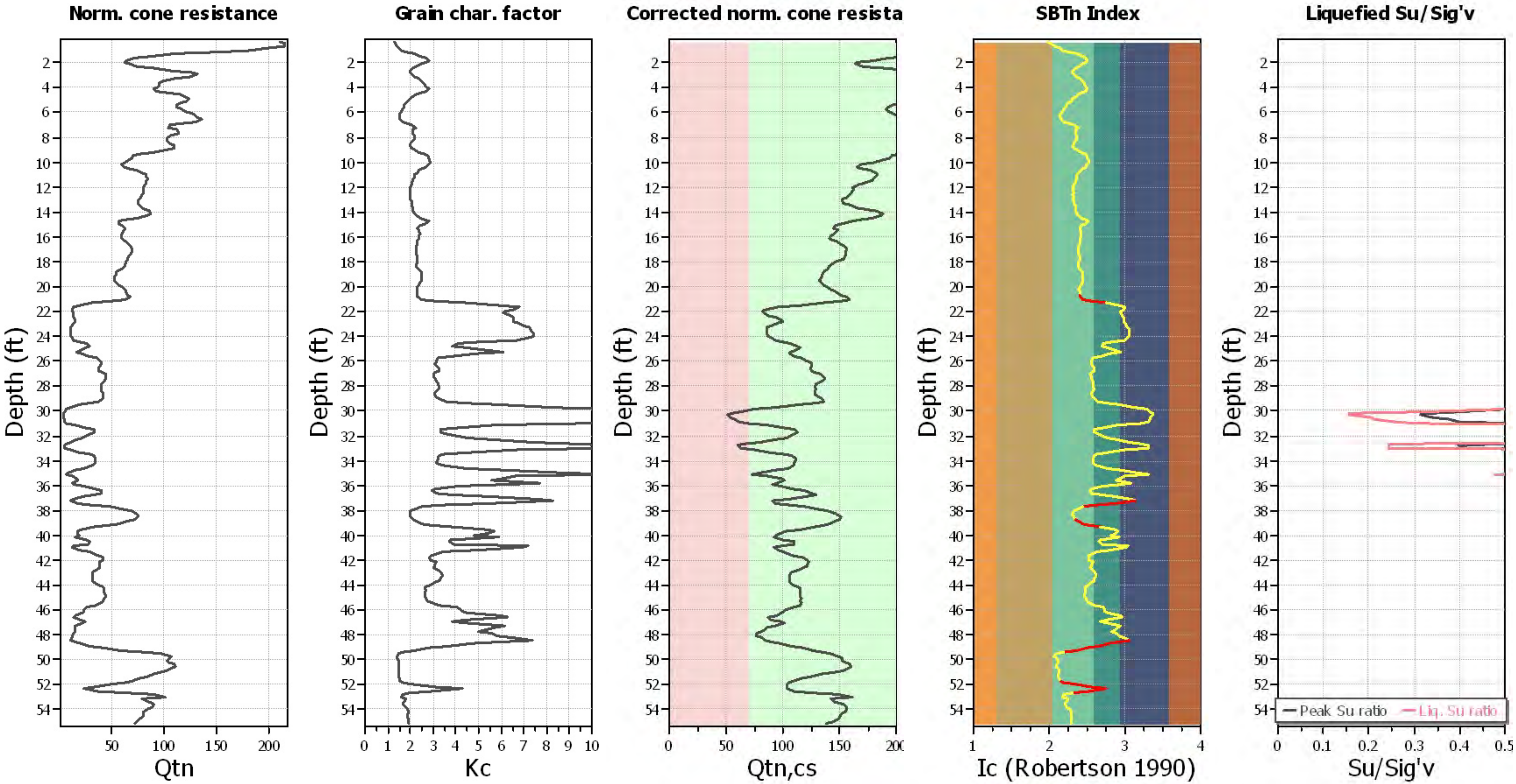
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

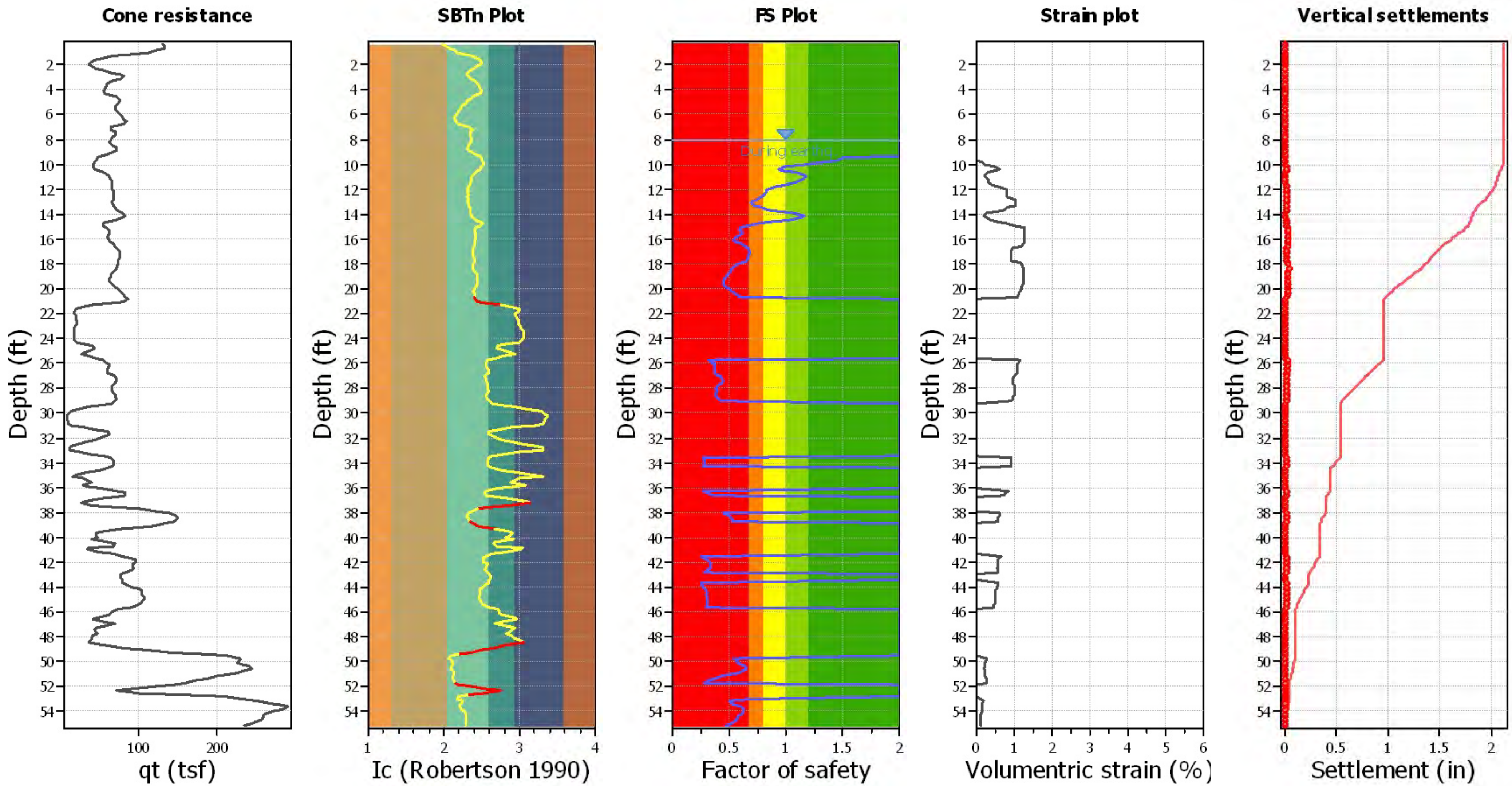
Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

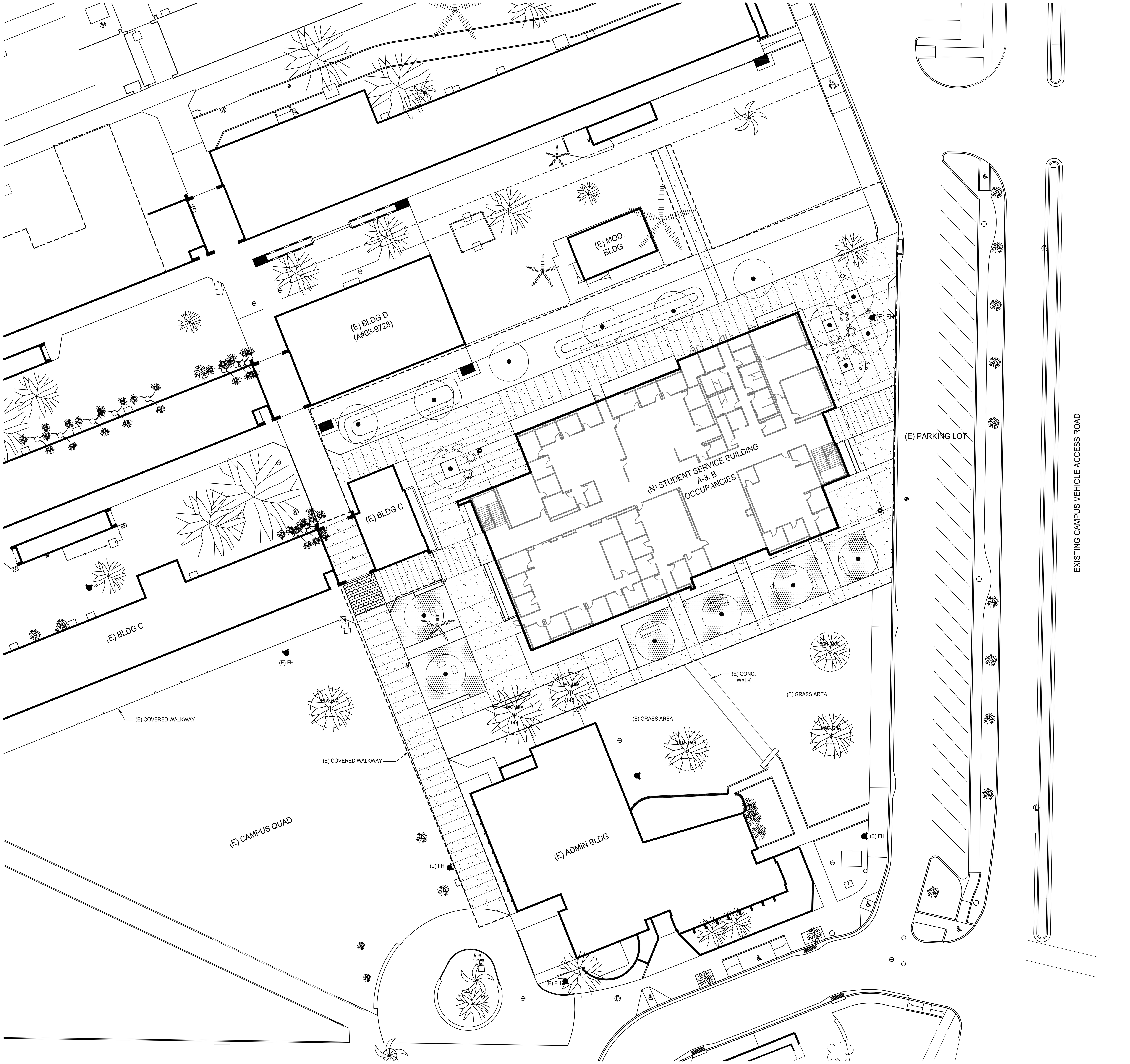
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.62	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	46.40 ft	Fill height:	N/A	Limit depth:	N/A

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain



ENLARGED SITE PLAN

SCALE: 1" = 20'-0"

0 5 10 20 40

NORTH

SITE PLAN LEGEND

CONCRETE PAVING
(N) CONCRETE PAVING
SEE LANDSCAPE PLANS

POST LIGHT

FH — FIRE HYDRANT

DDC — DOUBLE DETECTOR CHECK VALVE

PIV — POST INDICATOR VALVE

FDC — FIRE DEPARTMENT CONNECTION

GENERAL NOTES

1. VERIFY ALL EXISTING & FINISH GRADES, DIMENSIONS & SITE CONDITIONS BEFORE COMMENCING WORK AND REPORT ANY DISCREPANCIES TO THE ARCHITECT.
2. ALL GRADING WORK SHALL CONFORM TO APPLICABLE PROVISIONS OF THE UNIFORM BUILDING CODE, TITLE 24, AND LOCAL CODES OR ORDINANCES. IN THE EVENT OF CONFLICTING PROVISIONS, ALWAYS CONFORM TO THE STRICTER REQUIREMENTS.
3. DETERMINE NECESSARY SUBGRADE ELEVATIONS AND CONSTRUCT SMOOTH TRANSITIONS BETWEEN FINISHED GRADES. FINISHED GRADE ELEVATIONS ADJACENT TO BUILDING PERIMETERS TO BE 6" BELOW FINISHED FLOOR ELEVATIONS, U.N.O.
4. ALL CONCRETE PAVING TO BE MEDIUM BROOM FINISH UNLESS NOTED OTHERWISE.
5. CONTRACTOR TO VERIFY THAT ALL BARRIERS IN THE PATH OF TRAVEL HAVE BEEN REMOVED OR WILL BE REMOVED UNDER THIS PROJECT, AND PATH OF TRAVEL COMPLIES WITH CBC 11B-206.
6. LOCATIONS OF ALL UTILITIES SHOWN ARE APPROXIMATE. CONTRACTOR SHALL EXERCISE EXTREME CAUTION IN EXCAVATING AND TRENCHING TO AVOID INTERCEPTING EXISTING PIPING OR CONDUITS. THE ARCHITECT IS NOT RESPONSIBLE FOR THE LOCATION OF UNDERGROUND UTILITIES OR STRUCTURES WHETHER OR NOT SHOWN OR DETAILED AND INSTALLED BY OTHER CONTRACTS. THE CONTRACTOR SHALL IMMEDIATELY NOTIFY THE OWNER SHOULD SUCH UNIDENTIFIED CONDITIONS BE DISCOVERED. THESE DRAWINGS AND SPECIFICATIONS DO NOT INCLUDE NECESSARY COMPONENTS FOR CONSTRUCTION SAFETY.
7. COMPLY WITH CALIFORNIA FIRE CODE CHAPTER 33 - FIRE SAFETY DURING CONSTRUCTION AND DEMOLITION.
8. CONTRACTOR OPERATIONS SHALL NOT BLOCK, HINDER, IMPEDE OR OTHERWISE INHIBIT THE USE OF REQUIRED EXITS AT ANY TIME. CONTRACTOR SHALL MAINTAIN UNOBSTRUCTED ACCESS TO FIRE EXTINGUISHERS, FIRE HYDRANTS, TEMPORARY FIRE PROTECTION FACILITIES, STAIRWAYS AND OTHER ACCESS ROUTES FOR FIRE FIGHTING EQUIPMENT AND/OR PERSONNEL.

ABBREVIATIONS

A.D.	AREA DRAIN
C.B.	CATCH BASIN
C.D.	CONDENSATE DRAIN
C.J.	CONTROL JOINT
C.O.	CLEAN OUT
D.S.	DOWNSPOUT
(E)	EXISTING
E.J.	EXPANSION JOINT
F.D.C.	FIRE DEPARTMENT CONNECTION
F.F.	FINISHED FLOOR ELEVATION
F.G.	FINISHED GRADE
F.L.	FLOW LINE
G.N.V.	GROUND NOT VISIBLE
I.E.	INVERT ELEVATION
M.H.	MANHOLE
(N)	NEW
U.N.O.	UNLESS NOTED OTHERWISE
P.A.	PLANTER AREA
P.D.	PLANTER DRAIN
R.D.	ROOF DRAIN
S.D.	STORM DRAIN
T.C.	TOP OF CURB
TCN	TOP OF CONCRETE
T.G.	TOP OF GRATE
T.W.	TOP OF WALL
W.G.	WALKWAY GUTTER

tBP
architecture planning interiors

tBP/Architecture
4611 Teller Avenue
Newport Beach, CA 92660
ph: 949.673.0300 fx: 949.732.3895

architect

consultant

IDENTIFICATION STAMP
DIV. OF THE STATE ARCHITECT
OFFICE OF REGULATION SERVICES

APPL - 03-

AC ______ FLS ______ SS ______

DATE ______

DIVISION OF THE STATE ARCHITECT
700 N. ALAMEDA STREET, SUITE 5-500
LOS ANGELES, CA 90012
ph: (213) 897-3995

agency

COMPTON COLLEGE
STUDENT SERVICES BLDG.

COMPTON COMMUNITY COLLEGE DISTRICT
1111 E. ARTESIA BLVD.
COMPTON, CA 90221

owner

tBP project number: 20987.00

file name:

drawn by: checked by:

date: 09. 31. 2018

rev: date: description:

drawing title:
ENLARGED SITE PLAN

drawing no.:
AS-3

drawing of

Steven Haigler
Vice President, Administrative Services
Compton Community College District
1111 East Artesia Boulevard,
Compton, CA 90221

February 21, 2019

Subject: **Engineering Geology and Seismology Review for Compton College – New Student Services Building
1111 East Artesia Boulevard, Compton, CA
CGS Application No. 03-CGS3749**

Dear Mr. Haigler:

In accordance with your request and transmittal of documents received on December 12, 2018, the California Geological Survey has reviewed the engineering geology and seismology aspects of the consulting report prepared for Compton College. It is our understanding this project involves construction of a new Student Services building. This review was performed in accordance with Title 24, California Code of Regulations, 2016 California Building Code (CBC) and followed CGS Note 48 guidelines. We reviewed the following report:

> **Geotechnical Investigation Report, Proposed New Student Services Building, Compton College Campus, 1111 East Artesia Boulevard, Compton, California:** United-Heider Inspection Group, 22620 Goldencrest Drive, Suite 114, Moreno Valley, California 92553; company Project No. 10-18469PW, report dated November 13, 2018, 35 pages, 4 appendices.

Based on our review, the consultants provide a thorough and well-documented assessment of engineering geology and seismology issues with respect to the proposed improvements. The principal concerns identified by the consultants are the potential for strong ground shaking, seismically induced settlement, and hydrocollapse. They recommend design spectral acceleration parameters of $\mathbf{S_{DS} = 1.116}g$ and $\mathbf{S_{D1} = 0.611}g$, which are considered reasonable. The consultants report up to **2.11 inches of total seismic settlement** and **0.9 to 1.1 inches of differential seismic settlement**. In addition, they estimate the potential for **1 inch of hydrocollapse** at the site. The consultants also report the site is in the inundation areas for Whitter Narrows Dam and Sepulveda Dam. Their evaluation indicates deep-seated slope instability is not a design concern for the project.

In conclusion, ***the engineering geology and seismology issues at this site are adequately assessed in the referenced report, and no further information is requested***. If you have any further questions about this review letter, please contact the primary reviewer at (213) 239-0883.

Respectfully submitted,

Dr. Erik K. Frost
Engineering Geologist
PG 9273, CEG 2704

CERTIFIED ENGINEERING GEOLOGIST
Erik K. Frost
No. 2704
STATE OF CALIFORNIA

Concur:

Jennifer Thornburg
Senior Engineering Geologist
PG 5476, CEG 2240

CERTIFIED ENGINEERING GEOLOGIST
Jennifer Thornburg
No. 2240
STATE OF CALIFORNIA

Enclosures:

Note 48 Checklist Review Comments
Keyed to: *Note 48 - Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings*

Copies to:

Stephen E. Jacobs, *Certified Engineering Geologist*, and Param Piratheepan, *Registered Geotechnical Engineer*
United-Heider Inspection Group, 26620 Goldencrest Drive, Suite 114, Moreno Valley, CA 92553

Hung Cheng, *Architect*
tBP/Architecture, 4611 Teller Avenue, Newport Beach, CA 92660

Ted Beckwith, Senior Structural Engineer
Division of State Architect, 355 South Grand Avenue, Suite 2100, Los Angeles, CA 90071

Note 48 Checklist Review Comments

In the numbered paragraphs below, this review is keyed to the paragraph numbers of California Geological Survey Note 48 (October, 2013 edition), *Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings.*

Project Location

1. Site Location Map, Street Address, County Name: Adequately addressed.
2. Plot Plan with Exploration Data with Building Footprint: Adequately addressed.
3. Site Coordinates: Adequately addressed. Latitude and Longitude provided in report: 33.878698°N, 118.209314°W

Engineering Geology/Site Characterization

4. Regional Geology and Regional Fault Maps: Adequately addressed.
5. Geologic Map of Site: Not provided, but not considered critical for this project.
6. Subsurface Geology: Adequately addressed. The consultants report the site is underlain by Holocene to late Pleistocene age soils. They report groundwater was encountered at a depth of 46 feet feet below the ground surface (bgs).
7. Geologic Cross Sections: Not provided, but not considered critical for this project.
8. Active Faulting & Coseismic Deformation Across Site: Adequately addressed. The consultants report the site is not located in an Alquist-Priolo Earthquake Fault Zone. They also report there are no active or potentially active faults that traverse the site.
9. Geologic Hazard Zones (Liquefaction & Landslides): Adequately addressed. The consultants report the site is in a Zone of Required Investigation (ZORI) for liquefaction.
10. Geotechnical Testing of Representative Samples: Adequately addressed.
11. Geological Consideration of Grading Plans and Foundation Plans: Adequately addressed. The consultants recommend replacing existing fill soils and native soils with engineered fill to a minimum depth of 5 feet below foundations or 5 feet below existing grade (whichever is deeper). They recommend soils used for engineered fill should have an expansion index of less than 50. The CGS notes the expansion indices of soils tested at the site and the adjacent Instructional Building #2 site exceed 50. The consultants also provide recommendations for surface drainage to mitigate the potential for hydrocollapse (Item 31). These recommendations appear to reasonably consider the site geology.

    The consultants recommend founding the proposed building on either a mat foundation or a conventional shallow spread footing foundation system supported by a ground improvement method. Based on information from the project architect, it is our understanding that a mat foundation is planned for the proposed building, and that the site soils will not be improved. If a ground improvement program will be implemented, the consultants are reminded that additional reports should be submitted prior to and at the completion of ground improvement.

Seismology & Calculation of Earthquake Ground Motion

12. Evaluation of Historic Seismicity: Adequately addressed. The consultants provide a summary of historical seismicity in the region.

13. Classify the Geologic Subgrade (Site Class): Adequately addressed. The consultants classify the site soil profile as Site Class D, Stiff Soil. The data presented appear to support this conclusion.
14. General Procedure Seismic Parameters: Adequately addressed. The consultants report the following parameters derived from a map-based analysis:
    $S_S = 1.674g$ and $S_1 = 0.611g$
    $S_{DS} = 1.116g$ and $S_{D1} = 0.611g$
15. Seismic Design Category: Adequately addressed. The consultants report $S_1 < 0.75g$ and assign the site to Seismic Design Category D.
16. Site-Specific Ground Motion Analysis: Not applicable.
17. Deaggregated Seismic Source Parameters: Adequately addressed.
18. Time-Histories of Earthquake Ground Motion: Not applicable.

Liquefaction/Seismic Settlement Analysis

19. Geologic Setting for Occurrence of Seismically Induced Liquefaction: Adequately addressed. The consultants report a historically highest groundwater elevation of 8 feet bgs and analyze the potential for liquefaction.
20. Seismic Settlement Calculations: Adequately addressed. The consultants conduct liquefaction triggering and seismic settlement analyses for boring B-1 using an earthquake magnitude of 7.3, a PGA_M of $0.62g$, and a groundwater elevation of 8 feet bgs. They also provide analyses for CPT-1H, which were previously conducted for the adjacent Instructional Building #2. They report **total seismic settlement of 2.11 inches** (based on data from CPT-1H), and estimate **differential seismic settlement of 0.9 to 1.1 inches**. The data presented appear to support these conclusions.
21. Other Liquefaction Effects: Adequately addressed. The consultants summarize historical occurrences of ground damage associated with liquefaction. They report the 1933 Long Beach earthquake produced ground cracks approximately 1/2 mile east of the site, but that most severe damage associated with ground cracks occurred in formerly marshy areas along Compton Creek and former courses of the Los Angeles River. They note this earthquake produced much less severe ground damage at the site, because the site is outside of these formerly marshy areas.
22. Mitigation Options for Liquefaction: Not applicable.

Slope Stability Analysis

23. Geologic Setting for Occurrence of Landslides: Adequately addressed. The consultants report the site is relatively flat, and that there are no significant slopes nearby. As such, they conclude the possibility of earthquake-induced landsliding at the site is negligible. The data presented appear to support this conclusion.
24. Determination of Static and Dynamic Strength Parameters: Not applicable.
25. Determination of Pseudo-Static Coefficient (Keq): Not applicable.
26. Identify Critical Slip Surfaces for Static and Dynamic Analyses: Not applicable.
27. Dynamic Site Conditions: Not applicable.
28. Mitigation Options/Other Slope Failure: Not applicable.

Other Geologic Hazards or Adverse Site Conditions

29. Expansive Soils: Adequately addressed. The consultants report the near-surface soils have a medium expansion potential (EI = 56).

30. Corrosive/Reactive Geochemistry of the Geologic Subgrade: Adequately addressed. The consultants report negligible sulfate exposure in the site soils. They also report the site soils are non-corrosive to foundation elements.
31. Conditional Geologic Assessment: Selected geologic hazards addressed by the consultant are listed below:
    C. Flooding: Adequately addressed. The consultants report **the site is in the inundation areas for Whittier Narrows Dam and Sepulveda Dam**, and that the potential for earthquake-induced flooding exists at the site.
    G. Hydrocollapse: Adequately addressed. The consultants report potentially collapsible soils were identified at the adjacent Instructional Building #2. Based on this prior data, they estimate **approximately 1 inch of collapse settlement may occur at the site**. The data presented appear to support this conclusion.
    H. Regional Subsidence: Adequately addressed. The consultants report the site is mapped within or near an area of potential land subsidence due to oil and gas withdrawal.

Report Documentation

32. Geology, Seismology, and Geotechnical References: Adequately addressed.
33. Certified Engineering Geologist: Adequately addressed.
    Stephen E. Jacobs, Certified Engineering Geologist #1307
34. Registered Geotechnical Engineer: Adequately addressed.
    Param Piratheepan, Registered Geotechnical Engineer #2826

UNITED-HEIDER
INSPECTION GROUP
An ETS Company

CGS App # 03-CGS3749

April 22, 2019

To: Ms. Linda Owens
Director of Facilities
Compton Community College District
1111 East Artesia Blvd.
Compton, CA 90221

Subject: Supplemental Recommendations
Existing Library Building B Piles Demolition
Proposed New Student Services Building
Compton College Campus
1111 E. Artesia Boulevard
Compton, CA 90221

United - Heider Inspection Group Project No. 10-18469PW

Reference: United-Heider Inspection Group (November 13, 2018), Geotechnical Investigation Report, Proposed New Student Services Building, Compton College Campus, 1111 E. Artesia Boulevard, Compton, CA 90221, United - Heider Inspection Group Project No. 10-18469PW.

Dear Ms. Owens,

Pursuant to the email request from Tim Hall of tBP/Architecture, Inc., United-Heider Inspection Group is pleased to submit the following supplemental recommendations for the Existing Library Building B Piles Demolition. We understand that Architects/Structure Designers requested the geotechnical design recommendation for the limit of existing pile demolition for the Existing Library Building B and benching and sloping recommendation for the overexcavation.

The other conclusions and recommendations presented in United-Heider Inspection Group's Preliminary Geotechnical Investigation Report (November 2018) remain applicable unless modified herein.

Existing Library Building B Piles Demolition

We have reviewed the existing Library Building As-Built plans from the DSA State Records Center and Grading Plans for the Proposed New Student Services Building. We understand the following foundation and grading limits are established for the New Student Services Building; finish floor (FF) - 61.30'; Top of Mat Foundation - 58.30' (FF

– 36"); Bottom of Mat Foundation = 56.30' (FF – 60") and Bottom of Overxcavation - 51.30' (FF-10'). The existing library Building B Piles should be demolished and removed up to the limits of bottom of overexcavation (i.e., 51.30'). Demolition and removal of piles up to the limits of bottom overexcavation will provide an average of 5 ft of fill below the mat foundation. We believe 5 ft of compacted fill between unremoved pile portions and bottom of mat foundation will remediate any stiffness differences in the support or differential settlement issues.

Benching and Sloping

Based on our review of Demolition Plan C 1.1 and Architectural Drawing AS-4, overexcavation extends to a maximum depth of 10½ ft below existing ground and except Building C which was supported on grade beams and piles/caissons, no buildings or other structure surcharge were observed within 10 ft of excavation. As the Building C is supported on piles and located about 7½ ft away from the excavation (as shown in AS-4), Building C is considered creating a non-surcharging condition for the excavation.

Onsite near surface soil may be classified as Cal OSHA soil Type B in accordance with appendix A to subpart P of part 1926. The maximum allowable slope for the New Student Service Building site excavation shall not be steeper than 1H:1V. When surcharge loads from stored material or equipment, operating equipment, or traffic are present, a competent person shall determine the degree to which the actual slope must be reduced below the maximum allowable slope, and shall assure that such reduction is achieved. If the excavation is within 10 ft from surcharge loads from any other adjacent structures other than Building C, contractor should notify the geotechnical engineer of record to evaluate the situation and provide appropriate recommendation. A competent person should daily observe the sloped excavation for any distress and notify the geotechnical engineer of record when distress is observed. Sloping or benching for excavations greater than 20 feet deep shall be designed by a registered professional engineer.

We appreciate the opportunity to be of service to the Compton Community College District. If there is any further question regarding this project, please do not hesitate to contact this office.

Respectfully submitted,
United-Heider Inspection Group

REGISTERED PROFESSIONAL ENGINEER
PARAM PIRATHEEPAN
GE 2826
Exp. 09/30/20
GEOTECHNICAL
STATE OF CALIFORNIA

Param Piratheepan, GE
Principal Geotechnical Engineer

UNITED-HEIDER
INSPECTION GROUP
An ETS Company

CGS App # 03-CGS3749

September 5, 2019

To: Ms. Linda Owens
Director of Facilities
Compton Community College District
1111 East Artesia Blvd.
Compton, CA 90221

Subject: Supplemental Recommendations
Preparing Non-yielding/firm Excavation Bottom
Proposed New Student Services Building
Compton College Campus
1111 E. Artesia Boulevard
Compton, CA 90221

United - Heider Inspection Group Project No. 10-18469PW

Reference: United-Heider Inspection Group (November 2018), Geotechnical Investigation Report, Proposed New Student Services Building, Compton College Campus, 1111 E. Artesia Blvd., Compton, CA 90221, dated November 13, 2018.

Dear Ms. Owens,

Pursuant to the email request from Sherri Philips of PCM3, Inc., United-Heider Inspection Group is pleased to submit the following supplemental earthwork recommendations for preparing a non-yielding/firm overexcavation bottom for the Proposed New Student Services Building. We understand that PCM3 want to use a cheaper alternative instead of using oversize rock blanket that was used during Instructional Building #1 Site grading.

The other conclusions and recommendations presented in United-Heider Inspection Group's Preliminary Geotechnical Investigation Report (November 2018) remain applicable unless modified herein.

Non-yielding/firm Excavation Bottom

Upon excavation/overexcavation to the recommended depths, subgrade soils at the removal bottoms should be moisture-conditioned as needed and recompacted to a minimum of 90 percent relative compaction (per ASTM D1557).

If 90 percent relative compaction cannot be achieved at the excavation bottom due to the anticipated very moist, plastic, and difficult to compact soil conditions, it may be necessary to utilize a cement-modified soil (CMS), a subbase solution that blends cement and water with native (insitu) soils to improve undesirable soil properties. It may be desirable to use 5 percent cement blended into the soil to a minimum depth of 12 inches as shown in the attached cross-section. Specialty contractor should review the field conditions on-site to determine the appropriate cement content and depth of the cement modified soil.

We appreciate the opportunity to be of service to the Compton Community College District. If there is any further question regarding this project, please do not hesitate to contact this office.

Respectfully submitted,
United-Heider Inspection Group

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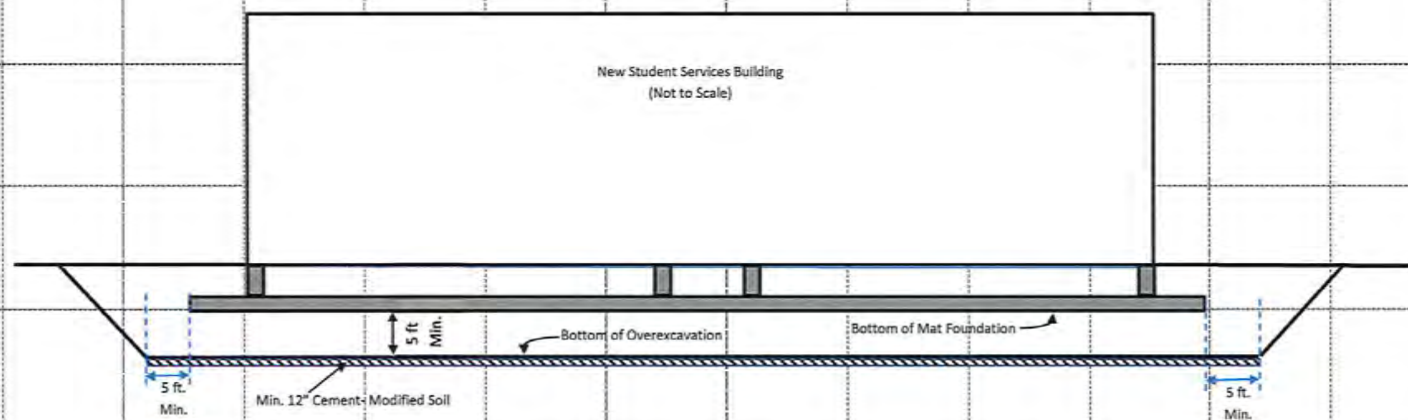

Non-yielding/firm Excavation Bottom

Proposed New Student Services Building

Compton College Campus, 1111 East Artesia Blvd., Compton, CA 90221

UNITED-HEIDER

Project No: 10-18469PW
Date: September 5, 2019

Scale: Not to Scale