6280 Riverdale Street
San Diego, CA 92120
(877) 215-4321 | oneatlas.com

March 14, 2022

Atlas No. 10-57575PW
Report No. 4

MS. LINDA OWENS, CHIEF FACILITY OFFICER
COMPTON COLLEGE DISTRICT
1111 EAST ARTESIA BOULEVARD
COMPTON, CA 90221

Subject: CGS Application No. 03-CGS5153
RESPONSE TO REVIEW COMMENTS
Geotechnical Investigation Report, Physical Education Complex Replacement Compton Community College District, CA
1111 East Artesia Boulevard
Compton, California 90221

References:

1) California Department of Conservation, California Geological Survey, 2022, Engineering Geology and Seismology Review for Compton College – Physical Education Complex Replacement, 1111 East Artesia Boulevard, Compton, CA. CGS Application No. 03-CGS5153, DSA Application No. 03-121755, dated February 4, 2022.
2) Atlas Technical Consultants, 2021, Addendum Geotechnical and Geohazard Report, Physical Education Complex Replacement, Compton Community College District, Compton, CA. Project No. 10-57575PW Report No. 2, dated September 7, 2021.
3) Atlas Technical Consultants, 2021, Geotechnical Investigation Report, Physical Education Complex Replacement, Compton Community College District, Compton, CA. Project No. 10-57575PW, Report No. 1, dated July 7, 2021.
4) Atlas Technical Consultants, 2021, CPT Test Results, Physical Education Complex Replacement, ComptonCommunity College District,Compton, CA. Project No. 10-57575PW, Report No. 3, October 14 2021.

Dear Ms. Owens:

Atlas Technical Consultants prepared this letter to respond to the referenced review comments from the California Department of Conservation's California Geological Survey (CGS) for the subject project.

In the review letter dated February 4, 2022 (Reference 1), CGS has requested additional information. For convenience, a copy of the review letter is attached. Our responses have been prepared per the CGS Comment Letter and based on: Title 24, California Code of Regulations, 2019 California Building Code (CBC) and followed CGS Note 48 Guidelines and in consideration of responses to these comments from the specialty ground improvement contractor, Keller North America (KNA).

Comment No. 7: *Subsurface Geology:* ***Additional information requested****. The consultant report the site contains up to 5 feet of fill material consisting of clays, silts, and sands, underlain by young alluvial deposits consisting of loose to dense silts and sands. They report groundwater was encountered at a depth of 44 feet below the existing ground surface (bgs) during their investigation. The consultants utilized information collected from eleven hollow stem auger borings drilled to a maximum depth of 61.5 feet bgs. However, CGS notes that the design of ground improvements planned for the site (refer to item 22) relies on the data from three cone penetration test (CPT) soundings and these explorations are not discussed or provided. The consultant and/or specialty ground improvement design build contractor are requested to provide the logs of these CPT soundings for our review.*

Response: Please see Attachment CC.

CPT Logs indicate some clay and silty-clay layers (based on SBT Concept: Soil Behavior Type). However, in the seismic and liquefaction settlement analyses that already have been performed based on SPT Borings, presented in Reference 3, due to the observed silty layers, we conservatively assumed that these clay layers behave very close to silty layers during seismic loads.

Comment No. 10: *Consideration of Geology in Geotechnical Engineering Recommendations:* ***Additional information is requested****. Considering the potential static settlement due to structural loads and potential for liquefaction and seismic settlement induced by design earthquake, the consultants recommend that ground improvement should be performed to allow for use of shallow foundation system for support of the PE building, pool mechanical building and swimming pool. They provide preliminary recommendations for ground improvement by installation of Vibro Stone Columns (VSCs) and Deep Soil Mixing (DSM) (discussed further in Item 22) to satisfy recommended shallow foundation bearing capacities and settlement limits. In their original report dated July 7, 2021, the consultants recommend over-excavation of soils to a minimum depth of 5 feet beneath the building pads (extending 5 feet laterally, if feasible) to improve the performance of the interior slabs-on-grade for the buildings. In their addendum report dated September 7, 2021, the consultants recommend that 2 feet of the new engineered fill should be placed beneath the bottom of the pool and atop the mitigated/improved soil to provide relative uniform support below the pool. While these preliminary site grading recommendations provided by the consultants appear to be reasonable, the consultants are requested to clarify whether the original over-excavation recommendations for the building pads are superseded by the planned VSC or DSM soil mitigation, or to provide updated recommendations for the site grading to provide adequate support for interior slabs with consideration of the planned ground improvement. The consultants also provide recommendations for support of the light poles around the pool on deep foundations. Additional discussion regarding the deep foundation recommendations is provided in Item 11B.*

Response: ATLAS recommended overexcavation and backfilling of the soil to have a uniform subgrade after ground mitigation:

- PE Building Pad and Foundation System: 5 feet of overexcavation and backfilling
- Below the bottom of the Pool: 2 feet of overexcavation and backfilling
- Pool House: 2 feet of overexcavation and backfilling

The recommended depth of overexcavation and backfilling may be verified after review of the field and/or Lab tests results after the soil mitigation.

Comment No. 11B: *Conditional Geotechnical Topics – Deep Foundations:* ***Additional information is requested.*** *In their addendum report, the consultants provide recommended values of downward side friction and lateral resistance of soils to be used in design of "concrete shaft" foundations for support of light poles around the pool. They also recommend the soil resistance should be neglected for the upper portion of the shaft within the undocumented fill soils. However, the consultants have not reported if the soil resistance values are ultimate or allowable, have not provided recommended uplift capacities, have not provided any recommendations for minimum diameter and length of shafts, nor any construction/installation procedures to maintain integrity of the excavations and concrete. Also, based on the configuration of DSM columns shown in the overall ground improvement plan provided by KNA North America (KNA), it appears the proposed light poles may be located beyond the DSM ground improvement zone for the swimming pool/pool mechanical building and therefore may be subject to loss of bearing/lateral capacity due to liquefaction. Therefore, for the consultants are requested to demonstrate, using site soil data, and considering the liquefied soil conditions, that the proposed concrete shafts will develop the recommended skin friction capacity for design loads and sufficient lateral resistance without excessive lateral deformation.*

Response: The poles are considered to be supported by side friction resistance of the bearing soil in the upper 8 feet of soil, as well as by lateral resistance for overturning. The seismic and liquefaction settlement analyses show that generally the upper 8 feet of the soil is not located within the liquefaction zone (the historically highest ground water table is at the depth of 8 feet). The poles around the pool area have relatively small vertical and lateral loads provided by the Structural Engineer of the project. Therefore, the non-liquefiable and bearing upper soil layers can provide the recommended relatively conservative soil resistance without DSM. Since the recommendations provided in Sections 4 and 5 of Reference 2 neglect much of the shallow soils, the lateral resistance is therefore more conservative as the shallow soils still provide resistance.

The recommended soil resistance parameters are allowable, and the uplift capacity is considered to be half of the downward capacity, based on the side friction resistance (200 psf).

The allowable passive resistance when the ground surface is level, may be assumed to be equal to the pressure developed by a fluid with a density of 200 pcf, to a maximum allowable value of 2,000 psf.

The minimum recommended diameter is 3 feet. The minimum recommended length of the concrete shafts in the half northern portion of the pool is 6 feet and in the half southern portion of the pool is 8 feet. (assumption: center to center, at least three diameters of the shaft)

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

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

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

Comment No. 12: *Evaluation of Historic Seismicity:* ***Additional information is requested.*** *The consultants depict the location of the site relative to sites of historical earthquake-generated liquefaction. They report no property damage or human losses were reported in the City of Compton area based on the referenced provided. However, CGS notes significant damage occurred in the site area as a result of the 1933 Lon Beach earthquake. We therefore request the consultants to review and discuss records of liquefaction in the site vicinity along with the performance of Compton Junior High School during the 1933 Long Beach earthquake and provide additional references, as needed.*

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

College (CDMG, California Geology, March 1973, p. 57). Other buildings in Compton with reported major damage included the Young Hotel and Aranbe Hotel (Daily News with photos from Orange County Register).

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

Comment No. 13: *Classify the Geology Subgrade (Site Class):* ***Additional information is requested.*** *The consultants classify the site soil profile for design purposes; however, they do not provide any data or calculations to justify this designation in accordance with ASCE 7-16, section 20.4. The consultants should specifically address how the selected Site Class was determined and provide supporting data and calculations.*

Response: We evaluated the site class in general conformance with ASCE 7-16, Section 20.4. In our calculations we used data from the deepest borings: B-4 and B-10 with the depth of 61½ feet and 56½ feet respectively. For the blow counts below the bottom of the borings up to the depth of 100 feet we assumed that the blow counts are close to the values found at the bottom of the borings, (conservatively we disregarded the typical increase in blow counts with depth), please see Attachment AA.

Please note that the calculation is based on the on-site soil blow counts (as-is situation); however, the project (PE Building, Pools and Pool house) will be constructed on the mitigated soil that will have more blow counts than the used values in our calculations

Comment No. 15: *Site-Specific Ground Motion Hazard Analysis:* ***Additional information may be needed.*** *The site-specific ground motion analysis presented appears to be reasonable for the Site Class D soil profile. However, based on the response to Item 13, the consultants may need to present revised analyses if a different Site Class is assigned.*

Response: Please see the response to Comment 13.

Comment No. 22: *Mitigation Options for Liquefaction/Seismic Settlement:* ***Additional information is requested.*** *Due to the potential for seismic settlement and surface manifestation resulting from liquefaction, the consultants recommend performing ground improvement to mitigate those hazards and facilitate the use of shallow foundations for support of proposed* *structures. As noted previously in Item 10, the consultants provide preliminary recommendations for ground improvement by installation of VSCs and/or DSM to mitigate liquefaction and reduce the potential static and seismic settlement at the locations of proposed improvements. CGS notes the preliminary recommendations the consultants provide include minimum area replacement ratios (ARRs), column dimensions and plan layout requirements, minimum bearing capacities, and maximum differential settlement criteria for design of the VSC/DSM systems. They recommend a specialty ground improvement contractor should perform the detailed design and draft plans for the selected ground improvement system, and the consultants provide preliminary recommendations for field testing requirements, quality assurance procedures, and final acceptance criteria for the VSC/DSM ground improvement.*

The ground improvement design submittals from the specialty ground improvement design-build contractor, KNA, provide the detailed VSC and DSM ground improvement design packages for the proposed PE building and pool mechanical building/swimming pool, respectively, including copies of their geotechnical calculations and a draft set of plans for each type of proposed ground improvement. Based on our review, CGS requests KNA and/or the consultants provide further information to address the following concerns regarding their design and plans for ground improvement:

General:
As noted in Item 7, it appears that KNA has relied on analysis of data from three CPT soundings for design of the VSC and DSM ground improvement planned for the project, but no discussion or original logs of these explorations have been provided for our review.

<u>VSC Ground Improvement for PE Building:</u>
The consultants refer to the requirements of 2019 CBC Section 1813A within their recommendations for VSCs to be installed under the entire building/structure footprint and to extend beyond the footprint of structure/foundation at least half the depth of the VSCs with a minimum of 10 feet or an approved alternative. They also cite 2019 CBC Section 1813A and recommend a minimum of four VSCs (or approved equivalent) should be located under each isolated or continuous/combined footing and that VSCs under the shallow foundations should be located symmetrically around the centroid of the footing or load. However, based on our review of VSC plan layout provided by KNA, it appears the design layout of VSCs does not satisfy the consultants' recommendations or the 2019 CBC requirements for extent of VSCs beyond the building perimeter, nor for multiple/symmetrical location of VSCs under foundations.

CGS observes that the calculations of VSC bearing capacity provided by KNA do not consider the presence of VSC columns (ARR = 0 and KNA reports they "used the pre-treatment soil parameters for this computation"). However, CGS notes the KNA calculations are based on the input of a significant value of shear strength for the soils (input as an effective soil cohesion with no friction) and we request justification for this value based on the available geotechnical data for the site.

CGS notes KNA has estimated the potential static settlement of the PE building supported by VSCs based on their consideration of a uniform surcharge load applied over the entire footprint of the building. However, this does not adequately address the potential static settlement of individual foundations supporting the PE building, and CGS requests the calculations and design of the VSC system be updated as appropriate to address and consider static settlements of foundations in accordance with the recommendations of the geotechnical consultants.

<u>DSM Ground Improvement for Pool Mechanical Building/Swimming Pool</u>:
CGS notes KNA provides discussion of QA/QC requirements and acceptance criteria for the DSM ground improvement in their design submittal and on Sheet KNA-2 of the shop drawings that generally appear to be reasonable and appropriate. However, a complete set of specifications for the installation, testing, and performance of the DSM system should be drafted and provided for CGS review. The specifications should include fully detailed and well-defined acceptance criteria for evaluation of the successful completion of the ground improvement and satisfaction of design and performance objectives.

The specifications for verification of DSM quality should include requirements that the selection of locations for confirmation coring and selection of core samples for UCS testing are subject to review and approval of the Geotechnical Engineer of the Record (GEOR) for the project.

The consultants recommend that coring for verification should be performed on at least 2% of the DSM columns, which appears to be reasonable. However, CGS observes that a lower percentage of cores is shown in the Notes and Details on Sheet KNA-2 of the KNA shop drawings. KNA is requested to update these Notes to conform with the geotechnical recommendations. In addition, KNA is requested to revise their sampling statement and indicate that the coring locations should be selected by the GEOR for the project.

CGS requests KNA to report the typical and maximum DSM grid panel spacing considered in their design and plan layout of DSM columns to justify the ARR value critical to design of the system. We note that the DSM column diameter and overlap dimensions are indicated on the shop drawing sheet KNA-3 but also request the panel spacing dimensions also be clearly indicated on the plans.

Additionally, in fulfilling the role as GEOR for the project, the geotechnical consultants should submit formal documentation of their review of the contractors' VSC and DSM design and plans that includes an explicit statement regarding their opinion of the conformance of the design with their geotechnical recommendations.

<u>Response:</u> Please see Attachments DD and BB.

Item No. 31 H. *Regional Subsidence:* ***Adequately addressed.*** *The consultants report the site lies either within, on near, an area of potential land subsidence due to withdrawal of oil and gas from nearby oil and gas fields, which appears to be reasonable based on the references presented.*

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

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

Respectfully submitted,
Atlas Technical Consultants LLC

REGISTERED PROFESSIONAL ENGINEER
MEHRAB JESMANI
GE 3175
EXP. 09/30/23
GEOTECHNICAL
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL GEOLOGIST
DOUGLAS A. SKINNER
No. 2472
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Mehrab Jesmani, PhD, PE, GE, M. ASCE
ATLAS, Senior Engineer/Project Manager

Douglas A. Skinner, CEG 2472
ATLAS, Senior Geologist

MJ:DAS:ER

Attachments:

- Attachment AA: Atlas' site class calculation
- Attachment BB: Atlas' letter of review of the soil mitigation plan
- Attachment CC: Atlas' CPT results report
- Attachment DD: KNA's responses and documents

Distribution:
Ms. Linda Owens at: lowens@compton.edu
Ms. Sheri Phillips at: sphillips@pcm3.com
Mr. Hraztan Zeitlian at: hraztan@struere

ATTACHMENT AA
SITE CLASS CALCULATION

ATLAS

Site Class Calculations

Project No.	10-57575PW	Project Name	Compton CC-PE Building
Boring No.	B-4		

Layer Top ft	Layer Bottom ft	Measured Blow Count	Sampler Type Ring/SPT	Correction Factor*	Energy Correction**	Corrected Blow Count	Layer Thickness/N
0	5	10	Assumed SPT	1	1.25	12.5	0.40
5	10	13	SPT with Autohammer	1	1.25	16.3	0.31
10	15	15	Ring with Autohammer	0.65	1.25	12.2	0.41
15	20	14	SPT with Autohammer	1	1.25	17.5	0.29
20	25	26	Ring with Autohammer	0.65	1.25	21.1	0.24
25	30	7	SPT with Autohammer	1	1.25	8.8	0.57
30	35	23	Ring with Autohammer	0.65	1.25	18.7	0.27
35	40	22	SPT with Autohammer	1	1.25	27.5	0.18
40	45	23	Ring with Autohammer	0.65	1.25	18.7	0.27
45	50	10	SPT with Autohammer	1	1.25	12.5	0.40
50	55	25	Ring with Autohammer	0.65	1.25	20.3	0.25
55	60	36	SPT with Autohammer	1	1.25	45.0	0.11
60	75	36	Assumed SPT	1	1.25	45.0	0.33
75	100	36	Assumed SPT	1	1.25	45.0	0.56
						SUM:	4.57

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

**A correction of 1.25 was used for Autohammer

		Site Class
N average:	21.9	D

Input
Calculated

Table 1

ATLAS

Site Class Calculations

Project No.	10-57575PW	**Project Name**	Compton CC-PE Building
Boring No.	B-10		

Layer Top ft	Layer Bottom ft	Measured Blow Count	Sampler Type Ring/SPT	Correction Factor*	Energy Correction**	Corrected Blow Count	Layer Thickness/N
0	5	10	Assumed SPT	1	1.25	12.5	0.40
5	10	12	Ring with Autohammer	0.65	1.25	9.8	0.51
10	15	5	SPT with Autohammer	1	1.25	6.3	0.80
15	20	23	Ring with Autohammer	0.65	1.25	18.7	0.27
20	25	5	SPT with Autohammer	1	1.25	6.3	0.80
25	30	7	Ring with Autohammer	0.65	1.25	5.7	0.88
30	35	13	SPT with Autohammer	1	1.25	16.3	0.31
35	40	18	Ring with Autohammer	0.65	1.25	14.6	0.34
40	45	11	SPT with Autohammer	1	1.25	13.8	0.36
45	50	32	Ring with Autohammer	0.65	1.25	26.0	0.19
50	55	17	SPT with Autohammer	1	1.25	21.3	0.24
55	60	26	Ring with Autohammer	0.65	1.25	21.1	0.24
60	75	26	Assumed SPT	1	1.25	32.5	0.46
75	100	26	Assumed SPT	1	1.25	32.5	0.77
						SUM:	6.57

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

**A correction of 1.25 was used for Autohammer

		Site Class
N average:	**15.2**	**D**

Input
Calculated

Table 2

ATTACHMENT BB

ATLAS' LETTER OF REVIEW OF THE SOIL MITIGATION PLAN

14457 Meridian Parkway
Riverside, CA 92518
(951) 697-4777. | oneatlas.com

March 14, 2022

Atlas No. 10-57575PW
Report No. 5

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

Subject: **Review of the Soil Mitigation Plan**
Compton College PE Complex Replacement
Compton Community College District
1111 East Artesia Boulevard
Compton, CA 90221

References:
1) Atlas Technical Consultants, 2021, Addendum Geotechnical and Geohazard Report, Physical Education Complex Replacement, Compton Community College District, Compton, CA. Project No. 10-57575PW Report No. 2, dated September 7, 2021.
2) Atlas Technical Consultants, 2021, Geotechnical Investigation Report, Physical Education Complex Replacement, Compton Community College District, Compton, CA. Project No. 10-57575PW, Report No. 1, dated July 7, 2021.
3) CGS's Comments Letter, Comment No. 22, Page 8, in regard to a formal documentation of Atlas review of the contractor's (KNA) VSC and DSM design and plans, dated February 4, 2022 (attached)

CGS Application No. 03-CGS5153, DSA Application No. 03-121755

Dear Ms. Owens:

In accordance with the CGS Comment No. 22 (Reference 3), Atlas Technical Consultants (Atlas) has reviewed the Vibro Stone Column Design provided in Attachment A' and the Deep Soil Mixing Design provided in Attachment B', the design and plans, prepared by Keller North America (KNA). Our review was based on the geotechnical and geohazard aspects of the reviewed documents (Attachments A' and B') and was to verify that they are in general conformance with the recommendations provided in References 1 and 2.

Based on our review and to the best of our knowledge and understanding, it is our opinion that the proposed soil mitigation design and plans, provided in Attachments A' and B', including the depth of the mitigated soil, diameter, length, spacing and area replacement ratio (ARR) of the proposed Vibro Stone Columns and Deep Soil Mixing Columns have been prepared in general conformance with the recommendations provided References 1 and 2. ATLAS recommends performing necessary tests (field and lab) on the mitigated soil (DSM and VRSC) to evaluate the behavior of the mitigated soil. Based on these tests results and analyses, the preliminary recommendations for the soil mitigations in references 1 and 2 and the design presented in the Attachments A' and B' may need to be modified (e.g., adding some additional rows of DSM and/or VRSC).

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

Respectfully submitted,
Atlas Technical Consultants LLC

REGISTERED PROFESSIONAL ENGINEER MEHRAB JESMANI GE 3175 EXP. 09/30/23 GEOTECHNICAL STATE OF CALIFORNIA

REGISTERED PROFESSIONAL GEOLOGIST DOUGLAS A. SKINNER No. 2472 CERTIFIED ENGINEERING GEOLOGIST STATE OF CALIFORNIA

Mehrab Jesmani, PhD, PE, GE 3175	Douglas A. Skinner, PG, CEG 2472
Senior Engineer	Senior Geologist

MJ:DAS:ER

Attachments:

- Attachment A': Keller North America, 2022, Compton Community College (Phase 1) Vibratory Replacement Stone Columns (VRSC) Shop Drawings – Overall Ground Improvement Plan Sheet, KNA-3: dated February 28, 2022

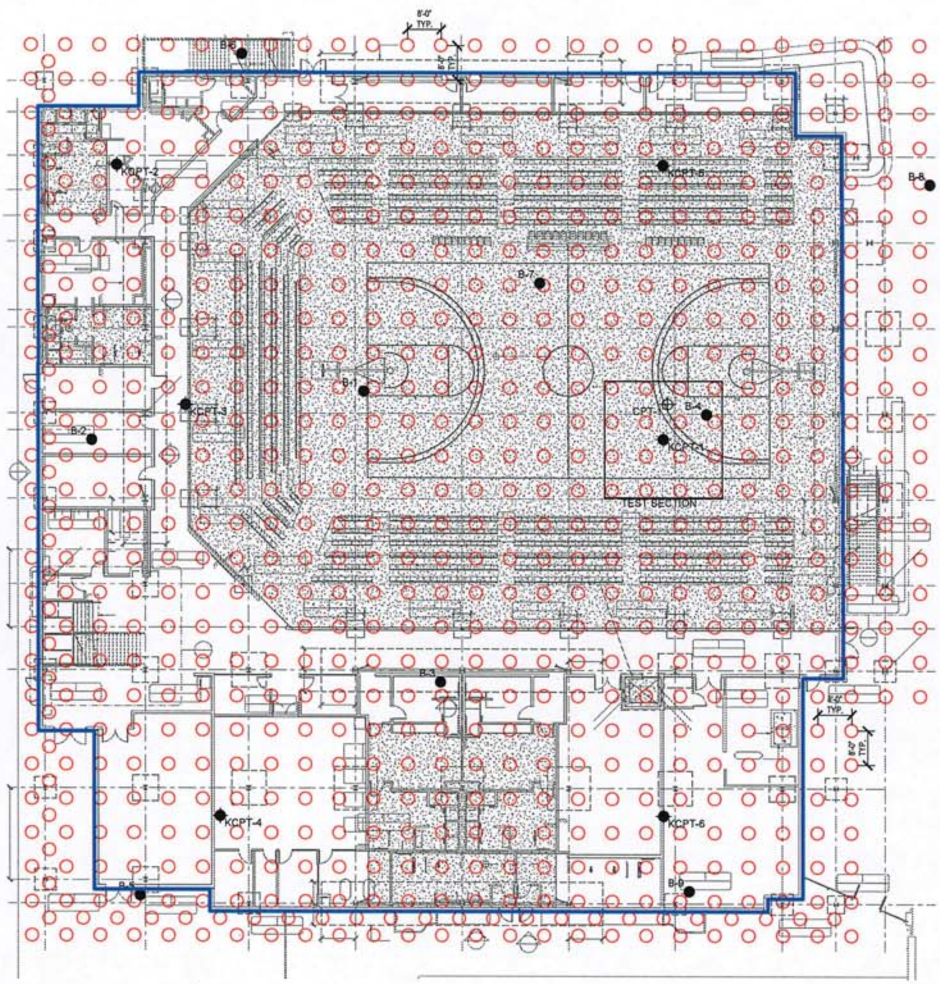
- Attachment B': Keller North America, 2022, Compton Community College (Phase 2) Deep Soil Mixing (DSM) Shop Drawings – Overall Ground Improvement Plan Sheet KNA-3P: dated February 28, 2022

Distribution:
Ms. Linda Owens at: lowens@compton.edu
Ms. Sheri Phillips at: sphillips@pcm3.com
Mr. Hraztan Zeitlian at: hraztan@struere

ATTACHEMENT A'

KNA’S VRCS SHOP DRAWINGS SHEET KNA-3

2/14/2022 6:20:28 PM



LEGEND:

- Ø36" Vibratory Replacement Stone Columns (VRSC). Install 23' from the existing ground surface. Approximate existing ground surface elevation: 55 feet.
- CPT-X Existing CPT location
- B-X Existing Boring location
- KCPT-X Proposed Post-Treatment CPT locations
- Building Footprint

STRUERE
ADVANCED ARCHITECTURE
3324 GRAND VIEW
LOS ANGELES, CALIFORNIA 90066
TELEPHONE (310) 749-7649
E-MAIL HRAZTAN@STRUERE.COM
WWW.STRUERE.COM

KELLER
17461 DERIAN AVENUE
SUITE 106
IRVINE, CALIFORNIA
92614
WWW.KELLER-NA.COM

VRSC ENGINEER OF RECORD:

IDENTIFICATION STAMP
DIV. OF THE STATE ARCHITECT
APP. 03-124753 INC: #
REVIEWED FOR
SS ☐ FLS ☐ ACS ☐
DATE:

Compton College

COMPTON COLLEGE
PHYSICAL EDUCATION
COMPLEX REPLACEMENT

1111 EAST ARTESIA BLVD.
COMPTON, CA 90221-5393

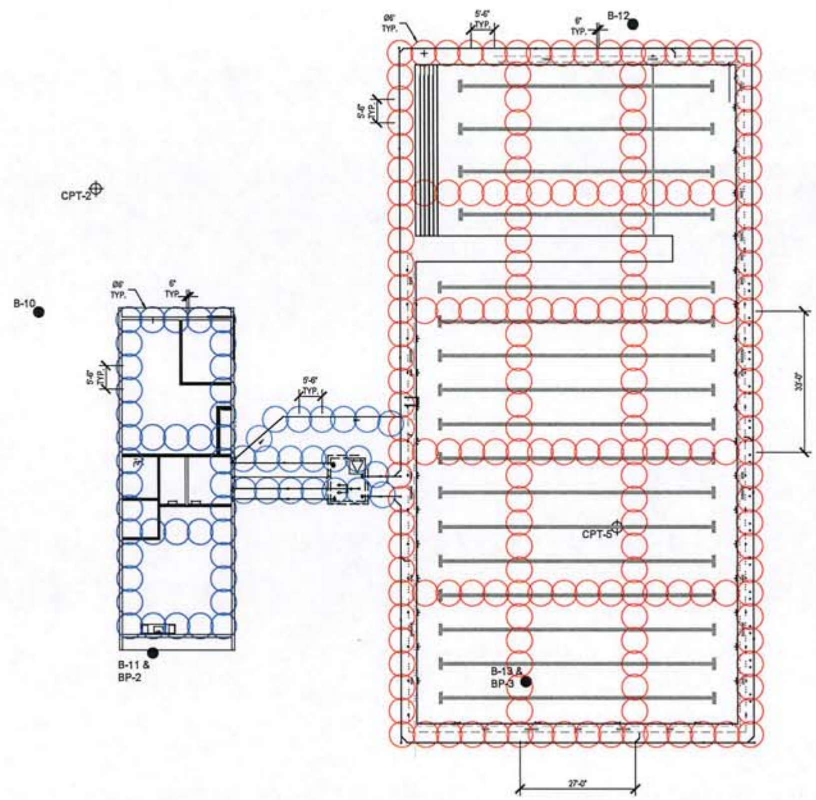
ISSUE DESCRIPTION	
DSA SUBMITTAL	22.02.28

SHEET TITLE
OVERALL GROUND
IMPROVEMENT PLAN
KNA-3

ATTACHEMENT B'

KNA'S DSM SHOP DRAWINGS SHEET KNA-3P

2/14/2022 6:20:28 PM



LEGEND:

Ø6' DEEP SOIL MIX COLUMN (DSM).
Install 49' from the existing ground surface.
Approximate existing ground surface elevation: 55 feet.

Ø6' DEEP SOIL MIX COLUMN (DSM).
Install 19' from the existing ground surface.
Approximate existing ground surface elevation: 55 feet.

CPT-X Existing CPT location

B-X Existing Boring location

STRUERE
ADVANCED ARCHITECTURE

KELLER
17461 DERIAN AVENUE
SUITE 106
IRVINE, CALIFORNIA
92614
WWW.KELLER-NA.COM

DSM ENGINEER OF RECORD:

Compton College

COMPTON COLLEGE
PHYSICAL EDUCATION
COMPLEX REPLACEMENT

1111 EAST ARTESIA BLVD,
COMPTON, CA 90221-5393

ISSUE DESCRIPTION	
DSA SUBMITTAL	22.03.28

SHEET TITLE
OVERALL GROUND IMPROVEMENT PLAN
KNA-3P

ATTACHMENT CC

ATLAS' CPT RESULTS REPORT

CPT TESTS RESULTS

PHYSICAL EDUCATION COMPLEX REPLACEMENT
COMPTON COMMUNITY COLLEGE DISTRICT

Compton, CA

PREPARED FOR:

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

PREPARED BY:

Atlas Technical Consultants LLC
14457 Meridian Parkway
Riverside, CA 92518

October 14, 2021

14457 Meridian Parkway
Riverside, CA 92518
(951) 697-4777 | oneatlas.com

October 14, 2021

Atlas No. 10-57575PW
Report No. 3

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

Subject: **CPT Tests Results, Compton College PE Complex Replacement, Compton Community College District, 1111 East Artesia Boulevard, Compton, California.**

Reference: Atlas Technical Consultants, 2021, Geotechnical Investigation Report, Physical Education Complex Replacement, Compton Community College District, Compton, CA, Project No. 10-57575PW, Dated: July 7.

Atlas Technical Consultants, 2021, Addendum to the Geotechnical Investigation Report, Compton College PE Complex Replacement, Compton Community College District, 1111 East Artesia Boulevard, Compton, California, Dated September 7.

Dear Ms. Owens:

Atlas Technical Consultants is pleased to present this report (Report No. 3) for the proposed Physical Education Complex Replacement, Compton College located at 1111 East Artesia Boulevard in the City of Compton, California.

This report has been prepared to present the results of our calculation for the total seismic settlement of the onsite soil based on the field CPT Tests (performed on September 3, 2021).

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

Respectfully submitted,
Atlas Technical Consultants LLC

REGISTERED PROFESSIONAL ENGINEER
MEHRAB JESMANI
GE 3175
EXP. 09/30/23
GEOTECHNICAL
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL GEOLOGIST
DOUGLAS A. SKINNER
No. 2472
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Mehrab Jesmani, PhD, PE, GE 3175
Senior Engineer

Douglas A. Skinner, PG, CEG 2472
Senior Geologist

MJ:DS

Distribution: Sheri Phillips, sphillips@pcm3.com
Hraztan Zeitlian, hraztan@struere.com

1. Field Investigation

Three CPT tests were performed on September 3, 2021. CPT-1 and CPT-2 were performed in close proximity to Borings B-4 and B-10 respectively. CPT-5 was close to B-13.

2. Results

Based on the results of the three CPT tests and utilizing three methods: IB-2008, BI-2014 and Moss (2006) we performed seismic settlement analysis: seismic dry settlement (for the soil above the historically highest ground water table) and liquefaction settlement. The assumed seismic parameters have been: Earthquake Magnitude= 7.3 and Peak Ground Acceleration= 0.802g. Table 1 presents the results of our calculations:

Table 1 – Total Seismic Settlements (CPT Test Results)

CPT No.	Total Seismic Settlement (in)		
	IB-2008	BI-2014	Moss-2006
1	4.2	3.5	3.2
2	5.8	5.0	4.7
5	5.9	4.8	4.4

The results indicate a largest total seismic settlement on the order of about 6 inches. Figures 2 to 16 present a summary of the field results and our seismic settlement analysis.

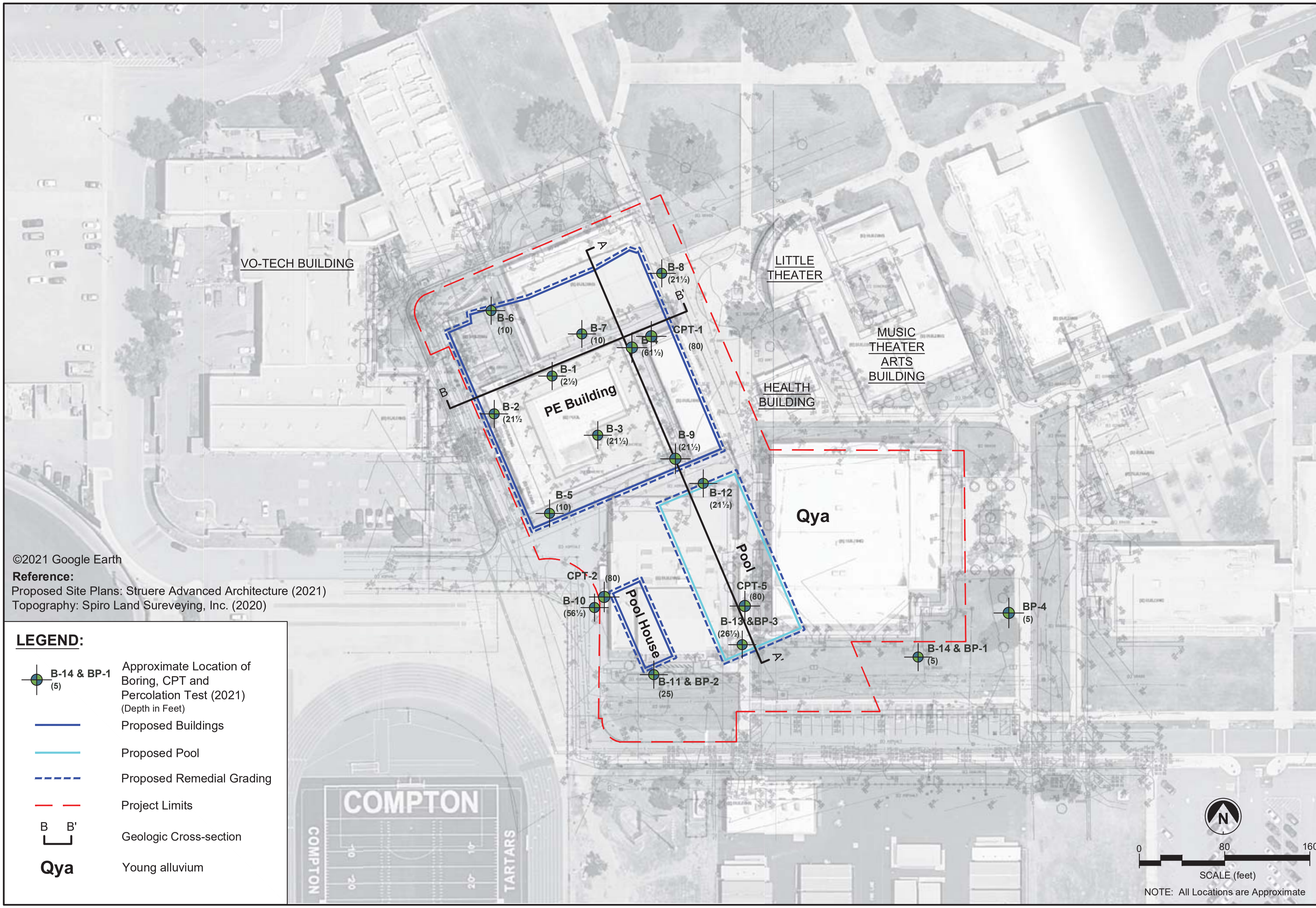
APPENDIX I
FIGURES

GEOTECHNICAL MAP

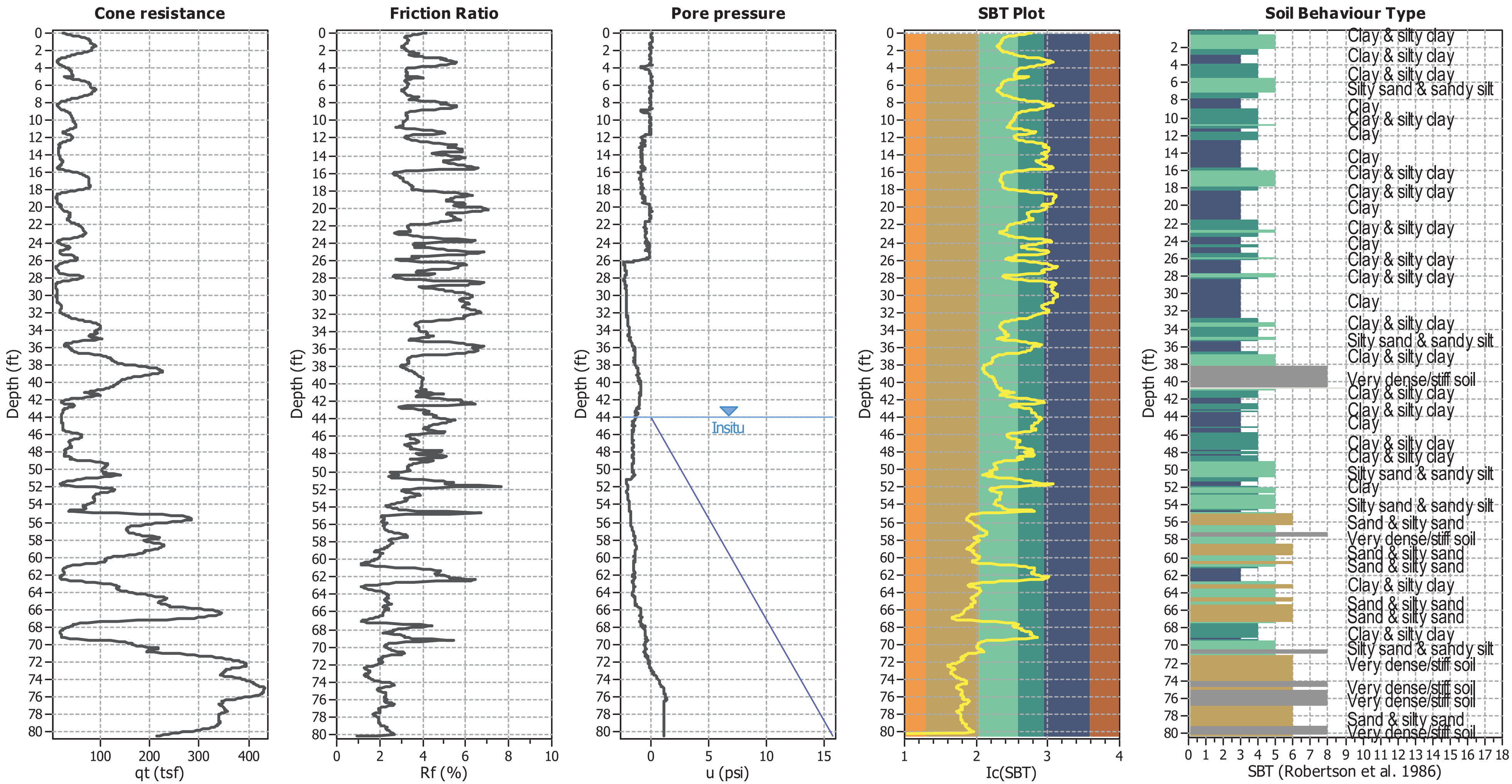
Physical Education Complex Replacement
Compton, California

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

Figure: 1



CPT basic interpretation plots



Input parameters and analysis data

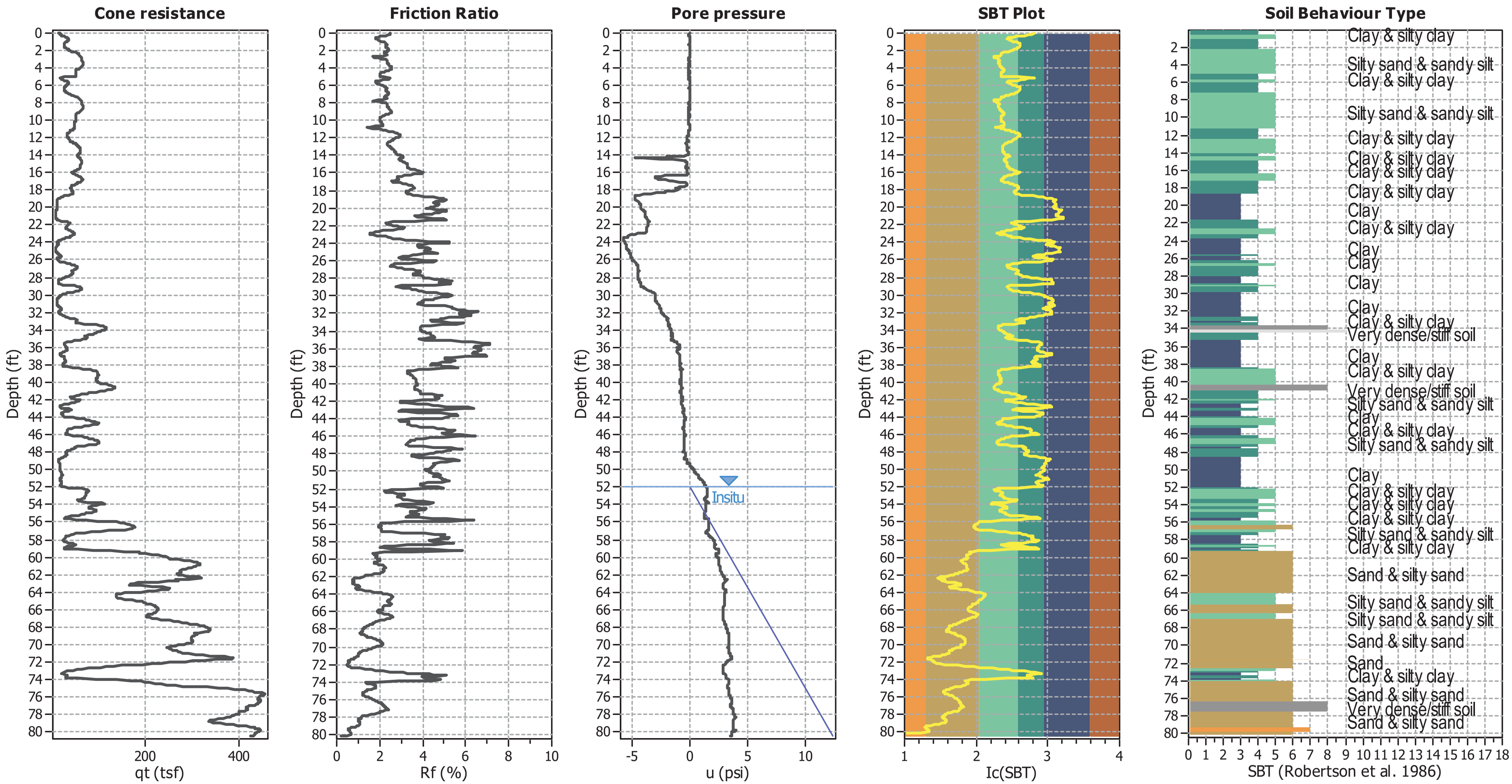
Analysis method:	Moss et al. (2006)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	Moss et al. (2006)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.80	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	44.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

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

Figure - 2

CPT basic interpretation plots



Input parameters and analysis data

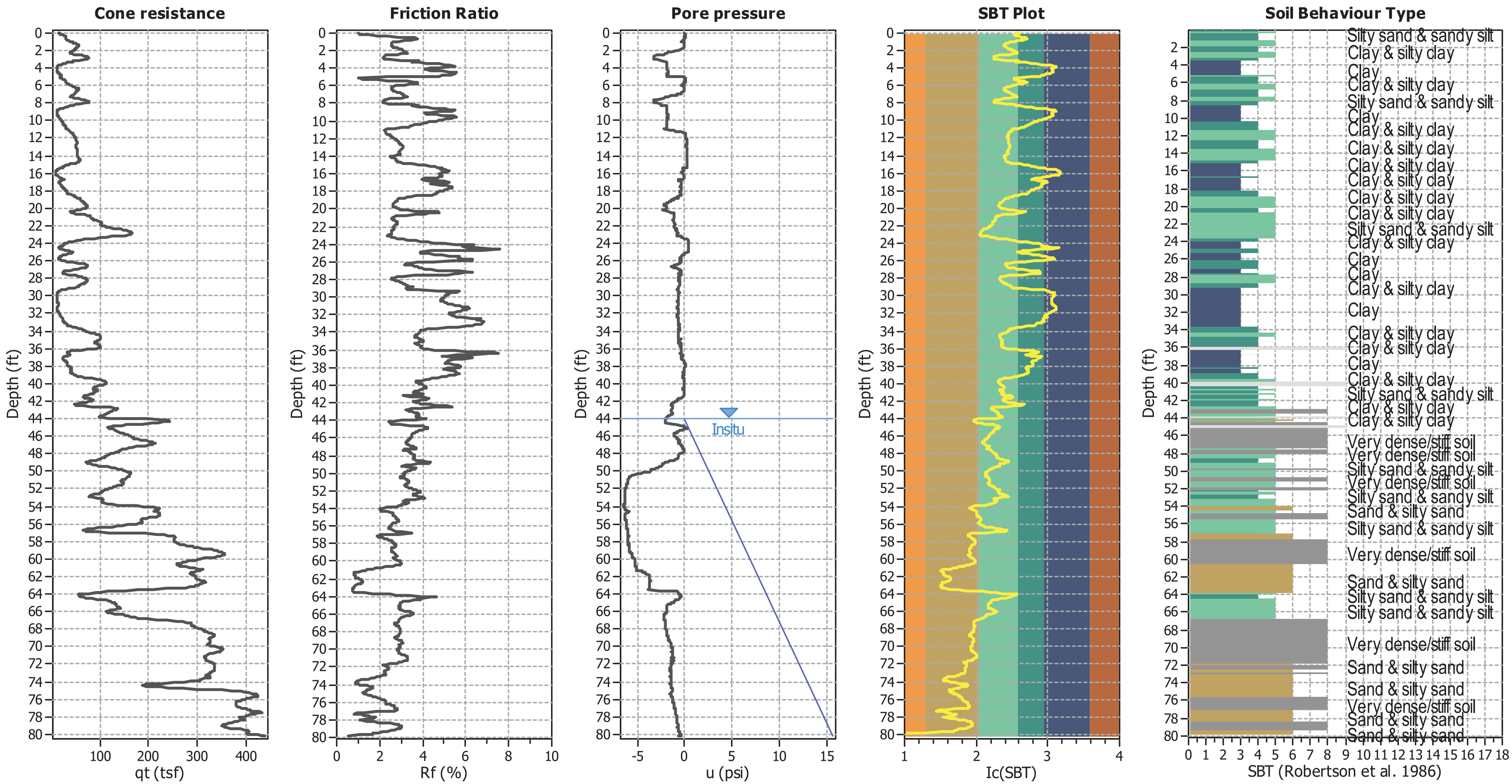
Analysis method:	Moss et al. (2006)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	Moss et al. (2006)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.80	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	52.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

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

Figure - 3

CPT basic interpretation plots



Input parameters and analysis data

Analysis method:	Moss et al. (2006)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	Moss et al. (2006)	Average results interval:	3	Transition detect. applied:	No
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:	0.80	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	44.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

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

Figure - 4

Atlas Technical Consultants
6280 Riverdale Street, San Diego, CA 92120

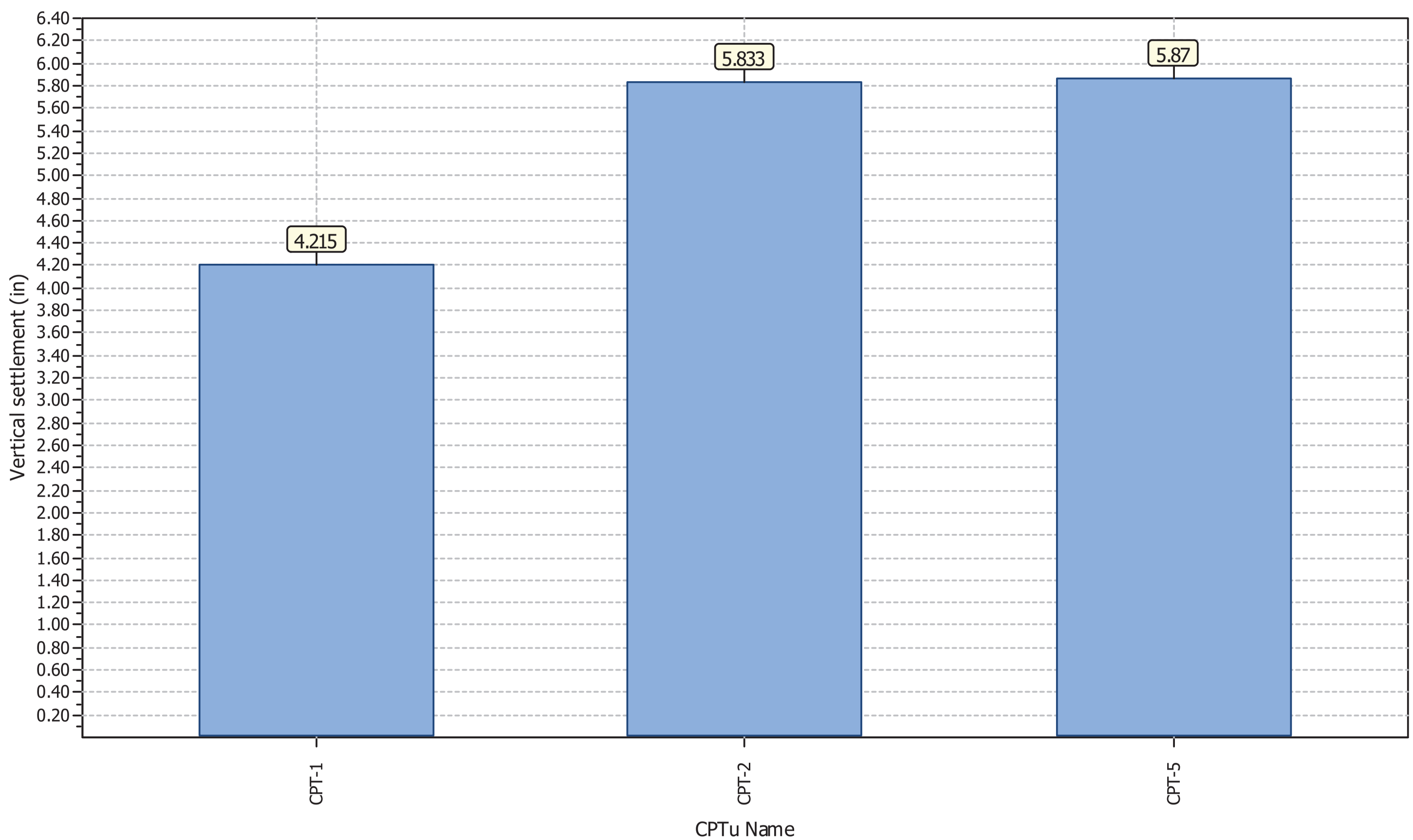
https://www.oneatlas.com/

Project title : 10-57575PW Compton CCD PE Complex GI

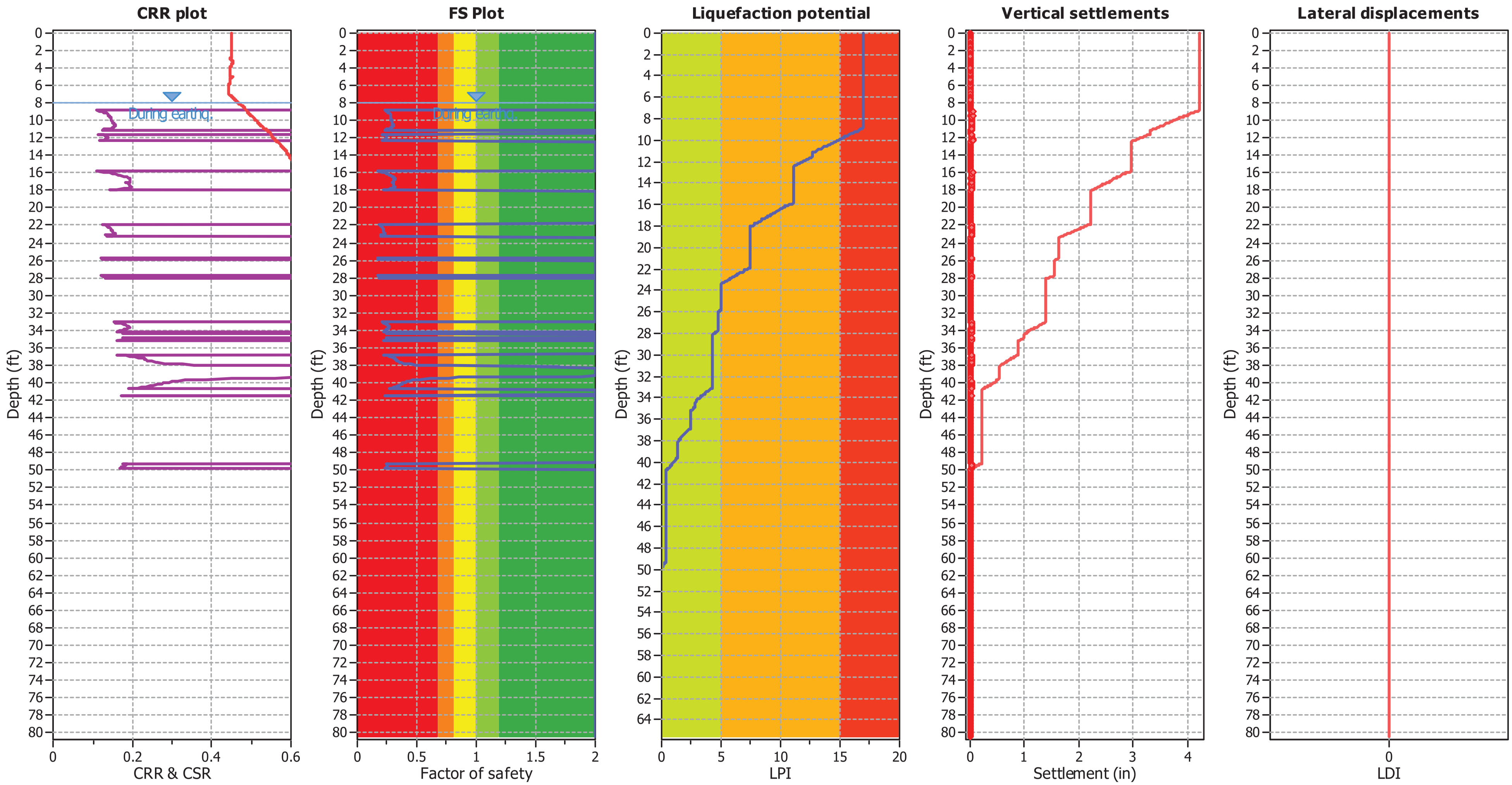
Location : Compton, CA

Analysis Method : I&B (2008)

Overall vertical settlements report



Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	R&W (1998)	Average results interval:	3	Transition detect. applied:	No
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:	0.80	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	44.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

F.S. color scheme

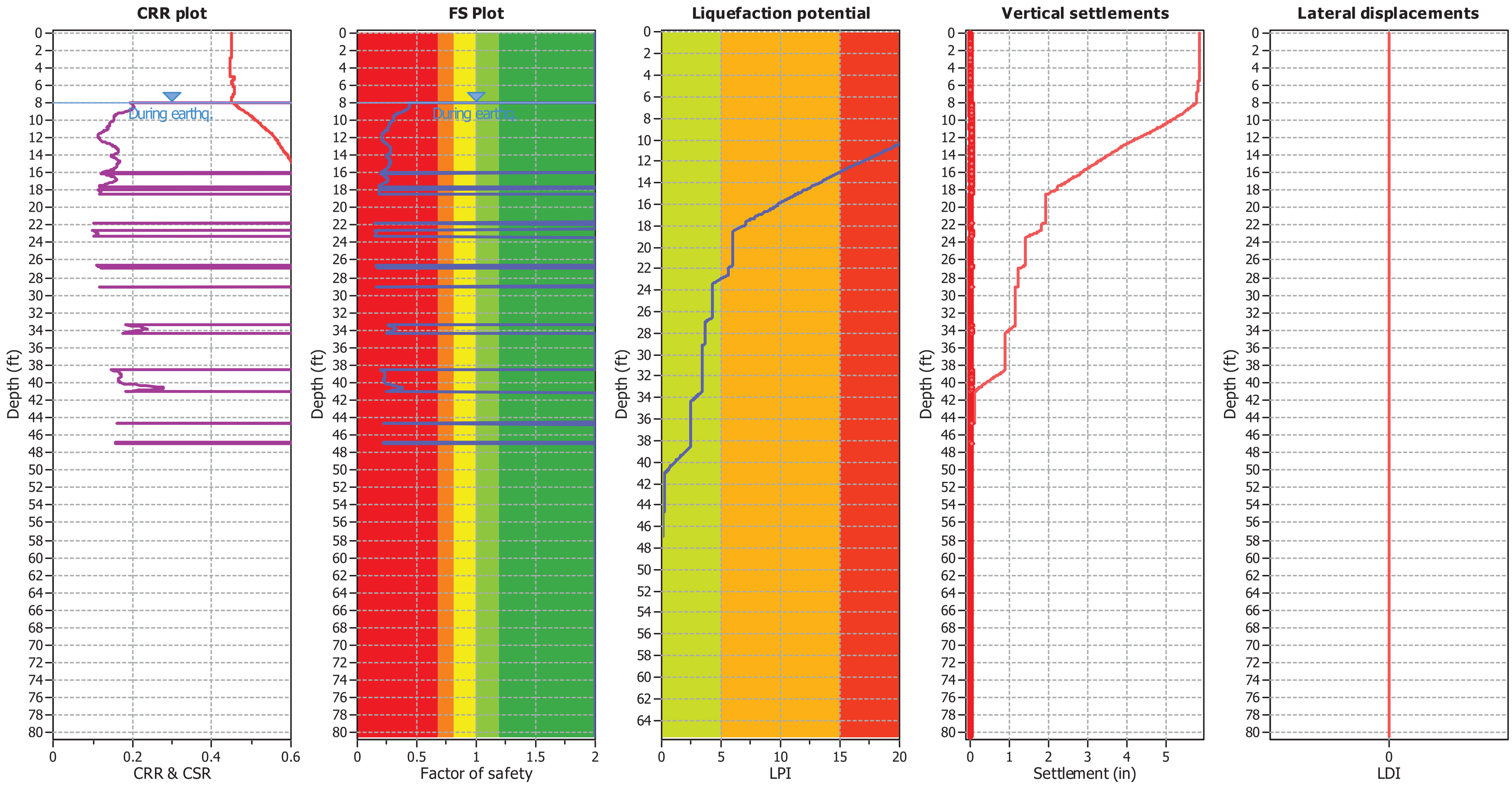
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

Figure - 6

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	R&W (1998)	Average results interval:	3	Transition detect. applied:	No
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:	0.80	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	52.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

F.S. color scheme

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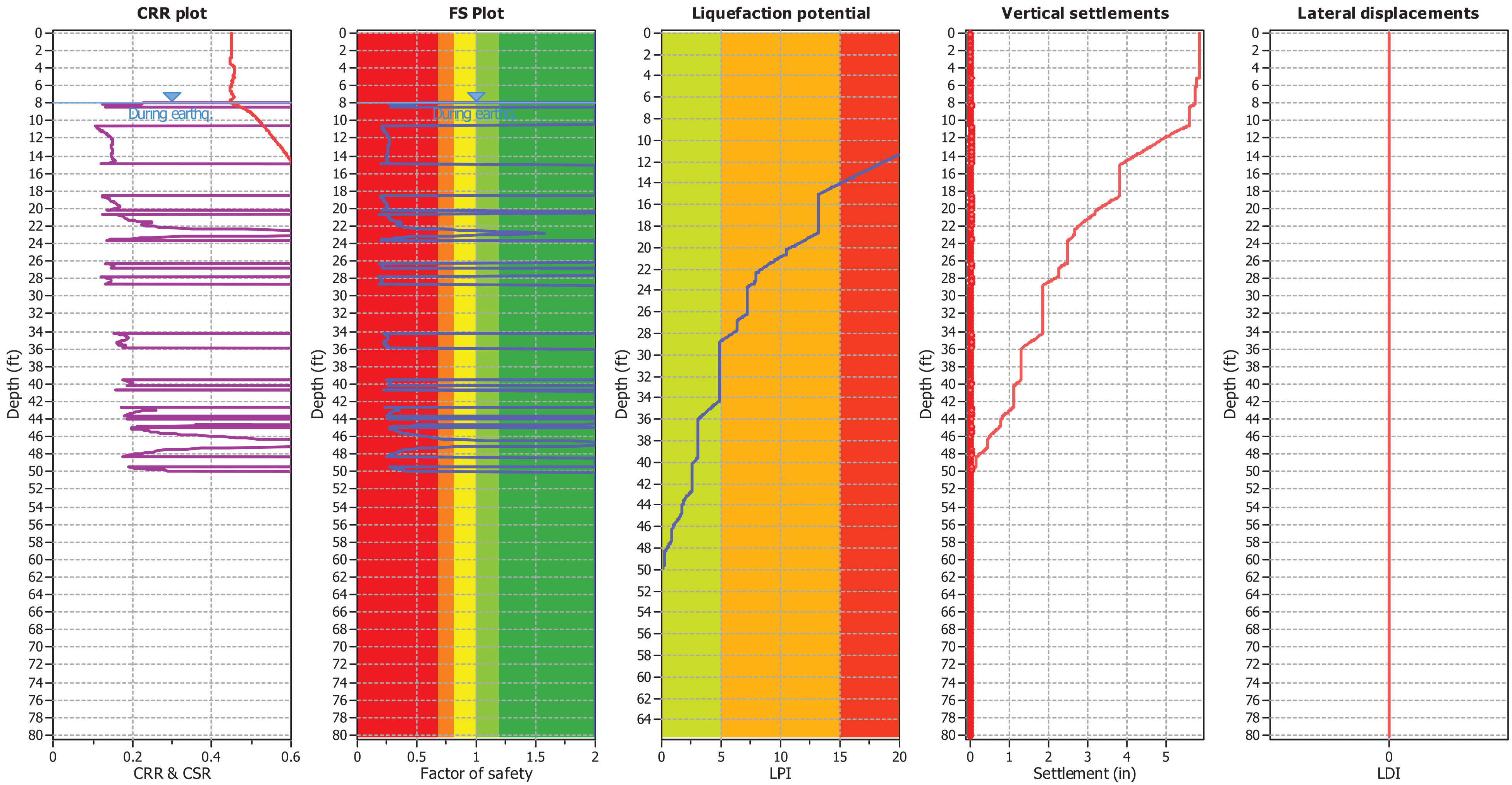
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

Figure - 7

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	R&W (1998)	Average results interval:	3	Transition detect. applied:	No
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:	0.80	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	44.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

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LPI color scheme

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- High risk
- Low risk

Figure - 8

Atlas Technical Consultants
6280 Riverdale Street, San Diego, CA 92120

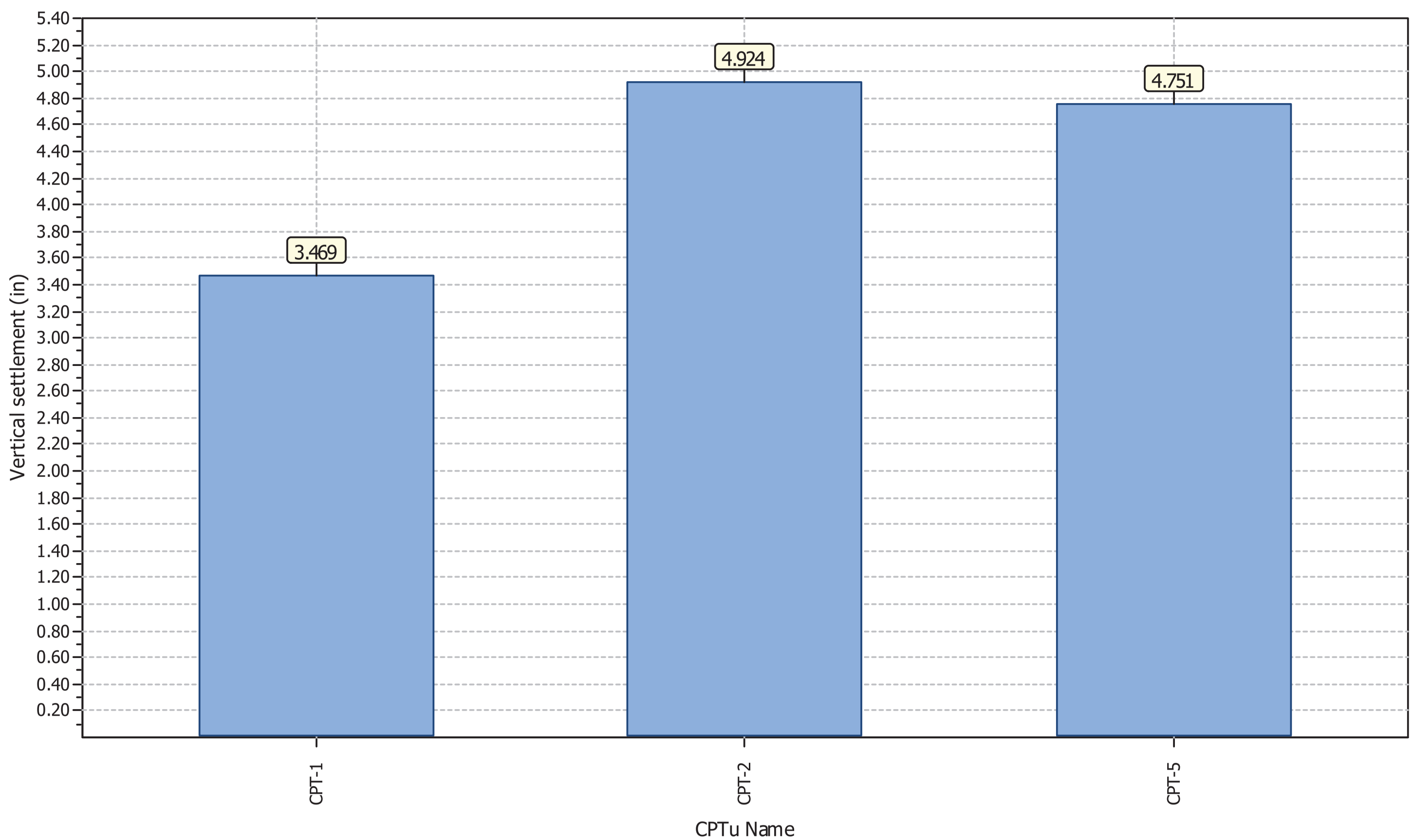
https://www.oneatlas.com/

Project title : 10-57575PW Compton CCD PE Complex GI

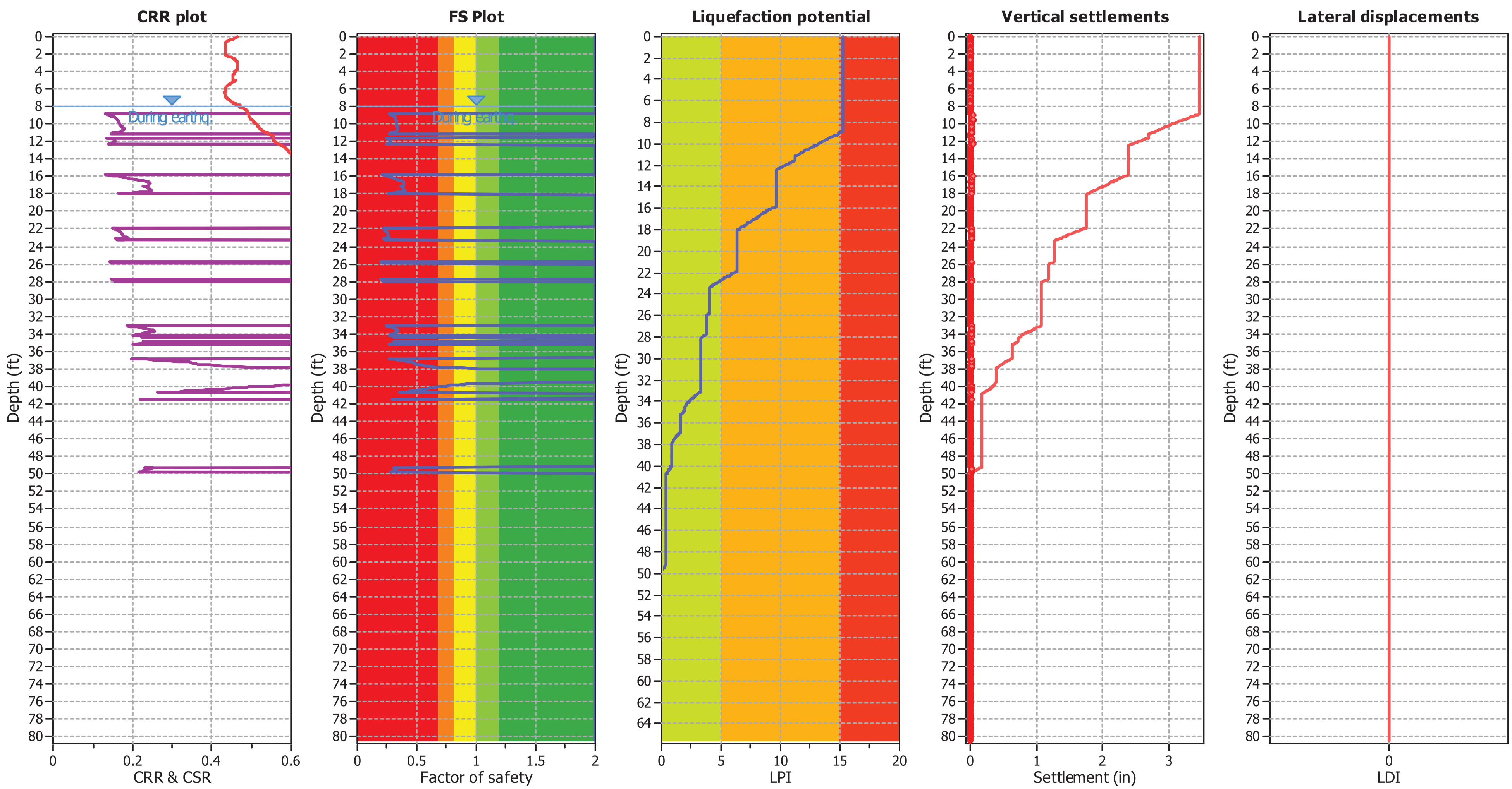
Location : Compton, CA

Analysis Method : B&I (2014)

Overall vertical settlements report



Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
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:	0.80	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	44.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

F.S. color scheme

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- Very likely to liquefy
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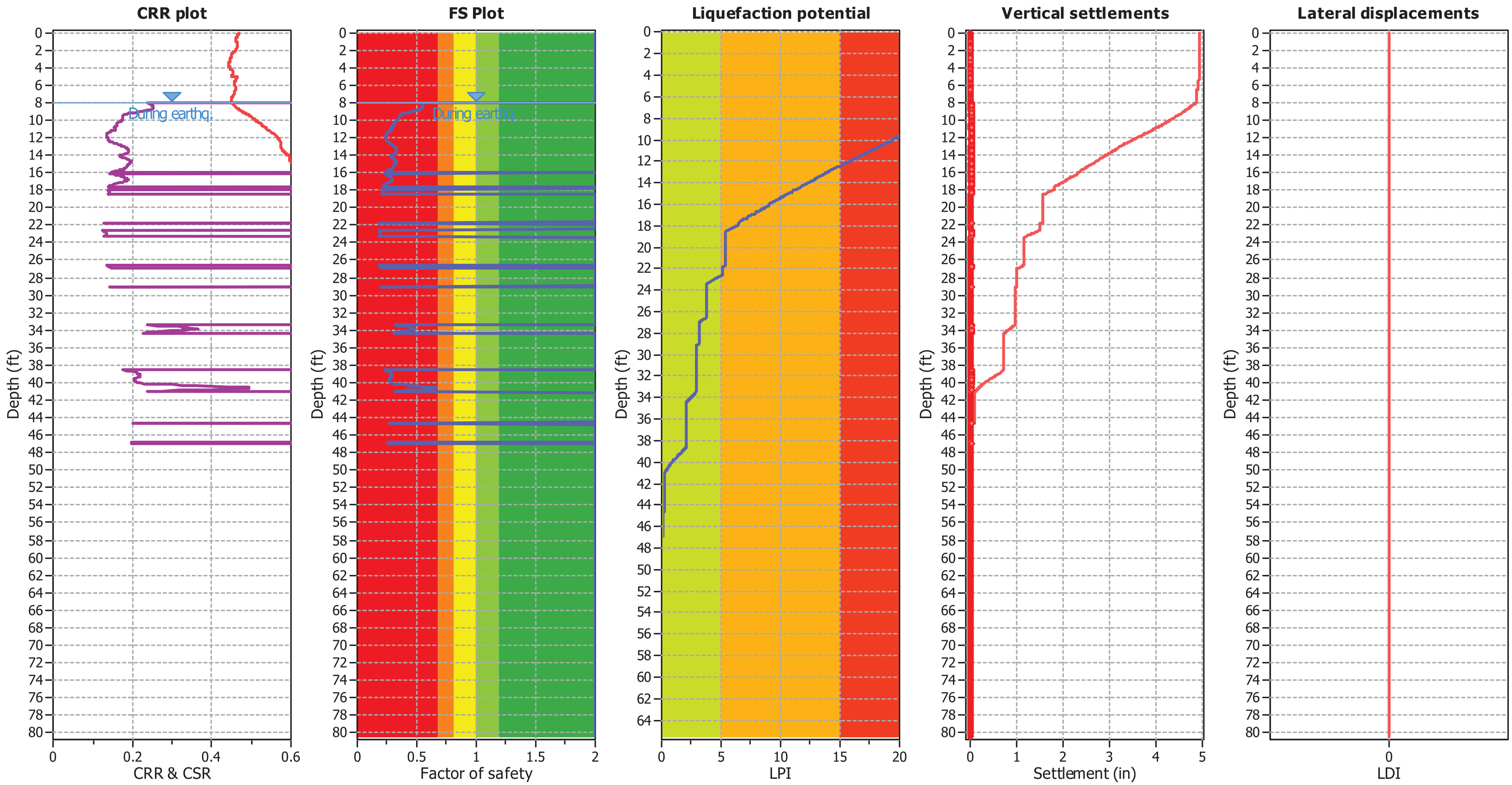
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

Figure - 10

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
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:	0.80	Use fill:	No	Limit depth applied:	Yes
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LPI color scheme

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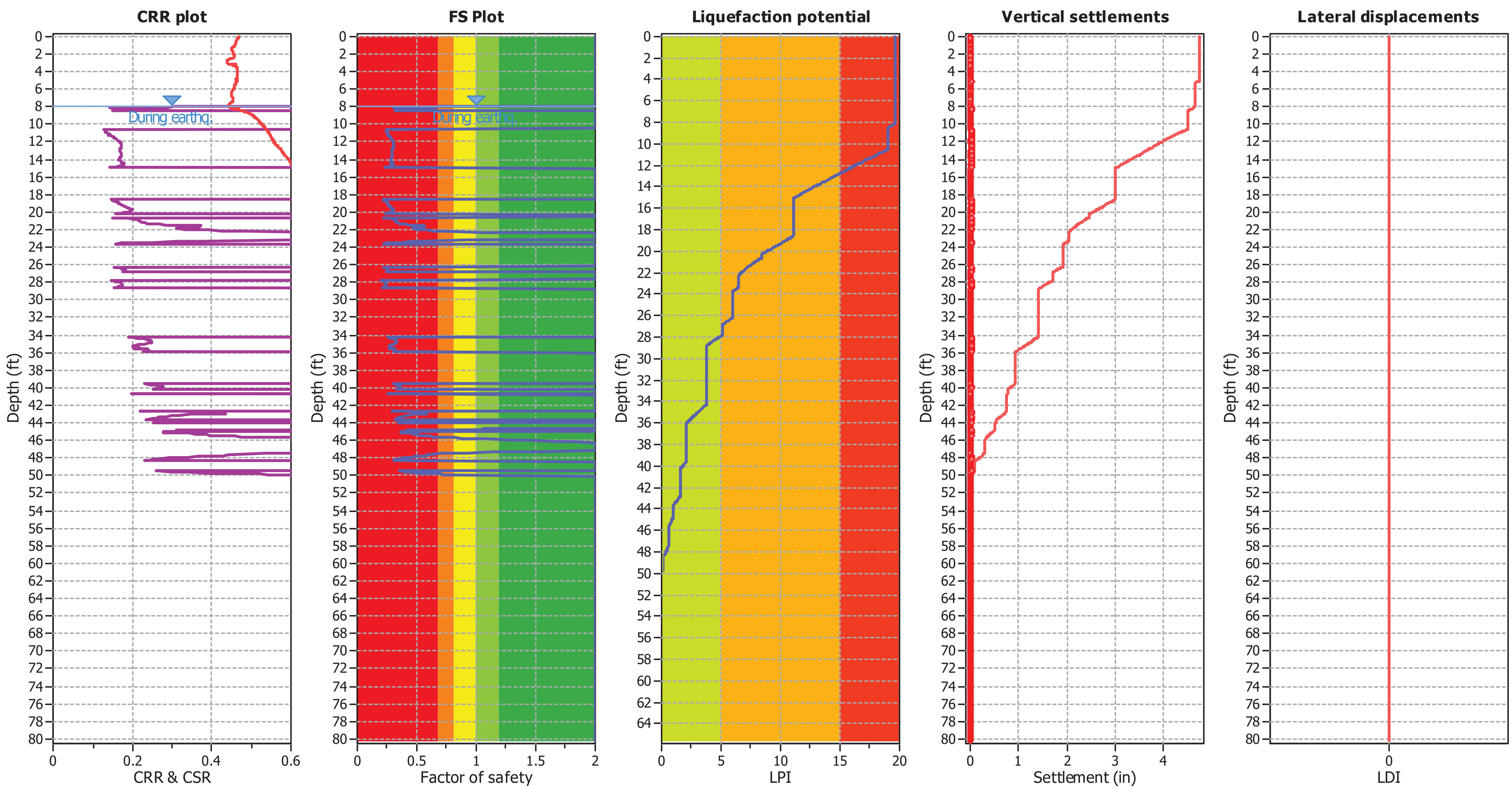
Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
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:	0.80	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	44.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

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- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

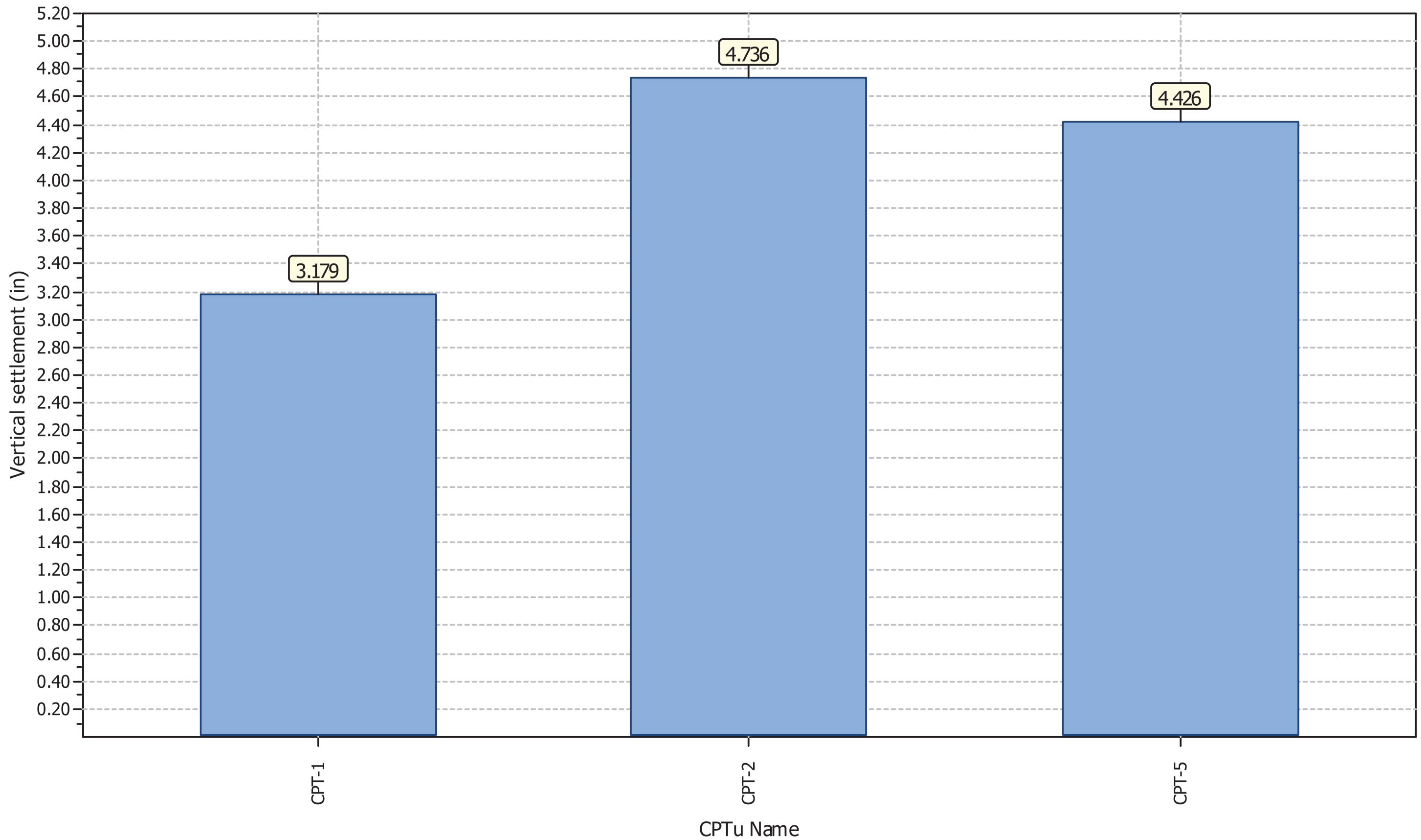
Figure - 12

Project title : 10-57575PW Compton CCD PE Complex GI

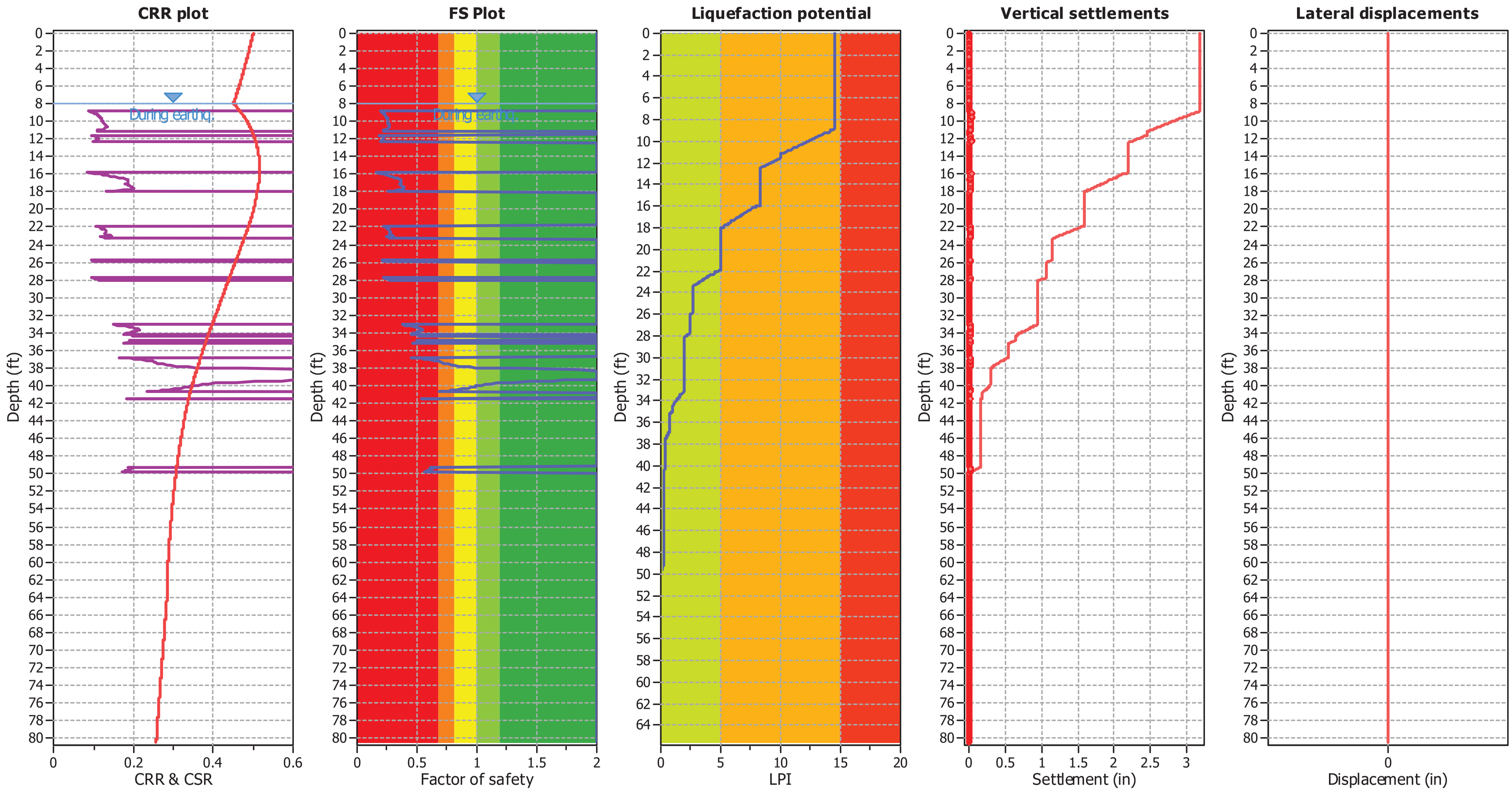
Location : Compton, CA

Analysis Method : Moss et al. (2006)

Overall vertical settlements report



Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Moss et al. (2006)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	Moss et al. (2006)	Average results interval:	3	Transition detect. applied:	No
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:	0.80	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	44.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

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LPI color scheme

- Very high risk
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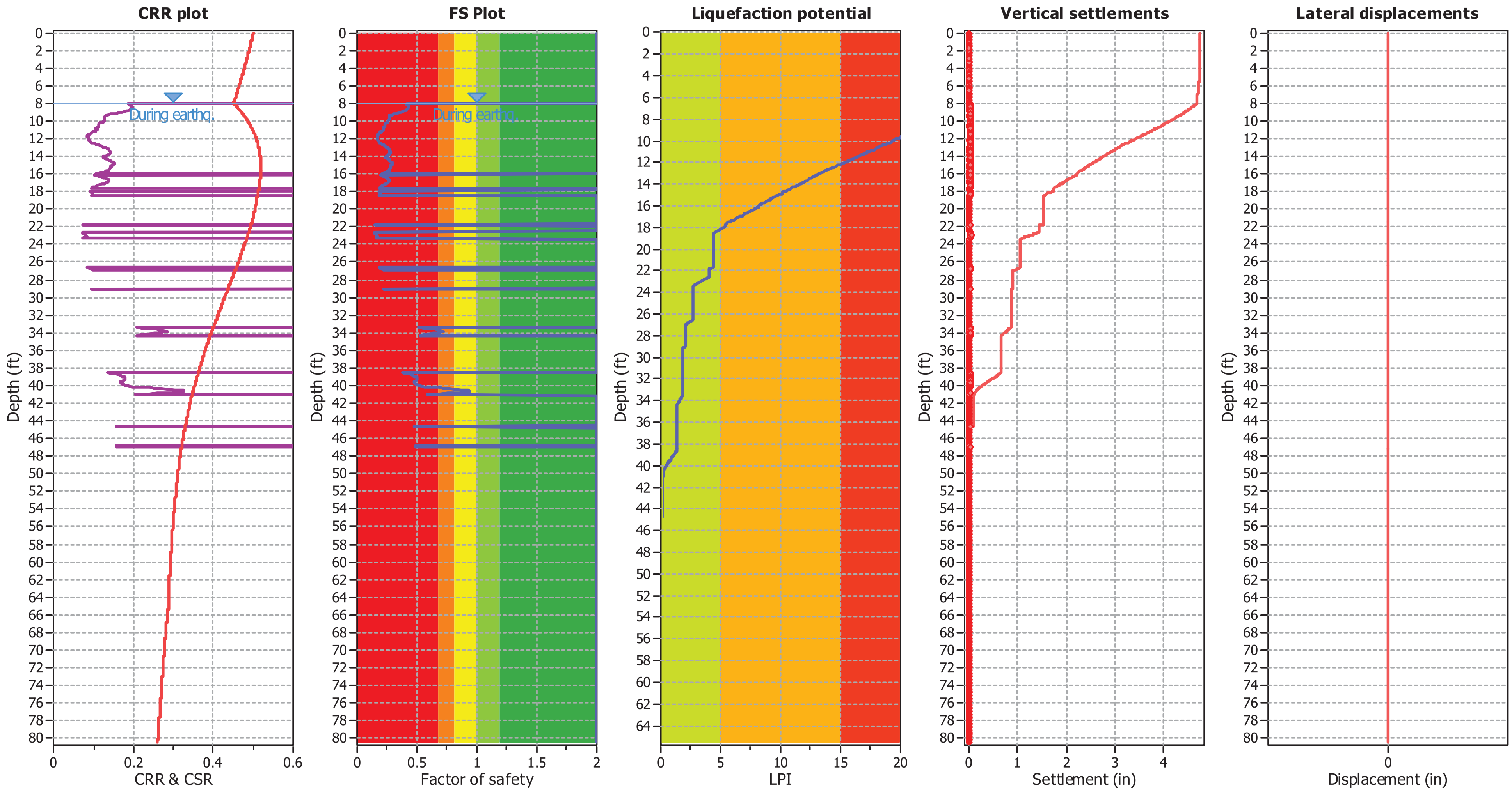
- Low risk

Figure - 14

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Moss et al. (2006)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	Moss et al. (2006)	Average results interval:	3	Transition detect. applied:	No
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:	0.80	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	52.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

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LPI color scheme

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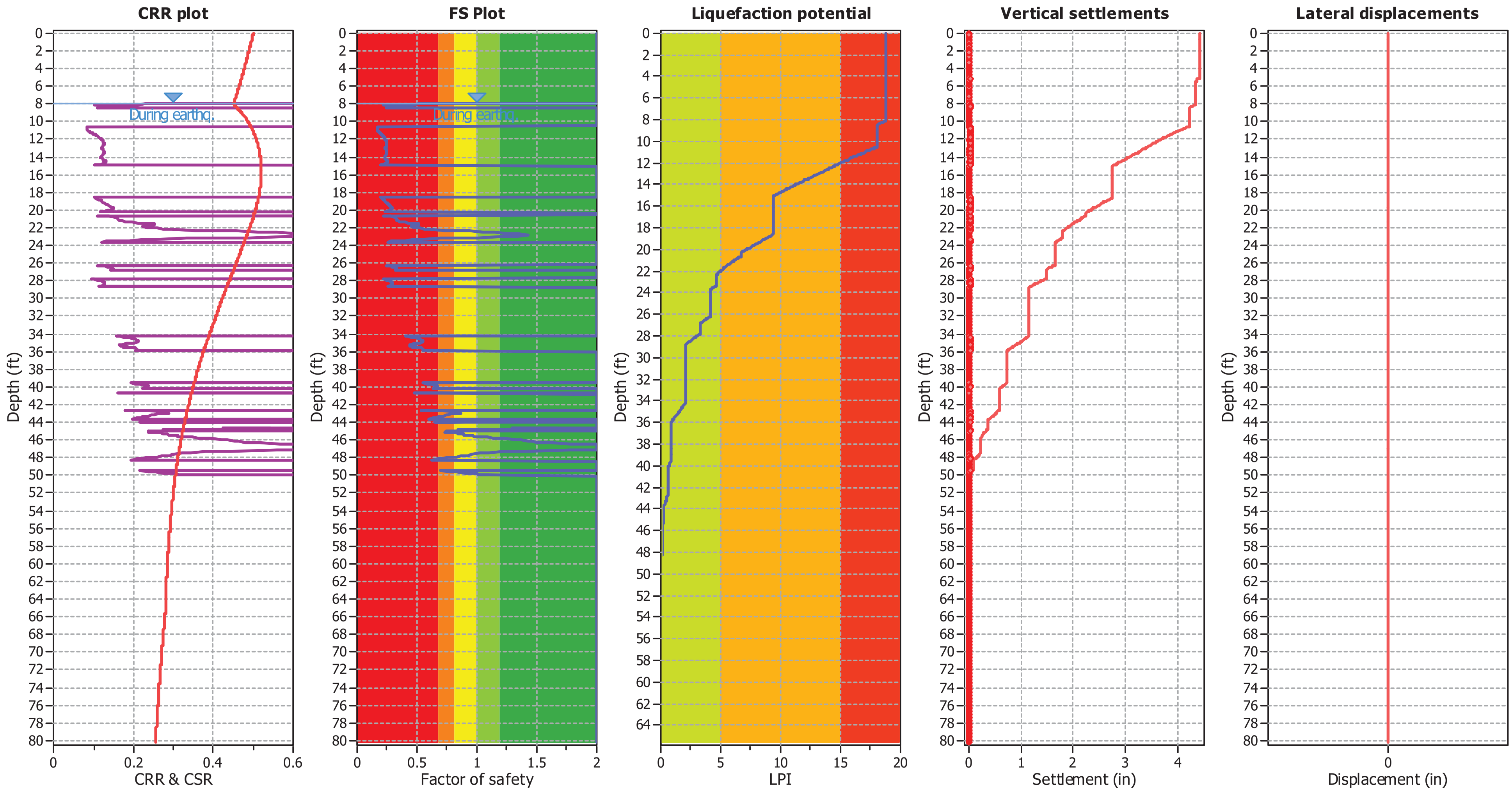
- Low risk

Figure - 15

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Moss et al. (2006)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	Moss et al. (2006)	Average results interval:	3	Transition detect. applied:	No
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:	0.80	Use fill:	No	Limit depth applied:	Yes
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LPI color scheme

- Very high risk
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Figure - 16

ATTACHMENT DD

KNA’S RESPONSES AND DOCUMENTS

March 9, 2022

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

Attention: Ms. Sheri Phillips
Project Manager
PCM2, Inc.

Subject: California Geological Survey (CGS) engineering geology and seismology review for Compton College – Physical Education Complex replacement

Keller North America (Keller) has reviewed the comments made by the California Geological Survey regarding Keller's design submittals for both proposed ground improvement systems for the structures at the project site. Although several comments have been made by CGS, please note that only those which refer to Keller's vibro stone column (VSC) and deep soil mixing (DSM) design submittals are addressed here. Keller's comment responses can be found below in **blue**.

Please note that the designs provided herein has been prepared for the exclusive use of Keller, with the special equipment and production procedure, for our client under the following strict limitations:

1. Only Keller may construct the work described by the design and
2. The design may not be used by others for any purpose.

Keller appreciates the opportunity to be of service. Please feel free to contact the undersigned at (909) 393-9300 with any questions, comments, or concerns.

Respectfully submitted,

REGISTERED PROFESSIONAL ENGINEER
SUNIL ARORA
C72352
EXP. 06/30/22
CIVIL
STATE OF CALIFORNIA

Sunil Arora, P.E.
Project Executive
Keller North America

David Chae
Assistant Project Manager
Keller North America

Bailey Uy
Engineer
Keller North America

California Geological Survey (CGS) engineering geology and seismology review for Compton College – Physical Education Complex replacement – Keller responses

22. Mitigation Options for Liquefaction/Seismic Settlement: **Additional information is requested.** Due to the potential for seismic settlement and surface manifestation resulting from liquefaction, the consultants recommend performing ground improvement to mitigate those hazards and facilitate the use of shallow foundations for support of proposed structures. As noted previously in Item 10, the consultants provide preliminary recommendations for ground improvement by installation of **VSCs and/or DSM** to mitigate liquefaction and reduce the potential static and seismic settlement at the locations of proposed improvements. CGS notes the preliminary recommendations the consultants provide include minimum area replacement ratios (ARRs), column dimensions and plan layout requirements, minimum bearing capacities, and maximum differential settlement criteria for design of the VSC/DSM systems. They recommend **a specialty ground improvement contractor should perform the detailed design and draft plans for the selected ground improvement system**, and the consultants provide preliminary recommendations for field testing requirements, quality assurance procedures, and final acceptance criteria for the VSC/DSM ground improvement.

The ground improvement design submittals from the specialty ground improvement design-build contractor, KNA, provide the detailed VSC and DSM ground improvement design packages for the proposed PE building and pool mechanical building/swimming pool, respectively, including copies of their geotechnical calculations and a draft set of plans for each type of proposed ground improvement. Based on our review, **CGS requests KNA and/or the consultants provide further information to address the following concerns regarding their design and plans for ground improvement**:

<u>General</u>:

- As noted in Item 7, it appears that KNA has relied on analysis of data from three CPT soundings for design of the VSC and DSM ground improvement planned for the project, but no discussion or original logs of these explorations have been provided for our review.

  It is noted that the referenced CPT explorations were performed after the Atlas's initial report was published. It is our understanding that Atlas will provide the CPT files and may amend their report to include these new explorations.

  Keller has reviewed all explorations and laboratory testing data performed for the project and has elected to perform our analysis based on the provided CPTs because the resolution of layering obtained by the CPT data has been deemed to be more accurate for this site.

  Due to their proximity to the proposed structures, Keller has used CPT 1 as a basis for the VSC ground improvement design at the proposed PE Building and CPTs 2 and 5 as a basis

for the DSM ground improvement design at the proposed Mechanical Building and Swimming Pool. Revised submittals for the VSC and DSM ground improvement designs can be found in Attachments A and B respectively.

Based on Atlas' experience on the project site, it is Keller's understanding that there may be more variation in the soil profile than what is portrayed in the CPTs. Therefore, Keller has used a conservative depth of treatment as provided by Atlas.

<u>VSC Ground Improvement for PE Building</u>:

- The consultants refer to the requirements of 2019 CBC Section 1813A within their recommendations for VSCs to be installed under the entire building/structure footprint and to extend beyond the footprint of structure/foundation at least half the depth of the VSCs with a minimum of 10 feet or an approved alternative. They also cite 2019 CBC Section 1813A and recommend a minimum of four VSCs (or approved equivalent) should be located under each isolated or continuous/combined footing and that VSCs under the shallow foundations should be located symmetrically around the centroid of the footing or load. However, based on our review of VSC plan layout provided by KNA, it appears the design layout of VSCs does not satisfy the consultants' recommendations or the 2019 CBC requirements for extent of VSCs beyond the building perimeter, nor for multiple/symmetrical location of VSCs under foundations.

  As determined in the CPTs, the site is mostly comprised of silts and clays. Because the liquefiable layers are relatively thin and discontinuous, it is Keller's opinion that a minimum of one row of VSCs outside of the building footprint is adequate. Additionally, per Atlas's addendum report dated September 7, 2021, in locations where it is not be physically possible to construct an additional row of columns outside of the building footprint, the secondary row of VSCs can be added within the perimeter grid. That is, adding an additional column within the proposed grid along the entire perimeter of the subject area/zone. This secondary column should be located at the center of each of the four VSCs.

  Based on our conversations with the project team, it is Keller's understanding that there are existing underground utilities along the project west and project south sides of the building; VSCs along these sides cannot be installed as initially proposed and the columns in conflict would need to be relocated. Thus, Keller has elected to relocate these columns inside the building footprint.

  As noted in Keller's VSC submittal, Keller uses a stiffness ratio of 6 as the basis of our design. This parameter can be considered to represent the strain compatibility between the in-situ soil and the stone column. Because the difference in stiffness ratio is minimal and Keller's proposed VSC design is not required to increase the allowable bearing pressure to high values (6 ksf or more), Keller's proposed VSC design will result in improvement of the surrounding soils such that the modulus of subgrade reaction, long-term settlement, and post-earthquake settlement can be considered uniform throughout. Thus, VSCs do not need to be placed directly under foundational elements. As noted in Keller's submittal, this assumption shall be verified by virtue of six (6) post-construction CPTs.

- CGS observes that the calculations of VSC bearing capacity provided by KNA do not consider the presence of VSC columns (ARR = 0 and KNA reports they "used the pre-treatment soil parameters for this computation"). However, CGS notes the KNA calculations are based on the input of a significant value of shear strength for the soils (input as an effective soil cohesion with no friction) and we request justification for this value based on the available geotechnical data for the site.

  Based on Keller's review of Atlas's geotechnical report, it is Keller's understanding that a single direct shear test has been performed by Atlas on a sample collected from boring B-4 at a depth of 11 to 11.5 feet. Because boring B-4 lies within the footprint of the proposed PE building, the boring looks to have a relatively uniform soil profile, and the adjacent borings look to be relatively consistent, the friction angle and cohesion values established by this test can be considered representative of the in-situ soil parameters underneath the proposed PE building.

  Thus, Keller has used a friction angle, ϕ, of 38° and cohesion, c, of 320 psf for our bearing capacity calculations. As shown in the proposed VSC layout, foundational elements are not always underlain by stone columns. To account for this, Keller has conservatively computed the bearing capacity underneath all footing types without considering the reinforcing effects of the proposed VSCs and additional strength gained by the VSC installation process.

  For an allowable bearing pressure of 3000 psf, all footing types for the proposed PE building exceed the generally accepted minimum factor of safety of 3. Details of these computations can be found in Attachment C.

- CGS notes KNA has estimated the potential static settlement of the PE building supported by VSCs based on their consideration of a uniform surcharge load applied over the entire footprint of the building. However, this does not adequately address the potential static settlement of individual foundations supporting the PE building, and CGS requests the calculations and design of the VSC system be updated as appropriate to address and consider static settlements of foundations in accordance with the recommendations of the geotechnical consultants.

  Keller has computed the static settlements under each of the footings using the commercially available software CPeT-IT v.3.6.1.5 by GeoLogismiki. For this analysis, Keller has assumed a uniform load of 3000 psf (corresponding to a max load of 48 kips for F1 footings, 108 kips for F2 footings, 147 kips for F3 footings, and 271 kips for F4 footings) as this is the maximum allowed bearing pressure allowed for spread footings on top of VSCs per Atlas's addendum report. Based on Keller's review of the structural plans and loads as provided by Brandow and Johnson on February 10, 2022, no footings exceed these loads.

Additionally, Keller did not consider the reinforcing effects of the proposed VSCs and additional strength gained by the VSC installation process to simplify the static settlement analysis.

As seen in Attachment D, the pre-treatment settlements range between 0.04 inches and 0.20 inches. As these values are lower than the post-treatment static settlement criteria of 0.75 inches, it is Keller's opinion that the VSCs are not required underneath foundational elements to meet the static settlement criteria prescribed by Atlas.

<u>DSM Ground Improvement for Pool Mechanical Building/Swimming Pool</u>:

- CGS notes KNA provides discussion of QA/QC requirements and acceptance criteria for the DSM ground improvement in their design submittal and on Sheet KNA-2 of the shop drawings that generally appear to be reasonable and appropriate. However, a complete set of specifications for the installation, testing, and performance of the DSM system should be drafted and provided for CGS review. The specifications should include fully detailed and well-defined acceptance criteria for evaluation of the successful completion of the ground improvement and satisfaction of design and performance objectives.

  Specifications for the installation, testing, and performance of the DSM system for both Atlas and CGS review are provided in Attachment E.

- The specifications for verification of DSM quality should include requirements that the selection of locations for confirmation coring and selection of core samples for UCS testing are subject to review and approval of the Geotechnical Engineer of the Record (GEOR) for the project.

  Keller has updated our shop drawings to reflect this comment.

- The consultants recommend that coring for verification should be performed on at least 2% of the DSM columns, which appears to be reasonable. However, CGS observes that a lower percentage of cores is shown in the Notes and Details on Sheet KNA-2 of the KNA shop drawings. KNA is requested to update these Notes to conform with the geotechnical recommendations. In addition, KNA is requested to revise their sampling statement and indicate that the coring locations should be selected by the GEOR for the project.

  Keller has updated both our submittal and shop drawings to reflect this comment.

- CGS requests KNA to report the typical and maximum DSM grid panel spacing considered in their design and plan layout of DSM columns to justify the ARR value critical to design of the system. We note that the DSM column diameter and overlap dimensions are indicated on the shop drawing sheet KNA-3 but also request the panel spacing dimensions also be clearly indicated on the plans.

  Keller has updated our shop drawings to reflect this comment.

Additionally, in fulfilling the role as GEOR for the project, **the geotechnical consultants should submit formal documentation of their review of the contractors' VSC and DSM design and plans** that includes an explicit statement regarding their opinion of the conformance of the design with their geotechnical recommendations.

Enclosed attachments

Attachment A – Vibro stone column design revised submittal
Attachment B – Deep soil mixing design revised submittal
Attachment C – Bearing capacity calculations
Attachment D – Static settlements under footings
Attachment E – Deep soil mixing sample specification

Attachment A

Vibro stone column revised submittal

Keller North America
17461 Derian Avenue, Suite 106
Irvine, CA 92614
Tel : 909-393-9300
Fax : 909-393-0036

Vibro Stone Columns Design
Physical Education Building of Compton Community College
Revision 1

1111 East Artesia Boulevard
Compton, California

Submitted to:
PCM3, Inc.
Compton CCD Office

Submitted by:
Keller North America

March 7, 2022

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PCM3
Compton CCD Office

Attention: Ms. Sheri Phillips
Subject: Vibro Stone Column (VSC) Ground Improvement Design

Keller North America (Keller) is pleased to present the following design submittal for ground improvement for the proposed buildings at this project site. The purpose of the ground improvement program is to enhance the safety, stability, and serviceability of the proposed structures. This is accomplished by increasing the strength of the ground to the point where the ground can safely support the anticipated structures under static loads as well as during and after the design level earthquake. Additional information is provided in the attached report.

The design provided herein has been prepared for the exclusive use of Keller, with the special equipment and production procedure, for our client under the following strict limitations:

1. Only Keller may construct the work described by the design and
2. The design may not be used by others for any purpose.

Keller appreciates the opportunity to be of service. Please feel free to contact the undersigned at (909) 393-9300 with any questions, comments, or concerns.

Respectfully submitted,

REGISTERED PROFESSIONAL ENGINEER
SUNIL ARORA
C72352
EXP. 06/30/22
CIVIL
STATE OF CALIFORNIA

David Chae, Assistant Project Manager	Sunil Arora, P.E. Project Executive	Bailey Uy Engineer

1. DESIGN SUMMARY

This project site is located at 1111 East Artesia Boulevard, Compton, California. Based on our review of the provided documents, the proposed construction is a 2-story Physical Education (PE) building supported by shallow spread footings with slab-on-grade.

Keller North America (Keller) proposes installing Vibro Stone Columns (VSC) to limit the total differential settlement to 2.88 inches over a horizontal distance of 40 feet (0.006 L). These columns shall have a minimum diameter of 36 inches, spaced in a square grid pattern, 8 feet on-center, and extend to a depth of 23 feet below the existing ground surface. The working grade for Keller will be near the existing ground elevation. The densification results will be verified by liquefaction analysis based on post-treatment CPTs. Our shop drawing plans are presented in Appendix A.

2. GROUND IMPROVEMENT DESIGN BASIS

This design is based on Keller's review of the following documents and performance requirements articulated by the project structural, geotechnical, and civil engineers. Although many documents were reviewed, only those which provided information that directly affects our design are listed below:

- Geotechnical Investigation Report, Physical Education Complex Replacement, Compton Community College District, by Atlas Technical Consultants, LLC, dated July 7, 2021
- Addendum Geotechnical and Geohazard Report, Physical Education Complex Replacement, Compton Community College District, by Atlas Technical Consultants, LLC, dated September 7, 2021
- CPT Data – Compton Site, CPT-1, CPT-2, and CPT-5, dated September 3, 2021
- Preliminary Foundation Schemes, by Brandow & Johnston, Inc., dated July 13, 2021

If any of these documents are changed or altered in any way, Keller should be notified, and the design may require modifications.

2.1 Subsurface Conditions

Based on our review of the Geotechnical Investigation Report by Atlas Technical Consultants LLC (Atlas), it is Keller's understanding that the site is generally underlain by about ½ foot of grass/topsoil/surficial fill and young alluvial deposits of Holocene to late Pleistocene age. These alluvial deposits are primarily comprised of inter-layered silty sands and sandy silts. In general, the near-surface sandy soil 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. Per Atlas's Geotechnical Investigation Report, groundwater was encountered at a depth between 44 feet to 52 feet below the existing ground surface.

2.2 Design Criteria

The ground improvement design criteria have been established by the project geotechnical and structural engineers and summarized in Table 1 below. Keller has reviewed the criteria and they appear typical and reasonable for this type of project.

Table 1: Design Input and Performance Criteria

	Criteria	Reference
Groundwater Level (Static)	44’ below grade	Atlas Technical Consultants, LLC
Groundwater Level (Seismic)	8’ below grade	
PGA_M (ASCE 7-16)	0.802 g	
Mw (ASCE 7-16)	7.3	
Depth of Liquefaction Analysis	50 feet	
Allowable Bearing Pressure	4,000 psf	
Total Post-treatment Differential Settlement	≤ 2.88 inch over 40 feet (0.006*L)	Brandow & Johnston, Inc. based on Table 12.13-3 of ASCE 7-16 for Risk Category III building

3. STATIC DESIGN

3.1 Foundation Bearing Capacity

Keller has verified the soil bearing capacity (shown in Table 2) of several typical spread footings based on Preliminary Foundation Schemes by Brandow and Johnston, Inc. Conservatively, Keller used the pre-treatment soil parameters for this computation. In conclusion, the VSC treated soil will provide adequate foundation bearing capacity. Please refer to Appendix B for computation details and the corresponding geometry illustration based the provided structural drawing. The calculated factors of safety meet or exceed the generally accepted minimum factor of safety of 3.

Table 2: Factor Safety against Soil Bearing Capacity Failure

Footing	Size	Bearing Capacity	Factor of Safety
F1	4-ft x 4-ft	3,000 psf	5.6
F2	6-ft x 6-ft		6.3
F3	7-ft x 7-ft		6.8
F4	9.5-ft x 9.5-ft		7.5

3.2 Static Settlement Estimation

Keller has computed the static settlements under each of the footings using the commercially available software CPeT-IT v.3.6.1.5 by GeoLogismiki considering the reinforcing effects of the proposed VSCs and additional strength gained by the VSC installation process. As seen in the table below, the pre-treatment settlements range between 0.04 inches and 0.20 inches. As these values are lower than the post-treatment static settlement criteria of 0.75 inches, Keller’s proposed VSC design will meet the static settlement criteria prescribed by Atlas. Please refer to Appendix C for computation details.

Table 3: Estimated Total Static Settlements

Footing ID	Dimensions	Computed Static Settlement, inch
F1	4-ft x 4-ft	0.04
F2	6-ft x 6-ft	0.09
F3	7-ft x 7-ft	0.13
F4	9.5-ft x 9.5-ft	0.20

As shown in this section, the computed static performance of the proposed foundation meets the expected design criteria.

4. SEISMIC DESIGN

5.1 VSC Densification Technical Background

The installation of stone columns at this site will seek to mitigate the liquefaction potential by densification, partial replacement, and reinforcement. We are proposing the implementation of vibro stone columns by the "dry bottom feed process". The degree of densification resulting from the installation of vibro stone columns is a function of many factors, including:

- Soil type, silt, and clay content,
- Uniformity of soil gradation,
- Plasticity of the soil,
- Pre-treatment relative densities,
- Vibrator type and energy output,
- Stone shape and durability,
- Stone column area and spacing between stone columns.

Note that soils with more than about 25% fines (passing through #200 sieve) or with 5% clayey particles may NOT be densifiable. To estimate the degree of densification improvement required to meet the liquefaction-induced settlement acceptance criteria (Table 1), Keller will perform liquefaction analysis on post-treatment CPTs.

5.2 Estimation of Densification from VSC

Baez (1995) describes a procedure for the estimation of stone column parameters (column diameter and spacing between columns) required to achieve certain post improvement penetration values in sands and silty sands. Based on this procedure and Keller's proprietary data base we have determined that a 36" diameter VSC at 8' by 8' grid pattern, with an equivalent area replacement ratio of 11% is expected to meet the liquefaction mitigation performance requirements.

Based on Atlas Technical Consultants experience on the project site there may be more variation in the soil profile then what is portrayed in the CPT. Therefore, Keller is using conservative depth of treatment as provided by the project GEOR.

Keller has reviewed the SPT boring data, and the SPT-based liquefaction analysis performed by Atlas Technical Consultants, LLC. Since the resolution of layering obtained from the CPT data is deemed to be more accurate, Keller has proceeded with the CPT-based liquefaction analysis.

Keller has estimated the CPT-based post-treatment liquefaction-induced settlement using triggering method of Robertson (NCEER R&W 1998) with settlement method proposed by Zhang et al. (2002), shown in Table 4. Please refer to Appendix D for detailed computations.

Table 4: Post-Treatment Liquefaction-induced Settlement

Exploration	**Post-treatment Liquefaction Settlement (inch)**
CPT-1	1.39

5.3 VSC Densification Verification

The acceptance criteria of the stone column treatment will be based on verifying densification by means of six (6) post-treatment CPT tests performed by Atlas Technical Consultants, LLC. Please refer to Keller's shop drawing for proposed post-treatment CPT locations.

Post-treatment CPTs shall be located close to (preferably within 10 feet) the pre-treatment CPTs whenever possible, so that Ic from pre-treatment CPTs can be used for post-treatment liquefaction analysis. Ic values after stone column treatment often shift to lower values, suggesting the soil becomes coarser and less plastic. But the stone column treatment does not change the soil type and therefore the original Ic values should be used in liquefaction analyses (Nguyen et al. 2014). This can be achieved by correcting (or shifting) the post-treatment Ic back to the pre-treatment Ic.

CPTs will be performed at the center point between four adjacent stone columns. A minimum of 7 days (preferably 14 days or more if possible) shall pass after installation of stone columns before CPT testing is conducted. This will allow the dissipation of the excess pore water pressure induced by the vibrator.

The CPTs will be analyzed for liquefaction triggering and settlement using the design methods described earlier. If the initial CPTs does not meet acceptance criteria, additional CPTs may be performed later to allow for additional porewater pressure dissipation and aging. Additional CPTs may also be performed to better define the limits of any non-conforming work. If this CPT testing shows area where the post-improvement liquefaction differential settlement is not met, additional stone columns may be installed at locations to achieve the performance specification. Keller may elect to perform its own additional site exploration at any time and for any reason during the course of the project.

5. CONSTRUCTION

Method: VSC technique uses specialty purpose-built depth vibrators to densify and reinforce the soils while constructing a VSC of an average 36-inch diameter. The installation process consists of imparting energy by means of vibrations that are generated close to the tip of the vibrator and are produced by rotating eccentric weights mounted on a shaft. An electric motor turns the eccentric weights. Follower tubes are added to achieve the design depth. The follower tube has visible markings at regular increments that enable measurement of penetration and re-penetration depths. If the vibrator encounters refusal, then the ground improvement design engineer shall review this location to determine if additional work is necessary. Predrilling may be employed with a 24-inch or 30-inch diameter auger. The intent of predrilling is to loosen the soil to increase the penetration rate of vibrator. The depth of predrilling may be up to the designed tip of VSC.

Bottom Feed: For this project, Keller plans to utilize the bottom-feed method of VSC construction. The vibrator will then advance to the design depth and the vibrator is lifted in stages as the stone is fed through a side pipe and expelled at the tip of the vibrator. Installation of VSC by the bottom feed method displaces the ground. Some heave or settlement may occur across the areas worked.

Equipment: Major support equipment anticipated to be utilized for VSC construction are:

- Vibrator Hung Caterpillar 365C excavator
- Drill Rig for pre-drilling
- Generator to power the vibrator
- Air Compressor to push gravel through the follower tube
- Loader to move gravel from stockpile to skip bucket (hanging from crane)
- Keller S23 Bottom Feed Vibrator System

Following VSC installation, excess material shall be removed by others. A minimum of the top 12-24 inches disturbed soil shall be excavated with compacted engineering fill by others. Geotechnical Engineer of Record may elect to use onsite material within the treated area for scarification and re-compaction. General Contractor will perform building examinations of adjacent buildings and monitoring of adjacent buildings, as needed.

6. QA/QC

Keller will supply a full-time quality control (QC) representative during our VSC installation. The quality control representative will observe all pertinent data with respect to the installation. This information includes but is not limited to the depth, the approximate amount of stone introduced into the cavity and the amperage drawn by the vibrator during installation. Attention is required to ensure that Keller is getting adequate amperage (average peak of approximately 160A) while constructing the columns and maintenance of the average theoretical diameter of 36 inch. The average diameter of the column is calculated from the stone volume utilized for the respective column. One loader bucket holds approximately 2 cubic yards of material. Depth of the column will be checked with the markings on the vibrator.

It is common for the owner or general contractor to supply an independent inspection agency to observe our installation. The following guidelines are intended to aid any 3rd party quality assurance (QA) representatives in their inspections:

Location: Each VSC will have a designated number indicated on our shop drawings. The VSC should be located in the field by the field engineer using pin flags with numbers corresponding to those shown on the shop drawings. The center of installed VSC shall be within 6 inches of the design location.

Depth of Treatment: Markings along the shaft of the vibrator assembly indicate the depth of penetration. The drill rigs may also have depth indicators that will be verified periodically. Inspection personnel should not approach an open hole or operated machinery without first obtaining the permission of Keller's field superintendent.

Amperage: Amperage is a measure of electrical current. The amperage drawn by the vibrator during installation is a measure of the amount of compaction effort that has been applied to the stone and surrounding soil matrix. More precisely, the amperage draw is a direct measure of current required by the electric motor of the vibrator to keep the system in equilibrium. The higher the current, the more the resistance of the particles around the vibrator tip. In general, high amperage readings indicate a high degree of compaction and stiff matrix soils, while very low amperage readings indicate that the matrix soils are less dense, and a lower degree of compaction is achieved within the stone. Very high amperages should not be maintained for long periods of time, as this can cause vibrator damage.

Materials: Aggregates used for VSC construction shall consist of clean coarse aggregate conforming to the gradation specified in Table 5. Crushed concrete materials from demolition of an existing structure may be substituted with approval of the Keller ground improvement design engineer. The material shall have a minimum durability index of 40 when tested in accordance with California Test Method 229.

Table 5: Aggregate Gradation Requirement

Sieve Size	**Percentage Passing**
2”	100
1"	90-100
½”	5-80
No.4	0-3

7. SHOP DRAWINGS

Our shop drawing in **Appendix A** depicts our proposed soil improvement plans of VSC for the proposed Physical Education Building. An As-Built Drawing with any field changes will be provided upon completion of VSC work.

8. REFERENCES

Baez, J.I. and G.R. Martin (1993) "Advances in the Design of Vibro Systems for the Improvement of Liquefaction Resistance," Symposium of Ground Improvement, Vancouver Geotechnical Society, Vancouver, B.C.

Baez J.I. (1995) “A Design Model for the Reduction of Soil Liquefaction by Vibro-Stone Columns,” Ph.D. Dissertation, University of Southern California.

Boulanger, R. W., and I. M. Idriss. "CPT and SPT based liquefaction triggering procedures." Report No. UCD/CGM.-14 1 (2014).

Cetin K.O., Bilge H.T., Wu J., Kammerer A. and Seed R.B., [2009]. “Probabilistic Models for Cyclic Straining of Saturated Clean Sands.” J. Geotech. and Geoenv. Engrg., 135[3], 371-386

Martin, G.R. and M. Lew (1999) “Recommended Procedures for Implementation of DMG Special Publication 117 –Guidelines for Analyzing and Mitigating Liquefaction in California-“ Southern California Earthquake Center.

Nguyen, Thang V., Lisheng Shao, James Gingery, and Peter Robertson. "Proposed modification to CPT-based liquefaction method for post-vibratory ground improvement." In *Geo-Congress 2014: Geo-characterization and Modeling for Sustainability*, pp. 1120-1132. 2014.

Rayamajhi, Deepak, Scott A. Ashford, Ross W. Boulanger, and Ahmed Elgamal. "Dense granular columns in liquefiable ground. I: shear reinforcement and cyclic stress ratio reduction." Journal of Geotechnical and Geoenvironmental Engineering 142, no. 7 (2016): 04016023.

Youd, T.L. and I.M. Idriss (1997) “Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils,” NCEER Technical Publication 97-0022.

Zhang, G., P. K. Robertson, and R. WI Brachman. "Estimating liquefaction-induced ground settlements from CPT for level ground." Canadian Geotechnical Journal 39, no. 5 (2002): 1168-1180.

Appendix A
Keller Shop Drawing

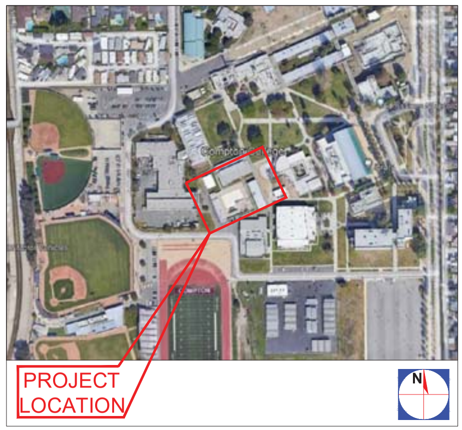
COMPTON COMMUNITY COLLEGE (PHASE 1)

VIBRATORY REPLACEMENT STONE COLUMNS (VRSC)

SHEET INDEX

KNA-1	TITLE SHEET
KNA-2	VRSC NOTES AND DETAILS
KNA-3	OVERALL GROUND IMPROVEMENT PLAN

VICINITY MAP



PROJECT ADDRESS

1111 EAST ARTESIA BOULEVARD
COMPTON, CALIFORNIA
90221

GEOTECHNICAL NOTES

Commentary

Keller North America (KNA) has been contracted to design and construct Stone Columns to support the foundation of the proposed buildings. This design submittal is as on the following information:

1. Geotechnical Investigation Report, Physical Education Complex Replacement, Compton Community College District, prepared by Atlas Technical Consultants, LLC, dated July 7th, 2021.
2. Addendum Geotechnical and Geohazard Report. Physical Education Complex Replacement, Compton Community College District, prepared by Atlas Technical Consultants, LLC, dated September 7th, 2021.
3. CPT Data - Compton Site, CPT-1, CPT-2, and CPT-5, dated September 3rd, 2021.
4. Preliminary Foundation Schemes, prepared by Brandow & Johnston, Inc., dated July 13th, 2021

*If any of these documents are changed or altered in any way, KNA may need to modify our design.

Underground Service Alert of Southern California
Call: TOLL FREE 800-422-4133
TWO WORKING DAYS BEFORE YOU DIG
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DATE CALLED:

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ADVANCED ARCHITECTURE
3324 GRAND VIEW
LOS ANGELES, CALIFORNIA 90066
TELEPHONE (310) 748-7649
E-MAIL HRAZTAN@STRUERE.COM
WWW.STRUERE.COM

KELLER
17461 DERIAN AVENUE
SUITE 106
IRVINE, CALIFORNIA
92614
WWW.KELLER-NA.COM

VRSC ENGINEER OF RECORD:

IDENTIFICATION STAMP
DIV. OF THE STATE ARCHITECT
APP. 03-121755 INC. #
REVIEWED FOR
SS ☐ FLS ☐ ACS ☐
DATE:

Compton College

COMPTON COLLEGE
PHYSICAL EDUCATION
COMPLEX REPLACEMENT

1111 EAST ARTESIA BLVD,
COMPTON, CA 90221-5393

ISSUE DESCRIPTION	
DSA SUBMITTAL	22.02.28

SHEET TITLE
TITLE SHEET
KNA-1

2/14/2022 6:20:28 PM

2/14/2022 6:20:28 PM

GROUND IMPROVEMENT GENERAL NOTES:

1. Keller North America (KNA) will be provided with a stable and relative level working surface. The working surface shall be constructed and manage by others such that KNA equipment can efficiently access and travel the site. KNA is not responsible for returning the site to its original grade or condition.
2. The GENERAL CONTRACTOR shall confirm that the proposed operation does not conflict with future improvement such as structural, mechanical, plumbing, and electrical prior to installation.
3. An underground service alert must be obtained 2 days before starting work.
4. All permits shall be procured and paid for by the GENERAL CONTRACTOR.
5. KNA will provide a qualified full-time quality control (QC) representative. This representative is titled KNA superintendent, foreman, or KNA field engineer. Third party testing or inspection shall be provided by others.
6. Stone columns shall be installed to design depth or auger/vibrator refusal.
7. Locating, protecting and rerouting/removal of all utilities are the responsibility of others. KNA is not responsible for damage to existing utilities.
8. After the completion of ground improvement work, others are responsible for protection of the work. Proper site drainage to prevent ponding of water at the area of the stone columns and control coordination of earthwork activities shall be managed such that existing stone columns are not damaged.
9. The stone column locations shown on these drawings are only for ground improvement layouts. These plans should not be used for foundation layout. Refer to the "for construction" structural package for specific foundation dimensions and locations.
10. Foundations shall not be poured until a final verification and approval by the project Geotechnical Engineer of Record.
11. Alternate structural shapes, material, and details cannot be used unless reviewed and approved by the ground improvement engineer, DSA & CGS.

GROUND IMPROVEMENT SPECIFIC NOTES:

1. The Ground Improvement Engineer is the registered professional engineer whose stamp is on these drawings.
2. KNA has relied on various documents for design. Those documents are listed in the ground improvement design report submitted with these drawings. These drawings should not be separated from that design report. If any of the reference documents listed in KNA's design report change in any way, KNA shall be notified and provided the opportunity to review the proposed changes and update this design as needed.
3. KNA must be notified immediately if information included in these plans or in the ground improvement design report conflicts with the project structural or architectural drawings.
4. The stone column properties used for design are provided in the design report.
5. A licensed surveyor (provided by CONTRACTOR) will accurately stake and identify to KNA the site control points for proper stone column layout by KNA, and show on the plans before installation begin. All stone columns shall be installed within six (6) inches of the location shown on these plans, unless otherwise approved by the Ground Improvement Design Engineer. KNA retains the sole authority to modify stone column locations due to constructability and site constraints. KNA shall provide an accurate method to allow the inspector to verify the as-built location of the stone columns during construction.
6. Practical stone column refusal is defined as failure to penetrate 1 foot in 1 minute by vibrator below groundwater table.
7. The stone column design shall be verified using the methods described in the stone column verification testing notes.
8. Stone columns shall be constructed to a nominal diameter of 36 inches. Some variability of this diameter should be expected due to variations in soil density.
9. The ground improvement engineer is the registered professional engineer whose stamp is on these drawings.

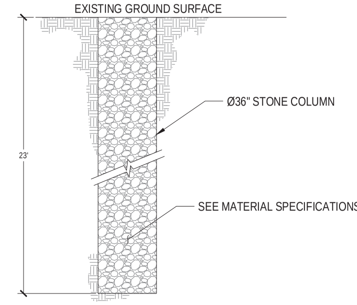
STONE COLUMN VERIFICATION NOTES:

1. The acceptance criteria of the Stone Column Treatment will be based on verifying densification by means of six (6) post-ground improvement CPT tests performed by Atlas Technical Consultants, LLC.
2. Post-improvement CPTs shall be located close to (preferably within 10 ft) the pre-construction CPTs whenever possible. CPTs will be performed at the center point between two adjacent stone columns.
3. A minimum of 7 days (preferably 14 days or more if practicable) shall pass after installation of stone columns before CPT testing is conducted. This will allow the dissipation of the excess pore water pressure induced by the vibrator.
4. The CPT's will be analyzed for liquefaction triggering and settlement using the design methods described in the stone column design report.
5. If the initial CPTs do not achieve the maximum settlement criteria, additional CPTs may be performed that allow for additional pore water pressure dissipation and aging. Additional CPTs may also be performed to better define the limits of an non-conforming work. After CPT testing is complete, additional stone columns may be installed at locations that to achieve the performance specification.
6. KNA may elect to perform its own additional site exploration at anytime and for any reason during the course of the project.

MATERIALS:

1. Aggregates used for VRSC construction shall consist of clean coarse aggregate conforming to the gradation specified below.
2. Crushed concrete materials from demolition of an existing structure maybe substituted with approval of the Keller Ground Improvement Design Engineer. The material shall have a minimum durability index of 40 when tested in accordance with California Test Method 229.

Sieve Size	Percentage Passing
2"	100
1"	90-100
1/2"	5-80
No. 4	0-3



TYPICAL STONE COLUMN SECTION
NOT TO SCALE

STRUERE
ADVANCED ARCHITECTURE
3324 GRAND VIEW
LOS ANGELES, CALIFORNIA 90066
TELEPHONE (310) 748-7649
E-MAIL HRAZTAN@STRUERE.COM
WWW.STRUERE.COM

KELLER
17461 DERIAN AVENUE
SUITE 106
IRVINE, CALIFORNIA
92614
WWW.KELLER-NA.COM

VRSC ENGINEER OF RECORD:

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DATE: ____

Compton College

COMPTON COLLEGE
PHYSICAL EDUCATION
COMPLEX REPLACEMENT

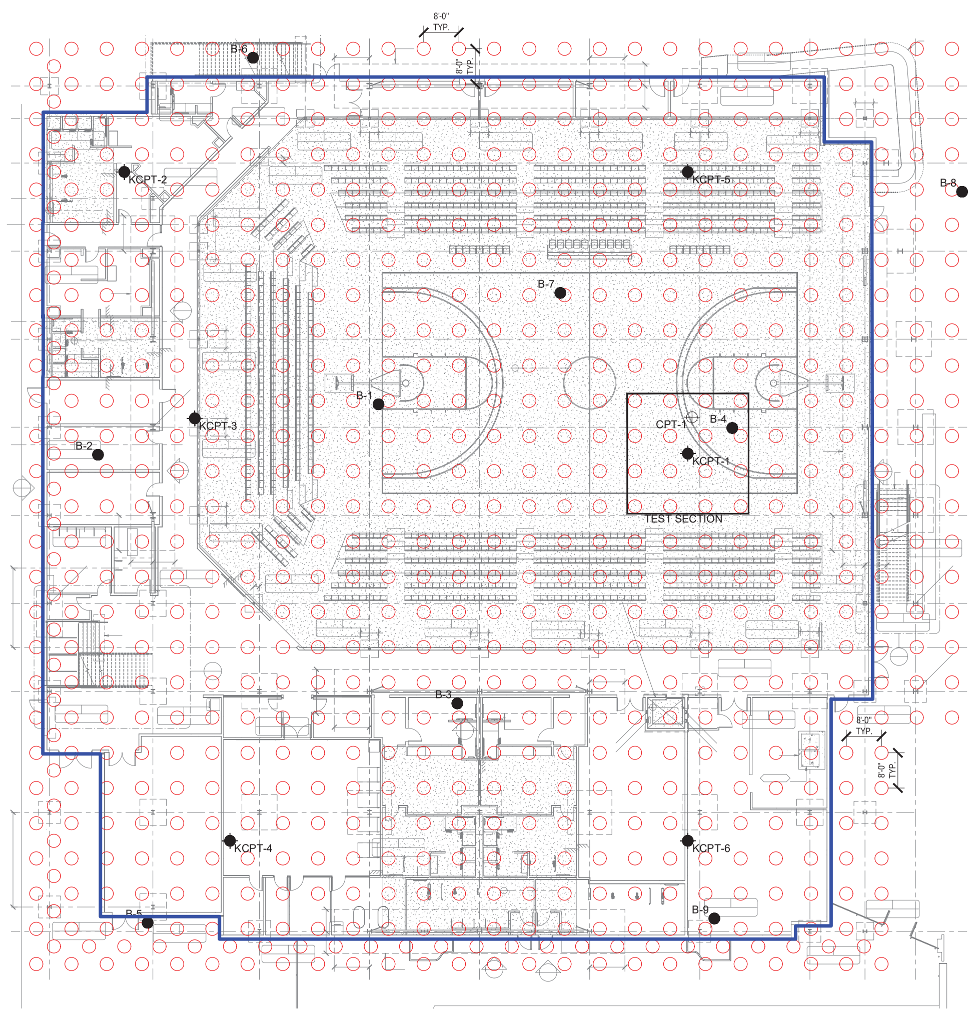
1111 EAST ARTESIA BLVD,
COMPTON, CA 90221-5393

ISSUE DESCRIPTION	
DSA SUBMITTAL	22.02.28

SHEET TITLE
VRSC NOTES AND DETAILS
KNA-2

LEGEND:

- Ø36" Vibratory Replacement Stone Columns (VRSC). Install 23' from the existing ground surface. Approximate existing ground surface elevation: 55 feet.
- CPT-X Existing CPT location
- B-X Existing Boring location
- KCPT-X Proposed Post-Treatment CPT locations
- Building Footprint



STRUERE
ADVANCED ARCHITECTURE
3324 GRAND VIEW
LOS ANGELES, CALIFORNIA 90066
TELEPHONE (310) 748-7649
E-MAIL HRAZTAN@STRUERE.COM
WWW.STRUERE.COM

KELLER
17461 DERIAN AVENUE
SUITE 106
IRVINE, CALIFORNIA
92614
WWW.KELLER-NA.COM

VRSC ENGINEER OF RECORD:

IDENTIFICATION STAMP
DIV. OF THE STATE ARCHITECT
APP. 03-121755 INC. #
REVIEWED FOR
SS ☐ FLS ☐ ACS ☐
DATE:

Compton College

COMPTON COLLEGE
PHYSICAL EDUCATION
COMPLEX REPLACEMENT

1111 EAST ARTESIA BLVD,
COMPTON, CA 90221-5393

ISSUE DESCRIPTION	
DSA SUBMITTAL	22.02.28

SHEET TITLE
OVERALL GROUND IMPROVEMENT PLAN
KNA-3

1" = 10'-0"

Appendix B
Foundation Bearing Capacity Check

VIBRO PIER GROUP BEARING CAPACITY CALCULATION

Project name	Compton Community Coll
Project location	
Project number	OP0013298
Date	2.18.2022
Designed by	MBU
Reviewed by	

Input Parameters

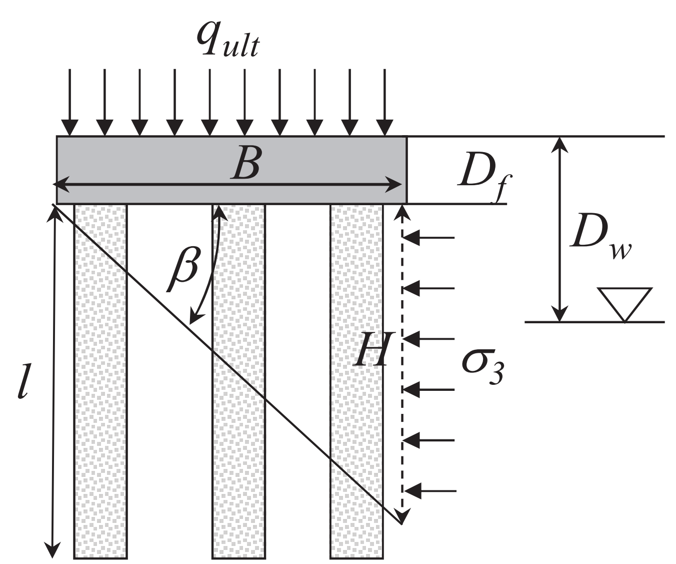
Strip or Square	-	Square	
Footing width	B	4	ft
Footing length	L	4	ft
Depth of embedment	D_f	2	ft
Area replacement ratio	ARR	0.00	
Adjacent surcharge	σ_{sur}	0	psf
Water table depth	D_w	8	ft
Moist unit weight	γ_{moist}	120	pcf
Saturated unit weight	γ_{sat}	120	pcf
Soil friction angle	ϕ_{soil}	38	°
Soil cohesion	c_{soil}	320	psf
Stress concentration factor	n	0	
Soil elastic modulus (at H/2)	E	250,000	psf
Stone friction angle	ϕ_{stone}	45	°
Treatment Depth	l	23	ft

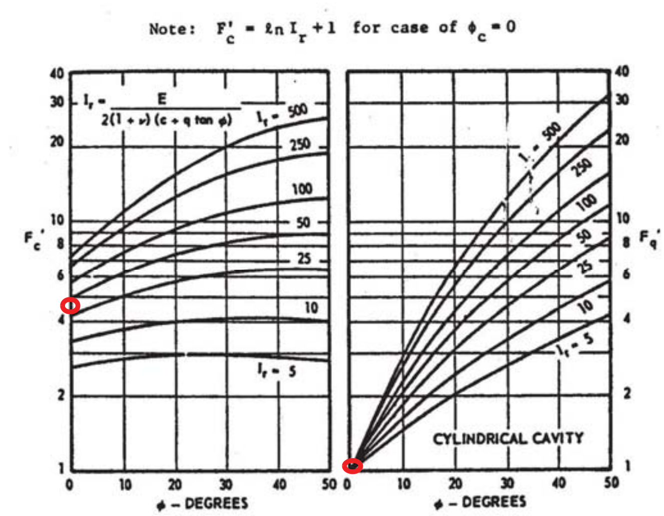
Calculated Parameters

Aggregate pier stress factor	μ_{SC}	0.0	
Soil stress factor	μ_{SOIL}	1.0	
Composite friction angle	ϕ_{avg}	38	°
Composite cohesion	c_{avg}	320	psf
Failure plane angle	β	64	°
Vertical interface length	H	8.2	ft
Passive coefficient	K_p	4.20	
Average vertical effective stress	σ_{vo}'	714	psf
Mean normal effective stress	q	2238	psf
Rigidity Index	I_R	45	
Confinement stress	σ_3	3,678	psf

Additional Input Parameters (for square footings)

Vesic cohesion factor	F_c'	4.5	
Vesic mean stress factor	F_q'	1	





From FHWA, 1983 (after Vesic)

Ultimate bearing pressure	q_{ult}	**16,773**	**psf**
Allowable bearing pressure	q_{des}	**3000**	**psf**

Factor of safety	**FS**	**5.6**

VIBRO PIER GROUP BEARING CAPACITY CALCULATION

Project name	Compton Community Coll
Project location	
Project number	OP0013298
Date	2.18.2022
Designed by	MBU
Reviewed by	

Input Parameters

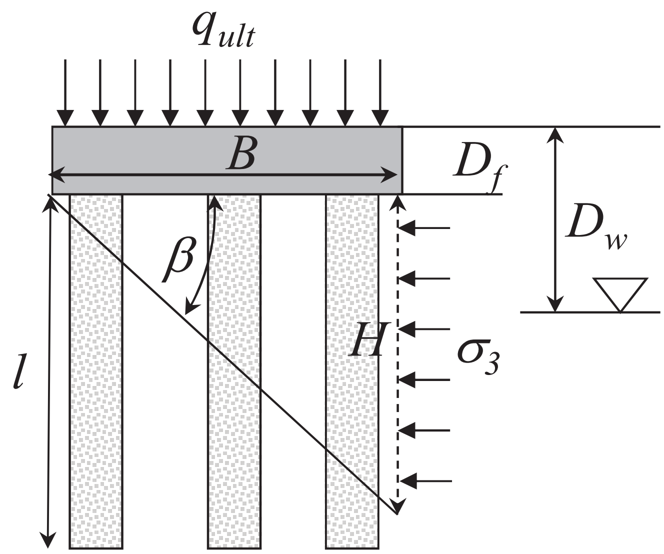
Strip or Square	-	Square	
Footing width	B	6	ft
Footing length	L	6	ft
Depth of embedment	D_f	2	ft
Area replacement ratio	ARR	0.00	
Adjacent surcharge	σ_{sur}	0	psf
Water table depth	D_w	8	ft
Moist unit weight	γ_{moist}	120	pcf
Saturated unit weight	γ_{sat}	120	pcf
Soil friction angle	ϕ_{soil}	38	°
Soil cohesion	c_{soil}	320	psf
Stress concentration factor	n	0	
Soil elastic modulus (at H/2)	E	250,000	psf
Stone friction angle	ϕ_{stone}	45	°
Treatment Depth	l	23	ft

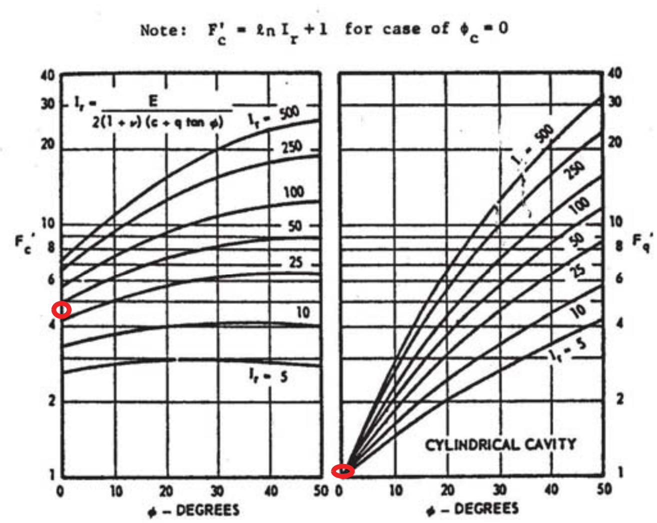
Calculated Parameters

Aggregate pier stress factor	μ_{SC}	0.0	
Soil stress factor	μ_{SOIL}	1.0	
Composite friction angle	ϕ_{avg}	38	°
Composite cohesion	c_{avg}	320	psf
Failure plane angle	β	64	°
Vertical interface length	H	12.3	ft
Passive coefficient	K_p	4.20	
Average vertical effective stress	σ_{vo}'	877	psf
Mean normal effective stress	q	2751	psf
Rigidity Index	I_R	37	
Confinement stress	σ_3	4,191	psf

Additional Input Parameters (for square footings)

Vesic cohesion factor	F_c'	4.5	
Vesic mean stress factor	F_q'	1	





From FHWA, 1983 (after Vesic)

Ultimate bearing pressure	q_{ult}	**18,932**	**psf**
Allowable bearing pressure	q_{des}	**3000**	**psf**

Factor of safety	**FS**	**6.3**

VIBRO PIER GROUP BEARING CAPACITY CALCULATION

Project name	Compton Community Coll
Project location	
Project number	OP0013298
Date	2.18.2022
Designed by	MBU
Reviewed by	

Input Parameters

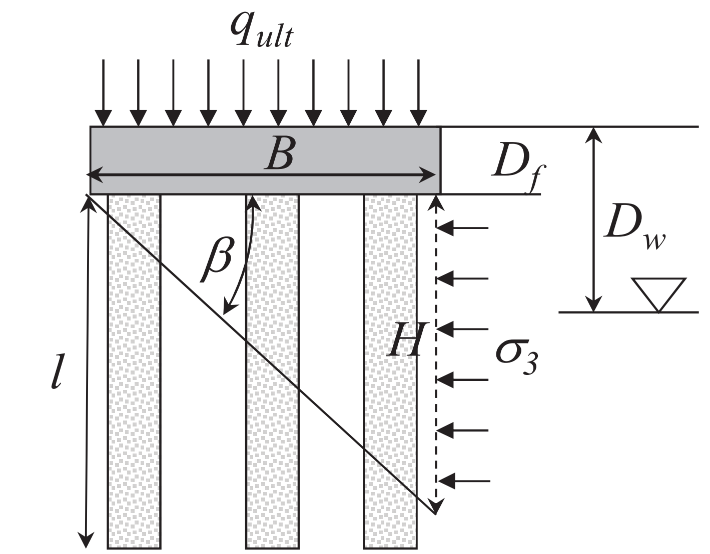
Strip or Square	-	Square	
Footing width	B	7	ft
Footing length	L	7	ft
Depth of embedment	D_f	2.5	ft
Area replacement ratio	ARR	0.00	
Adjacent surcharge	σ_{sur}	0	psf
Water table depth	D_w	8	ft
Moist unit weight	γ_{moist}	120	pcf
Saturated unit weight	γ_{sat}	120	pcf
Soil friction angle	ϕ_{soil}	38	°
Soil cohesion	c_{soil}	320	psf
Stress concentration factor	n	0	
Soil elastic modulus (at H/2)	E	250,000	psf
Stone friction angle	ϕ_{stone}	45	°
Treatment Depth	l	23	ft

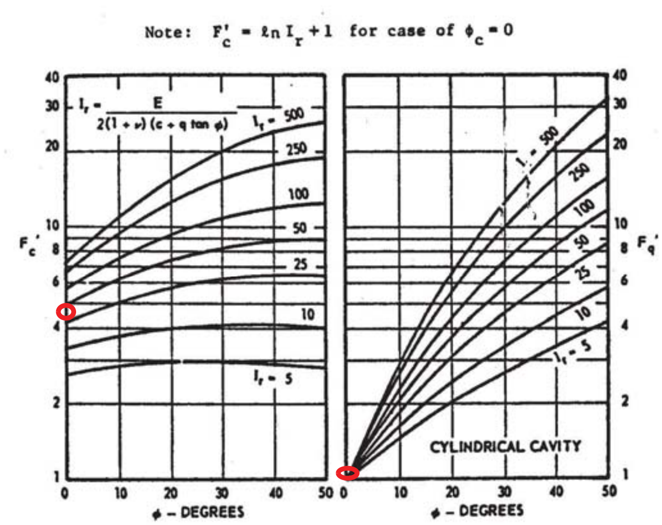
Calculated Parameters

Aggregate pier stress factor	μ_{SC}	0.0	
Soil stress factor	μ_{SOIL}	1.0	
Composite friction angle	ϕ_{avg}	38	°
Composite cohesion	c_{avg}	320	psf
Failure plane angle	β	64	°
Vertical interface length	H	14.4	ft
Passive coefficient	K_p	4.20	
Average vertical effective stress	σ_{vo}'	991	psf
Mean normal effective stress	q	3107	psf
Rigidity Index	I_R	34	
Confinement stress	σ_3	4,547	psf

Additional Input Parameters (for square footings)

Vesic cohesion factor	F_c'	4.5	
Vesic mean stress factor	F_q'	1	





From FHWA, 1983 (after Vesic)

Ultimate bearing pressure	q_{ult}	**20,426**	**psf**
Allowable bearing pressure	q_{des}	**3000**	**psf**

Factor of safety	**FS**	**6.8**

VIBRO PIER GROUP BEARING CAPACITY CALCULATION

Project name	Compton Community Coll
Project location	
Project number	OP0013298
Date	2.18.2022
Designed by	MBU
Reviewed by	

Input Parameters

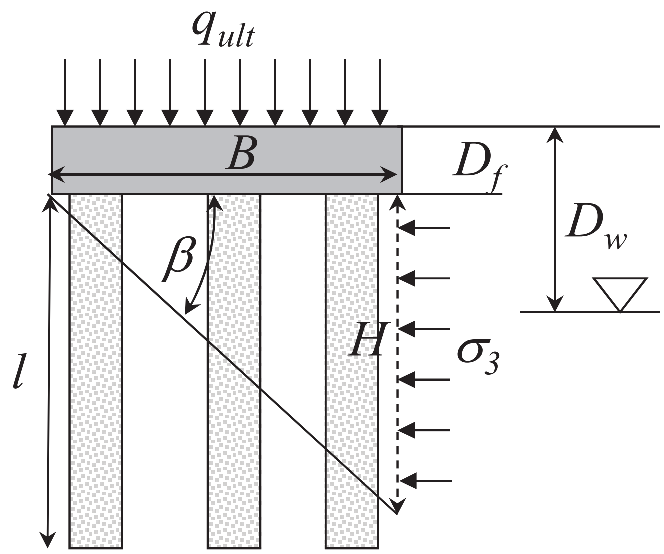
Strip or Square	-	Square	
Footing width	B	9.5	ft
Footing length	L	9.5	ft
Depth of embedment	D_f	2.5	ft
Area replacement ratio	ARR	0.00	
Adjacent surcharge	σ_{sur}	0	psf
Water table depth	D_w	8	ft
Moist unit weight	γ_{moist}	120	pcf
Saturated unit weight	γ_{sat}	120	pcf
Soil friction angle	ϕ_{soil}	38	°
Soil cohesion	c_{soil}	320	psf
Stress concentration factor	n	0	
Soil elastic modulus (at H/2)	E	250,000	psf
Stone friction angle	ϕ_{stone}	45	°
Treatment Depth	l	23	ft

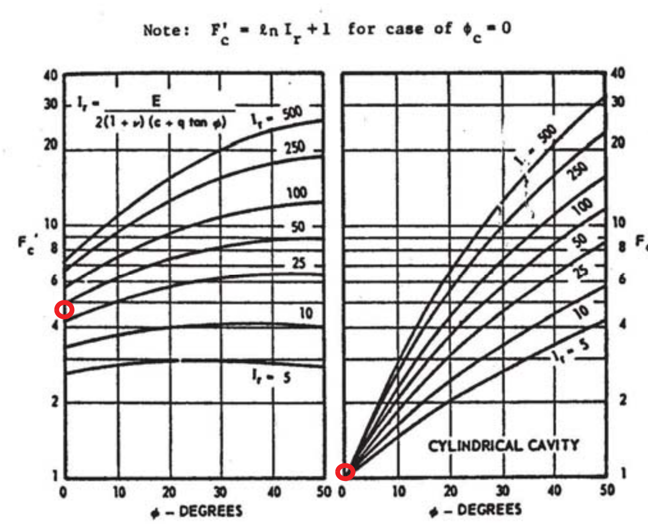
Calculated Parameters

Aggregate pier stress factor	μ_{SC}	0.0	
Soil stress factor	μ_{SOIL}	1.0	
Composite friction angle	ϕ_{avg}	38	°
Composite cohesion	c_{avg}	320	psf
Failure plane angle	β	64	°
Vertical interface length	H	19.5	ft
Passive coefficient	K_p	4.20	
Average vertical effective stress	σ_{vo}'	1156	psf
Mean normal effective stress	q	3624	psf
Rigidity Index	I_R	29	
Confinement stress	σ_3	5,064	psf

Additional Input Parameters (for square footings)

Vesic cohesion factor	F_c'	4.5	
Vesic mean stress factor	F_q'	1	



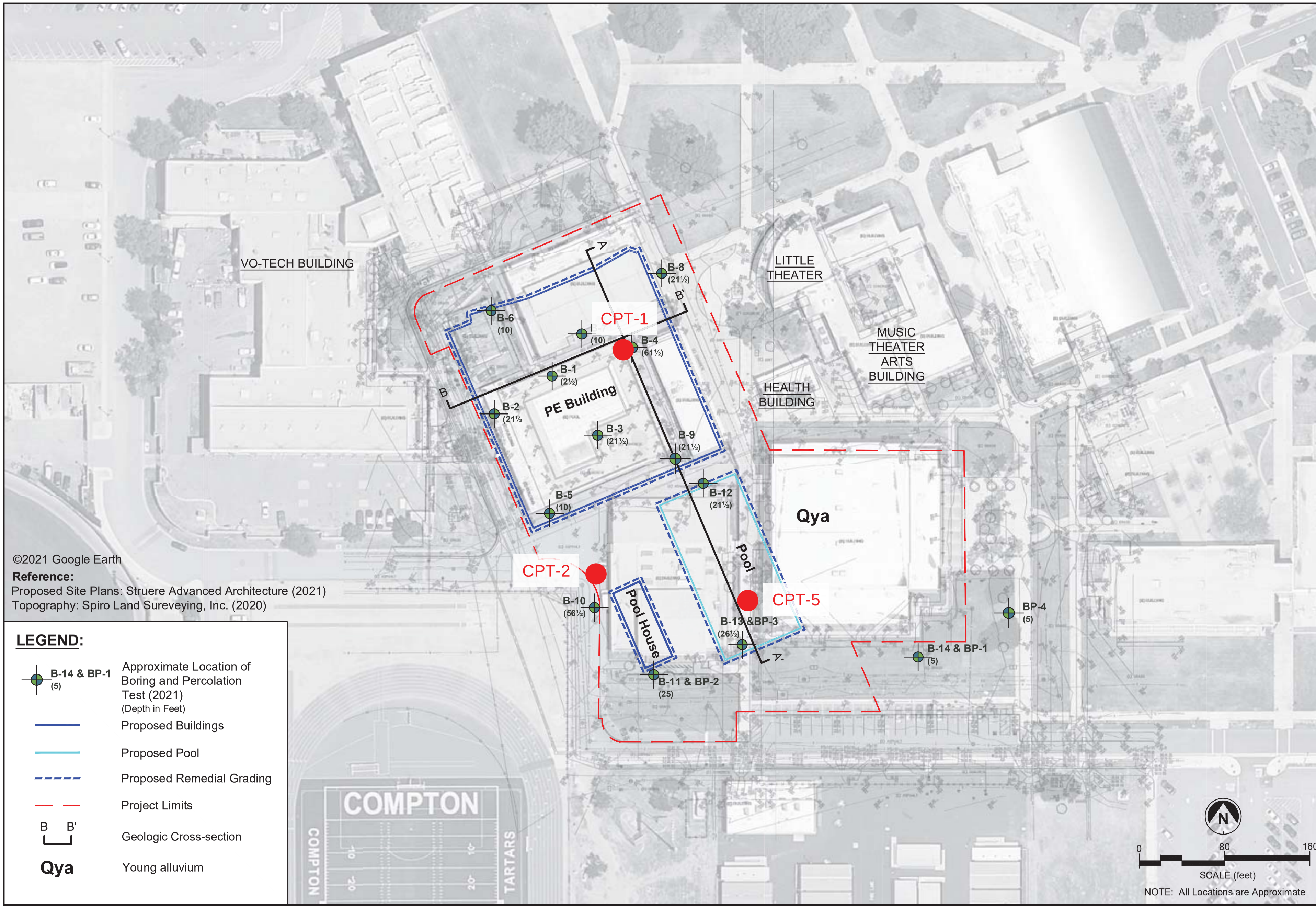


From FHWA, 1983 (after Vesic)

Ultimate bearing pressure	q_{ult}	**22,600**	**psf**
Allowable bearing pressure	q_{des}	**3000**	**psf**

Factor of safety	**FS**	**7.5**

Appendix C
Static Settlement based on Predicted Pre-treatment CPTs



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

GEOTECHNICAL MAP

Physical Education Complex Replacement
Compton, California

ATLAS

Figure:
2

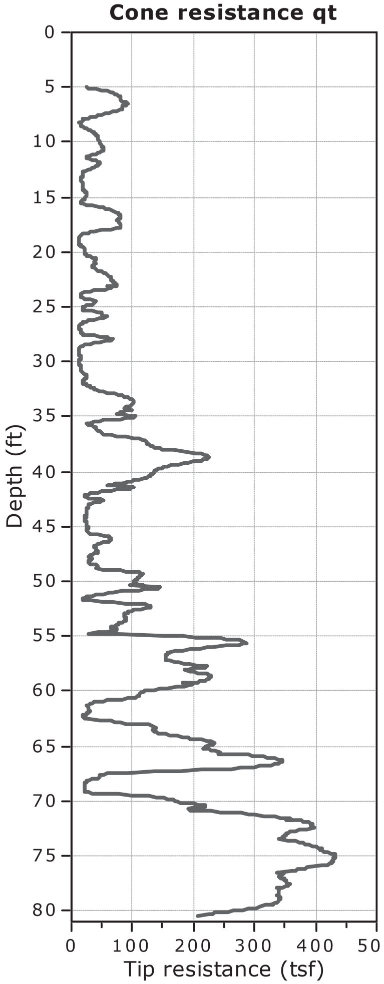
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: ATLAS
Location: 1111 E. Artesia Blvd, Compton, CA

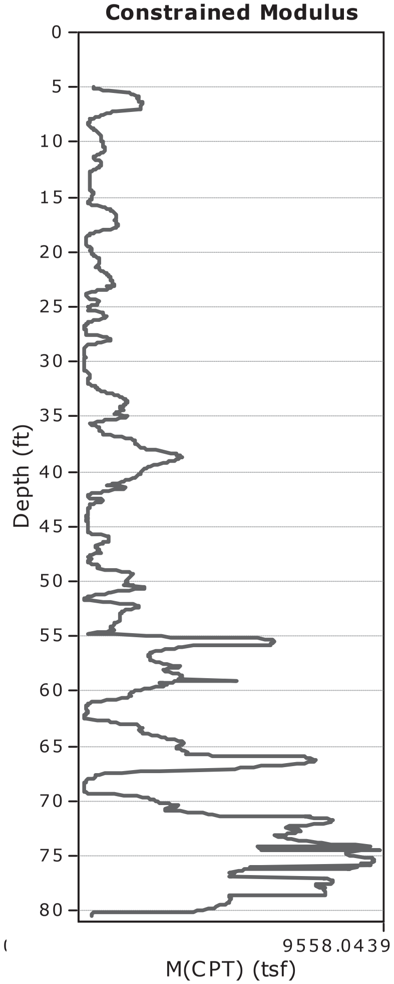
CPT: CPT-1
Total depth: 80.45 ft, Date: 9/3/2021
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown

Settlements calculation according to theory of elasticity*

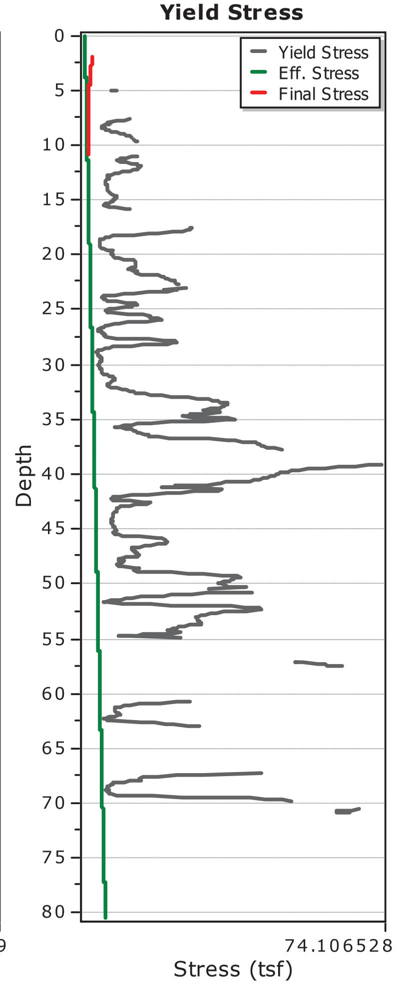
Cone resistance qt



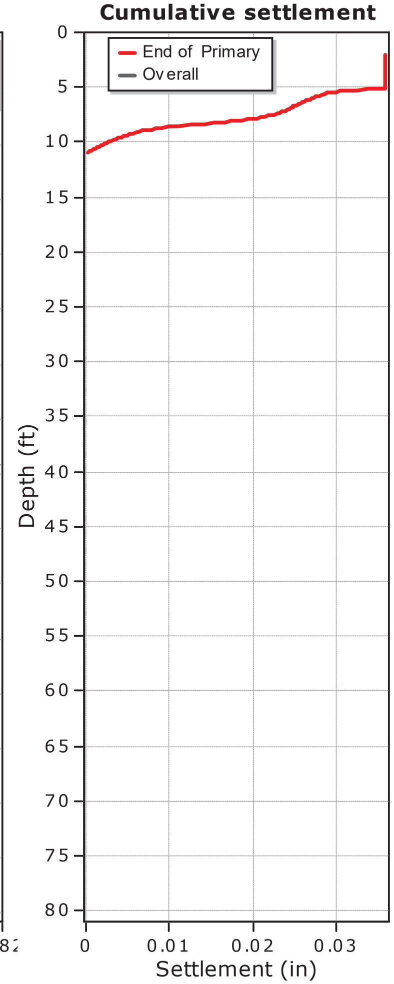
Constrained Modulus



Yield Stress



Cumulative settlement



Calculation properties

Footing type: Rectangular
Footing width: 4.00 (ft)
L/B: 1.0
Footing pressure: 1.50 (tsf)
Embedment depth: 2.00 (ft)
Footing is rigid: No
Remove excavation load: No
Apply 20% rule: Yes
Calculate secondary settlements: No
Time period for primary consolidation: N/A
Time period for second. settlements: N/A

* Primary settlement calculation is performed according to the following formula:

$$S = \sum \frac{\Delta\sigma_v}{M_{CPT}} \Delta z$$

* Secondary (creep) settlement calculation is performed according to the following formula:

$$S = C_\alpha \cdot \Delta z \cdot \log(t/t_p)$$

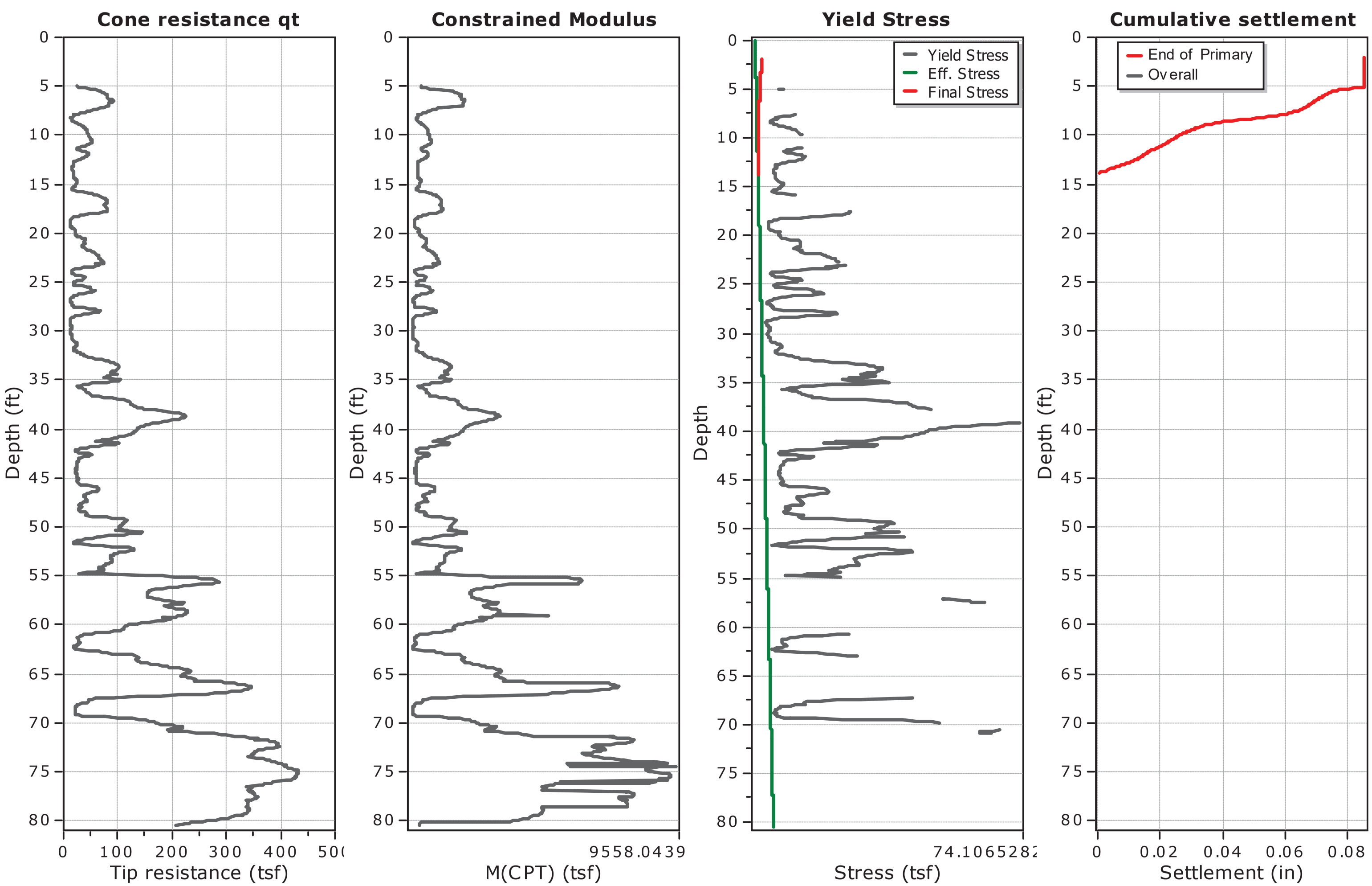
where t_p is the duration of primary consolidation

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

Project: ATLAS
Location: 1111 E. Artesia Blvd, Compton, CA

CPT: CPT-1
Total depth: 80.45 ft, Date: 9/3/2021
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown

Settlements calculation according to theory of elasticity*



Calculation properties

Footing type: Rectangular
Footing width: 6.00 (ft)
L/B: 1.0
Footing pressure: 1.50 (tsf)
Embedment depth: 2.00 (ft)
Footing is rigid: No
Remove excavation load: No
Apply 20% rule: Yes
Calculate secondary settlements: No
Time period for primary consolidation: N/A
Time period for second. settlements: N/A

* Primary settlement calculation is performed according to the following formula:

$$S = \sum \frac{\Delta\sigma_v}{M_{CPT}} \Delta z$$

* Secondary (creep) settlement calculation is performed according to the following formula:

$$S = C_\alpha \cdot \Delta z \cdot \log(t/t_p)$$

where t_p is the duration of primary consolidation

Kehoe Testing and Engineering
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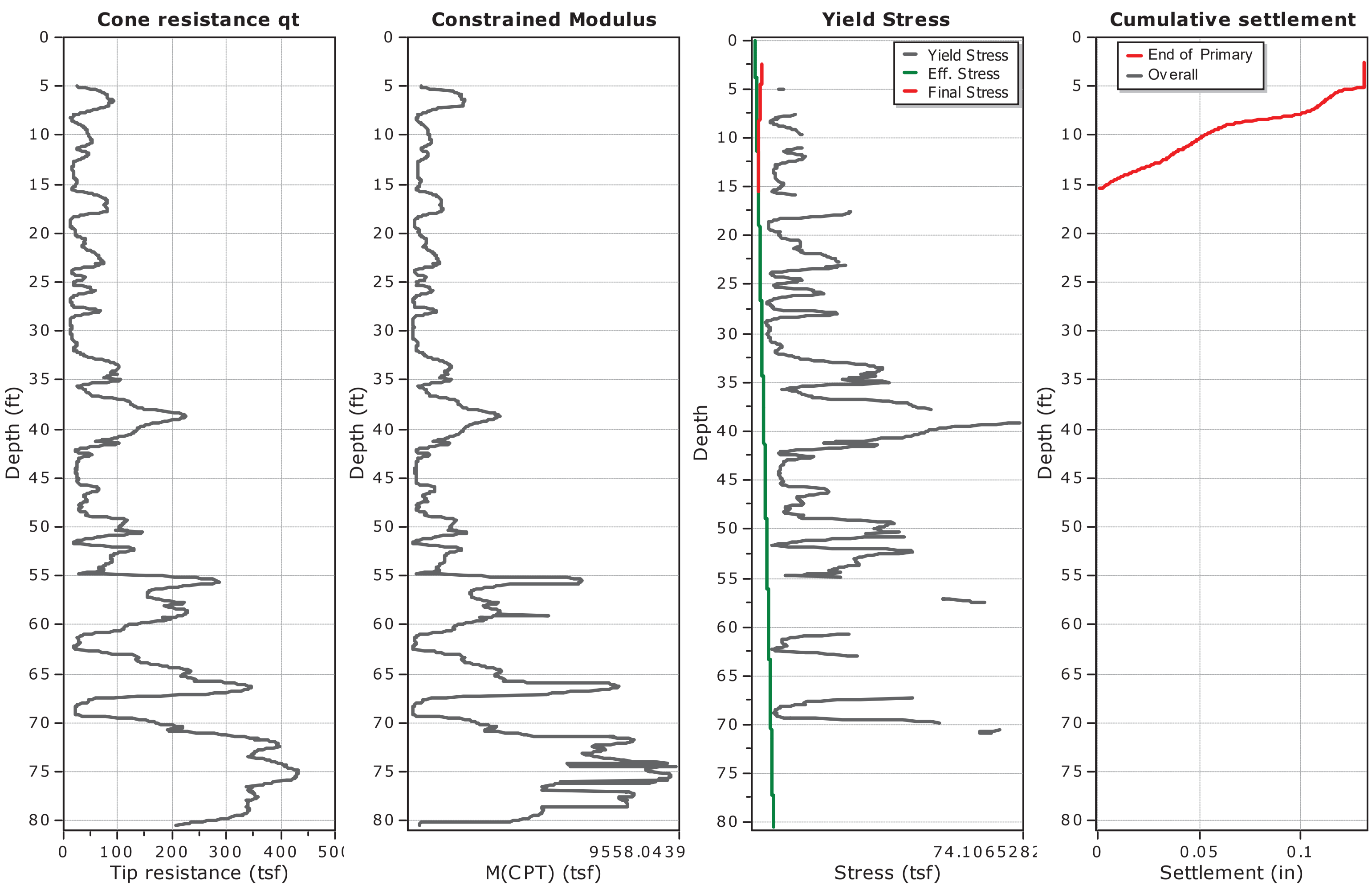
steve@kehoetesting.com
www.kehoetesting.com

Project: ATLAS
Location: 1111 E. Artesia Blvd, Compton, CA

CPT: CPT-1
Total depth: 80.45 ft, Date: 9/3/2021
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown

Settlements calculation according to theory of elasticity*



Calculation properties

Footing type: Rectangular
Footing width: 7.00 (ft)
L/B: 1.0
Footing pressure: 1.50 (tsf)
Embedment depth: 2.50 (ft)
Footing is rigid: No
Remove excavation load: No
Apply 20% rule: Yes
Calculate secondary settlements: No
Time period for primary consolidation: N/A
Time period for second. settlements: N/A

* Primary settlement calculation is performed according to the following formula:

$$S = \sum \frac{\Delta\sigma_v}{M_{CPT}} \Delta z$$

* Secondary (creep) settlement calculation is performed according to the following formula:

$$S = C_\alpha \cdot \Delta z \cdot \log(t/t_p)$$

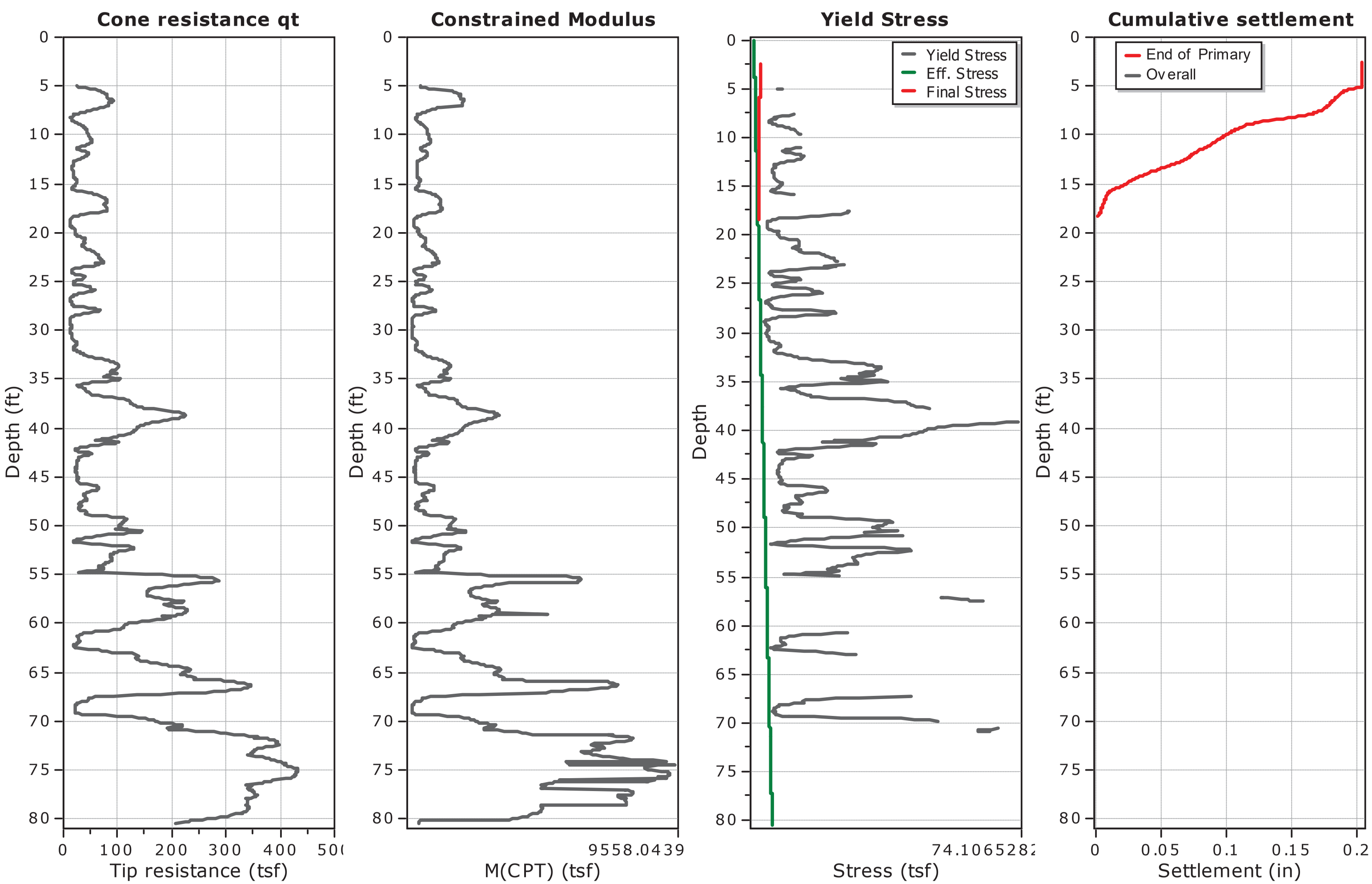
where t_p is the duration of primary consolidation

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

Project: ATLAS
Location: 1111 E. Artesia Blvd, Compton, CA

CPT: CPT-1
Total depth: 80.45 ft, Date: 9/3/2021
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown

Settlements calculation according to theory of elasticity*



Calculation properties

Footing type: Rectangular
Footing width: 9.50 (ft)
L/B: 1.0
Footing pressure: 1.50 (tsf)
Embedment depth: 2.50 (ft)
Footing is rigid: No
Remove excavation load: No
Apply 20% rule: Yes
Calculate secondary settlements: No
Time period for primary consolidation: N/A
Time period for second. settlements: N/A

* Primary settlement calculation is performed according to the following formula:

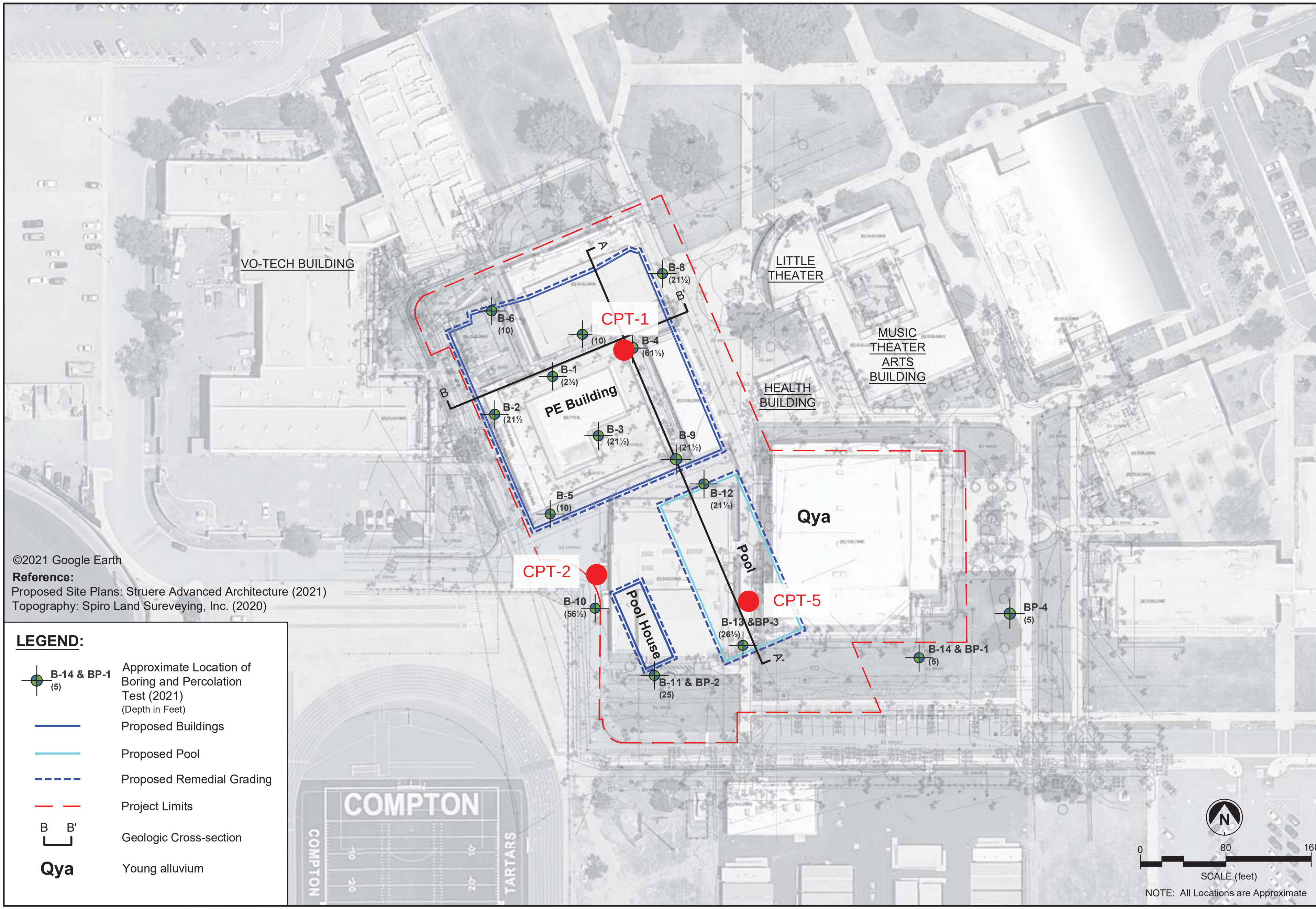
$$S = \sum \frac{\Delta\sigma_v}{M_{CPT}} \Delta z$$

* Secondary (creep) settlement calculation is performed according to the following formula:

$$S = C_\alpha \cdot \Delta z \cdot \log(t/t_p)$$

where t_p is the duration of primary consolidation

Appendix D
Pre- and Post-treatment Liquefaction Analysis



©2021 Google Earth

Reference:
Proposed Site Plans: Struere Advanced Architecture (2021)
Topography: Spiro Land Sureveying, Inc. (2020)

LEGEND:

B-14 & BP-1 (5)	Approximate Location of Boring and Percolation Test (2021) (Depth in Feet)
	Proposed Buildings
	Proposed Pool
	Proposed Remedial Grading
	Project Limits
B B'	Geologic Cross-section
Qya	Young alluvium

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

GEOTECHNICAL MAP
Physical Education Complex Replacement
Compton, California

ATLAS

Figure:
2

Liquefaction Analysis and Stone Column Mitigation

Project: OP0013298 - Compton Community College

CPT ID: CPT-1 (PE Building) Surface Elev.: 0.0 ft (use 0 ft to plot depth instead of elevation)

by: Bailey Uy

Date: 09.07.21

LIQUEFACTION ANALYSIS PARAMETERS

Triggering Method =	Robertson (NCEER R&W 1998)	
Vol. Settlement Method =	Zhang et al. (2002)	
Depth of GW During CPT =	45.00	ft
Depth of GW During Earthquake =	8.00	ft
Depth of Fill =	0.00	ft
Unit Weight of fill =	120	pcf
PGApre =	0.802	g
Mw =	7.30	
Ic Threshhold =	2.6	
Use Kσ ? =	Yes	

ADVANCED LIQUEFACTION PARAMETERS

Use Ic Transition Zones?	Yes			
Transition zone (dIc / dz) =	0.7	Ic/ft	Manual Trans Zones?:	No
Cliq TZ dIc =	0.04		Use Manual Thin Layer Cor.? =	No
Min. Trans. Zone Points:	4			
Ic_min and max =	1.9	3		

Perform Ground Improvement Analysis?: **Stone Columns**

STONE COLUMN DESIGN PARAMETERS

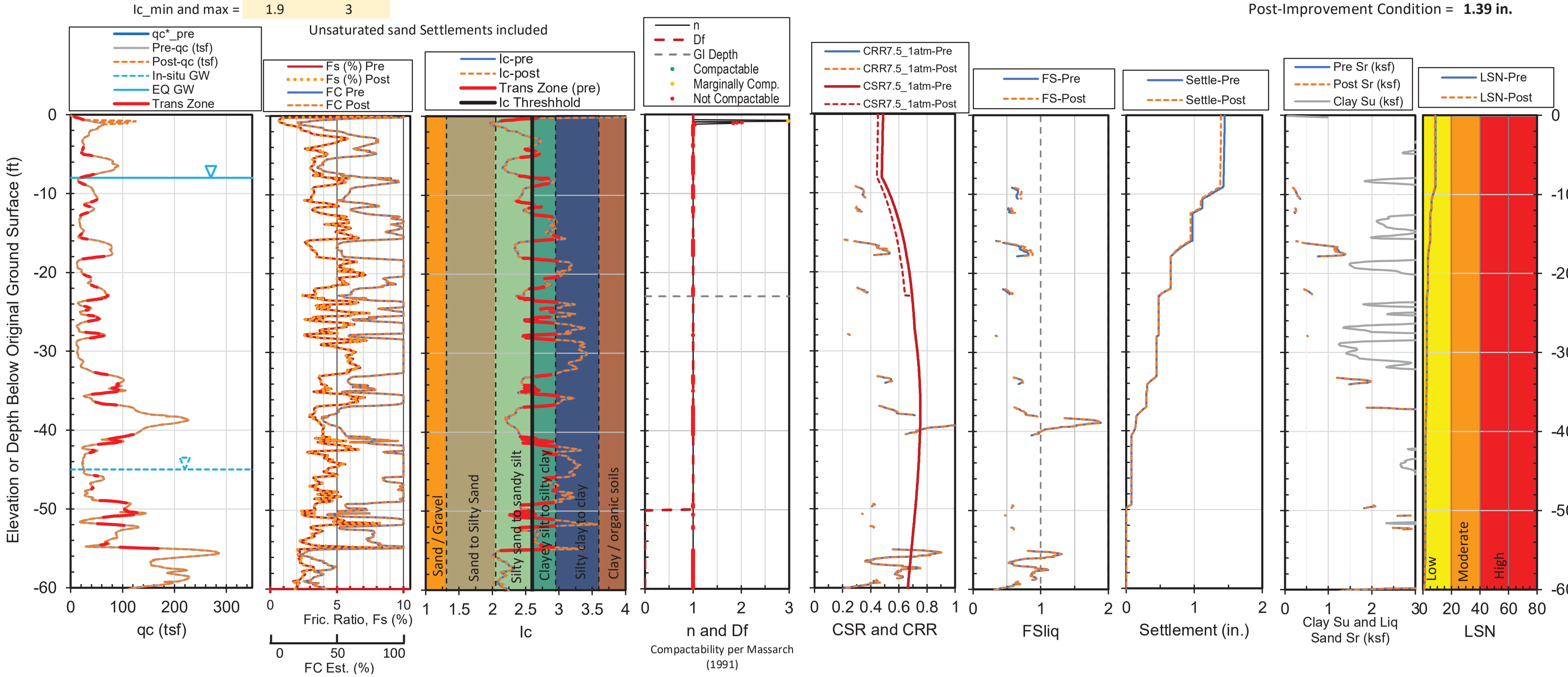
Depth Below Existing Grade =	23	ft	
Stone Column Diameter, D =	36	inch	
Stone Column spacing, S =	8	ft	
Square or Triangular Layout =	Square		
ARR =	11.0	%	
Baez for HBI =	HBI		
HBI Baez Scaling Factor (BSF):	Single	1	
Post Ic Shift	Use pre		
Gr =	6		
Rrd = PGApost/PGApre =	0.928		
PGApost in Impr. Zone =	0.744	g	

Volumetric Settlement Results:

Existing (Pre-Treatment) Condition = **1.45 in.**

Post-Improvement Condition = **1.39 in.**

Unsaturated sand Settlements included



Attachment B

Deep soil mixing revised submittal

Keller North America
17461 Derian Avenue, Suite 106
Irvine, CA 92614
Tel : 909-393-9300
Fax : 909-393-0036

Deep Soil Mixing Design
Pool and Pool Building of Compton Community College
Revision 1

1111 East Artesia Boulevard
Compton, California

Submitted to:
PCM3, Inc.
Compton CCD Office

Submitted by:
Keller North America

March 4, 2022

North America's Leader in Geotechnical Construction
www.Keller-na.com
Equal Opportunity Employer

Keller North America

PCM3
Compton CCD Office

Attention: Ms. Sheri Phillips
Subject: Deep Soil Mixing Design
Compton Community College Pool House and Swimming Pool

Keller North America (Keller) is pleased to present the following design submittal for ground improvement for the proposed buildings at this project site. The purpose of the ground improvement program is to enhance the safety, stability, and serviceability of the proposed structures. This is accomplished by increasing the strength of the ground to the point where the ground can safely support the anticipated structures under static loads as well as during and after the design level earthquake. Additional information is provided in the attached report.

The design provided herein has been prepared for the exclusive use of Keller, with the special equipment and production procedure, for our client under the following strict limitations:

1. Only Keller may construct the work described by the design and
2. The design may not be used by others for any purpose.

Keller appreciates the opportunity to be of service. Please feel free to contact the undersigned at (909) 393-9300 with any questions, comments, or concerns.

Respectfully submitted,

REGISTERED PROFESSIONAL ENGINEER
SUNIL ARORA
C72352
EXP. 06/30/22
CIVIL
STATE OF CALIFORNIA

David Chae,
Assistant Project Manager

Sunil Arora, P.E.
Project Executive

Bailey Uy
Engineer

1. DESIGN SUMMARY

This project site is located at 1111 East Artesia Boulevard, Compton, California. Keller proposes Deep Soil Mixing (DSM) as the ground improvement element to provide sufficient foundation support to the proposed Pool Building and swimming pool. The current site is approximately at an elevation of 55 feet. Table 1 summarizes the design depth of DSM for each proposed structure.

Based on Atlas Technical Consultants experience on the project site there may be more variation in the soil profile then what is portrayed in the CPT. Therefore, Keller is using conservative depth of treatment as provided by the project GEOR.

Table 1: Design Depth of DSM

Area	Approximate Existing Site Elevation, ft	Approximate Tip Elevation of DSM, ft	Approximate Depth (ft)
Swimming Pool	55	6	49
Pool Building	55	36	19

Deep Soil Mixing (DSM)

DSM construction involves using high torque equipment to mechanically mix grout with native soils to create a nearly homogeneous mixture of weak concrete called soilcrete. DSM is a top-down construction technique. As the mixing tool is advanced into the soil, grout slurry is pumped through the hollow stem of the shaft and injected into the soil at the tip and through the tool. The auger flights and mixing blades on the tool blend the soil with grout in a pug-mill fashion. When the design depth is reached, the tool is withdrawn to the surface. Left behind are stabilized soil mixed columns. Often predrilling can be used to simplify the disposal of construction spoils and waste soil. Depending on project requirements DSM can be used to improve ~10% to ~90% of the soil in each area.

The target average 28-days unconfined compressive strength is 150 psi. Keller plans to use 200 kg/m^3 at the beginning of the mixing operation and observe the wet soilcrete strength development to adjust cement dosage accordingly.

Figure 1: Construction of DSM

2. GROUND IMPROVEMENT DESIGN BASIS

This design is based on Keller's understanding of the following project documents and performance requirements articulated by the project structural engineer and geotechnical engineer. Although many documents were reviewed, only those which provided information that directly affects our design are listed below.

- Geotechnical Investigation Report, Physical Education Complex Replacement, Compton Community College District, by Atlas Technical Consultants, LLC, dated July 7, 2021
- Addendum Geotechnical and Geohazard Report, Physical Education Complex Replacement, Compton Community College District, by Atlas Technical Consultants, LLC, dated September 7, 2021
- CPT Data – Compton Site, CPT-1, CPT-2, and CPT-5, dated September 3, 2021
- Preliminary Foundation Schemes, by Brandow & Johnston, Inc., dated July 13, 2021

If any of these documents are changed or altered in any way, Keller should be notified, and the design may require modifications.

2.1 Subsurface Conditions

Based on our review of the Geotechnical Investigation Report by Atlas Technical Consultants LLC (Atlas), it is Keller's understanding that the site is generally underlain by about ½ foot of grass/topsoil/surficial fill and young alluvial deposits of Holocene to late Pleistocene age. These alluvial deposits are primarily comprised of inter-layered silty sands and sandy silts. In general, the near-surface sandy soil 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. Per Atlas's Geotechnical Investigation Report, groundwater was encountered at a depth between 44 feet to 52 feet below the existing ground surface.

2.2 Design and Performance Requirements

The ground improvement design criteria have been established by the project geotechnical and structural engineers and summarized in Table 1 below. Keller has reviewed the criteria and they appear typical and reasonable for this type of project.

Table 2: Design and Performance Criteria

	Criteria	**Reference**
Groundwater Level (Static)	44' below grade	Atlas Technical Consultants, LLC
Groundwater Level (Seismic)	8' below grade	
PGA_M (ASCE 7-16)	0.802 g	
M_w (ASCE 7-16)	7.3	
Depth of Liquefaction Analysis	50 feet	
Post-treatment Liquefaction-induced Differential Settlement for Swimming Pool	≤ 0.5 inch over 154 feet	Aquatic Design Group
Post-treatment Liquefaction-induced Differential Settlement for Pool Building	≤ 3.6 inch over 40 feet (0.075*L)	Brandow & Johnston, Inc. based on Table 12.13-3 of ASCE 7-16 for Risk Category II building

3. DSM DESIGN

The target average 28-days unconfined compressive strength (UCS) of the DSM is 150 psi.

3.1 Foundation Bearing Capacity of Pool Building

The minimum required area replacement ratio (A_r) of DSM is 30%, per Atlas' Addendum Geotechnical and Geohazard Report. Per Keller's shop drawing as seen in Appendix A, the actual area replacement ratio, A_r, of the proposed Pool Building is 50%. Keller checks the bearing capacity of DSM columns against crushing under seismic condition as follows:

Working Pressure (p) = 8,000 psf/A_r = 16,000 psf
Factor Safety (FS) = UCS/p = 150 psi / 16,000 psf = 1.3

3.2 Seismic Design of DSM for Swimming Pool and Pool Building

The design of soil mixing cells to mitigate liquefaction-induced settlement relies on the reinforcement effects, as published by Nguyen, et al. (2013). The minimum design A_r of the DSM over the proposed pool building and swimming pool is approximately 30%.

Nguyen (2013) suggested incorporating, R_{rd}, the ratio of shear stress reduction for improved and unimproved case when analyzing post construction liquefaction potential to account for the shear reinforcement effect of DSM Grid. With the $A_r = 30\%$ and the soilcrete to soil shear modulus ratio of $G_r = 30$, the calculated shear stress reduction factor yields $R_{rd} = 0.229$.

Therefore, the post-treatment PGA can be computed as $PGA_{post} = PGA_{pre} \times R_{rd}$. Here in this chapter, the key computation equations are listed.

R_{rd} is given by the following equation:

$$R_{rd} = \min\left\{\frac{1}{G_r \cdot \left[A_r \cdot C_G \cdot \gamma_r + \frac{1}{G_r} \cdot (1 - A_r)\right]}, 1\right\}$$

where, G_r = average stiffness ratio, A_r = area replacement ratio

C_G = equivalent shear factor computed as the shear stiffness of the DSM grid system:

$$C_G = 1 - 0.5\sqrt{1 - A_r}$$

γ_r = shear strain ratio between DSM and soil:

$$\gamma_r = \left[1 - (1 - A_r)^{1.3} \cdot \left(\frac{G_r - 1}{185}\right)^{0.4}\right] \cdot \min\left(\frac{H}{S}, 1\right)$$

Based on the Geotechnical Investigation Report by Atlas Technical Consultants LLC., dated July 7th, 2021, the ground motion input used in Keller's post-treatment liquefaction-induced settlement analysis is:

- $M_w = 7.3$
- $PGA_{post} = 0.229 \times 0.802g = 0.184g$ (within the treatment length of DSM)

The post-treatment liquefaction-induced settlement analysis is included in Appendix B of this submittal. Table 3 below summarizes the computed results for each structure of this project:

Table 3: Pre- and Post-treatment Liquefaction-Induced Settlement Analysis

Area		**Pre-treatment Liquefaction-Induced Settlement (inch)**	**Post-treatment Liquefaction-Induced Settlement (inch)**
Pool Building	CPT-2	2.59	0.68
Swimming Pool	CPT-5	2.79	0.09

4. DSM CONSTRUCTION

4.1 Layout

Keller will provide an AutoCAD shop drawing for each DSM column coordinate overlaid on the site Civil drawing. Keller understands that the general contractor will be responsible and use a licensed surveyor to provide Keller with controlled points and survey benchmarks before installation and will prepare as-built drawings after completion. DSM columns will be installed within 6 inches of the design locations as shown in the Keller shop drawing.

4.2 Sequence of Work

Once a stable working platform has been established as shown in Keller shop drawing. DSM columns will be constructed.

4.3 Predrill

To minimize the mixing tool damage and maintaining soil mixing quality, Keller may pre-drill holes or excavate for better mixing quality. The holes will be filled with soilcrete up to the working elevation during the mixing stage.

4.4 Soil Mixing

In general, soil mixing operation parameters, such as mixing shaft speed, penetration rate, batching grout specific gravity (sg), and pumping rate will be determined based on our lab mixing result and our experience and will be fine-tuned at the beginning of mixing column production. The design cement content in place (cement weight/[soil volume + grout volume]) will start from approximately 200 kg/m^3 with grout slurry specific gravity (sg) of 1.45. Keller engineers may adjust the cement content and grout sg based on the field sample strength development.

4.4.1 Vertical Alignment

Vertical alignment of the mix tool stroke will be controlled by the drill rig operator. Two measurements of verticality will be monitored. These are the fore-aft and left-right vertical mast positions. Verticality will be measured by a level as measured on the mixing tool prior to penetration. Intermittent measurements will be made as may be necessary during mixing operations.

4.4.2 Mixing Shaft Speed

The mixing shaft speed which is anticipated to be ranging between 20-50 RPM and shall be adjusted to accommodate a constant rate of mixing shaft penetration based on the degree of drilling difficulty. The mixing shaft speed can be adjusted according to drilling difficulty. The mixing shaft speed can be adjusted to aid mixing of the soil column when needed or to assist penetration in hard drilling. Mixing shaft speed will be recorded.

4.4.3 Penetration Rate

In order to ensure adequate mixing, the penetration rate of the mixing shaft shall be maintained at about 1.0 to 3.0 feet/minute during penetration. The penetration rate and maximum depth of each stroke shall be recorded by Keller's data acquisition system.

4.4.4 Grout Take

The grout slurry flow per vertical foot of the column will be adjusted to the requirements of the design mix. Progressive cavity pumps will be used to transfer the grout from the mixing plant to the mixing rig. Flow monitoring devices will be installed in the grout line to detect any line blockage and monitor flow, total injected grout per column and grout pressure. These parameters will be recorded.

Inevitably some variations of the grout take will occasionally occur due to field conditions. It is anticipated that a grout flow rate between 50 to 250 GPM will be used during penetration. Keller's Data Acquisition System (DAQ) can automatically adjust the grout flow rate as a function of the penetration rate and maintain the pre-set cement dosage prescribed by the design engineer.

4.4.5 Withdrawal Rate

The mixing shaft will be withdrawn at a rate of 6 to 12 feet per minute.

4.4.6 Obstruction/ Mixing Shaft Refusal

Keller will use a data acquisition system to monitor the mixing shaft penetration and the shaft rotation resistance in terms of the hydraulic pressure. The DAQ system will calculate and plot the Drilling Index as a function of depth, a mixing parameter to detect penetration resistance and refusal depth. Keller will set up the penetration criteria based on the site measurement. In case of underground obstruction, such as abandoned footings, piles, utilities, etc., the general contractor will be responsible to remove obstructions and backfilled with sandy soil prior to Keller mixing operation.

4.5 Material

Cement: Cement will be furnished by Keller and conform to ASTM C150 "Standard Specification for Portland Cement," Type II/V or equivalent. The cement will be adequately protected from moisture and contamination while in transit to and in storage at the job site. Reclaimed cement or cement containing lumps or deleterious matter will not be used.

Water: Water for the slurry will be fresh, free of deleterious substances that adversely affect the strength and mixing properties of the slurry, furnished by others.

4.6 Equipment

4.6.1 Batching Equipment

The batch plant shall consist of in-line eductor (jet valve) mixers. Dry materials shall be stored in tankers and/or silos and fed to the mixers for shearing and circulation. The resulting grout slurry will be transferred to a surge tank for continuous agitation and to supply the in-situ soil mixing rig. Grout slurry quality will be assured by frequent testing prior to injection into the soil.

4.6.2 Mixing Equipment

Single shaft mixing equipment that mechanically mixes the soil and cement slurry for the full dimensions of the column will be used for the Work. We anticipate using hydraulic drill rigs for the soil mixing operations. This rig is capable of up to > 150,000 ft-lbs. of torque at > 20 rpm. The working shaft rate of rotation ranges between 20 and 60 rpm. The mixing shaft will have mixing augers and/or blades (paddles) configured in such a manner so that they are capable of thoroughly blending the in-situ soils and cement slurry. The power source for driving the mixing shafts will be sufficient to maintain the required mix tool (shaft) rotation speed in revolutions per minute and penetration/ withdrawal rates from the ground surface to the maximum depth required. The design target Blade Rotation Number (BRN), defined as the number of blades cut in each 1.0-meter soil) will be at least 300.

The DSM equipment will be equipped with devices to assure vertical alignment in two planes (90 degrees in plan from each other): fore-aft and left-right. The DSM equipment will be equipped with a real-time display of depth, rotation speed, grout flow rate; grout specific gravity, cumulative grout injected, and grout pressure for each soil mix column. The cement will be mixed with water within the jet valve to create a 1.45 sg mix +/- 0.1. Note that sg can be changed by Keller based on UCS data and field conditions. No mixing operation will be allowed if the DAQ system not functioning.

4.6.3 Pumping Equipment

Grout slurry will be supplied to the drill using large size Moyno pumps. These pumps will be sized and powered so that design volumes and pressures can be maintained up to 1,000 feet away from the batching facility. It is anticipated that a continuous grout slurry flow of 150 gallons per minute at 100 psi to the drill rig will be necessary

4.6.4 Equipment Location

The batching and pumping facility will be set up central to both in situ soil mixing areas. This will eliminate the need to move the plant once it is established.

5. QA/QC

Following the installation of DSM columns, verification testing will include:

- Unconfined compressive test on wet soils mixed samples
- Unconfined compressive test on cored samples
- Review of production DAQ logs

5.1 Wet Soils Mixed Samples

Wet Soil mix samples will be retrieved and cast into molds for one column per rig/shift, at one random depth, typically near the end of each shift. Samples will be retrieved using an in situ wet sampler immediately after column construction and shall consist of no fewer than 8 specimens. Soil clods greater than 10% of the mold diameter will be screened off. Appropriate curing techniques shall be implemented until testing based on ASTM D 1632.

Unconfined compression testing shall be performed by an approved laboratory in pairs of specimens at 7 days. If the 7-days specimens do not reach the desired strength according to the lab test curve, another pair of specimens will be tested at 14 days, 28 days, and if needed at 56 days. All specimens at 28 days and available 56-days of age will be tested and used in the statistical calculation. The Unconfined Compressive Strength (UCS) shall be determined by ASTM D1633 “Standard Test Methods for Compressive Strength of Molded Soil-Cement Cylinders”. Sulfur caps shall be required in the UCS tests to minimize the end effects on the test specimen. The advantage of the wet sampling is that Keller can get an early trend of the soilcrete strength development without waiting to the end of the project for coring and can make early decisions in the field program to add additional soil mixing column s if necessary.

If wet grab strengths at 7 days of age are greater than the average required unconfined compressive strength, additional tests may be omitted at the discretion of the GEOR. Wet grab samples will be kept on-site (approximately 3 days) for an initial set before being shipped to the lab.

5.2 Core Samples

Keller will core 2% of the DSM columns.

All core locations shall be randomly selected, to collect core samples for unconfined compression testing. Coring will start after the soilcrete has gained adequate strength and verified by the strength development from the wet sample tests. The double-tubes coring method, with the utilization of vibrators to assist the core to depth, can be used instead of the conventional coring technique. At minimum three (3) samples from each core will be extracted. Keller anticipates 4 specimens trimmed from each core hole to be tested by ASTM D1633.

Uniformity of mixing shall be evaluated by the geotechnical engineer of record (GEOR) based on the continuous core samples recovered. The continuous core holes shall extend the entire depth of the DSM column. Estimated recovery of 80 percent for each 5-foot-long segment of a boring and at least 90 percent when averaged over all core runs within a single boring shall be achieved. The lumps of unimproved soils shall not exceed 20 percent of the total volume of any 5-foot core segment from a boring. If the core recovery below the anticipated value due to the gravel particles in the soilcrete matrix, Keller shall be allowed to utilize a downhole camera or other approved methods to verify the core hole.

Keller will calculate the average 28-day UCS value from all core samples and wet grab samples. No more than 5 percent of all specimens tested shall exhibit an unconfined compressive strength of less than 150 psi at 28 days. A ceiling, the not-to-exceed value of four times the average unconfined compressive strength (i.e., 600 psi) shall be used for individual specimens in calculating the average strength achieved in the field from each coring and wet sample and for the entire project.

If the acceptance criteria are not achieved in a designated area, Keller may be given the opportunity to conduct additional UCS test on soilcrete specimens on 56 days of age, site exploration, coring, sampling, downhole imaging, and strength testing from the additional cured specimen to better define the average design strength at Keller's preference and expense. If a designated area is rejected, Keller shall submit a Remixing or Mitigation plan.

At the end of the project, to not unnecessary delay subsequent activities by waiting for a 28-day test result, correction of early strength gain will be used to approve the DSM work. However, this correlation will not relieve the contractor of the responsibility to achieve average 28-days strength of 150 psi. Based on FHWA (2013) guidelines, the following UCS aging factor correlations will be applied to this job:

- 28: 3-day, 1.72
- 28: 7-day, 1.35
- 28: 14-day, 1.15

A site-specific correlation between 3-days and 28-days strength may be used to supersede this correlation if in the opinion of the Engineer the site-specific correlation is more appropriate.

5.3 Production DAQ Logs

During the soil mixing production, Keller will review the wet soilcrete strength development as well as production column mixing logs and may add additional soil mixing columns if the soilcrete strength is below the target average UCS values as listed above.

6. SHOP DRAWINGS

Our shop drawing in **Appendix A** depicts our proposed soil improvement plans of DSM for the proposed pool building and swimming pool. An As-Built Drawing with any field changes will be provided upon completion of DSM work.

7. REFERENCES

Bruce, Mary Ellen C., Ryan R. Berg, James G. Collin, George M. Filz, Masaaki Terashi, David S. Yang, and Sa Geotechnical. Federal Highway Administration design manual: Deep mixing for embankment and foundation support. No. FHWA-HRT-13-046. The United States. Federal Highway Administration. Offices of Research & Development, 2013.

Filz, G. M., and E. Templeton. "Design guide for levee and floodwall stability using deep-mixed shear walls.”." New Orleans District and Hurricane Protection Office, US Army Corps of Engineers, Final Report Contract W912P8-07 (2011): 0031.

Nguyen, T.V., Rayamajhi, D., Boulanger, R.W., Ashford, S.A., Lu, J., Elgamal, A. and Shao, L. (2013). "Design of DSM grids for liquefaction remediation." Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 139(11), 1923-1933

Appendix A
Deep Soil Mixing Design Shop Drawing

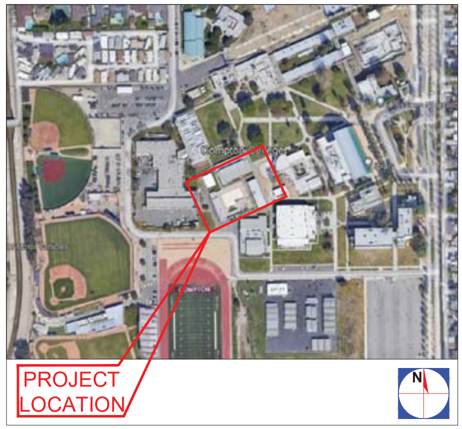
COMPTON COMMUNITY COLLEGE
(PHASE 2)

DEEP SOIL MIXING (DSM)

SHEET INDEX

KNA-1P	TITLE SHEET
KNA-2P	DSM NOTES
KNA-3P	OVERALL GROUND IMPROVEMENT PLAN

VICINITY MAP



PROJECT ADDRESS

1111 EAST ARTESIA BOULEVARD

COMPTON, CALIFORNIA

90221

GEOTECHNICAL NOTES

Commentary

Keller North America (KNA) has been contracted to design and construct Stone Columns to support the foundation of the proposed buildings. This design submittal is as on the following information:

1. Geotechnical Investigation Report, Physical Education Complex Replacement, Compton Community College District, prepared by Atlas Technical Consultants, LLC, dated July 7th, 2021.
2. Addendum Geotechnical and Geohazard Report. Physical Education Complex Replacement, Compton Community College District, prepared by Atlas Technical Consultants, LLC, dated September 7th, 2021.
3. CPT Data - Compton Site, CPT-1, CPT-2, and CPT-5, dated September 3rd, 2021.
4. Preliminary Foundation Schemes, prepared by Brandow & Johnston, Inc., dated July 13th, 2021

*If any of these documents are changed or altered in any way, KNA may need to modify our design.

Underground Service Alert of Southern California
Call: TOLL FREE 800-422-4133
TWO WORKING DAYS BEFORE YOU DIG
RE-NOTIFY EVERY TWO WEEKS
TICKET #
DATE CALLED:

STRUERE
ADVANCED ARCHITECTURE
3324 GRAND VIEW
LOS ANGELES, CALIFORNIA 90066
TELEPHONE (310) 748-7649
E-MAIL HRAZTAN@STRUERE.COM
WWW.STRUERE.COM

KELLER
17461 DERIAN AVENUE
SUITE 106
IRVINE, CALIFORNIA
92614
WWW.KELLER-NA.COM

DSM ENGINEER OF RECORD:

IDENTIFICATION STAMP
DIV. OF THE STATE ARCHITECT
APP. 03-121755 INC: #
REVIEWED FOR
SS ☐ FLS ☐ ACS ☐
DATE:

Compton College

COMPTON COLLEGE
PHYSICAL EDUCATION
COMPLEX REPLACEMENT

1111 EAST ARTESIA BLVD,
COMPTON, CA 90221-5393

ISSUE DESCRIPTION	
DSA SUBMITTAL	22.02.28

SHEET TITLE
TITLE SHEET

KNA-1P

2/14/2022 6:20:28 PM

2/14/2022 6:20:28 PM

GROUND IMPROVEMENT GENERAL NOTES:

1. OTHERS are to provide a dry, stable, and relative level working platform. It is KNA North America's (KNA) understanding that the working grade will be near existing grade of Elv. 55. The working surface shall be constructed and managed by others such that KNA's equipment can safely track and efficiently work under its own weight without the need for steel plates or crane mats.
2. The GENERAL CONTRACTOR shall confirm that the proposed operation does not conflict with future improvement such as structural, mechanical, plumbing, and electrical prior to DSM installation.
3. An underground service alert must be obtained 2 days before starting work.
4. All permits shall be procured and paid for by the OWNER, other than transportation permits required for KNA's mobilization and demobilization.
5. All encroachment permits within the public right of way and letters of permission from private owners must be obtained by the OWNER.
6. KNA will provide a qualified full-time quality control (QC) representative. This representative is either KNA's Superintendent/Foreman/or Field Engineer. Third party testing and/or inspection shall be provided by OTHERS.
7. Locating, protecting and rerouting/removal of all utilities are the responsibility of OTHERS. KNA is not responsible for damage to existing utilities.
8. After the completion of Ground Improvement work, OTHERS are responsible for the protection of DSM columns. Proper site drainage to prevent ponding of water at the area of the soil-mixed columns and control coordination of earthwork activities shall be managed such that existing soil-mixed columns are not damaged.
9. The DSM locations shown on the approved construction drawings are only for Ground Improvement layouts. These plans should not be used for foundation layout.
10. All post-improvement testing including frequency and criteria for soil-mixed columns are noted on the plans and design submittal.
11. Foundations shall not be poured until approved by the project Geotechnical Engineer of Record.
12. Alternate structural shapes, material, and details cannot be used unless reviewed and approved by the ground improvement engineer, DSA & CGS.

DSM VERIFICATION NOTES:

1. The acceptance of the work shall be based on demonstrating that the in-place mixing of grout with the treatment soils has achieved the average design strength requirements. Soilcrete strengths shall be determined statistically by wet (grab) sample and core samples. Confirmation sample collection and testing will be conducted by KNA. Samples shall be collected by KNA using wet sampling and continuous core sampling techniques described below. Test shall be performed at the frequencies described below.
2. Wet Soil mix samples will be retrieved and cast into molds for one column per rig/shift, at one random depth, typically near the end of each shift. Samples will be retrieved using an in situ wet sampler immediately after column construction and shall consist of no fewer than 8 specimens. These samples shall be tested in pairs: two at seven (7) days, two at fourteen days (14), two at twenty eight (28) days and two at fifty six (56) days if necessary. Soil clods greater than 10% of the mold diameter will be screened off. Appropriate curing techniques shall be implemented until testing based on ASTM D 1632.
3. Unconfined compression testing shall be performed by an approved laboratory working directly for the OWNER. Samples shall be tested in pairs starting at 7-days. If the 7-day specimens do not reach the desired strength according to the lab test curve, another pair of specimens will be tested at 14 days, 28 days, and if needed at 56 days. All specimens at 28 days and available 56-days of age will be tested and used in the statistical calculation.
4. If wet grab strengths at 7 days of age are greater than the average required unconfined compressive strength, additional tests may be omitted at the discretion of the GEOR. Web grab samples will be kept on-site (approximately 3 days) for an intial set before being shipped to the lab.
5. The Unconfined Compressive Strength (UCS) shall be determined by ASTM D1633 "Standard Test Methods for Compressive Strength of Molded Soil-Cement Cylinders". Sulfur caps shall be required in the UCS tests to minimize the end effects on the test specimen. The advantage of the wet sampling is that KNA can get an early trend of the soilcrete strength development without waiting to the end of the project for coring and can make early decisions in the field program to add additional soil mixing columns if necessary.
6. KNA will core 2% of the production DSM columns.
7. All core locations shall be randomly selected and the selection of locations for confirmation coring and selection of core samples for UCS testing are subject to review and approval of the Geotechnical Engineer of Record (GEOR) for the project.
8. The double-tubes coring method, with the utilization of vibrators to assist the core to depth, can be used instead of the conventional coring technique. At minimum three (3) samples from each core will be extracted. KNA anticipates 4 specimens trimmed from each core hole and tested by ASTM D1633.
9. KNA will calculate the average 28-day UCS value from all core samples and wet grab samples. The target average 28 days UCS value shall be 150 psi or greater.
10. No more than 5 percent of all specimens tested shall exhibit an unconfined compressive strength of less than 150 psi at 28 day of age. A ceiling, the not-to-exceed value of four times the average unconfined compressive strength (i.e., 600 psi) shall be used for individual specimens in calculating the average strength achieved in the field from each coring and wet sample and for the entire project.
11. If the acceptance criteria is not achieved in a designated area, KNA may be given the opportunity to conduct additional UCS test on soilcrete specimens on 56 days of age, site exploration, coring, sampling, downhole imaging, and strength testing from the additional cured specimen to better define the average design strength at KNA's preference and expense. If a designated area is rejected, KNA shall submit a Remixing or Mitigation plan.
12. Uniformity of mixing shall be evaluated by the Geotechnical Engineer of Record (GEOR) based on the continuous core samples recovered. The continuous core holes shall extend the entire depth of the DSM column. Estimated recovery of 80 percent for each 5-foot-long segment of a boring and at least 90 percent when averaged over all core runs within a single boring shall be achieved. The lumps of unimproved soils shall not exceed 20 percent of the total volume of any 5-foot core segment from a boring. If the core recovery below the anticipated value due to the gravel particles in the soilcrete matrix, KNA shall be allowed to utilize a downhole camera or other approved methods to verify the core hole. This may include additional cores in the same column.
13. At the end of the project, to not unnecessary delay subsequent activities by waiting for 28 days test result, a correction of early strength gain will be used to approve the soil-mixed column work. However, this correlation will not relieve the contractor of the responsibility to achieve average 28 days strength. Based on FHWA (2013) guidelines, the following UCS aging factor correlations will be applied to this job:
   a. 28: 3 day, 1.72
   b. 28: 7 day, 1.35
   c. 28: 14 day, 1.15
13. A site-specific correlation between 3 days and 28 days strength may be used to supersede this correlation if in the opinion of the Engineer, the site-specific correlation is more appropriate.

DSM CONSTRUCTION:

1. OWNER will provide to KNA, at least four (4) control points. KNA will provide an AutoCAD Shop Drawing for all DSM columns overlaid on the site Civil drawing and stake all DSM locations.
2. DSM columns will be installed within 6 inches of the design locations as shown in the KNA shop drawing. Construction tolerances:
   a. Plan location ±6 inches
   b. Verticality ±1% of plumb
3. KNA retains the sole authority to modify DSM column locations due to constructability and/or site constraints. KNA will prepare as-built drawings after completion.
4. Once a stable working platform has been established as shown in KNA Shop Drawing. DSM columns will be constructed sequentially based on a pattern dictated in the Field. KNA requires access to all DSM locations at all times to maximize efficiency.
5. To minimize the mixing tool damage and maintaining soil mixing quality, KNA may pre-dill holes or excavate for better mixing quality. The holes will be filled with soilcrete up to the working elevation during the mixing stage.
6. In general, soil mixing operation parameters, such as mixing shaft speed, penetration rate, batching grout specific gravity, and pumping rate will be determined based on our lab mixing results and our experience and will be fine-tuned at the beginning of mixing column production. The design cement content in place (cement weight/[soil volume + grout volume]) will start from approximately 200 kg/m^3 with grout slurry specific gravity of 1.45. KNA's Engineers may adjust the cement content and specific gravity based on the field sample strength development.
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8. The mixing shaft speed which is anticipated to be ranging between 20-50 RPM and shall be adjusted to accommodate a constant rate of mixing shaft penetration based on the degree of drilling difficulty. The mixing shaft speed can be adjusted according to drilling difficulty. The mixing shaft speed can be adjusted to aid mixing of the soil column when needed or to assist penetration in hard drilling. Mixing shaft speed will be recorded.
9. In order to ensure adequate mixing, the penetration rate of the mixing shaft shall be maintained at about 1.0 to 3.0 feet/minute during penetration but will vary based on actual site conditions. The penetration rate and maximum depth of each stroke shall be recorded by KNA's data acquisition system (DAQ).
10. The grout slurry flow per vertical foot of the column will be adjusted to the requirements of the design mix. Progressive cavity pumps will be used to transfer the grout from the mixing plant to the mixing rig. Flow monitoring devices will be installed in the grout line to detect any line blockage and monitor flow, total injected grout per column and grout pressure. These parameters will be recorded.
11. Inevitably some variations of the grout take will occasionally occur due to field conditions. It is anticipated that a grout flow rate between 50 to 250 GPM will be used during penetration. KNA's Data Acquisition System (DAQ) can automatically adjust the grout flow rate as a function of the penetration rate and maintain the pre-set cement dosage prescribed by the design engineer.
12. The mixing shaft will be withdrawn at a rate of 6 to 12 feet per minute during the re-stroke operation and complete removal of the mixing shaft from the ground thus mixed.
13. KNA will use a data acquisition system to monitor the mixing shaft penetration and the shaft rotation resistance in terms of the hydraulic pressure. The DAQ system will calculate and plot the Drilling Index as a function of depth, a mixing parameter to detect penetration resistance and refusal depth. KNA will set up the penetration criteria based on the site measurement. In case of underground obstruction, such as abandoned footings, piles, utilities, etc., the general contractor will be responsible to remove obstructions and backfilled with sandy soil prior KNA mixing operation.
14. Cement will be furnished by KNA and conform to ASTM C150 "Standard Specification for Portland Cement," Type II/V or equivalent. The cement will be adequately protected from moisture and contamination while in transit to and in storage at the job site. Reclaimed cement or cement containing lumps or deleterious matter will not be used.
15. Water for the slurry will be fresh, free of deleterious substances that adversely affect the strength and mixing properties of the slurry, furnished by the OTHERS.
16. The batch plant shall consist of in-line eductor (jet valve) mixers. Dry materials shall be stored in tankers and/or silos and fed to the mixers for shearing and circulation. The resulting grout slurry will be transferred to a surge tank for continuous agitation and to supply the in-situ soil mixing rig. Grout slurry quality will be assured by frequent testing prior to injection into the soil.
17. Single shaft mixing equipment that mechanically mixes the soil and cement slurry for the full dimensions of the column will be used for the work. We anticipate using hydraulic drill rigs for the soil mixing operations. This rig is capable of up to > 150,000 ft-lbs. of torque at > 20 rpm. The working shaft rate of rotation ranges between 20 and 50 rpm. The mixing shaft will have mixing augers and/or blades (paddles) configured in such a manner so that they are capable of thoroughly blending the in-situ soils and cement slurry. The power source for driving the mixing shafts will be sufficient to maintain the required mix tool (shaft) rotation speed in revolutions per minute and penetration/ withdrawal rates from the ground surface to the maximum depth required. The design target Blade Rotation Number (BRN, defined as the number of blades cut in each 1.0-meter soil) will be at least 300.
18. The DSM equipment will be equipped with devices to assure vertical alignment in two planes (90 degrees in plan from each other): fore-aft and left-right. The DSM equipment will be equipped with a real-time display of depth, rotation speed, grout flow rate; grout specific gravity, cumulative grout injected, and grout pressure for each soil mix column. The cement will be mixed with water within the jet valve to create a 1.45 specific gravity mix +/- .1. No mixing operation will be only allowed if the DAQ system not functioning.
19. Grout slurry will be supplied to the drill using large size Moyno pumps. These pumps will be sized and powered so that design volumes and pressures can be maintained up to 1,000 ft away from the batching facility. It is anticipated that a continuous grout slurry flow of 150 gallons per minute at 100 psi to the drill rig will be necessary.
20. The batching and pumping facility will be set up at a central location to areas all structures. This will eliminate the need to move the plant once it is established.

STRUERE
ADVANCED ARCHITECTURE
3324 GRAND VIEW
LOS ANGELES, CALIFORNIA 90066
TELEPHONE (310) 748-7649
E-MAIL HRAZTAN@STRUERE.COM
WWW.STRUERE.COM

KELLER
17461 DERIAN AVENUE
SUITE 106
IRVINE, CALIFORNIA
92614
WWW.KELLER-NA.COM

DSM ENGINEER OF RECORD:

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REVIEWED FOR
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DATE: ________

Compton College

COMPTON COLLEGE
PHYSICAL EDUCATION
COMPLEX REPLACEMENT

1111 EAST ARTESIA BLVD,
COMPTON, CA 90221-5393

ISSUE DESCRIPTION	
DSA SUBMITTAL	22.02.28

SHEET TITLE
DSM NOTES AND DETAILS
KNA-2P

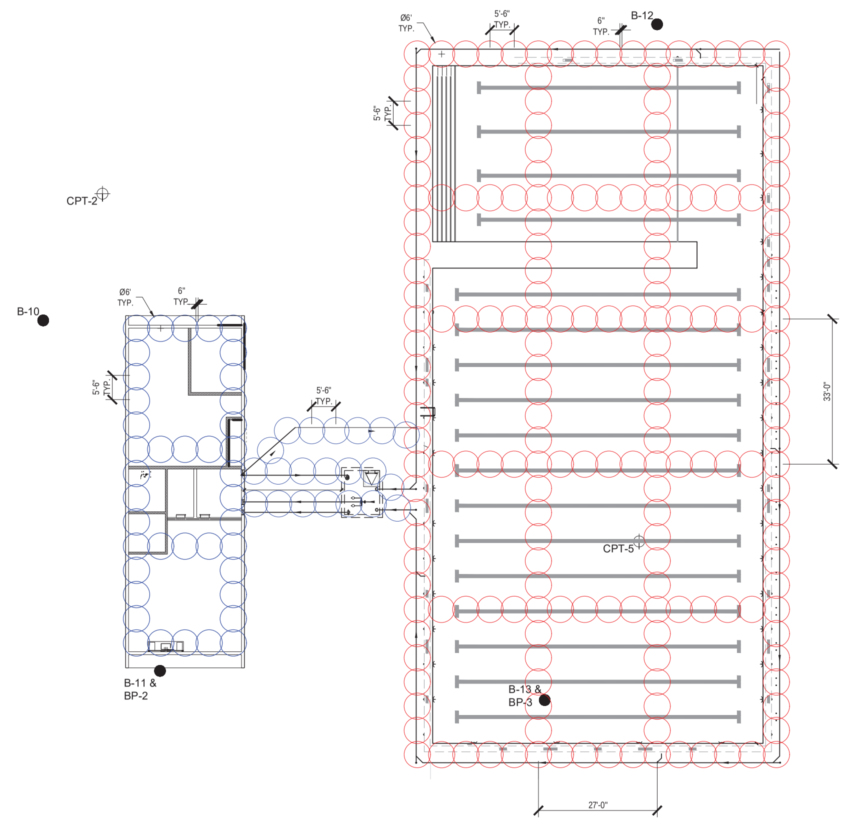
LEGEND:

Ø6' DEEP SOIL MIX COLUMN (DSM).
Install 49' from the existing ground surface.
Approximate existing ground surface elevation: 55 feet.

Ø6' DEEP SOIL MIX COLUMN (DSM).
Install 19' from the existing ground surface.
Approximate existing ground surface elevation: 55 feet.

CPT-X Existing CPT location

B-X Existing Boring location



STRUERE
ADVANCED ARCHITECTURE
3324 GRAND VIEW
LOS ANGELES, CALIFORNIA 90066
TELEPHONE (310) 748-7649
E-MAIL HRAZTAN@STRUERE.COM
WWW.STRUERE.COM

KELLER
17461 DERIAN AVENUE
SUITE 106
IRVINE, CALIFORNIA
92614
WWW.KELLER-NA.COM

DSM ENGINEER OF RECORD:

IDENTIFICATION STAMP
DIV. OF THE STATE ARCHITECT
APP. 03-121755 INC. #
REVIEWED FOR
SS ☐ FLS ☐ ACS ☐
DATE: ______

Compton College

COMPTON COLLEGE
PHYSICAL EDUCATION
COMPLEX REPLACEMENT

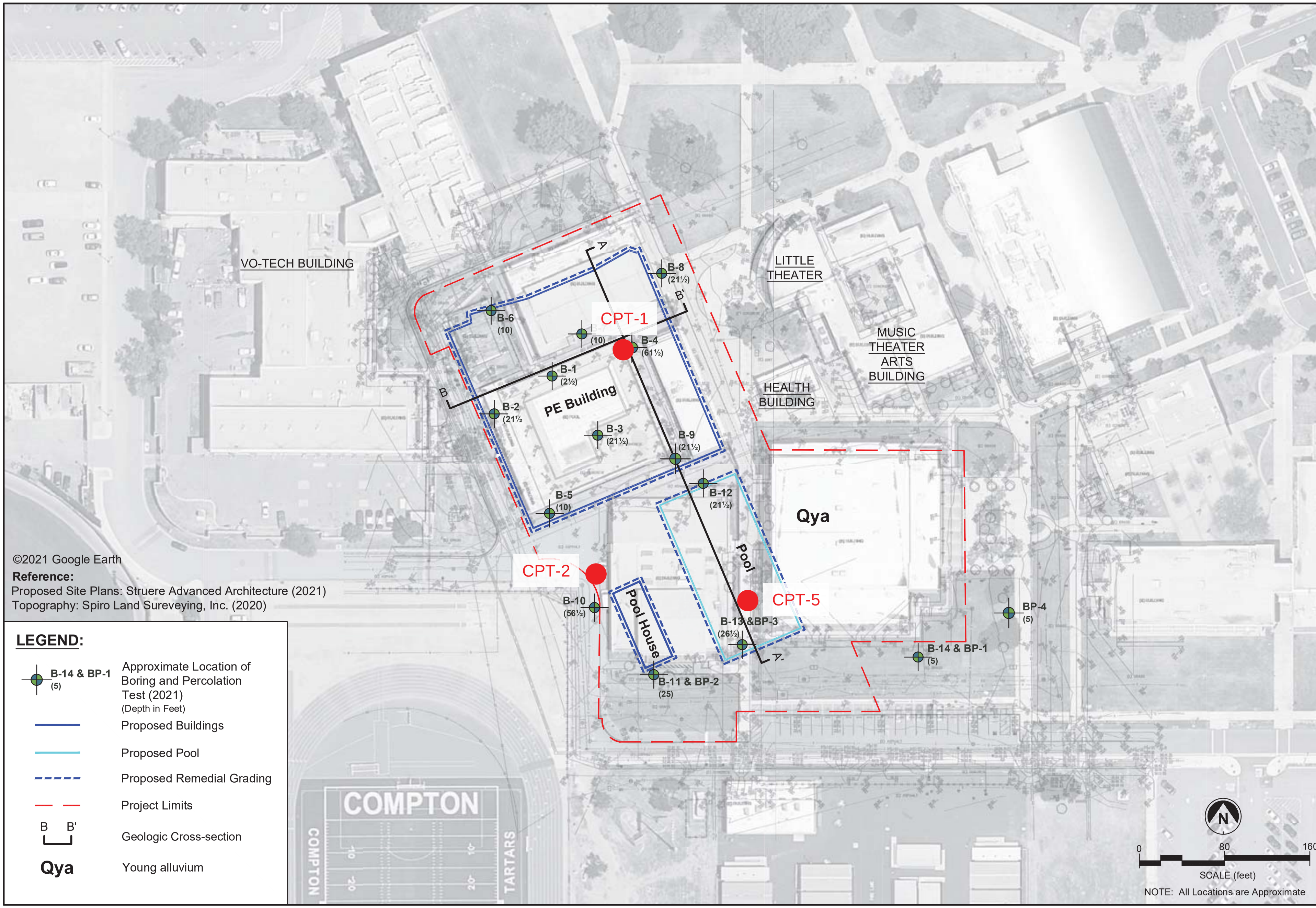
1111 EAST ARTESIA BLVD,
COMPTON, CA 90221-5393

ISSUE DESCRIPTION	
DSA SUBMITTAL	22.02.28

SHEET TITLE
OVERALL GROUND IMPROVEMENT PLAN
KNA-3P

2/14/2022 6:20:28 PM

Appendix B
Pre- and Post-treatment Liquefaction-induced Settlement Computation



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

GEOTECHNICAL MAP
Physical Education Complex Replacement
Compton, California

ATLAS

Figure: 2

Liquefaction Analysis and Deep Soil Mixing Mitigation

Project: OP0013298 - Compton Community College

CPT ID: CPT-2 (Pool Building)   Surface Elev.: 0.0 ft (use 0 ft to plot depth instead of elevation)

by: Bailey Uy

Date: 09.07.21

Perform Ground Improvement Analysis?: **Deep Soil Mixing**

LIQUEFACTION ANALYSIS PARAMETERS

Triggering Method =	Robertson (NCEER R&W 1998)	
Vol. Settlement Method =	Zhang et al. (2002)	
Depth of GW During CPT =	44.00	ft
Depth of GW During Earthquake =	8.00	ft
Depth of Fill =	0.00	ft
Unit Weight of fill =	120	pcf
PGApre =	0.802	g
Mw =	7.30	
Ic Threshhold =	2.6	
Use Kσ ? =	Yes	

ADVANCED LIQUEFACTION PARAMETERS

Use Ic Transition Zones?	Yes			
Transition zone (dIc / dz) =	0.65	Ic/ft	Manual Trans Zones?:	No
Cliq TZ dIc =	0.03		Use Manual Thin Layer Cor.? =	No
Min. Trans. Zone Points:	4			
Ic_min and max =	1.9	3		

DSM GRID DESIGN PARAMETERS

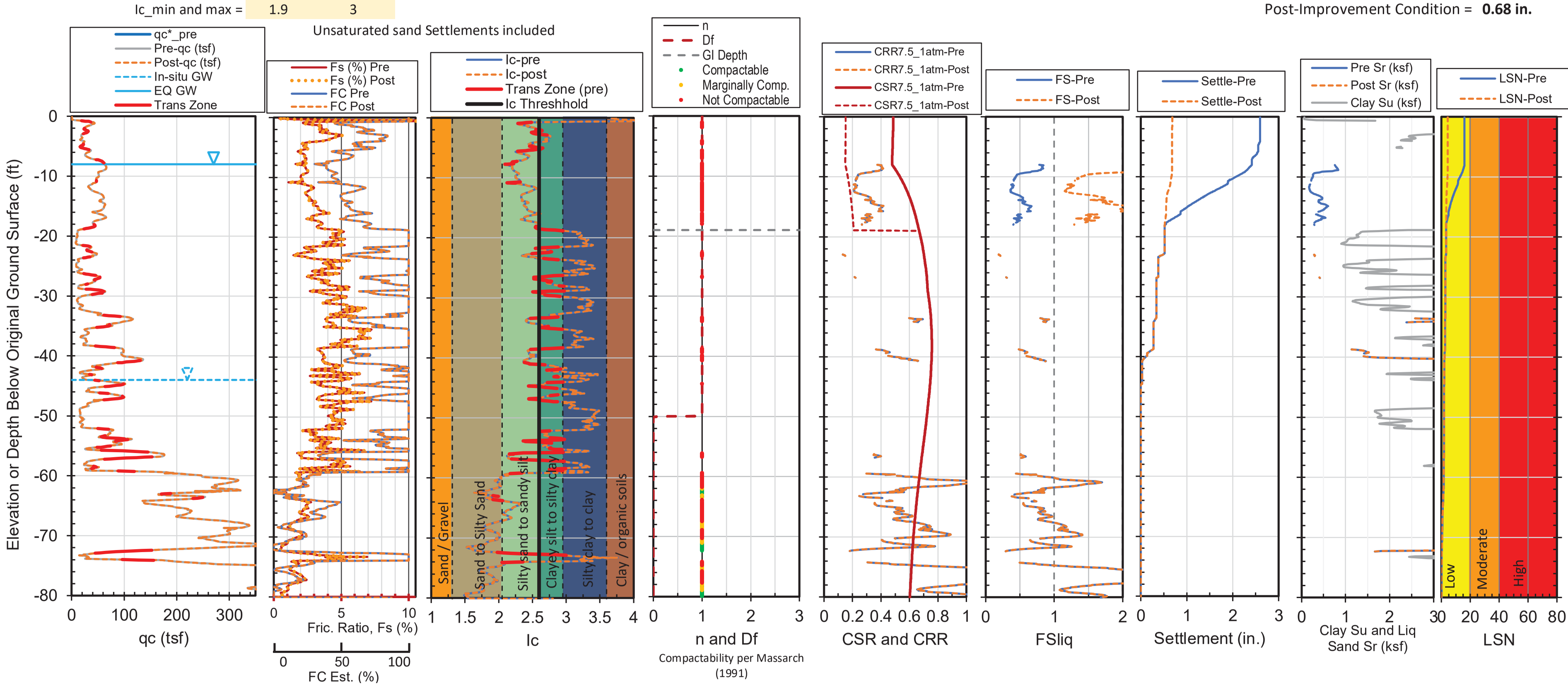
Depth Below Existing Grade =	19	ft
ARR =	30	%
S =	28	ft
Gr =	30	
Rrd = PGApost/PGApre =	0.314	
PGApost in Impr. Zone =	0.252	g

Volumetric Settlement Results:

Existing (Pre-Treatment) Condition = **2.59 in.**

Post-Improvement Condition = **0.68 in.**

Unsaturated sand Settlements included



Liquefaction Analysis and Deep Soil Mixing Mitigation

Project: OP0013298 - Compton Community College

CPT ID: CPT-5 (Swimming Pool) Surface Elev.: 0.0 ft (use 0 ft to plot depth instead of elevation)

by: Bailey Uy

Date: 09.07.21

Perform Ground Improvement Analysis?: **Deep Soil Mixing**

LIQUEFACTION ANALYSIS PARAMETERS

Triggering Method =	Robertson (NCEER R&W 1998)	
Vol. Settlement Method =	Zhang et al. (2002)	
Depth of GW During CPT =	44.00	ft
Depth of GW During Earthquake =	8.00	ft
Depth of Fill =	0.00	ft
Unit Weight of fill =	120	pcf
PGApre =	0.802	g
Mw =	7.30	
Ic Threshhold =	2.6	
Use Kσ ? =	Yes	

ADVANCED LIQUEFACTION PARAMETERS

Use Ic Transition Zones?	Yes			
Transition zone (dIc / dz) =	0.65	Ic/ft	Manual Trans Zones?:	No
Cliq TZ dIc =	0.04		Use Manual Thin Layer Cor.? =	No
Min. Trans. Zone Points:	4			
Ic_min and max =	1.9	3		

DSM GRID DESIGN PARAMETERS

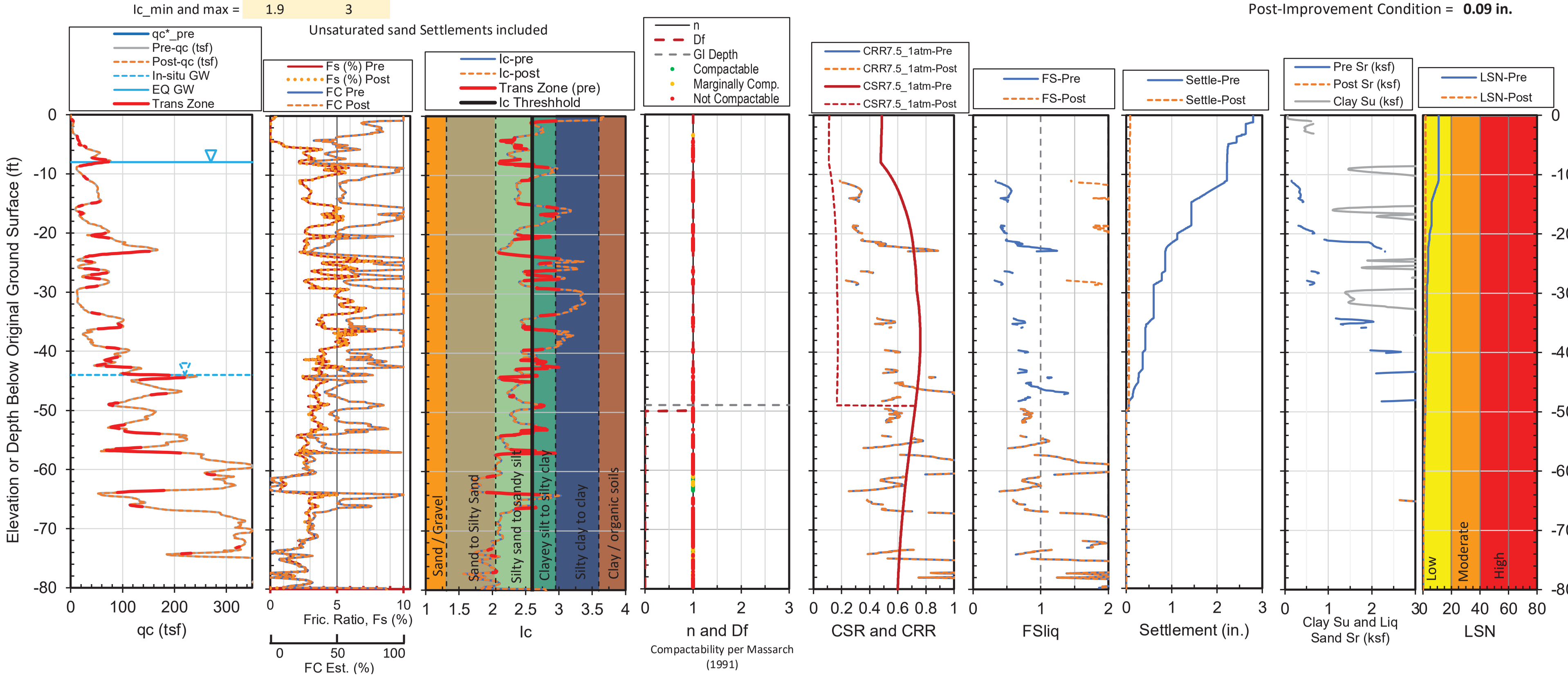
Depth Below Existing Grade =	49	ft
ARR =	30	%
S =	28	ft
Gr =	30	
Rrd = PGApost/PGApre =	0.229	
PGApost in Impr. Zone =	0.184	g

Volumetric Settlement Results:

Existing (Pre-Treatment) Condition = **2.79 in.**

Post-Improvement Condition = **0.09 in.**

Unsaturated sand Settlements included



Attachment C

Bearing capacity calculations

VIBRO PIER GROUP BEARING CAPACITY CALCULATION

Project name	Compton Community Coll
Project location	
Project number	OP0013298
Date	2.18.2022
Designed by	MBU
Reviewed by	

Input Parameters

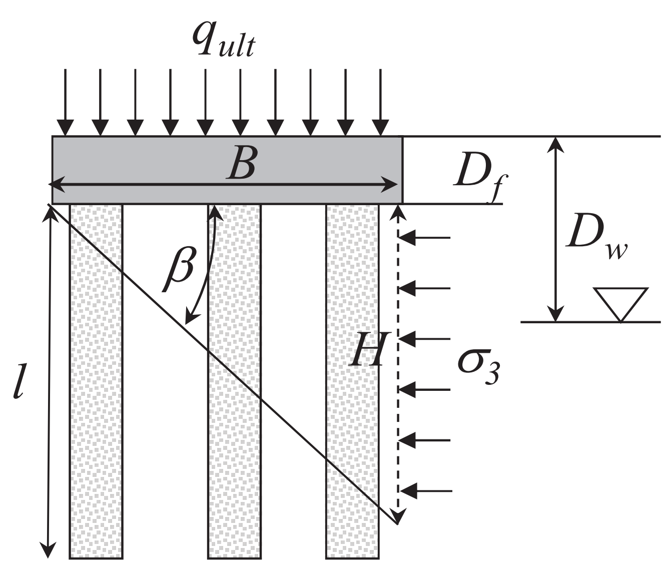
Strip or Square	-	Square	
Footing width	B	4	ft
Footing length	L	4	ft
Depth of embedment	D_f	2	ft
Area replacement ratio	ARR	0.00	
Adjacent surcharge	σ_{sur}	0	psf
Water table depth	D_w	8	ft
Moist unit weight	γ_{moist}	120	pcf
Saturated unit weight	γ_{sat}	120	pcf
Soil friction angle	ϕ_{soil}	38	°
Soil cohesion	c_{soil}	320	psf
Stress concentration factor	n	0	
Soil elastic modulus (at H/2)	E	250,000	psf
Stone friction angle	ϕ_{stone}	45	°
Treatment Depth	l	23	ft

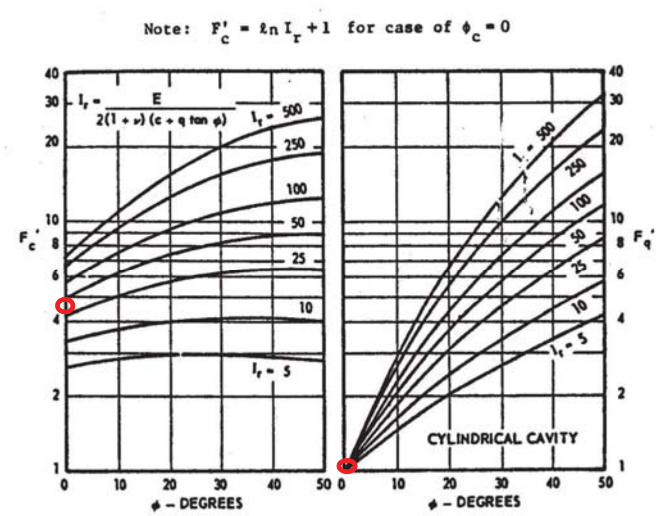
Calculated Parameters

Aggregate pier stress factor	μ_{SC}	0.0	
Soil stress factor	μ_{SOIL}	1.0	
Composite friction angle	ϕ_{avg}	38	°
Composite cohesion	c_{avg}	320	psf
Failure plane angle	β	64	°
Vertical interface length	H	8.2	ft
Passive coefficient	K_p	4.20	
Average vertical effective stress	σ_{vo}'	714	psf
Mean normal effective stress	q	2238	psf
Rigidity Index	I_R	45	
Confinement stress	σ_3	3,678	psf

Additional Input Parameters (for square footings)

Vesic cohesion factor	F_c'	4.5	
Vesic mean stress factor	F_q'	1	





From FHWA, 1983 (after Vesic)

Ultimate bearing pressure	q_{ult}	**16,773**	**psf**
Allowable bearing pressure	q_{des}	**3000**	**psf**

Factor of safety	**FS**	**5.6**

VIBRO PIER GROUP BEARING CAPACITY CALCULATION

Project name	Compton Community Coll
Project location	
Project number	OP0013298
Date	2.18.2022
Designed by	MBU
Reviewed by	

Input Parameters

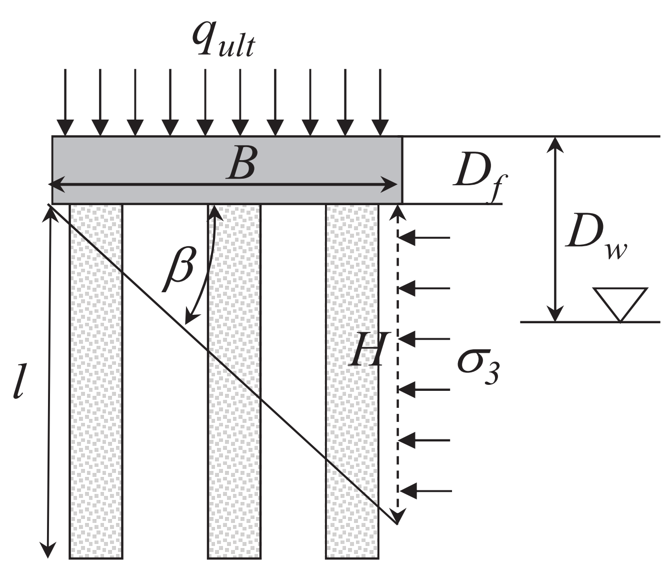
Strip or Square	-	Square	
Footing width	B	6	ft
Footing length	L	6	ft
Depth of embedment	D_f	2	ft
Area replacement ratio	ARR	0.00	
Adjacent surcharge	σ_{sur}	0	psf
Water table depth	D_w	8	ft
Moist unit weight	γ_{moist}	120	pcf
Saturated unit weight	γ_{sat}	120	pcf
Soil friction angle	ϕ_{soil}	38	°
Soil cohesion	c_{soil}	320	psf
Stress concentration factor	n	0	
Soil elastic modulus (at H/2)	E	250,000	psf
Stone friction angle	ϕ_{stone}	45	°
Treatment Depth	l	23	ft

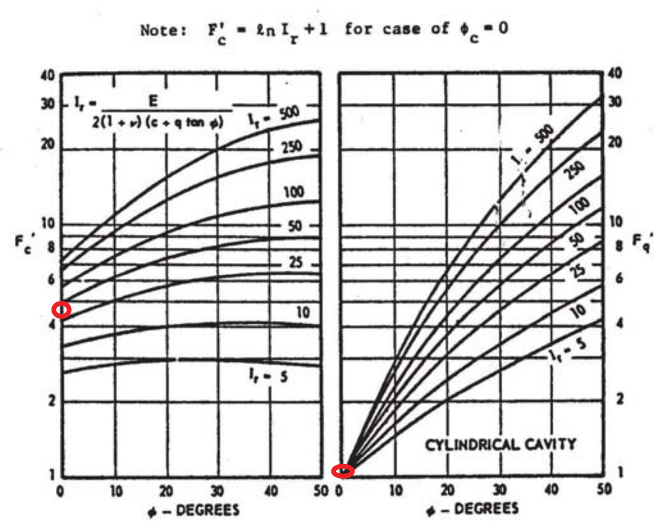
Calculated Parameters

Aggregate pier stress factor	μ_{SC}	0.0	
Soil stress factor	μ_{SOIL}	1.0	
Composite friction angle	ϕ_{avg}	38	°
Composite cohesion	c_{avg}	320	psf
Failure plane angle	β	64	°
Vertical interface length	H	12.3	ft
Passive coefficient	K_p	4.20	
Average vertical effective stress	σ_{vo}'	877	psf
Mean normal effective stress	q	2751	psf
Rigidity Index	I_R	37	
Confinement stress	σ_3	4,191	psf

Additional Input Parameters (for square footings)

Vesic cohesion factor	F_c'	4.5	
Vesic mean stress factor	F_q'	1	





From FHWA, 1983 (after Vesic)

Ultimate bearing pressure	q_{ult}	**18,932**	**psf**
Allowable bearing pressure	q_{des}	**3000**	**psf**

Factor of safety	**FS**	**6.3**

VIBRO PIER GROUP BEARING CAPACITY CALCULATION

Project name	Compton Community Coll
Project location	
Project number	OP0013298
Date	2.18.2022
Designed by	MBU
Reviewed by	

Input Parameters

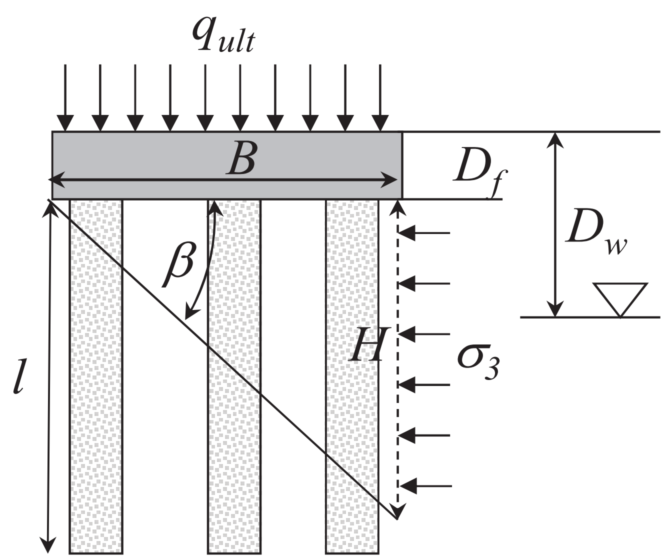
Strip or Square	-	Square	
Footing width	B	7	ft
Footing length	L	7	ft
Depth of embedment	D_f	2.5	ft
Area replacement ratio	ARR	0.00	
Adjacent surcharge	σ_{sur}	0	psf
Water table depth	D_w	8	ft
Moist unit weight	γ_{moist}	120	pcf
Saturated unit weight	γ_{sat}	120	pcf
Soil friction angle	ϕ_{soil}	38	°
Soil cohesion	c_{soil}	320	psf
Stress concentration factor	n	0	
Soil elastic modulus (at H/2)	E	250,000	psf
Stone friction angle	ϕ_{stone}	45	°
Treatment Depth	l	23	ft

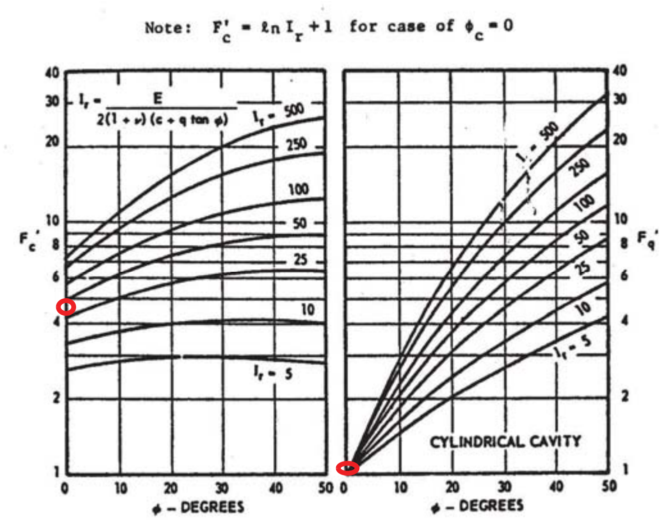
Calculated Parameters

Aggregate pier stress factor	μ_{SC}	0.0	
Soil stress factor	μ_{SOIL}	1.0	
Composite friction angle	ϕ_{avg}	38	°
Composite cohesion	c_{avg}	320	psf
Failure plane angle	β	64	°
Vertical interface length	H	14.4	ft
Passive coefficient	K_p	4.20	
Average vertical effective stress	σ_{vo}'	991	psf
Mean normal effective stress	q	3107	psf
Rigidity Index	I_R	34	
Confinement stress	σ_3	4,547	psf

Additional Input Parameters (for square footings)

Vesic cohesion factor	F_c'	4.5	
Vesic mean stress factor	F_q'	1	





From FHWA, 1983 (after Vesic)

Ultimate bearing pressure	q_{ult}	**20,426**	**psf**
Allowable bearing pressure	q_{des}	**3000**	**psf**

Factor of safety	**FS**	**6.8**

VIBRO PIER GROUP BEARING CAPACITY CALCULATION

Project name	Compton Community Coll
Project location	
Project number	OP0013298
Date	2.18.2022
Designed by	MBU
Reviewed by	

Input Parameters

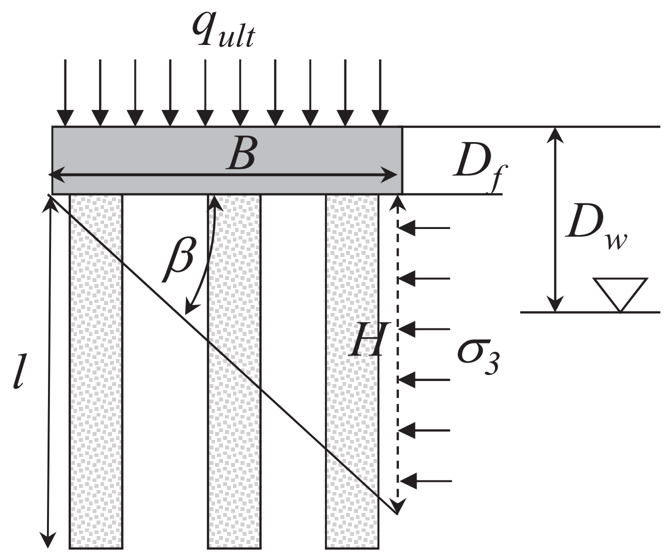
Strip or Square	-	Square	
Footing width	B	9.5	ft
Footing length	L	9.5	ft
Depth of embedment	D_f	2.5	ft
Area replacement ratio	ARR	0.00	
Adjacent surcharge	σ_{sur}	0	psf
Water table depth	D_w	8	ft
Moist unit weight	γ_{moist}	120	pcf
Saturated unit weight	γ_{sat}	120	pcf
Soil friction angle	ϕ_{soil}	38	°
Soil cohesion	c_{soil}	320	psf
Stress concentration factor	n	0	
Soil elastic modulus (at H/2)	E	250,000	psf
Stone friction angle	ϕ_{stone}	45	°
Treatment Depth	l	23	ft

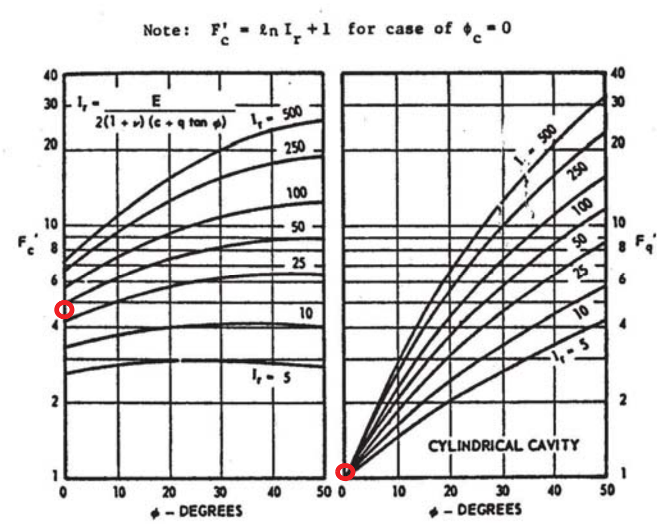
Calculated Parameters

Aggregate pier stress factor	μ_{SC}	0.0	
Soil stress factor	μ_{SOIL}	1.0	
Composite friction angle	ϕ_{avg}	38	°
Composite cohesion	c_{avg}	320	psf
Failure plane angle	β	64	°
Vertical interface length	H	19.5	ft
Passive coefficient	K_p	4.20	
Average vertical effective stress	σ_{vo}'	1156	psf
Mean normal effective stress	q	3624	psf
Rigidity Index	I_R	29	
Confinement stress	σ_3	5,064	psf

Additional Input Parameters (for square footings)

Vesic cohesion factor	F_c'	4.5	
Vesic mean stress factor	F_q'	1	





From FHWA, 1983 (after Vesic)

Ultimate bearing pressure	q_{ult}	**22,600**	**psf**
Allowable bearing pressure	q_{des}	**3000**	**psf**

Factor of safety	**FS**	**7.5**

Attachment D

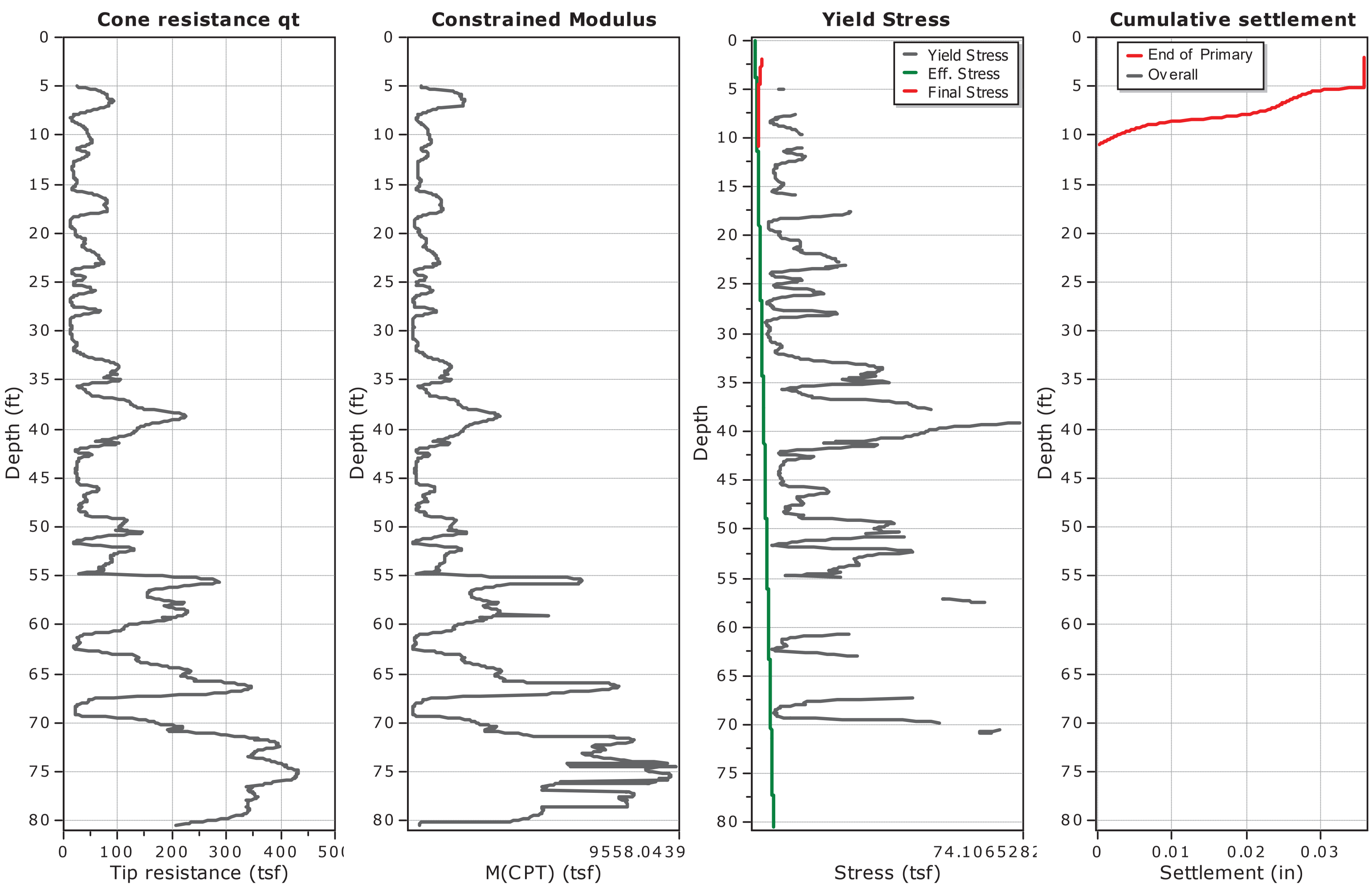
Static settlements under footings

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

Project: ATLAS
Location: 1111 E. Artesia Blvd, Compton, CA

CPT: CPT-1
Total depth: 80.45 ft, Date: 9/3/2021
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown

Settlements calculation according to theory of elasticity*



Calculation properties

Footing type: Rectangular
Footing width: 4.00 (ft)
L/B: 1.0
Footing pressure: 1.50 (tsf)
Embedment depth: 2.00 (ft)
Footing is rigid: No
Remove excavation load: No
Apply 20% rule: Yes
Calculate secondary settlements: No
Time period for primary consolidation: N/A
Time period for second. settlements: N/A

* Primary settlement calculation is performed according to the following formula:

$$S = \sum \frac{\Delta\sigma_v}{M_{CPT}} \Delta z$$

* Secondary (creep) settlement calculation is performed according to the following formula:

$$S = C_\alpha \cdot \Delta z \cdot \log(t/t_p)$$

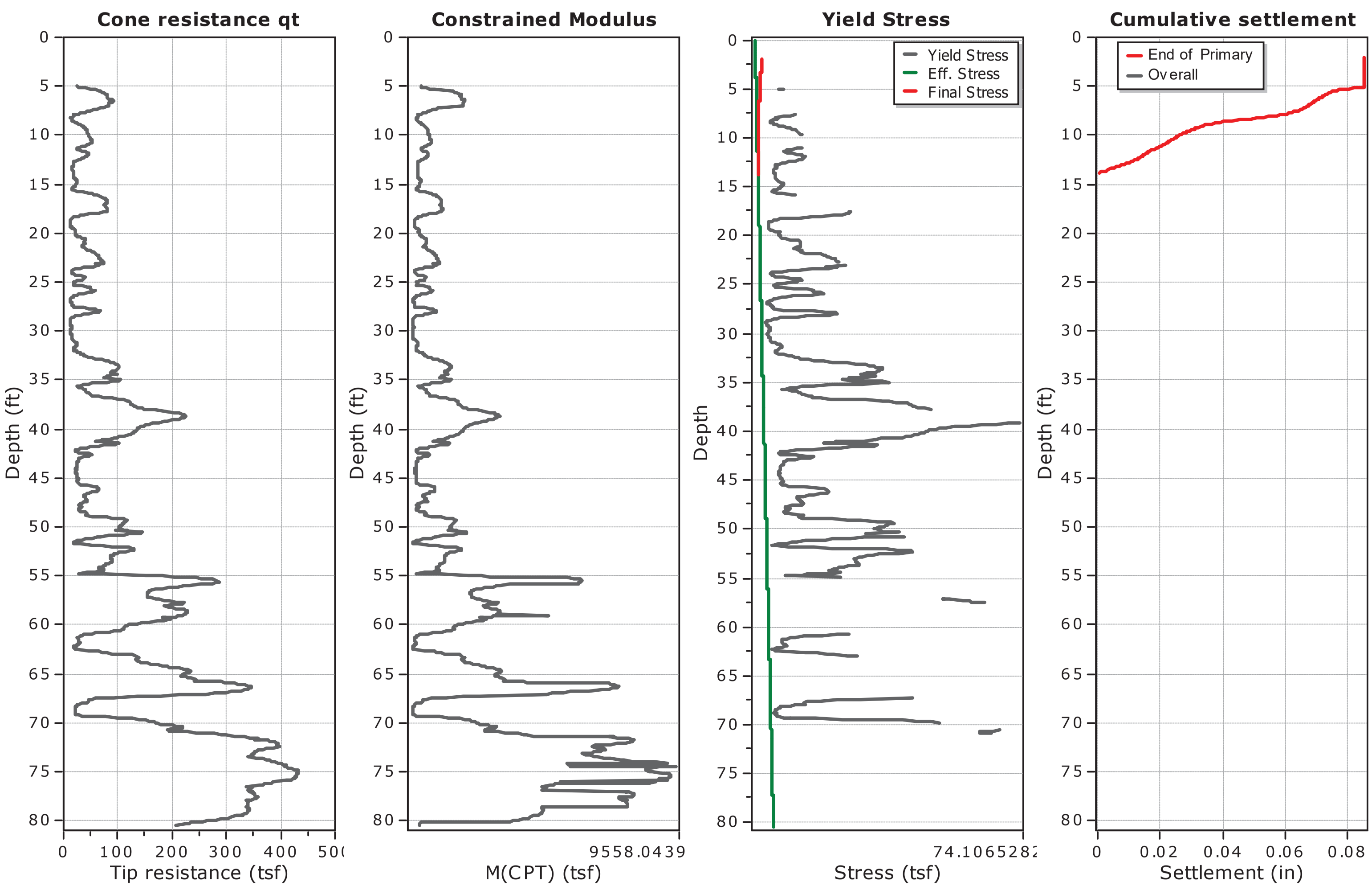
where t_p is the duration of primary consolidation

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

Project: ATLAS
Location: 1111 E. Artesia Blvd, Compton, CA

CPT: CPT-1
Total depth: 80.45 ft, Date: 9/3/2021
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown

Settlements calculation according to theory of elasticity*



Calculation properties

Footing type: Rectangular
Footing width: 6.00 (ft)
L/B: 1.0
Footing pressure: 1.50 (tsf)
Embedment depth: 2.00 (ft)
Footing is rigid: No
Remove excavation load: No
Apply 20% rule: Yes
Calculate secondary settlements: No
Time period for primary consolidation: N/A
Time period for second. settlements: N/A

* Primary settlement calculation is performed according to the following formula:

$$S = \sum \frac{\Delta\sigma_v}{M_{CPT}} \Delta z$$

* Secondary (creep) settlement calculation is performed according to the following formula:

$$S = C_\alpha \cdot \Delta z \cdot \log(t/t_p)$$

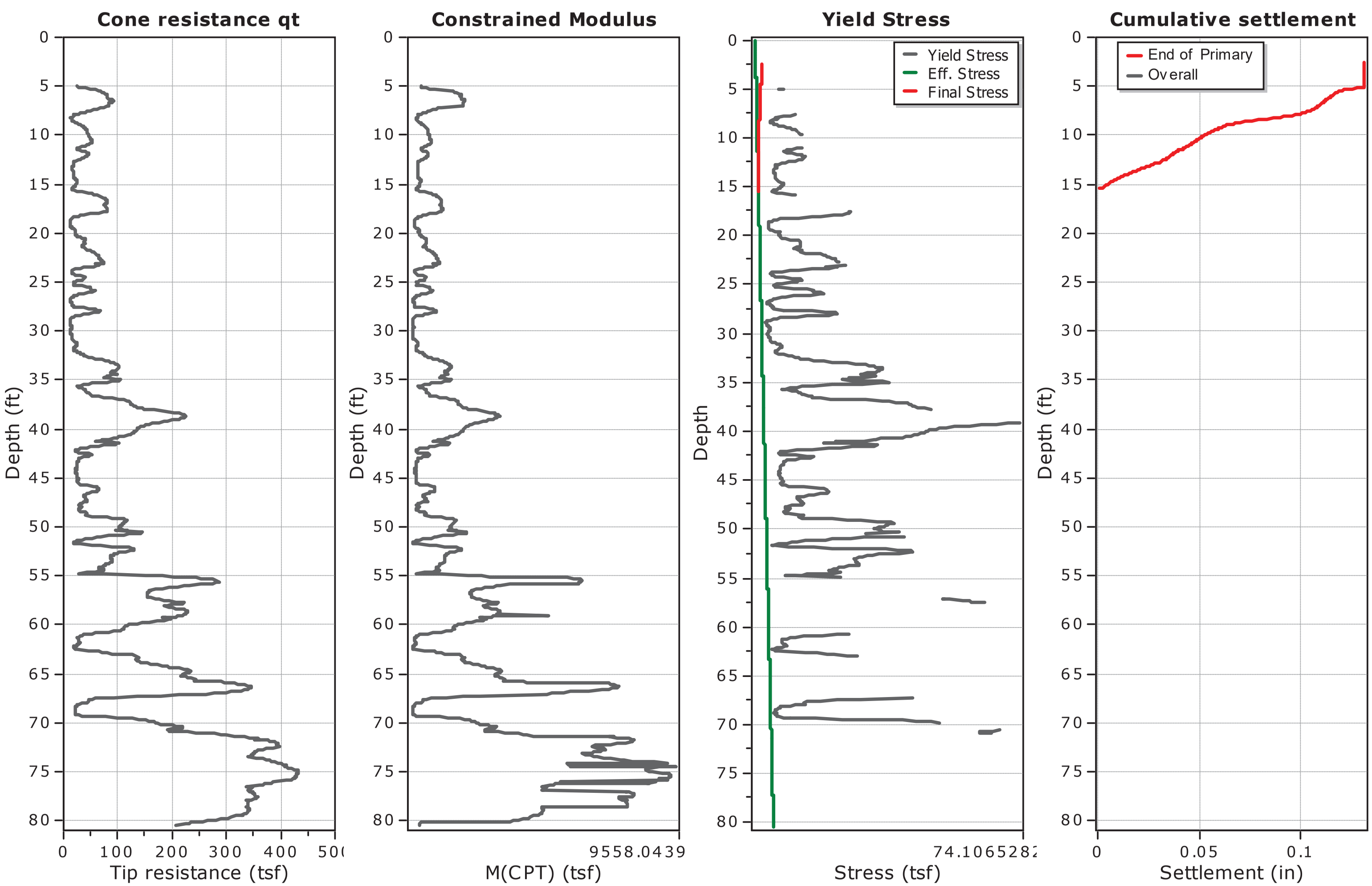
where t_p is the duration of primary consolidation

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

Project: ATLAS
Location: 1111 E. Artesia Blvd, Compton, CA

CPT: CPT-1
Total depth: 80.45 ft, Date: 9/3/2021
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown

Settlements calculation according to theory of elasticity*



Calculation properties

Footing type: Rectangular
Footing width: 7.00 (ft)
L/B: 1.0
Footing pressure: 1.50 (tsf)
Embedment depth: 2.50 (ft)
Footing is rigid: No
Remove excavation load: No
Apply 20% rule: Yes
Calculate secondary settlements: No
Time period for primary consolidation: N/A
Time period for second. settlements: N/A

* Primary settlement calculation is performed according to the following formula:

$$S = \sum \frac{\Delta\sigma_v}{M_{CPT}} \Delta z$$

* Secondary (creep) settlement calculation is performed according to the following formula:

$$S = C_\alpha \cdot \Delta z \cdot \log(t/t_p)$$

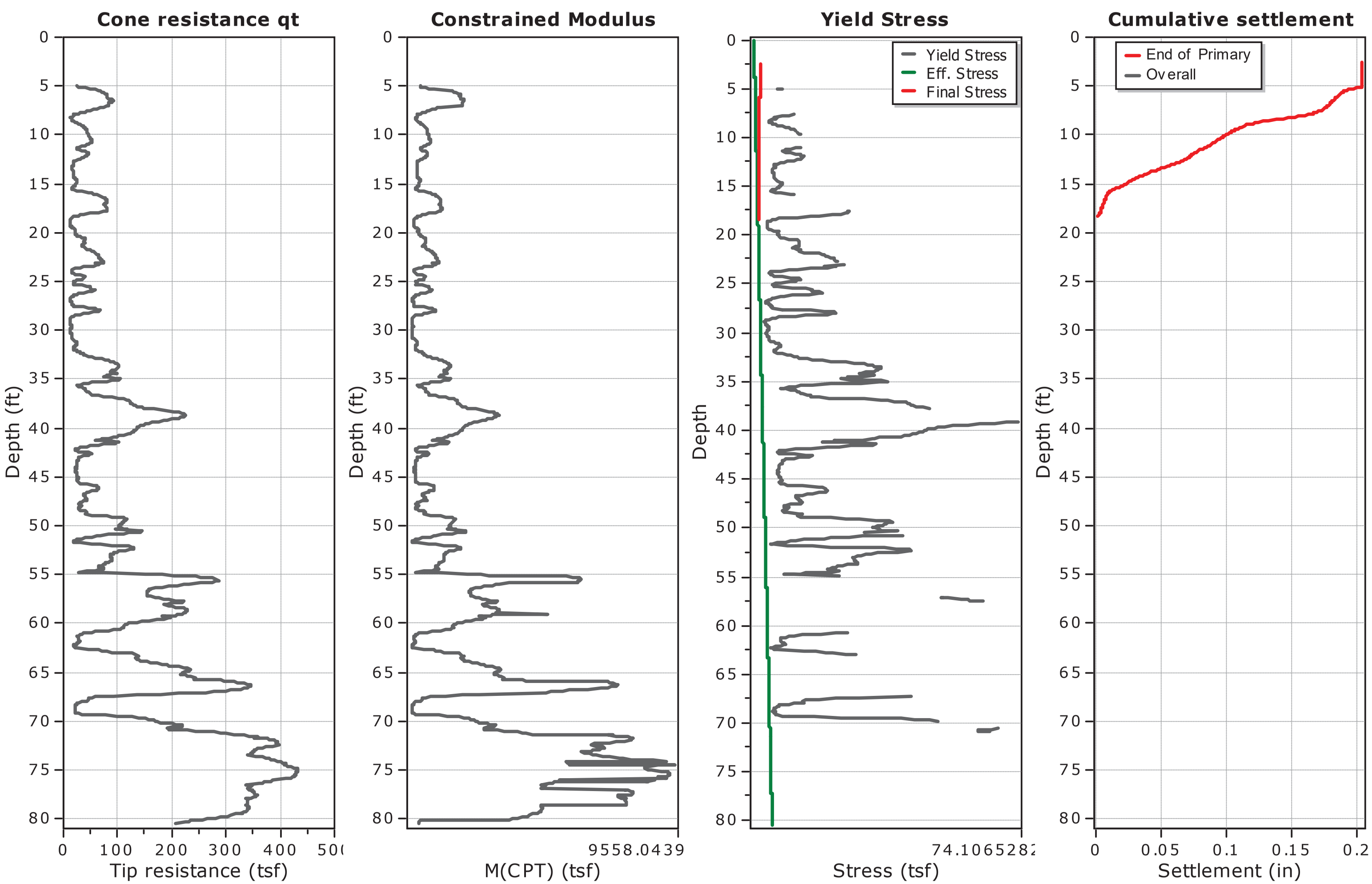
where t_p is the duration of primary consolidation

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

Project: ATLAS
Location: 1111 E. Artesia Blvd, Compton, CA

CPT: CPT-1
Total depth: 80.45 ft, Date: 9/3/2021
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown

Settlements calculation according to theory of elasticity*



Calculation properties

Footing type: Rectangular
Footing width: 9.50 (ft)
L/B: 1.0
Footing pressure: 1.50 (tsf)
Embedment depth: 2.50 (ft)
Footing is rigid: No
Remove excavation load: No
Apply 20% rule: Yes
Calculate secondary settlements: No
Time period for primary consolidation: N/A
Time period for second. settlements: N/A

* Primary settlement calculation is performed according to the following formula:

$$S = \sum \frac{\Delta\sigma_v}{M_{CPT}} \Delta z$$

* Secondary (creep) settlement calculation is performed according to the following formula:

$$S = C_\alpha \cdot \Delta z \cdot \log(t/t_p)$$

where t_p is the duration of primary consolidation

Attachment E

Deep soil mixing sample specification

SPECIFICATION

Deep Soil Mixing (DSM)

PART 1 - GENERAL

1.01 SCOPE

A. In accordance with the specifications contained in this Section and as shown on the Contract Drawings, the DSM Contractor shall furnish all plant, equipment, labor, and materials required to plan, develop mix design, and construct the Deep Soil Mixing (DSM) test section and production DSM at the locations and elevations indicated on the Contract Drawings and these specifications, and associated testing, monitoring, sampling, and recording to meet the performance requirements outlined in these Contract Documents.

B. The purpose of the DSM is to mitigate the liquefaction potential and limit differential settlement under the building area.

C. The scope of work shall include, but not limited to, the following:

1. Design for DSM.
2. Construction of the DSM.

1.02 RELATED DOCUMENTS

1.02.1. The DSM Contractor shall acknowledge that the following references have been received, read, and understood at the time of the bid.

1.03 DEFINITIONS

1. DSM: A soil-cement constructed by treating soils in situ by deep soil–cement mixing technology. The DSM shall consist of overlapping DSM columns in a single row or overlapping multiple columns.
2. Element: This is an inclusive term that refers to a DSM element produced by a single stroke of the mixing tools at a single equipment location. An element produced by a single-axis machine or a set of overlapping elements produced by a single stroke of a multiple shaft mixing tool is each considered an element. An element consisting of overlapping elements produced by a single stroke of a multiple-shaft mixing tool is sometimes referred to as a "panel".
3. Cement factor in place, cement dosage: Ratio of weight of dry cement to the volume of soil to be treated and the grout volume.
4. Grout: A stable mixture of water, Portland cement, and admixtures. The purpose of the grout is to assist in loosening the soils for penetration and optimum mixing, and upon setting, to strengthen the in situ soil.
5. Grout–soil ratio: A volumetric ratio of grout to in situ soil to be mixed.

6. Volume ratio: Ratio of the volume of slurry injected to the volume of soil mixing column.

7. Spoil Return: All materials including, but not limited to liquids, semi-solids, and solids that are discharged above the ground surface during, or as a result of the DSM process.

8. Obstruction: Man-made or man-placed objects or materials occurring at or below the ground surface which unavoidably stops the progress of work for more than one (1) hour despite the DSM Contractor's diligent efforts.

1.04 SUBMITTALS

A. Complete fabrication, assembly, and installation drawings, together with details and data governing materials used, and other accessories furnished, shall be submitted for acceptance in accordance with the procedures outlined in Section 1A, "General." Data submitted shall include, but not be limited to the following:

1. Qualifications Package.

2. Grout Mix: Proposed mix designs including all materials and quantities.

3. Cement: Mill certificates

4. Manufacture information for each admixture

5. Field Test Program.

6. Deep Mixing Work Plan.

7. Workplan for the Quality Control Program.

8. Sample Daily Quality Control Report.

9. Daily Quality Control Reports shall be submitted at the end of the next working day.

10. Calibrations: Submit all metering equipment calibration test results including mixing systems, delivery systems, alignment systems, and mixing tool rotational and vertical speed.

11. DSM Test Results: Submit all QC test results.

12. Record Drawings: Submit record drawings indicating the location of the DSM in terms of project coordinates.

1.05 QUALIFICATIONS OF THE DSM CONTRACTOR TO BE SUBMITTED WITH BID

A. General: The DSM Contractor shall submit a Qualifications Package that demonstrates DSM experience. One (1) Contractor, the DSM Contractor, shall perform all parts of the DSM installation. The DSM Contractor shall be experienced in all aspects of DSM design and construction, and shall furnish all necessary plant, materials, skilled labor, and supervision to complete the Contract. The DSM Contractor may be the Contractor bidding the job or a subcontractor.

B. Staff Experience: The DSM Contractor shall submit qualifications of the Project Superintendent, DSM Design Engineer, DSM Rig Operator(s), and DSM Mixing Plant Operator(s) to be utilized on the project. The Project Superintendent shall be authorized to act on behalf of the DSM Contractor. The Project Superintendent shall have at least five (5) years on-site experience managing DSM field operations of similar size and scope and shall have supervised at least two (2) projects within the past five (5) years employing the DSM technique proposed for this project. The Project Superintendent shall have experience and knowledge of all aspects of DSM as required for the project and shall be present at the worksite at all times during DSM operations. The DSM Design Engineer shall have at least five (5) years of experience in the design/QC of DSM systems. The DSM Design Engineer shall be a Civil and Structural or Geotechnical Engineer currently registered by the State of California. The DSM Design Engineer shall supervise review QC records and as-built drawings to confirm that the DSM work meets the design intent. The DSM Rig and Mixing Plant Operator(s) shall have at least three years of experience using the equipment selected for this project. DSM Contractor shall submit evidence of previous staff experience in the Qualifications Package Submittal. Personnel named in this package shall not be substituted without the express written consent of the Engineer.

C. Project Experience. The DSM Contractor shall submit evidence of experience and competence to design and construct the DSM. This evidence shall document that the DSM Contractor has at least five years of experience over the last ten years; and has completed at least five (5) projects of similar scope to this project. The DSM Contractor shall submit information on prior projects in the Qualifications Package Submittal to document their qualifications. The projects must have the following characteristics to qualify as acceptable projects. Failure of the Qualification Package to meet these requirements may result in the rejection of the DSM Contractor.

1. Satisfactorily completed at least five (5) school projects with DSA/CGS involvement for liquefaction mitigation using DSMs, during the last three years.

2. At least five (5) projects showing the independent and successful design and installation of structural DSM of similar or greater depth and length.

3. At least five (5) projects where the DSM Contractor implemented QA/QC programs during DSM treatment and used computerized data acquisition systems; and

4. An ongoing project may be used to satisfy the experience requirements provided the qualifying work has been completed and accepted by the owner.

5. Qualifications Package Submittal: The Qualifications Package shall include project and staff experience. For project experience, the DSM Contractor shall submit detailed information on previous projects in the format listed below. The architect may contact any of the listed references to verify the accuracy of the information. Failure to provide accurate and complete information may result in the invalidation of the listed project.

   a. Name of person in charge of the project for the Contractor.

   b. Name of the project.

   c. Location of the project.

d. Name of client/owner.

e. Name and telephone number of the person in charge of the project for the client. The contractor shall verify that all listed references and telephone numbers are current and complete.

f. A description of the project, including a detailed discussion of the work elements included in the construction.

D. For staff experience, the DSM Contractor shall submit the names and resumes of the Project Superintendent, DSM Design Engineer, DSM Rig Operator(s), and DSM Mixing Plant Operator(s) to be utilized on the project.

1.06 PERFORMANCE CRITERIA

A. Perform appropriate Ground Improvement beneath areas of structures listed above as stated under the scope of work to provide the following criteria upon successful completion of each.

1. Post Construction Liquefaction induced differential settlement shall be less than:

a. ≤ 2.40 inches over 40 feet for the Mechanical Building

b. ≤ 0.50 inches over 154 feet for the Swimming Pool

2. DSM should be constructed to a depth sufficient to satisfy the criteria above, as confirmed by the testing specified herein.

3. The minimum area replacement ratio is defined as: area of GI/ the tributary area for a GI shall be 30%.

1.07 BENCH-SCALE TEST PROGRAM

A. The DSM Contractor shall submit a field demonstration test program plan that contains descriptions of the construction procedures, equipment, and ancillary equipment to be used for mixing and grout proportioning and injection; mix design(s)/cement dosage(s) and associated soil strata to be evaluated; operational and material parameters to be monitored during the field demonstration test program; layout of the DSM test elements to be constructed; a summary of QC/QA samples to be collected and tested for the test program; and examples of the forms that will be used to document the work.

B. Laboratory testing shall be used to identify initial mix designs for the bench-scale test program. Bulk soil samples from the site shall be obtained by the contractor.

C. Based on the results of the lab bench scale test results, the DSM Contractor shall submit a deep mixing work plan for review and acceptance by the Engineer.

D. It is important to recognize the bench-scale testing result and field obtain strength data can vary. The type of drill the DSM contractor uses, the number of blades, and its rotation, penetration rate, and pump rate all play a key role in determining the strength result.

1.08 DEEP MIXING WORK PLAN

A. Based on the results of the field demonstration test program, the DSM Contractor shall submit a deep mixing work plan for review and acceptance by the Engineer. This plan shall include the following items:

1. Detailed descriptions of a sequence of construction, all construction procedures, equipments (catalog cut sheets), batching and storage equipment layout, ancillary equipment to be used to penetrate the ground, proportion, mix binders and inject and mix the site soils.

2. Proposed mix design(s), including cement, water, and admixtures, and their relative proportions, the required mixing time, water-to-cement ratio of the grout, cement factor in place, and volume ratio for a deep mixed element. The mix design shall be stamped andsigned by a Civil or Geotechnical Engineer who is currently registered by the State of California.

3. Proposed injection and mixing parameters, including mixing slurry rates, slurry pumping rates, air injection pressure, and volume flow rates, mixing tool rotational speeds, and penetration and withdrawal rates.

4. Methods for controlling and recording the verticality and the top and bottom elevation of each element.

5. Methods for monitoring the quality control parameters outlined in the quality control program and collecting samples for laboratory confirmation testing.

6. Methods for locating the DSM in the field and confirming that the DSM is plumb.

7. The anticipated cement dosages to achieve the acceptance criteria.

8. A proposed element numbering scheme.

9. Working drawings for the DSM elements showing the site location of the DSM project as well as the dimensions, layout, and locations of all DSM elements. Drawings shall indicate the identification number of every element if a multi-shaft mixing tool is used and every element if a single-auger mixing tool is used.

10. Sample Daily Quality Control Reports.

11. The GEOR or his representative shall perform the QC testing.

1.09 EXISTING UTILITIES

A. The General Contractor shall field locate and verify the locations of all utilities prior to starting work. The General Contractor shall notify the DSM Contractor, Engineer, and Owner's Project Manager of any utility locations that may be impacted and may require relocation.

PART 2 - PRODUCTS

2.01 MATERIALS

A. Grout: The material added to the blended in situ soils shall be a Portland cement grout. The purposes of the grout are to assist in loosening the soils for penetration and optimum mixing, and upon setting, to strengthen the in-situ soils. The grout shall be premixed in a mixing plant that combines dry materials and water in predetermined proportions. Ratios of the grout components shall be proposed by the DSM Contractor, confirmed during the field test program, and reviewed and accepted by the Architect. Once accepted, the grout composition shall not change unless requested in writing from the DSM Contractor and accepted in writing by the Engineer.

B. Cement: The cement used in preparing the grout shall conform to ASTM C 150 Type II/V PCC. The cement shall be adequately protected from moisture and contamination while in transit to and in storage at the job site. Reclaimed cement or cement containing lumps or deleterious matter shall not be used.

C. Water: Water used in mixing cement grout shall conform to ASTM C 1602.

D. Admixtures: Admixtures of softening agents, dispersions, retarders, or plugging or bridging agents may be added to the water or the grout to permit efficient use of materials and proper workability of the grout provided the DSM Contractor submits documentation demonstrating the effects of the admixture. Admixtures shall be accepted by the Engineer before use.

2.02 EQUIPMENT

A. Deep mixing equipment shall be of sufficient size, capacity, and torque to perform the required deep mixing to the desired depths. Characteristics of deep mixing equipment are as follows:

B. Drilling Equipment: The equipment shall be capable of advancing through previously installed elements to achieve designed overlapping or remixing as needed and be sufficient to maintain the necessary revolutions per minute and penetration rate at the maximum depth to achieve thorough mixing.

   1. The mixing and injection equipment shall be sufficient to adequately blend and distribute the binder with the in-situ soils to provide the required strength. The mixing shafts shall have mixing augers and blades (paddles) configured in such a manner so that they are capable of thoroughly blending the in-situ soils andgrout.

   2. The power source for driving the mixing shafts shall be sufficient to maintain the required revolutions per minute (RPM) and penetration rate from a stopped position at the maximum depth required.

C. Equipment Instrumentation: All equipment shall have monitoring equipment to permit accurate and continuous monitoring, recording, and controlling of mixing tool depth, location, binder volume flow rates, and density, binder injection pressures and quantities, tool rotational speeds, tool advancement, and withdrawal rates.

   1. The output from these sensors, the cement factor in place, and the number of mixing cuts per unit depth shall be visible in real-time to the operator and QC Inspector during

penetration and withdrawal.

2. The proposed display and monitoring systems shall be submitted and accepted by QC Inspector prior to use.

3. Calibration of this equipment shall be performed at the beginning of the project and the calibration data shall be submitted to QC Inspector. The calibration shall be repeated at intervals not to exceed two (2) months.

D. In the event that equipment instrumentation becomes partially or fully inoperable, the DSM Contractor shall repair the instrumentation system and bring it to a fully operational state. DSM construction is not allowed without instrumentation unless there is a safety hazard by not continuing the work. The DSM rig shall be equipped with electronic sensors built into the leads to determine vertical alignment in two (2) directions.

1. The sensors shall be calibrated at the beginning of the project and the calibration data shall be submitted to QC Inspector. The calibration shall be repeated at intervals not to exceed two (2) months.

2. The output from the sensors shall be routed to a console that is visible to the operator and QC Inspector during penetration. The console shall be capable of indicating the alignment angle in each plane.

E. Grout Mixing: Grout shall be premixed in a mixing plant, using a batch process, which combines dry materials and water in predetermined proportions. The mixing plant shall consist of a grout mixer, grout agitator, grout pump, and a computer control/measurement unit.

1. Dry materials shall be stored in silos. The dry materials shall be transported to the project site and blown into the on-site storage tanks using a pneumatic system.

2. The air evacuated from the storage tanks during the loading process shall be filtered before being discharged to the atmosphere.

3. Automatic batch scales or calibrated auger shall be used to accurately determine mix proportions for water and cement during grout preparation.

4. The dry admixtures, if used for mixing with water and cement, may be delivered to the mixing plant by a calibrated auger. However, the DSM Contractor shall demonstrate that the calibrated auger can deliver the quantity of dry admixture with accuracy equivalent to that measured and delivered by weight.

5. The grout density shall be measured by a mud balance or a mass flow meter before it is sent to the mixing rig.

6. Calibration of mixing components shall be done at the beginning of the project and repeated at intervals not to exceed two (2) months thereafter.

7. The cement shall be adequately protected from moisture and contamination while in transit to and in storage at the job site. Reclaimed cement or cement containing lumps or deleterious matter shall not be used.

8. Positive displacement pumps shall be used to transfer the grout from the mixing plant to the DSM equipment. The grout shall be delivered to each slurry–injection point by an individual positive displacement pump.

PART 3 - EXECUTION

3.01 FIELD TEST PROGRAM

A. Prior to production work, the DSM Contractor shall construct a test section to verify that the DSM Contractor's proposed equipment, procedures, and mix design can uniformly mix the onsite soils to the target depth(s) and achieve the product requirements outlined in the acceptance criteria. The field test program shall be used to optimize the various components of the DSM process, such as type of mixing equipment (e.g. single, double, or triple-axis), grout mix composition, rotational speed, penetration, and retraction rates, and confirm that results create soil-cement properties that meet required design criteria. The DSM Contractor shall construct more than one (1) test section if multiple mix designs/cement dosages are proposed.

B. The DSM Contractor can begin production work before test program results are available at its sole risk.

C. The DSM Contractor shall submit a plan drawing showing the location of the test section elements.

D. One (1) test section shall be constructed for each initial DSM mix design/cement dosage the DSM Contractor proposes to evaluate in the field demonstration test program and possibly use for the production DSM. A test section is defined as a continuous secant-type DSM section at least 15 linear feet long and of the depth and arrangement shown on the contract drawings. The cement dosage used for the accepted test section shall be required for use in the production DSM construction.

E. Equipment, procedures, accepted mix design, and element layout used on the test section shall be identical to those proposed for the production DSM construction.

F. The DSM Contractor shall perform full-depth core sampling and the related laboratory UCS testing for each test section in accordance with the Quality Control Program.

G. The DSM Contractor shall submit to the Engineer results of the test program and recommend grout mix, procedure, and equipment parameters based on those results. The DSM Contractor, at their expense, may be required to repeat construction of a test section if recommended parameters fall outside test requirements. The test program shall confirm that the resultant test section geometry and soil-cement properties meet the required design criteria before production work commences.

3.02 PRODUCTION DSM

A. The DSM Contractor shall proceed with construction of the production DSM after results of the field test program have been accepted by the Engineer. The DSM Contractor shall take all the risks, if he proceeds with the production prior to the approval of the field test program by the Engineer.

B. The DSM Contractor is responsible for the survey and site layout of each column to within 3 inches of the design coordinates. Due to the construction procedure of the Deep Soil Mixing operation, a daily re-layout of target locations is required. The layout can also be done by the DSM Contractor's QA/QC representative with oversight by a licensed professional engineer.

The General Contractor is responsible for grading a suitable working platform at the site capable of supporting the weight of the drill and other equipment and allowing movement from location to location without difficulty.

C. The DSM shall have essentially vertical elements and shall extend through the on-site soils to the elevations required by the contract drawings.

D. The completed DSM elements shall be a homogeneous mixture of grout and in-situ soils. Mixing is to be controlled by shaft rotational speed, drilling speed, and grout injection rate.

E. The DSM Contractor shall determine the average target DSM strength, thickness, and depth(s).

F. The overlap of elements and constant center-to-center spacing between adjacent elements shall conform to the contract drawings. A vertical alignment of 1.5 percent shall be maintained during the DSM installation.

G. Monitoring of construction parameters and confirmation testing will be used to verify that the acceptance criteria have been satisfied.

1. The DSM Contractor shall establish consistent procedures to be employed during DSM construction to ensure a relatively uniformly mixed product is created.

2. These procedures are to be defined in the deep mixing work plan and subsequently modified, if necessary, based on the results of the test section(s).

3. Prior to beginning production DSM installation, the DSM Contractor shall construct test section(s) in the area shown on the Contract Drawings, and results of the test section program shall be accepted by the Engineer.

4. The purpose of the test sections is to verify that the DSM Contractor's proposed equipment, procedures, and mix design can uniformly mix the on-site soils to the target depth(s) and achieve the required DSM strength(s).

5. Based on the evaluation of completed in–place test sections, the Engineer will determine if the test sections yield acceptable results and whether the DSM Contractor may proceed with the production DSM construction.

    a. The cement factor in place, equipment, installation procedures, and sampling and testing methods established during the test sections shall be used for the production DSM construction.

6. The DSM Contractor may request that the established cement factor in place, equipment, installation procedure, or test methods be modified. The Engineer may require additional testing or a new test section, at no additional cost to the owner, to verify that acceptable results can be achieved using the modification(s).

7. The DSM Contractor shall not employ cement factor in place, equipment, installation procedures, or sampling or testing methods unless accepted by the Engineer in writing.

H. The DSM Contractor shall conduct sampling and testing of the production DSM using the same methods employed during the test sections and in accordance with the requirements listed in the Quality Control Program.

1. For the production DSM construction, the following minimum frequency shall be instituted: Collect full–depth continuous core of the DSM for a minimum of 2% of the DSM locations.

2. Perform UCS tests on wet (grab) specimens in accordance with the requirements of the Quality Control Program.

3.03 MATERIAL ACCEPTANCE CRITERIA

A. The in-place grout mix together with the soils shall achieve:

1. A minimum unconfined compressive strength (UCS) at 28 days of 150 psiand shall be determined by ASTM D 1633 "Standard Test Method for Compressive Strength of Molded Soil-Cement Cylinders"

2. No more than ten (10) percent of the wet samples or ten (10) percent of the core samples tested for each building shall exhibit a UCS of less than 75 psi at 28 days.

3. A pattern of low-strength samples, such as at a constant depth will not be accepted.

4. Uniformity of soil-cement shall meet the requirement as outlined in Section 3.05.

3.04 GEOMETRIC ACCEPTANCE CRITERIA

A. The DSM Contractor shall accurately stake the location of the proposed DSM shown on the Contract Drawings before beginning installation.

B. The DSM shall be installed within the following geometric tolerances:

1. The horizontal alignment of the DSM shall be within three (3) inches of the planned location at the top of DSM.

2. The vertical inclination of the DSM shall be inclined no more than 1.5 percent from vertical.

3. The element overlaps and constant center-to-center spacing between adjacent elements shall conform to the contract plan and the vertical alignment of 1.5 percent shall be maintained during the DSM installation.

C. The DSM Contractor shall provide an adequate method to allow QC Inspector to verify the as-built location of the DSM during construction.

1. The DSM Contractor shall not be compensated for DSM sections that are located outside of the specified tolerances.

D. The equipment operator shall control the vertical alignment of the DSM element. Two (2) measures of the drill rig mast verticality shall be monitored, longitudinal and transverse to the

DSM alignment. The DSM elements shall be installed at an inclination deviating no more than 1.5 percent from vertical at any point.

E. DSM depths shall be determined by the DSM Contractor.

1. The equipment shall be adequately marked to allow QC Inspector to confirm the penetration depth during construction. The total depth of penetration shall be measured either by observing the length of the mixing shaft inserted below a reference point on the mast, or by subtraction of the exposed length of the shaft above the reference point from the total shaft length, or by electronic depth encoder.

2. The final depth of the stroke shall be noted and recorded on the Daily Quality Control Report by the DSM Contractor.

3. If rigs with varying mixing shaft lengths are used, the shortest shafts shall extend to the minimum District-accepted DSM depth(s) provided by the DSM Contractor.

3.05 UNIFORMITY OF MIXING ACCEPTANCE CRITERIA

A. Uniformity of mixing shall be evaluated by QC Inspector based on the full–depth core samples recovered by the DSM Contractor from the DSM.

1. To evaluate uniformity using core samples, all lengths of the unrecovered core shall be assumed to be unimproved soil.

2. Uniformity shall be determined by QC Inspector through inspection of core samples. A full-depth core is defined as a full–length continuous coring operation at a single location that extends from the top to the bottom of the DSM element.

3. Recovery shall be at least 85 percent for each full-depth core. In addition, continuous core recovery shall be at least 85 percent over any 5-foot core run. If 85% cannot be confirmed by coring in sandy or gravelly soils, the DSM Contractor at no extra cost may propose optical viewer logs to confirm uniformity.

4. Within a full-depth core, the sum length of unmixed or poorly mixed soil regions or lumps that extend entirely across the diameter of the core sample (2.5 inches) shall not exceed 10 percent of the entire recovered core length of a DSM element. In addition, lumps of unimproved soil shall not be more than 15 percent of the total volume of any 5-foot section interval of the full-depth core. Any individual or aggregation of lumps of unimproved soil shall not be larger than six (6) inches in the greatest dimension. If there are excessive mechanical damages to the recovered cores, the DSM Contractor is allowed to perform additional core(s) in the same or adjacent soil mixing column(s) at the DSM Contractor's expense.

B. If any section of the DSM is found not to satisfy the above criteria, the DSM Contractor shall mitigate (while injecting grout at the design grout ratio) the failed section of the DSM at no additional cost to the owner.

1. Unless otherwise determined by the Engineer, the extent of the failed section shall be considered to include all DSM sections constructed during all rig shifts that occurred between the times of construction when passing tests were achieved, i.e., within the

wet sampling interval. The DSM Contractor may conduct additional sampling and testing to better define the limits of the failed area at no additional cost to the owner.

2. The DSM Contractor shall submit a proposed remixing/repair plan of failed section(s) for review and acceptance by the Engineer.

3.06 OBSTRUCTIONS

A. The DSM Contractor shall be responsible to penetrate and mix some dense sand layers and stiff clay layers, which may need pre-drilling at no cost to the owner. If an obstruction is encountered that prevents pre-drilling advancement, the DSM Contractor shall immediately notify the Engineer and Owner's Project Manager and investigate the location and extent of the obstruction using methods accepted by the Engineer. The DSM Contractor shall propose remedial measures to clear the obstruction for acceptance by the Engineer and Owner's Project Manager. The DSM Contractor will be compensated for removal or clearing of obstructions with prior acceptance from the Engineer. If the element cannot be installed at the design location due to obstructions, the element shall be relocated as directed by the Engineer and Owner's Project Manager.

B. While the investigation for obstruction is underway, the DSM Contractor shall continue to install elements in areas away from the obstruction location. No stand–by delay will be allowed for equipment and operations during the investigation of obstruction.

C. The DSM Contractor shall be compensated for removal or clearing of unknown obstructions with prior acceptance by the Engineer.

3.07 GROUT PREPARATION

A. Dry binders shall be stored in silos and fed to mixers for agitation and shearing. In order to accurately control the mixing ratio of grout, the addition of water and cement shall be determined by weight using the automatic batch scales in the mixing plant, or the real-time grout-specific gravity measurement.

1. The admixtures, if used, for mixing with water and cement, can be delivered to the mixing plant by a calibrated auger. However, the DSM Contractor shall prove that the calibrated auger can deliver the quantity of dry admixture with accuracy equivalent to that measured and delivered by weight.

B. A minimum mixing time of three minutes and a maximum holding time of three (3) hours shall be enforced for the grout.

1. The specific gravity of the (grout) shall be determined during the design mix program. The specific gravity of the grout shall be checked by the DSM Contractor at least two times per shift per rig using the methods outlined in ASTM D 4380. If the grout is batched by the jet valve method, the specific gravity shall be measured in real-time during production. The specific gravity of the grout measured in the field shall not deviate by more than 3 percent of the calculated specific gravity for the design cement ratio.

2. If the grout density deviates by more than 3 percent, the DSM Contractor should recalibrate monitoring equipment and perform additional testing as required at no additional cost to the owner. The DSM Contractor may also adjust cement or water

quantities appropriately and retest at no additional cost to the owner.

3. The grout hold time shall be calculated from the beginning of the initial mixing.

4. The specific gravity measurements shall be indicated on the Daily Quality Control Report.

3.08 SOIL–GROUT MIXING

A. Installation of each element shall be continuous without interruption. If an interruption of more than one (1) hour occurs, the element shall be remixed (while injecting grout at the design grout ratio) for the entire height of the element at no additional cost to the owner.

B. The completed DSM shall be a uniform mixture of cement grout and the in situ soils.

1. Soil and grout shall be mixed together in place by specially designed blades on the mixing shafts.

2. The grout shall be pumped through the mixing shafts and injected from the bottom of the mixing tool. The mixing tool shall break up the soil and blend it with cement grout.

3. The mixing action of the tool shall blend, circulate, and knead the soil over the length of the element while mixing it in place with the grout.

4. Over any five (5)–foot section of an element, the lumps of unimproved soil shall not amount to more than fifteen (15) percent of the total volume of the DSM segment and any individual lump or aggregationof lumps of unimproved soil shall be no larger than six (6) inches in greatest dimension.

3.09 SHAFT ROTATIONAL SPEED AND PENETRATION/WITHDRAWAL RATE

A. The mixing shaft rotational speed (measured in RPMs) and penetration/withdrawal rates may be adjusted to achieve adequate mixing. The required rotational speeds and penetration/withdrawal rates for the various soil layers encountered shall be determined during the test sections.

B. The shaft rotational speed shall be adequate during penetration and withdrawal to achieve the design blade rotation number. The blade rotation number is defined as the number of the blade cut through one (1) meter of soil. The rotational speeds and penetration/withdrawal rates shall be recorded on the Daily Quality Control Report.

C. The cementing factor in place and the blade rotation number determined during the test section shall be used during the balance of the work. If these parameters are varied less than 85 percent from those determined during the test sections, the DSM section shall be remixed (while injecting grout at the design grout ratio) to a depth at least three (3) feet below the deficient zone at no additional cost to the owner.

D. The DSM Contractor may request that the established mixing parameters be modified during the production DSM installation. To verify acceptable results for the modified parameters, the Engineer may require additional testing or a new test section at no additional cost to the owner.

3.10 GROUT INJECTION RATE

A. The grout injection rate per vertical foot of the element shall be in accordance with the requirements of the design mix.

1. The required mix design and cement factor in place shall be determined during the test sections.

2. The grout injection rate shall be constantly monitored and controlled.

3. The DSM Contractor shall record the volume of grout injected for each three (3) vertical feet of each element on the Daily Quality Control Report.

B. If the volume of grout injected per vertical foot of element is less than eighty-five (85) percent of the amount required to meet the grout–soil ratio established during the test sections, the DSM shall be remixed and additional grout injected (at the design grout– soil ratio) to a depth at least three (3) feet below the deficient zone, at no additional cost to the owner.

C. The DSM Contractor may request that the established cement factor in place be modified during the production DSM installation.

1. To verify acceptable results for the modified cement factor in place, the Engineer may require additional testing or a new test section at no additional cost to the owner.

3.11 CONTROL OF SPOILS

A. The DSM Contractor shall control and process all spoils created during the DSM construction.

1. The areas designated by the owner shall be used for containment and processing of the spoils.

2. Positive means shall be provided for containing all spoil returns, flush water, and other waste materials within the work area.

3. All sedimentation and turbidity control measures required by applicable Federal, State, and local regulations shall be implemented. Precautions and measures shall be implemented to prevent any spoil returns or other waste material fromentering storm drain structures, drainage courses, or leaving the site via surface drainage. If spoil returns or other waste materials enter such areas, the DSM Contractor shall be responsible for immediately and completely cleaning and removing these materials to the acceptance of the owner and at no cost to the owner.

3.12 QUALITY CONTROL PROGRAM

A. General.

1. The DSM Quality Control (QC) Program shall be the responsibility ofthe DSM Contractor and shall include, as a minimum, the following components:

a. Construction of at least one (1) test section by the DSM Contractor;

b. Construction of additional test sections when the DSM Contractor proposes to evaluate multiple grout mix/cement dosages;

c. Field monitoring by the DSM Contractor of construction parameters during DSM construction;

d. Sample collection including full depth continuous coring, and wet sampling, along with testing performed by the DSM Contractor;

e. Reporting of the field monitoring, sampling, and any strength testing performed by the DSM Contractor.

2. The DSM Contractor shall provide all the personnel and equipment necessary to implement the Quality Control Program.

a. Prior to site mobilization, the DSM Contractor shall submit a detailed work plan for the Quality Control Program for review and acceptance by the Engineer.

b. The work plan shall include, as a minimum, a description of all procedures to be implemented, parameters to be monitored, tolerances for the parameters monitored, and the names of any subcontractors used for testing.

3. Following the test sections, the DSM Contractor may revise the Quality Control Program.

a. The established quality control procedures shall be maintained throughout the production DSM installation to ensure consistency in the DSM installation and to verify that the work complies with all requirements indicated in the Contract Plans and Specifications.

B. Sample Collection and Strength Testing

1. The acceptance of the work shall be based on demonstrating that the in-place grout mix together with the soils has achieved the strength and uniformity requirements.

2. Confirmation that the strength and uniformity requirements have been satisfied will be determined by a series of tests performed on samples collected by the DSM Contractor. Confirmation sample collection and testing shall include:

a. Full-depth continuous coring and testing: Full–depth continuous coring performed by the DSM Contractor and QA laboratory UCS testing conducted on the core samples by the GEOR laboratory.

b. Wet (grab) soil mix samples: Wet samples that are retrieved and cast into molds by the DSM Contractor and QC laboratory UCS testing by the GEOR laboratory.

c. Additional confirmation testing: The DSM Contractor, at its own expense, may perform borehole imaging.

3. Full–Depth Coring, Sampling and Testing: Continuous coring shall be performed for the full depth of the DSM by the DSM Contractor.

a. Full–depth samples obtained by the DSM Contractor shall have a diameter of at least 2.5 inches. The full–depth samples shall be obtained along an essentially vertical alignment located one–fourth of an element diameter from the element center.

b. Full–depth samples shall be retrieved using standard continuous coring techniques after the soil–grout mixture has hardened sufficiently.

c. For the continuous coring method, each core run shall be at least five (5) feet in length and contain at least five (5) test specimens with a length to diameter ratio of 2, or greater.

   1) A minimum recovery of 85 percent for each five (5)–foot–long core run shall be achieved. During coring, the elevation of the bottom of the holes shall be measured after each core run in the order that the core recovery for each run can be calculated. If 85% cannot be confirmed by coring in sandy or gravelly soils, the DSM Contractor at no extra cost may propose optical viewer logs to confirm uniformity.

   2) The DSM Contractor shall determine the time interval between element installation and coring except that the interval shall be no longer than required to conduct twenty-eight (28)–day strength testing.

d. Upon retrieval, the full–depth samples shall be logged and test specimen selection.

   1) Field logging will be performed to determine if the uniformity and recovery criteria have been satisfied.

   2) Following logging, select four (4) to ten (10) specimens from each full–depth sample recovered for QA UCS strength testing.

   3) Following logging and test specimen selection the entire full–depth sample, including the designated test specimens, shall be immediately sealed in plastic wrap to prevent drying. The designated test core specimens for QA testing will be transported to the GEOR laboratory.

   4) All core holes shall be filled with cement grout that will obtain a twenty-eight (28)–day strength equal to or greater than the strength of the DSM.

e. QA strength testing shall be conducted on core samples.

   1) The core samples shall be stored in a moist room in accordance with ASTM C 192 until the test date.

   2) UCS tests shall be conducted on core samples at the design target cure age in accordance with ASTM D 1633.

   3) The remaining portions of the full–depth samples that are not tested shall be retained by the DSM Contractor, until completion and acceptance of all DSM sections, for possible inspection and confirmation testing.

4. Wet Sample Collection and Testing: Wet (grab) samples shall be retrieved and cast into molds by the DSM Contractor from a minimum of one column per work shift per rig, at one random depth.

    a. Samples shall be retrieved using an in-situ wet sampler immediately after element construction and shall consist of no fewer than six (6) specimens per sampling event. The specimens shall be in (3)-inch by six (6)-inch cylindrical molded.

    b. UCS shall be conducted on wet specimens in pairs at selected ages in accordance with ASTM D 1633, including the design target cure age. Results of wet specimens tested before the design target cure age may be used to provide an early indication of DSM strength and the trend of strength increase with curing time, and to evaluate whether the work of the DSM Contractor can achieve the average target UCS criteria.

5. Daily Quality Control Report.

    a. The DSM Contractor shall submit Daily Quality Control Reports to the Engineer at the end of the next working day in an electronic file or by hard copy. The Daily Quality Control Report shall document the progress of the DSM construction, present the results of the QC parameter monitoring, and clearly indicate if the elements have met the acceptance criteria.

    b. The Daily Quality Control Report shall include as a minimum the results of the following QC parameter monitoring for each element:

        1) Rig number.

        2) Type of mixing tool.

        3) Date and time (start and finish) of element construction.

        4) Element number and reference drawing number.

        5) Element diameter.

        6) Element top and bottom elevations.

        7) Grout mix design designation.

        8) Slurry-specific gravity measurements.

        9) Description of obstructions, interruptions, or other difficulties during installation and how they were resolved.

    c. The Daily Quality Control Reports shall also include the following parameters recorded automatically or manually for each element at intervals no greater than four (4) feet and submitted in the form of either table or figures:

        1) Elevation in feet vs. real-time.

2) Shaft rotation speed in RPMs vs. real-time.

3) Penetration and withdrawal rates in feet per minute vs. real-time.

4) Grout injection rate in GPM vs. real-time.

5) Grout specific gravity vs. time.

6) The cement factor in place vs. depth.

7) The blade rotation number vs. depth.

3.13 AS-BUILT DRAWINGS

A. Following DSM construction, the DSM Contractor shall submit as-built drawings of the DSM in terms of project coordinates.

Designation: D 1632 – 96

Standard Practice for Making and Curing Soil-Cement Compression and Flexure Test Specimens in the Laboratory[1]

This standard is issued under the fixed designation D 1632; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ϵ) indicates an editorial change since the last revision or reapproval.

1. Scope

1.1 This practice covers the procedure for making and curing compression and flexure test specimens of soil-cement in the laboratory under accurate control of quantities of materials and test conditions.

1.2 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

C 127 Test Method for Specific Gravity and Absorption of Coarse Aggregate[2]

D 558 Test Methods for Moisture-Density Relations of Soil-Cement Mixtures[3]

D 559 Test Methods for Wetting-and-Drying Tests of Compacted Soil-Cement Mixtures[3]

D 560 Test Methods for Freezing-and-Thawing Tests of Compacted Soil-Cement Mixtures[3]

D 1633 Test Method for Compressive Strength of Molded Soil-Cement Cylinders[3]

D 1634 Test Method for Compressive Strength of Soil-Cement Using Portions of Beams Broken in Flexure (Modified Cube Method)[3]

D 1635 Test Method for Flexural Strength of Soil-Cement Using Simple Beam with Third-Point Loading[3]

D 4753 Specification for Evaluating, Selecting, and Specifying Balances and Scales for Use in Testing Soil, Rock, and Related Construction Materials[3]

E 11 Specification for Wire-Cloth Sieves for Testing Purposes[4]

3. Significance and Use

3.1 This practice is used to prepare soil-cement specimens for compressive and flexural strength testing in accordance with Method B of Test Method D 1633, Test Method D 1634, and Test Method D 1635.

3.2 This practice does not apply to soil-cement specimens prepared in commonly available molds, which are 4.0 in. (101.6 mm) in diameter and 4.584 in. (116.4 mm) in height. For these size specimens, Methods D 559 or Methods D 560 should be used for sample preparation. Compressive strength testing should be in accordance with Method A of Test Method D 1633.

4. Apparatus

4.1 *Compression Test Specimen Molds*—Molds (Fig. 1) having an inside diameter of 2.8 ± 0.01 in. (71 ± 0.25 mm) and a height of 9 in. (229 mm) for molding test specimens 2.8 in. (71 mm) in diameter and 5.6 in. (142 mm) high; machined steel top and bottom pistons having a diameter 0.005 in. (0.13 mm) less than the mold; a 6-in. (152-mm) long mold extension; and a spacer clip. At least two aluminum separating disks 1⁄16 in. (1.54 mm) thick by 2.78 in. (70.6 mm) in diameter shall be provided.

NOTE 1—Satisfactory molds may be made from cold-drawn, seamless steel tubing having a Rockwell hardness of approximately 85 HRB or from steel pipe machined on the inside. The 2.8 by 5.6-in. (71 by 142-mm) specimens fit many triaxial compression machines in service, and thus may be used for triaxial as well as unconfined compression tests.

4.2 *Flexure Test Specimen Molds*—Molds having inside dimensions of 3 by 3 by 11¼ in. (76.2 by 76.2 by 285.8 mm) (see Fig. 2 and Fig. 3) for molding specimens of the same size. The molds shall be so designed that the specimen will be molded with its longitudinal axis in a horizontal position. The parts of the molds shall be tight-fitting and positively held together. The sides of the molds shall be sufficiently rigid to prevent spreading or warping. The interior faces of the molds shall be plane surfaces with a permissible variation, in any 3-in. (76.2-mm) line on a surface, of 0.002 in. (0.051 mm) for new molds and 0.003 in. (0.076 mm) for molds in use. The distance between opposite sides shall be 3 ± 0.01 in. (76.20 ± 0.25 mm) for new molds, and 3 ± 0.015 in. (76.20 ± 0.38 mm) for molds in use. The height of the molds shall be 3 in. (76.20 mm)

[1] This practice is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.15 on Stabilization by Admixtures.

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[2] *Annual Book of ASTM Standards*, Vol 04.02.

[3] *Annual Book of ASTM Standards*, Vol 04.08.

[4] *Annual Book of ASTM Standards*, Vol 14.02.

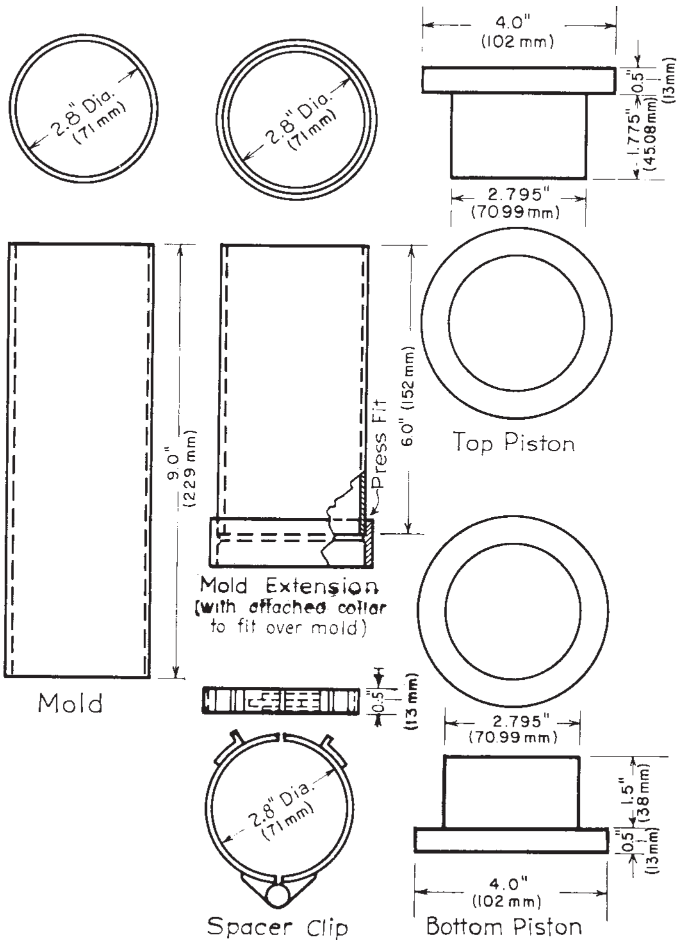

FIG. 1 Soil-Cement Cylinder Mold

with permissible variations of − 0.01 in. (−0.25 mm) and + 0.005 in. (+ 0.13 mm) for both new molds and for molds in use. Four ⅜-in. (9.52-mm) spacer bars and top and bottom machined steel plates shall be provided. The plates shall fit the mold with a 0.005-in. (0.13-mm) clearance on all sides.

NOTE 2—The molds shall be made of metal having a hardness not less than 85 HRB.

4.3 *Sieves*—2-in. (50-mm), ¾-in. (19.0-mm), No. 4 (4.75-mm) and No. 16 (1.18-mm) sieves conforming to the requirements of Specification E 11.

4.4 *Balances*—A balance or scale of 25-lb (12-kg) capacity, sensitive to 0.01 lb (0.0045 kg) and a balance of 1000-g capacity, sensitive to 0.1 g, both meeting the requirements of Specification D 4753.

4.5 *Drying Oven*—A thermostatically controlled drying oven capable of maintaining a temperature of 230 ± 9°F (110± 5°C) for drying moisture samples.

4.6 *Compression Testing Machine or Compression Frame*, having a capacity of approximately 60 000 lbf (267 kN) for compacting flexural test specimens and for optional use in compacting compression test specimens.

4.7 *Dropping-Weight Compacting Machine*—A controlled dropping-weight device of 15 lb (6.8 kg) for striking the top piston, for optional use in compacting compression test specimens (see Fig. 4 and Fig. 5). When this equipment is used, the top piston listed in 4.1 is made the foot of the compacting device.

4.8 *Compression Specimen Extruder*, consisting of a piston, jack, and frame for extruding specimens from the mold.

4.9 *Miscellaneous Equipment*—Tools such as trowel, spatula, pan, and the like, or a suitable mechanical device for thoroughly mixing the sample of soil-cement with water; graduate for measuring water, moisture sample cans, and the like.

4.10 *Tamping Rod*—A square-end cut, ½-in. (12.7-mm) diameter, smooth steel rod approximately 20 in. (510 mm) in length.

4.11 *Moist Room or Cabinet*—A moist room or cabinet capable of maintaining a temperature of 73.4 ± 3°F (23.0± 1.7°C) and a relative humidity of not less than 96 % for moist curing specimens.

5. Preparation of Materials

5.1 Bring materials to room temperature (preferably 65 to 75°F (18 to 24°C)) before beginning the tests.

5.2 Store cement in a dry place, in moisture-proof containers, preferably made of metal. Thoroughly mix the cement in order that the sample may be uniform throughout the tests. Pass it through a No. 16 (1.18-mm) sieve and reject all lumps.

5.3 The mixing water shall be free of acids, alkalies, and oils, and in general suitable for drinking.

5.4 Dry the soil sample, if damp when received from the field, until it becomes friable under a trowel. Drying may be in air or by use of drying apparatus such that the temperature of the sample does not exceed 140°F (60°C). Thoroughly break up the aggregations in such a manner as to avoid reducing the natural size of individual particles.

5.5 Sieve an adequate quantity of representative pulverized soil on the 2-in. (50-mm), ¾-in. (19.0-mm), and No. 4 (4.75-mm) sieves. Discard any aggregate retained on the 2-in. (50-mm) sieve. Remove aggregate passing the 2-in. (50-mm) sieve and retained on the ¾-in. (19.0-mm) sieve, and replace it with an equal mass of aggregate passing the ¾-in. (19.0-mm) sieve and retained on the No. 4 (4.75-mm) sieve. Obtain aggregate for replacement from the original sample.

NOTE 3—This practice for making soil-cement specimens for compression and flexure tests is used primarily with soil materials having not more than 35 % aggregate retained on the No. 4 (4.75-mm) sieve and not more than 85 % retained on the No. 40 (425-µm) sieve.

5.6 Soak the aggregate passing the ¾-in. sieve and retained on the No. 4 sieve in water for 24 h, remove, and surface dry. Determine the absorption properties in accordance with Test Method C 127.

5.7 Take a 100-g sample of the soil passing the No. 4 sieve and dry it in the drying oven to constant mass, and determine the water content of the sample to permit calculation of the quantity of water that shall be added to the soil-cement mixture to bring it to the proper water content for molding specimens.

5.8 Take a representative sample of sufficient size to make one flexure test specimen or three compression test specimens of the soil passing the No. 4 (4.75-mm) sieve and also of the fractions passing the ¾-in. (19.0-mm) sieve and retained on the No. 4 (4.75 mm) sieve, prepared as described in 5.4, 5.5, and 5.6.

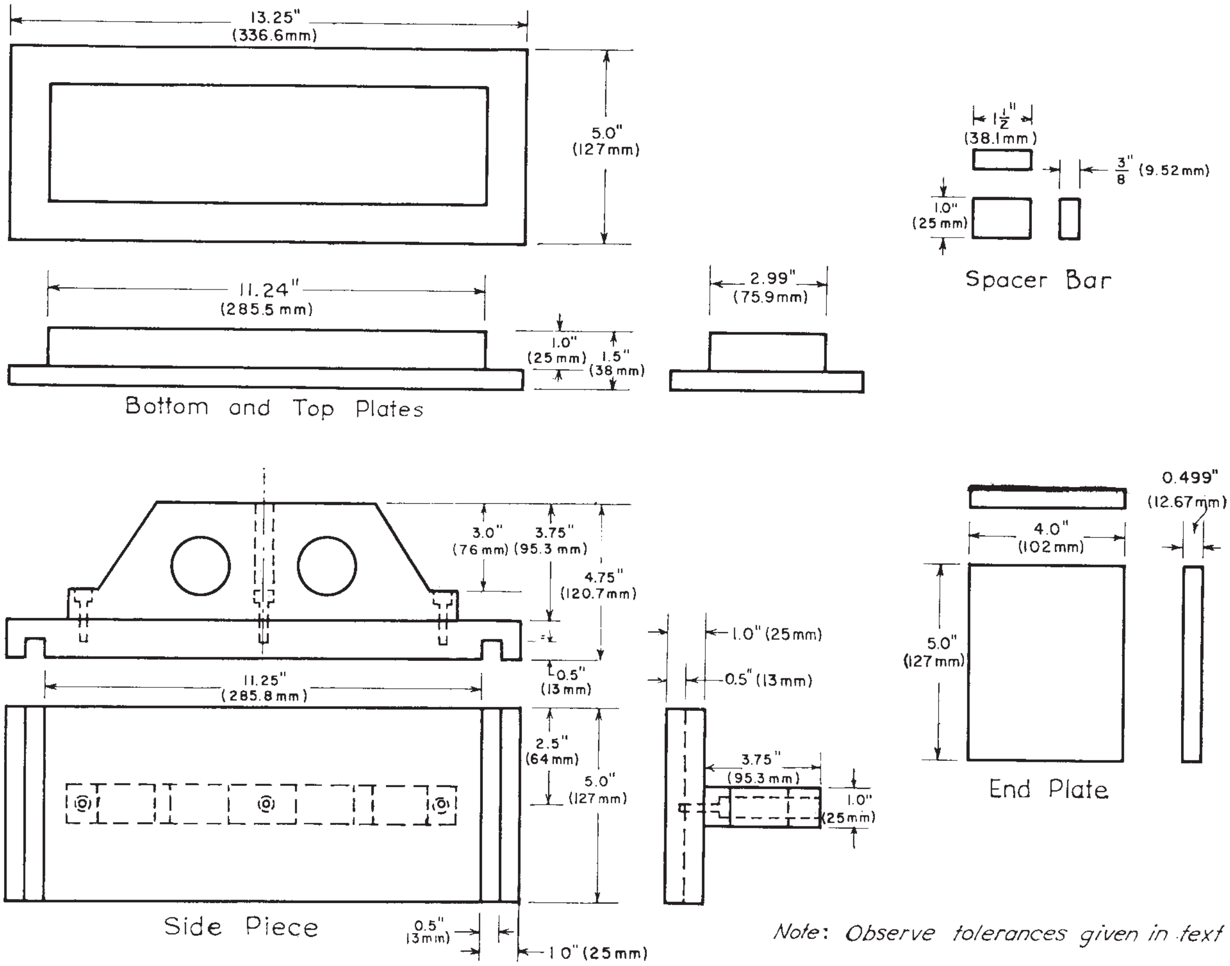

FIG. 2 Mold for Soil-Cement Beam for Flexure Test

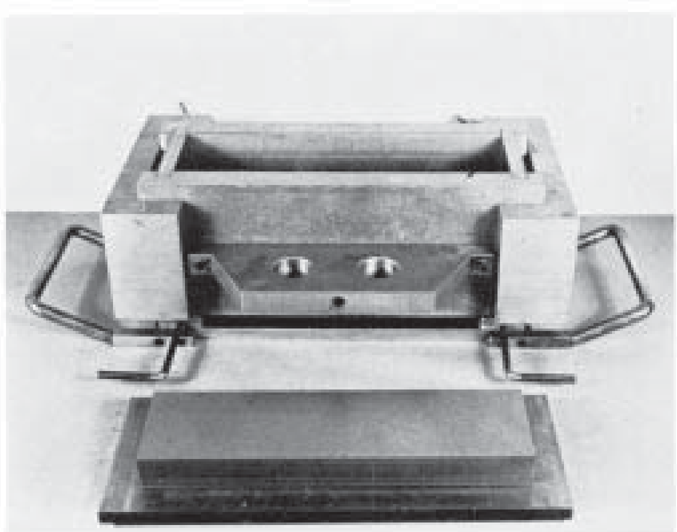

FIG. 3 Heavy Steel Mold and Top Plate for Making 3 by 3 by 11¼-in. (76.2 by 76.2 by 285.8-mm) Flexure Test Beam

6. Determining the Mass of Materials

6.1 Determine the mass to the nearest 0.01 lb (5 g) the designed quantities of soil passing the No. 4 (4.75-mm) sieve and aggregate passing the ¾-in. (19.0-mm) sieve and retained on the No. 4 sieve. Determine the mass to the nearest 1 g of the designed quantity of cement and measure the designed quantity of water to the nearest 1 mL.

NOTE 4—The designed quantities of soil, cement, and water are usually based on results obtained from ASTM tests. The "optimum" water content of the mixture and the "maximum" unit weight to which the specimens are compacted are determined by Test Methods D 558. The quantity of cement is usually sufficient to produce soil-cement of a quality suitable for road and runway base construction. This cement quantity is indicated by criteria established for interpreting the results obtained from Methods D 559 and Methods D 560.

7. Mixing Materials

7.1 *General*—Mix soil-cement either by hand or in a suitable laboratory mixer in batches of such size as to leave about 10% excess after molding test specimens. Protect this material against loss of water, determine the mass of a representative part of it and dry it in the drying oven to constant mass to determine the actual water content of the soil-cement mixture. When the soil-cement mixture contains aggregate retained on the No. 4 (4.75-mm) sieve, the sample for water content determination shall have a mass of at least 500 g and its mass shall be determined to the nearest gram. If the mixture does not contain aggregate retained on the No. 4 sieve, the sample shall have a mass of at least 100 g and its mass shall be

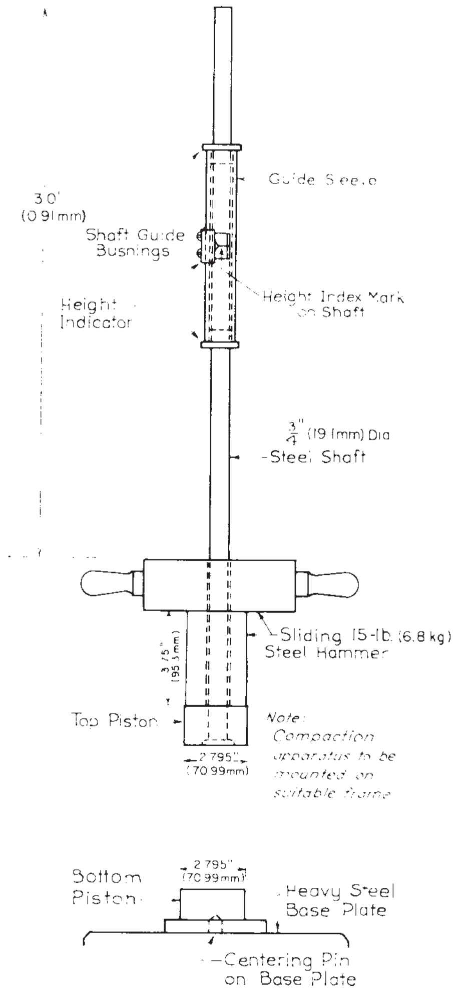

FIG. 4 Schematic Drawing of a Suitable Dropping-Weight Compacting Device

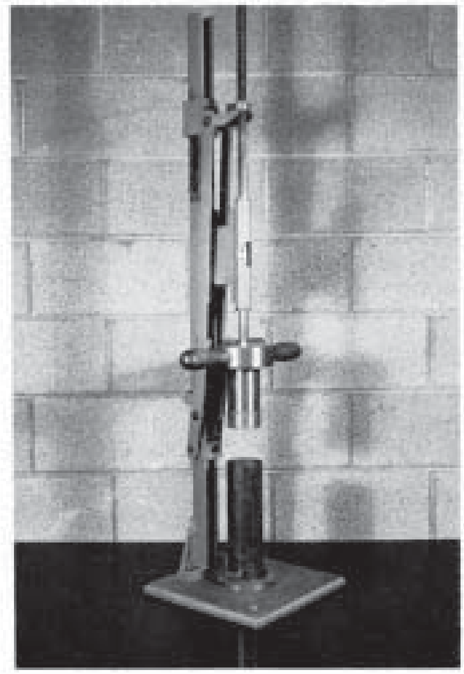

FIG. 5 Compacting Device Suitable for Making 2.8 by 5.6-in. (71 by 142 mm) Compression Test Cylinder

determined to the nearest 0.1 g.

7.2 *Hand Mixing*—Mix the batch in a clean, damp, metal pan or on top of a steel table, with a blunt bricklayer's trowel, using the following procedure:

7.2.1 Mix the cement and minus No. 4 (4.75-mm) soil until they are thoroughly blended.

7.2.2 Add water and mix the mass until it is thoroughly blended.

7.2.3 Add the saturated surface-dry coarse aggregate and mix the entire batch until the coarse aggregate is uniformly distributed throughout the batch.

7.3 *Machine Mixing*—Follow the sequence specified for hand mixing. To eliminate segregation, deposit machine-mixed soil-cement in a clean, damp, metal pan and remix with the trowel.

NOTE 5—The operation of mixing and compacting compression and flexure test specimens shall be continuous and the elapsed time between the addition of water and final compaction shall not exceed 30 min.

COMPRESSION TEST SPECIMENS

8. Size of Specimens

8.1 Compression test specimens shall be cylinders with a length equal to twice the diameter. This method provides for specimens 2.8 in. (71 mm) in diameter by 5.6 in. (142 mm) in length, but the same procedure may be used for molding larger or smaller specimens.

9. Molding Specimens

9.1 Lightly coat the mold and the two separating disks with commercial form oil. Hold the cylinder mold in place with the spacer clip over the bottom piston so that the latter extends about 1 in. (25 mm) into the cylinder.

9.2 Place a separating disk on top of the bottom piston and place the extension sleeve on top of the mold. Place in the mold a predetermined mass of the uniformly mixed soil-cement to provide a specimen of the designed unit weight when 5.6 in. (142 mm) high. When the soil-cement contains aggregate retained on the No. 4 (4.75-mm) sieve, carefully spade the mix around the mold sides with a thin spatula. Then compact the soil-cement initially from the bottom up by steadily and firmly forcing (with little impact) a square-end cut ½-in. (12.7-mm) diameter smooth steel rod repeatedly through the mixture from the top down to the point of refusal, distributing the roddings uniformly over the cross-section of the mold. Perform the operation carefully so as not to leave holes in clayey soil-cement mixtures. Repeat the process until the mass is packed out to a height of approximately 6 in. (150 mm).

9.3 Remove the extension sleeve and place a separating disk on the surface of the soil-cement. Remove the spacer clip supporting the mold on the bottom piston. Put the top piston in place and apply either a static load by the compression machine or a dynamic load by the compacting device until the specimen is 5.6 in. (142 mm) high.

9.4 Remove the pistons and separating disks from the mold assembly, but leave the specimen in the mold.

10. Curing Specimens

10.1 Cure the specimens in the molds in the moist room for 12 h, or longer if required, to permit subsequent removal from the molds using the sample extruder. Return the specimens to the moist room, but protect from dripping water for the specified moist curing period. Generally the specimens will be tested in the moist condition directly after removal from the moist room.

NOTE 6—Other conditioning procedures, such as soaking in water, air drying or oven drying, alternate wetting and drying, or alternate freezing and thawing, may be specified after an initial moist curing period. Curing and conditioning procedures shall be given in detail in the report.

11. Capping Specimens

11.1 Before testing, cap the ends of all compression specimens that are not plane within 0.002 in. (0.05 mm). Capped surfaces shall meet this same tolerance and shall be at right angles to the axis of the specimen.

11.2 Cap the specimens with gypsum plaster. The caps shall be as thin as practical and shall be aged sufficiently so that they will not flow or fracture when the specimen is tested (suggested time 3 h at 73°F (23°C)). During this period maintain the specimens at constant water content.

FLEXURE TEST SPECIMENS

12. Size of Specimen

12.1 Flexure test specimens shall be rectangular beams with a length as tested at least 2 in. (51 mm) greater than three times the depth. This procedure provides for beams 3 by 3 by 11¼ in. (76.2 by 76.2 by 285.8 mm), but the same procedures may be used for molding smaller or larger specimens.

13. Molding Specimens

13.1 Form the test specimens with the longitudinal axis horizontal. Lightly oil the mold parts and assemble with the sides and ends separated from the base plate by the ⅜-in. (9.53-mm) spacer bars, one placed at each corner of the mold.

13.2 Divide into three equal batches a predetermined mass of uniformly mixed soil-cement to make a beam of the designed unit weight. Place one batch of the material in the mold and level by hand. When the soil-cement contains aggregate retained on the No. 4 (4.75-mm) sieve, carefully spade the mix around the sides of the mold with a thin spatula. Compact the soil-cement initially from the bottom up by steadily and firmly, forcing (with little impact) a square-end cut ½-in. (12.7-mm) diameter smooth steel rod repeatedly through the mixture from the top down to the point of refusal. Approximately 90 roddings distributed uniformly over the cross section of the mold are required; take care so as not to leave holes in clayey soil-cement mixtures. Level this layer of compacted soil-cement by hand and place and compact layers two and three in an identical manner. The specimen at this time shall be approximately 3¾ in. (95 mm) high.

13.3 Place the top plate of the mold in position and remove the spacer bars. Obtain final compaction with a static load applied by the compression machine or compression frame until the designed height of 3.0 in. (76 mm) is reached.

13.4 Immediately after compaction, carefully dismantle the mold and remove the specimen onto a smooth, rigid, wood or sheet metal pallet.

NOTE 7—A suggested method for removing the specimen from the mold is to remove first the top and then the sides and end plates of the mold. The specimen is then resting on the bottom plate of the mold. The flat face of a carrying pallet is then placed against one side of the specimen and then the bottom mold plate, the specimen, and the pallet are rotated 90° so that the specimen rests on its side on the pallet. The bottom mold plate is then carefully removed.

14. Curing Specimens

14.1 Cure the specimens on pallets in the moist room and protect from free water for the specified moist curing period. Generally the specimen will be tested in the moist condition directly after removal from the moist room (see Note 6).

15. Capping Specimens

15.1 Before testing, cap areas, on opposite *sides* of the specimens as molded, that will come in contact with the load-applying block and supports and that are not plane within 0.002 in. (0.05 mm). Capped surfaces shall meet this same tolerance and shall be parallel to the horizontal axis of the specimen.

NOTE 8—Specimens are tested on their sides, with the original top and bottom surfaces as molded perpendicular to the testing machine bed. Specimens made in molds meeting the specifications in 3.2 generally will not require capping.

15.2 If capping is necessary, cap specimens with gypsum plaster. The caps shall be as thin as practical and shall be aged sufficiently so that they will not flow or fracture when the specimen is tested (suggested time 3 h at 73°F (23°C)). During this period maintain the specimens at constant water content.

REPORT

16. Report

16.1 The report shall include the following:

16.1.1 Gradation of soil as received and as used in making specimens,

16.1.2 Specimen identification number,

16.1.3 Designed water content,

16.1.4 Designed oven-dry unit weight,

16.1.5 Designed cement content,

16.1.6 Actual water content,

16.1.7 Actual oven-dry unit weight,

16.1.8 Actual cement content, and

16.1.9 Details of curing and conditioning periods.

17. Precision and Bias

17.1 This practice describes procedures for making and curing test specimens. Since there are no test values determined, a statement on precision and bias of the method is not applicable.

18. Keywords

18.1 flexural strength; soil-cement; soil stabilization; unconfined compressive strength

Standard Test Methods for Compressive Strength of Molded Soil-Cement Cylinders[1]

This standard is issued under the fixed designation D 1633; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

This standard has been approved for use by agencies of the Department of Defense.

1. Scope *

1.1 This test method covers the determination of the compressive strength of soil-cement using molded cylinders as test specimens.

1.2 Two alternative procedures are provided as follows:

1.2.1 *Method A*—This procedure uses a test specimen 4.0 in. (101.6 mm) in diameter and 4.584 in. (116.4 mm) in height. Height to diameter ratio equals 1.15. This test method made be used only on materials with 30 % or less retained on the ¾-in. (19.0-mm) sieve. See Note 3.

1.2.2 *Method B*—This procedure uses a test specimen 2.8 in. (71.1 mm) in diameter and 5.6 in. (142.2 mm) in height. Height to diameter ratio equals 2.00. This test method is applicable to those materials that pass the No. 4 (4.75-mm) sieve.

1.3 All observed and calculated values shall conform to the guidelines for significant digits and rounding established in Practice D 6026.

1.4 The values stated in inch-pound units are to be regarded as standard, except as noted in 1.4.1-1.4.3. The values given in parentheses are mathematical conversions to SI units, and are provided for information only and are not considered standard.

1.4.1 The gravitational system of inch-pound units is used when dealing with inch-pound units. In this system, the pound (lbf) represents a unit of force (weight), while the unit for mass is slugs.

1.4.2 The slug unit of mass is almost never used in commercial practice (density, scales, balances, etc.). Therefore, the standard unit for mass in this standard is either kilogram (kg) or gram (g), or both. Also, the equivalent inch-pound unit (slug) is not given.

1.4.3 It is common practice in the engineering/construction profession in the United States to use concurrently pounds to represent both a unit of mass (lbm) and of force (lbf). This use combines two separate system of units, the absolute system and the gravitational system. It is scientifically undesirable to combine the use of two separate sets of inch-pound units within a single standard. As stated in 1.4.2, this standard uses the gravitational system and does not present the slug unit for mass. However, the use of scales or balances recording pounds of mass (lbm) or the recording of density in lbm/ft^3 shall not be regarded as nonconformance with this standard.

1.5 *This standard does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

C 42 Test Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete[2]

D 559 Test Methods for Wetting-and-Drying Tests of Compacted Soil-Cement Mixtures[3]

D 560 Test Methods for Freezing-and-Thawing Tests of Compacted Soil-Cement Mixtures[3]

D 653 Terminology Relating to Soil, Rock, and Contained Fluids[3]

D 1632 Practice for Making and Curing Soil-Cement Compression and Flexure Test Specimens in the Laboratory[3]

D 2216 Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass[3]

D 3740 Practice for the Minimum Requirements for Agencies Engaged in the Testing and/or Inspection of Soil and Rock Used in Engineering Design and Construction[3]

D 4753 Specification for Evaluating, Selecting, and Specifying Balances and Scales for Use in Soil, Rock, and Construction Material Testing[3]

D 6026 Practice for Using Significant Digits in Calculating and Reporting Geotechnical Test Data[4]

E 4 Practices for Load Verification of Testing Machines[5]

3. Terminology

3.1 For definitions of terms used in this test method, refer to Terminology D 653.

4. Significance and Use

4.1 Method A makes use of the same compaction equipment

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[1] This test method is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.15 on Stabilization with Admixtures.

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[2] *Annual Book of ASTM Standards,* Vol 04.02.
[3] *Annual Book of ASTM Standards,* Vol 04.08.
[4] *Annual Book of ASTM Standards,* Vol 04.09.
[5] *Annual Book of ASTM Standards,* Vol 03.01.

***A Summary of Changes section appears at the end of this standard.**

and molds commonly available in soil laboratories and used for other soil-cement tests. It is considered that Method A gives a relative measure of strength rather than a rigorous determination of compressive strength. Because of the lesser height to diameter ratio (1.15) of the cylinders, the compressive strength determined by Method A will normally be greater than that for Method B.

4.2 Method B, because of the greater height to diameter ratio (2.00), gives a better measure of compressive strength from a technical viewpoint since it reduces complex stress conditions that may occur during the shearing of Method A specimens.

4.3 In practice, Method A has been more commonly used than Method B. As a result, it has been customary to evaluate or specify compressive strength values as determined by Method A. A factor for converting compressive strength values based on height to diameter ratio is given in Section 8.[6]

NOTE 1—The agency performing this test method can be evaluated in accordance with Practice D 3740. Not withstanding statements on precision and bias contained in this test method: the precision of this test method is dependent on the competence of the personnel performing it and the suitability of the equipment and facilities used. Agencies that meet the criteria of Practice D 3740 are generally considered capable of competent and objective testing. Users of this test method are cautioned that compliance with Practice D 3740 does not, in itself, ensure reliable testing. Reliable testing depends on many factors; Practice D 3740 provides a means of evaluating some of these factors.

5. Apparatus

5.1 *Compression Testing Machine*—This machine may be of any type having sufficient capacity and control to provide the rate of loading prescribed in 7.2. It shall conform to the requirements of Section 15 of Practices E 4. The testing machine shall be equipped with two steel bearing blocks with hardened faces (Note 2), one of which is a spherically seated head block that normally will bear on the upper surface of the specimen, and the other a plain rigid block on which the specimen will rest. The bearing faces shall be at least as large, and preferably slightly larger, than the surface of the specimen to which the load is applied. The bearing faces, when new, shall not depart from a plane by more than 0.0005 in. (0.013 mm) at any point, and they shall be maintained within a permissible variation limit of 0.001 in. (0.02 mm). In the spherically seated block, the diameter of the sphere shall not greatly exceed the diameter of the specimen and the center of the sphere shall coincide with the center of the bearing face. The movable portion of this block shall be held closely in the spherical seat, but the design shall be such that the bearing face can be rotated freely and tilted through small angles in any direction.

NOTE 2—It is desirable that the bearing faces of blocks used for compression testing of soil-cement have a hardness of not less than 60 HRC.

5.2 *Molds and Compaction Equipment*, in accordance with Test Methods D 559 or D 560 for Method A; Practice D 1632 for Method B.

6. Test Specimens

6.1 Mold the test specimens as follows:

6.1.1 *Method A*—Specimens are 4.0 in. (101.6 mm) in diameter and 4.584 in. (116.4 mm) in height and are molded in accordance with Test Methods D 559 or D 560.

6.1.2 *Method B*—Specimens are 2.8 in. (71.1 mm) in diameter and 5.6 in. (142.2 mm) in height and are molded in accordance with Practice D 1632.

NOTE 3—These methods may be used for testing specimens of other sizes. If the soil sample includes material retained on the 4.75-mm (No. 4) sieve, it is recommended that Method A be used, or that larger test specimens, 4.0 in. (101.6 mm) in diameter and 8.0 in. (203.2 mm) in height, be molded in a manner similar to Method B.

6.2 Moist cure the specimens in accordance with Practice D 1632.

6.3 At the end of the moist-cure period, immerse the specimens in water for 4 h.

6.4 Remove the specimens from the water and make compression tests as soon as practicable, keeping specimens moist by a wet burlap or blanket covering.

NOTE 4—Other conditioning procedures, such as air or oven drying, alternate wetting and drying, or alternate freezing and thawing may be specified after an initial moist curing period. Curing and conditioning procedures shall be given in detail in the report.

6.5 Check the smoothness of the faces with a straightedge. If necessary, cap the faces to meet the requirements of the section on Capping Specimens of Practice D 1632.

7. Procedure

7.1 Place the lower bearing block on the table or platen of the testing machine directly under the spherically seated (upper) bearing block. Place the specimen on the lower bearing block, making certain that the vertical axis of the specimen is aligned with the center of thrust of the spherically seated block. As this block is brought to bear on the specimen, rotate its movable portion gently by hand so that uniform seating is obtained.

7.2 Apply the load continuously and without shock. A screw power testing machine, with the moving head operating at approximately 0.05 in. (1 mm)/min when the machine is running idle, may be used. With hydraulic machines, adjust the loading to a constant rate within the limits of 20 ± 10 psi (140 ± 70 kPa)/s, depending upon the strength of the specimen. Record the total load at failure of the test specimen to the nearest 10 lbf (40 N).

8. Calculation

8.1 Calculate the unit compressive strength of the specimen by dividing the maximum load by the cross-sectional area.

NOTE 5—If desired, make allowance for the ratio of height to diameter (h/d) by multiplying the compressive strength of Method B specimens by the factor 1.10. This converts the strength for an h/d ratio of 2.00 to that for the h/d ratio of 1.15 commonly used in routine testing of soil-cement (see Section 4). This conversion is based on that given in Method C 42, which has been found applicable for soil-cement.

9. Report

9.1 The report shall include the following:

[6] For additional discussion on the significance and use of compressive strength results, see the *Soil-Cement Laboratory Handbook*, Chapter 4, Portland Cement Association, Skokie, IL, 1971, pp 31 and 32.

9.1.1 Specimen identification number,

9.1.2 Diameter and height, in. (mm),

9.1.3 Cross-sectional areas, in.^{2} (mm^2),

9.1.4 Maximum load, to the nearest 10 lbf (40 N),

9.1.5 Conversion factor for height to diameter ratio (see Note 4), if used,

9.1.6 Compressive strength, calculated to the nearest 5 psi (35 kPa),

9.1.7 Age of specimen, and

9.1.8 Details of curing and conditioning periods, and water content in accordance with Test Method D 2216 at the time of test.

10. Precision and Bias

10.1 The precision and bias of this test method have not been established by an interlaboratory test program. However, based on the test data that are available, the following may serve as a guide as to the variability of compressive strength test results.

10.1.1 Tests were performed in a single lab on 122 sets of duplicate specimens molded from 21 different soil materials. The average difference in strength on duplicate specimens was 8.1 % and the median difference was 6.2 %. These values are expressed as the percent of the average strength of the two specimens as follows:

$$\% \text{ Difference} = \frac{(\text{high value} - \text{low value})}{(\text{high value} + \text{low value})/2} \times 100 \quad (1)$$

The distribution of the variation is shown in Fig. 1. The data[7,8] cover a wide range of cement contents and compressive strengths.

11. Keywords

11.1 compressive strength; soil-cement; soil stabilization

[7] Packard, R. G., "Alternate Measures for Measuring Freeze-Thaw and Wet-Dry Resistance of Soil-Cement Mixtures," *Highway Research Bulletin*, 353, Transportation Research Board, 1962, pp 8–41.

[8] Packard, R. G., and Chapman, G. A., "Developments in Durability Testing of Soil-Cement Mixtures," Highway Research Record No. 36, Transportation Research Board, 1963, pp 97–122.

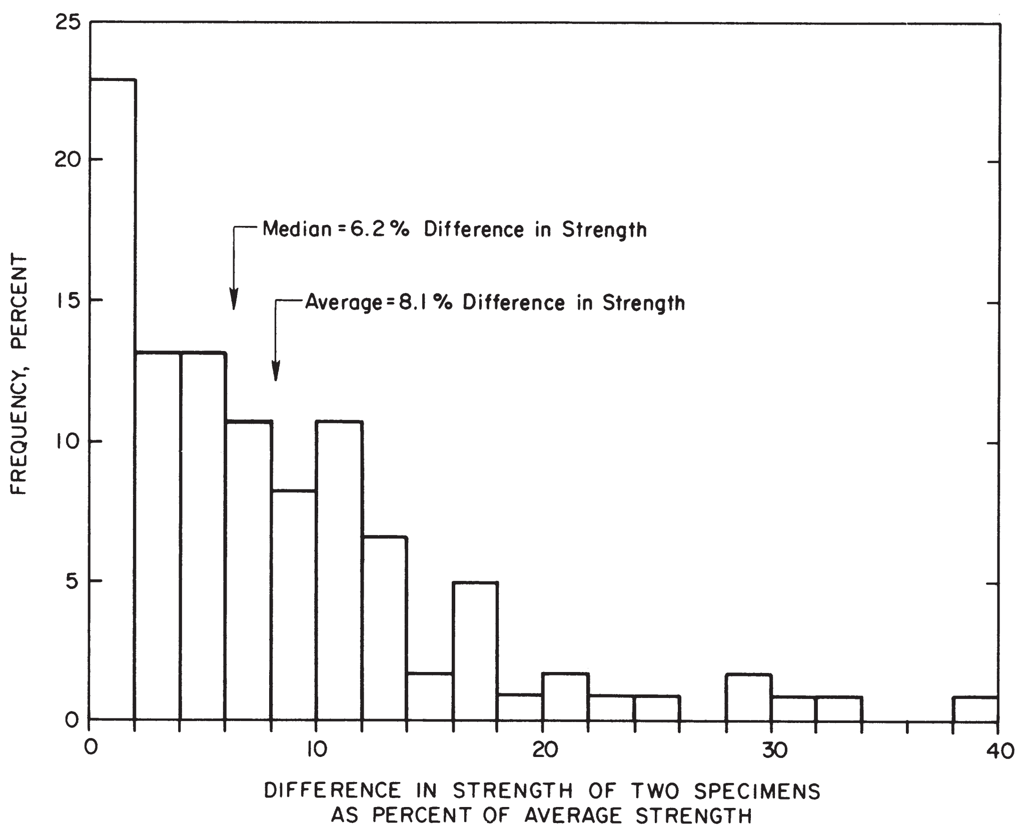


FIG. 1 Distribution of Variation of Test Results for 122 Sets of Duplicate Specimens

SUMMARY OF CHANGES

In accordance with Committee D18 policy, this section identifies the location of changes to this standard since the last edition (1996) that may impact the use of this standard.

(*1*) Changed title to clarify that two methods are presented.
(*2*) Added new sentence at the end of 1.2.1 to identify applicable materials.
(*3*) Added a new sentence at the end of 1.2.2 to identify applicable materials.
(*4*) Added new 1.3 to reference Practice D 6026.
(*5*) Revised 1.4 to clarify units used in the test method.
(*6*) Added Terminology D 653, Test Method D 2216, Specification D 4753, and Practice D 6026 to Section 2, Referenced Documents.
(*7*) Added new footnote 4 to reference *Annual Book of ASTM Standards*, Vol 04.09 and renumbered the remaining footnotes.
(*8*) Added new Section 3 on Terminology. Renumbered remaining sections.
(*9*) Added reference to Test Method D 2216 in 9.1.8.
(*10*) Changed "crushing" to "shearing" in 4.2.
(*11*) Changed "moisture" to "water" in 9.1.8.
(*12*) Prepared new Summary of Changes.

14457 Meridian Parkway
Riverside, CA 92518
(951) 697-4777. | oneatlas.com

March 14, 2022

Atlas No. 10-57575PW
Report No. 5

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

Subject: **Review of the Soil Mitigation Plan**
Compton College PE Complex Replacement
Compton Community College District
1111 East Artesia Boulevard
Compton, CA 90221

References:
1) Atlas Technical Consultants, 2021, Addendum Geotechnical and Geohazard Report, Physical Education Complex Replacement, Compton Community College District, Compton, CA. Project No. 10-57575PW Report No. 2, dated September 7, 2021.
2) Atlas Technical Consultants, 2021, Geotechnical Investigation Report, Physical Education Complex Replacement, Compton Community College District, Compton, CA. Project No. 10-57575PW, Report No. 1, dated July 7, 2021.
3) CGS's Comments Letter, Comment No. 22, Page 8, in regard to a formal documentation of Atlas review of the contractor's (KNA) VSC and DSM design and plans, dated February 4, 2022 (attached)

CGS Application No. 03-CGS5153, DSA Application No. 03-121755

Dear Ms. Owens:

In accordance with the CGS Comment No. 22 (Reference 3), Atlas Technical Consultants (Atlas) has reviewed the Vibro Stone Column Design provided in Attachment A' and the Deep Soil Mixing Design provided in Attachment B', the design and plans, prepared by Keller North America (KNA). Our review was based on the geotechnical and geohazard aspects of the reviewed documents (Attachments A' and B') and was to verify that they are in general conformance with the recommendations provided in References 1 and 2.

Based on our review and to the best of our knowledge and understanding, it is our opinion that the proposed soil mitigation design and plans, provided in Attachments A' and B', including the depth of the mitigated soil, diameter, length, spacing and area replacement ratio (ARR) of the proposed Vibro Stone Columns and Deep Soil Mixing Columns have been prepared in general conformance with the recommendations provided References 1 and 2. ATLAS recommends performing necessary tests (field and lab) on the mitigated soil (DSM and VRSC) to evaluate the behavior of the mitigated soil. Based on these tests results and analyses, the preliminary recommendations for the soil mitigations in references 1 and 2 and the design presented in the Attachments A' and B' may need to be modified (e.g., adding some additional rows of DSM and/or VRSC).

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

Respectfully submitted,
Atlas Technical Consultants LLC

REGISTERED PROFESSIONAL ENGINEER MEHRAB JESMANI GE 3175 EXP. 09/30/23 GEOTECHNICAL STATE OF CALIFORNIA

REGISTERED PROFESSIONAL GEOLOGIST DOUGLAS A. SKINNER No. 2472 CERTIFIED ENGINEERING GEOLOGIST STATE OF CALIFORNIA

Mehrab Jesmani, PhD, PE, GE 3175
Senior Engineer

Douglas A. Skinner, PG, CEG 2472
Senior Geologist

MJ:DAS:ER

Attachments:

- Attachment A': Keller North America, 2022, Compton Community College (Phase 1) Vibratory Replacement Stone Columns (VRSC) Shop Drawings – Overall Ground Improvement Plan Sheet, KNA-3: dated February 28, 2022

- Attachment B': Keller North America, 2022, Compton Community College (Phase 2) Deep Soil Mixing (DSM) Shop Drawings – Overall Ground Improvement Plan Sheet KNA-3P: dated February 28, 2022

Distribution:
Ms. Linda Owens at: lowens@compton.edu
Ms. Sheri Phillips at: sphillips@pcm3.com
Mr. Hraztan Zeitlian at: hraztan@struere

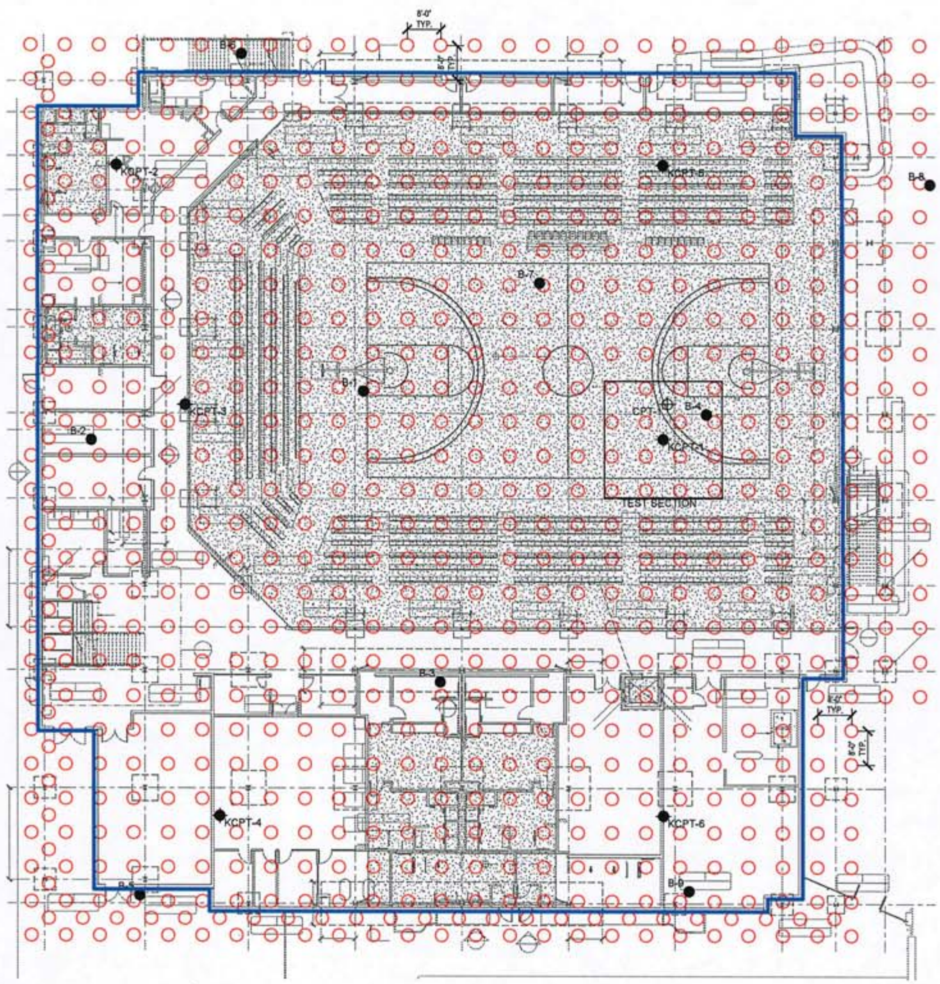
ATTACHEMENT A'

KNA’S VRCS SHOP DRAWINGS SHEET KNA-3

2/14/2022 6:20:28 PM

LEGEND:

- Ø36" Vibratory Replacement Stone Columns (VRSC). Install 23' from the existing ground surface. Approximate existing ground surface elevation: 55 feet.
- CPT-X Existing CPT location
- B-X Existing Boring location
- KCPT-X Proposed Post-Treatment CPT locations
- Building Footprint



STRUERE
ADVANCED ARCHITECTURE
3324 GRAND VIEW
LOS ANGELES, CALIFORNIA 90066
TELEPHONE (310) 749-7649
E-MAIL HRAZTAN@STRUERE.COM
WWW.STRUERE.COM

KELLER
17461 DERIAN AVENUE
SUITE 106
IRVINE, CALIFORNIA
92614
WWW.KELLER-NA.COM

VRSC ENGINEER OF RECORD:

IDENTIFICATION STAMP
DIV. OF THE STATE ARCHITECT
APP. 03-124753 INC: #
RE-REVIEWED FOR
SS ☐ FLS ☐ ACS ☐
DATE:

Compton College

COMPTON COLLEGE
PHYSICAL EDUCATION
COMPLEX REPLACEMENT

1111 EAST ARTESIA BLVD,
COMPTON, CA 90221-5393

ISSUE DESCRIPTION	
DSA SUBMITTAL	22.02.28

SHEET TITLE
OVERALL GROUND IMPROVEMENT PLAN
KNA-3

ATTACHEMENT B'

KNA'S DSM SHOP DRAWINGS SHEET KNA-3P

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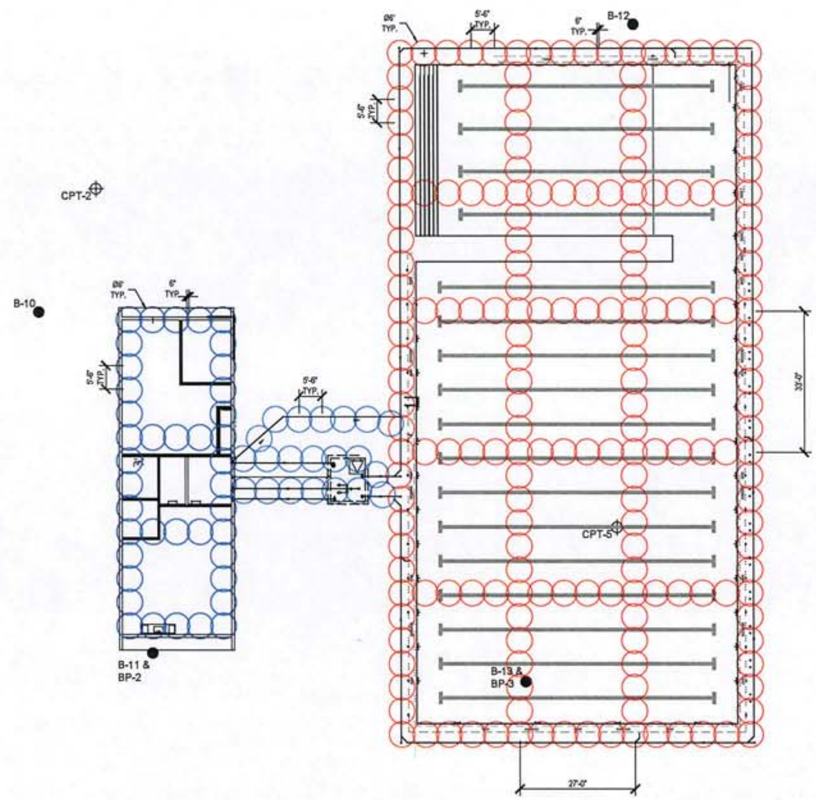
LEGEND:

Ø6' DEEP SOIL MIX COLUMN (DSM).
Install 49' from the existing ground surface.
Approximate existing ground surface elevation: 55 feet.

Ø6' DEEP SOIL MIX COLUMN (DSM).
Install 19' from the existing ground surface.
Approximate existing ground surface elevation: 55 feet.

CPT-X Existing CPT location

B-X Existing Boring location



STRUERE
ADVANCED ARCHITECTURE

KELLER
17461 DERIAN AVENUE
SUITE 106
IRVINE, CALIFORNIA
92614
WWW.KELLER-NA.COM

DSM ENGINEER OF RECORD:

Compton College

COMPTON COLLEGE
PHYSICAL EDUCATION
COMPLEX REPLACEMENT

1111 EAST ARTESIA BLVD.
COMPTON, CA 90221-5393

ISSUE DESCRIPTION	
DSA SUBMITTAL	22.02.28

SHEET TITLE
OVERALL GROUND IMPROVEMENT PLAN
KNA-3P

ADDENDUM GEOTECHNICAL AND GEOHAZARD REPORT

PHYSICAL EDUCATION COMPLEX REPLACEMENT
COMPTON COMMUNITY COLLEGE DISTRICT

PREPARED FOR:

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

PREPARED BY:

Atlas Technical Consultants LLC
14457 Meridian Parkway
Riverside, CA 92518

September 7, 2021

14457 Meridian Parkway
Riverside, CA 92518
(951) 697-4777. | oneatlas.com

September 7, 2021

Atlas No. 10-57575PW
Report No. 2

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

Subject: **Addendum to the Geotechnical Investigation Report**
Compton College PE Complex Replacement
Compton Community College District
1111 East Artesia Boulevard, Compton, California

Reference: Atlas Technical Consultants, 2021, Geotechnical Investigation Report, Physical Education Complex Replacement, Compton Community College District, Compton, CA, Project No. 10-57575PW, Dated: July 7.

Dear Ms. Owens:

Atlas Technical Consultants is pleased to present this addendum geotechnical and geohazard report for the proposed Physical Education Complex Replacement, Compton College located at 1111 East Artesia Boulevard in the City of Compton, California.

This addendum report has been prepared based on the questions that were sent to Atlas by the Structural Engineering Team, Brandon & Johnston, the Architect Team, Struere, Inc. and the Pool Design Team. Please note that the preliminary recommendations and information provided in this addendum report need to be verified by some field tests during construction.

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

Respectfully submitted,
Atlas Technical Consultants LLC

REGISTERED PROFESSIONAL ENGINEER
MEHRAB JESMANI
GE 3175
EXP. 09/30/23
GEOTECHNICAL
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL GEOLOGIST
DOUGLAS A. SKINNER
No. 2472
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Mehrab Jesmani, PhD, PE, GE 3175
Senior Engineer

Douglas A. Skinner, PG, CEG 2472
Senior Geologist

MJ:DS:ER:ds

Distribution: Sheri Phillips, sphillips@pcm3.com
Hraztan Zeitlian, hraztan@struere

1. Soil Mitigation and Ground Improvement (Preliminary Recommendation)

The preliminary recommendations of the soil mitigation addressed in this report is Deep Soil Mixing (DSM), with about 30 percent of area replacement ratio and/or Vibro Stone Columns (VSC) method with Soil Densification and with the minimum of about 11 percent of area replacement ratio. It shall be noted that generally the mitigated soils with vibro stone column estimates more settlement potential compared to the similar mitigated soil with deep soil mixing; so, the preliminary recommended depth and other parameters of the mitigated soil with vibro stone columns provided in this report need to be verified by the contractor based on the project requirements and the results of the field testing (e.g., cone penetration tests [CPT, scheduled 9/3/21]) and the contractor shall increase the depth of the mitigated soil with VSC when needed.

The preliminary recommendation of the diameter, overlap, grid size and area replacement ratio of the Deep Soil Mixing have been provided in the Referenced Geotechnical Investigation Report, Section 4.3.1.

The preliminary recommendations for the soil mitigation is Deep Soil Mixing and/or Vibro Stone Columns, provided in this report and the Referenced Geotechnical Investigation Report should be detailed by the specialty contractor and need to be verified in the field and during construction by the contractor based on the results of CPT (e.g., CPT scheduled 9/3/21) and other necessary tests. We recommend performing a test zone of mitigated soil in the project site and perform the necessary tests on the mitigated soil to evaluate the soil properties and behavior before and after mitigation.

It will be the responsibility of the contractor to provide the final configuration, design parameters, recommendations, and method of performance for the soil mitigation methods (e.g., deep soil mixing and/or vibro stone columns diameter, length, grid size, overlap, area replacement ratio, total required depth and thickness of the mitigated soil, etc.) to mitigate, static, liquefaction and seismic dry settlement within the zone and the depth of the mitigated soil and provide the satisfactory bearing capacity, subgrade modulus, safety factor and other required parameters to limit the settlements within the safe and acceptable range. The final selected method of soil mitigation should be monitored and not cause damage to the existing buildings, utilities, and other facilities and improvements in the site (due to vibration, etc).

Installation of ground improvements should be observed by the geotechnical consultant. A specialty contractor with expertise in VSC and DSM ground improvement methods should be consulted. This specialty contractor should review the boring and CPT logs and provide their approach, calculations, and specifications for ground improvements at the site.

1.1 Soil Mitigation and Ground Improvement (Preliminary Recommendation for Vibro Stone Columns)

The preliminary recommendations for Vibro Stone Columns can be as follow:

- Minimum Diameter: 3 feet
- Maximum Spacing: at 8 feet on center
- Minimum Area Replacement Ratio: 11%
- Extent beyond the foundation shall be at least half the depth of the VSCs with a minimum of 10 feet or an approved alternative.
- VSCs under the shallow foundation shall be located symmetrically around the centroid of the footing or load.
- Minimum of four vibro stone columns shall be under each isolated or continuous/combined footing or approved equivalent.
- If the proposed development is in close proximity to existing buildings and unable to extend the ground improvement beyond the buildings and/or pool foundations, the secondary row of VSCs can be added within the perimeter grid. That is, adding an additional column within the proposed grid along the entire perimeter of the subject area/zone. This secondary column should be located at the center of each of the four VSCs. However, based on our understanding for this project site, we do not see any concern with nearby buildings.
- Field verification should be provided to confirm the effectiveness of the ground improvements with regard to the settlement (static, liquefaction and seismic dry settlement) at the site. Field verification should consist of the advancement of CPTs and and/or other necessary tests upon completion of ground improvements. At a minimum, we recommend that CPTs be advanced at six locations within the building footprint, or one CPT for every 4,000 square feet of building area, whichever results in a greater number of test locations. The data should be provided to the geotechnical consultant of record who should perform seismic settlement/liquefaction analysis. This should be performed in order to confirm that the settlement criteria discussed herein are met and for the preparation of the final verified report.

It shall be noted that based on Section 1813A, 2019 CBC: Vibro Stone Columns for Ground Improvement (1813A.1, General), ground improvement shall be installed under the entire building/structure footprint and not under isolated foundation elements only.

1.2 Soil Mitigation and Ground Improvement (Preliminary Recommendation for Deep Soil Mixing)

The preliminary recommendations for Deep Soil Mixing (DSM) can be as follow:

- Minimum Diameter: 6 feet
- Minimum Area Replacement Ratio: about 30% (should cover the entire footprint: Building/Structure)
- Minimum 6 inches overlap
- Field verification should be provided to confirm the effectiveness of the ground improvements with regard to the settlement (static, liquefaction and seismic dry settlement) at the site. Field verification should consist of Data Acquisition (DAQ) Reports, Unconfined Compressive Strength (UCS) testing on wet obtained samples and cores. At a minimum, we recommend that wet obtained samples shall be taken once every rig shift and minimum 8 samples be made for testing at 7, 14, 28 and 56 days. If the design strength criteria are met at 7 or 14 days, no further testing is required by GEOR. Coring should be performed on 2% of the columns. All UCS testing will be performed by a designated lab working under GEOR. Specialty Contractor will provide as-built report to GEOR for review and preparing the final verification report for final acceptance of work.

2. PE Building: Isolated Spread Footings and Tie Beams and Combined/ Continuous Foundation with Soil Mitigation (Preliminary Recommendation)

Based on our understanding from the information provided by the structural engineers (from Table 12.13-3, ASCE 7-16, Risk Category III: 0.006L), the acceptable differential settlement (static plus earthquake and liquefaction), for the PE Building foundation system is 2.88 inches over a horizontal distance of 40 feet. In this case the recommended depth of the ground improvement and soil mitigation is about 23 feet. Preliminary design parameters are provided below for planning purposes. The specialty contractor designing the ground improvement should recommend the actual capacities, column layout, depth, spacing and other design parameters.

- Deep Soil Mixing allowable bearing capacity:

  Dead and Live: 6,000 psf
  Dead+Live+Seismci: 8,000 psf
  Subgrade Modulus: 125 pci
  Coefficient of Friction between foundation and DSM Column: approximately 0.35

- Vibro Stone Columns allowable bearing capacity:

  Dead and Live: 3,000 psf
  Dead+Live+Seismci: 4,000 psf
  Subgrade Modulus: 35 to 45 pci
  Coefficient of Friction between foundation and VSC: approximately 0.35

3. Pool Building with Soil Mitigation (Preliminary Recommendation)

Based on our understanding from the information provided by the structural engineers (from Table 12.13-3, ASCE 7-16) and for the pool building, the acceptable differential settlement (static plus earthquake and liquefaction) for the Risk Category II is 3.6 inches (0.0075L) and for Risk Category III is 2.4 inches (0.005L) over a horizontal distance of 40 feet, respectively. In this case the recommended depth of the ground improvement and soil mitigation is about 19 feet for the Risk Category II and 27 feet for Risk Category III. Preliminary design parameters are provided below for planning purposes. The specialty contractor designing the ground improvement should recommend the actual capacities, column layout, depth, spacing and other design parameters.

- Deep Soil Mixing allowable bearing capacity:

  Dead and Live: 6,000 psf
  Dead+Live+Seismci: 8,000 psf
  Subgrade Modulus: 125 pci
  Coefficient of Friction between foundation and DSM Column: approximately 0.35

- Vibro Stone Columns allowable bearing capacity:

  Dead and Live: 3,000 psf
  Dead+Live+Seismci: 4,000 psf
  Subgrade Modulus: 35 to 45 pci
  Coefficient of Friction between foundation and VSC: approximately 0.35

4. Pool with Soil Mitigation (Preliminary Recommendation)

- Based on our understanding from the information provided by the pool design team, the acceptable differential settlement can be considered as below:
  - ½ inch over a horizontal distance of 154 feet. For this case, the depth of the ground improvement and soil mitigation can be about 49 feet.
  - 1 inch over a horizontal distance of 154 feet. For this case, the depth of the ground improvement and soil mitigation can about 47 feet.
- The thickness of the fill observed in our boring logs at the pool location was 3 feet in the half-northern portion of the pool and 5 feet in the half-southern portion of the pool. (This fill will be removed due to the depth of the pool.)

Additional parameters that may be needed are provided as clarification or as a recommendation:

- Soil equivalent Fluid Pressure for Level Ground: Recommendations in Section 4.9 of the Referenced Geotechnical Investigation Report are still applicable for H=9 feet (height of the wall).
- Passive Pressure: Passive pressure of soil based on the Section 4.5 of the Referenced Geotechnical Investigation Report.

- Coefficient of Friction: Depends on the pool design
- Allowable Bearing Capacity of the mitigated soil under the pool (for deep soil mixing or vibro stone columns). Please see the information provided in Section 2 of this report.
- Approximate Soil Unit Weight: 115 pcf
- Seismic Increment for Soil Loads: Recommendations in Section 4.9 of the Referenced Geotechnical Investigation Report are still applicable for H=9 feet (height of the wall).
- Expansive Soil Characteristics: Discussed in the Referenced Geotechnical Investigation Report.
- Liquefaction and Groundwater Conditions: Discussed in the Referenced Geotechnical Investigation Report.
- Site Specific Seismic Values: Discussed in the Referenced Geotechnical Investigation Report.
- On the mitigated soil, to provide relatively uniform support below the pool, we recommend 2 feet of new engineered fill (comprised of aggregate base) compacted to 95 percent relative compaction (ASTM D1557). The pool design team can prepare the other requirements in detail.
- Relatively flexible connections and facilities for the pool are recommended to reduce the potential of water leakage during operation and under static and seismic load.

5. Light Pole Footing around the Pool (Concrete Shaft)

- Side friction resistance (for downward loads): 200 psf
- Lateral resistance for level ground surfaces: an equivalent fluid passive pressure of 200 pcf can be used up to a maximum of 2,000 psf.
- Due to the presence of the undocumented fill in the half northern portion of the pool, the upper 3 feet and in the half southern portion of the pool the upper 5 feet of the soil resistance shall be neglected.

6. Resistance to Lateral Loads

- Considering the recommendations provided in Section 4.5 of the Referenced Geotechnical Investigation Report, the frictional resistance and the passive resistance of the soils may be combined provided that the passive resistance is reduced by one-third.

7. Excavation for Buildings Pads, Foundations and Exterior Flat Work (Clarification Notes)

It should be noted that in the event of a major local earthquake, generally some damages to the project will occur and repairs to the damaged parts and portions should be anticipated.

However, we provided recommendations for over excavation minimum of 5 feet beneath the buildings pads (with 5 feet beyond if feasible) to improve the performance of the interior slab-on-grade. To further reduce the potential for repairs after an earthquake, the soil mitigation using ground improvement is considered to include the entire footprint of the buildings (e.g., inside the buildings, below the slab-on-grade) as discussed in the previous sections.

- Generally, for the sidewalks and flat works, minimum of 2 feet of over excavation is recommended.
- Other requirements shall be based on the Referenced Geotechnical Investigation Report.

GEOTECHNICAL INVESTIGATION REPORT

PHYSICAL EDUCATION COMPLEX REPLACEMENT
COMPTON COMMUNITY COLLEGE DISTRICT

Compton, CA

PREPARED FOR:

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

PREPARED BY:

Atlas Technical Consultants LLC
14457 Meridian Parkway
Riverside, CA 92518

July 7, 2021

ATLAS

14457 Meridian Parkway
Riverside, CA 92518
(951) 697-4777. | oneatlas.com

July 7, 2021

Atlas No. 10-57575PW
Report No. 1

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

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

Dear Ms. Owens:

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

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

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

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

Respectfully submitted,
Atlas Technical Consultants LLC

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

REGISTERED PROFESSIONAL GEOLOGIST
DOUGLAS A. SKINNER
No. 2472
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Mehrab Jesmani, PhD, PE, GE
Senior Engineer

Douglas A. Skinner, PG, CEG
Senior Geologist

MJ:DS:ds

Distribution: sphillips@pcm3.com

CONTENTS

1. INTRODUCTION 1
   1.1 Site Location and Description 1
   1.2 Proposed Development 1
   1.3 Purpose and Scope 1
2. GEOLOGIC AND GEOTECHNICAL FINDINGS 3
   2.1 Regional Geology 3
   2.2 Subsurface Conditions 4
      2.2.1 Expansion Potential 4
      2.2.2 Corrosivity Potential 4
      2.2.3 Excavatability 5
   2.3 Groundwater 6
3. FAULTING, SEISMICITY AND SEISMIC HAZARDS 6
   3.1 Faulting and Primary Seismic Hazards 6
      3.1.1 Regional Seismicity 8
   3.2 Secondary Seismic Hazards 8
      3.2.1 Liquefaction 8
      3.2.2 Seismically Induced Settlement 9
      3.2.3 Earthquake-Induced Lateral Displacement 9
      3.2.4 Surface Manifestations of Liquefaction 10
      3.2.5 Seismically Induced Landslide 10
      3.2.6 Hydro-Collapsible Soils 10
      3.2.7 Other Hazards 10
4. CONCLUSIONS AND RECOMMENDATIONS 11
   4.1 Earthwork 11
      4.1.1 Site Preparation 11
      4.1.2 Excavation/Overexcavation in Building Pad Area and the Exterior Flatwork Area for Slab-On-Grade 12
      4.1.3 Fill Placement and Compaction 13
      4.1.4 Fill Materials 13
   4.2 CBC Seismic Design Parameters 14
   4.3 Soil Treatment 16
      4.3.1 Deep Soil Mixing, Preliminary Recommendations 16
      4.3.2 Settlement of the Treated soil 16
      4.3.3 Continuous Foundation System Supported by Deep Soil Mixing (DSM) Columns 16
   4.4 Minor Footings 17
   4.5 Resistance to Lateral Loads 17
   4.6 Slab-On-Grade 17
      4.5.1 Exterior Concrete 18
   4.7 Moisture/Vapor Mitigation for Concrete Floor Slab-on-Grade 18
   4.8 Temporary Excavations 18
   4.9 Minor Retaining Wall 19

4.10 Surface Drainage ... 20
4.11 Trench Backfill ... 20
4.12 Preliminary Pavement Section ... 21
4.12.1 Asphalt Concrete (AC) Pavement ... 21
4.12.2 Portland Cement Concrete Pavement ... 22
4.13 General Note for Concrete and Rebar Recommendation ... 23
4.14 Percolation Test ... 23
4.15 Construction Observation and Testing ... 25
4.16 Limitations ... 25

5. REFERENCES ... 25

TABLES

Table 1 – Corrosion Results (UHIG, 2015) ... 5
Table 2 – Characteristics and Estimated Earthquakes for Regional Faults ... 7
Table 3 – California Building Code Seismic Design Parameters ... 14
Table 4 – 2019 California Building Code / ASCE 7-16 Site-Specific Parameters ... 15
Table 5 – Earth Pressures for Retaining Walls ... 19
Table 6 – Asphalt Pavement Sections ... 21
Table 7 – Vehicular PCC Pavement Sections ... 22
Table 8 – Deep Borehole Percolation Rate Test Results ... 24
Table 9 – Shallow Borehole Percolation Rate Test Results ... 24
Table 10 – Reduction Factors ... 24

FIGURES

Figure 1 – Site Vicinity Map
Figure 2 – Geotechnical Map
Figure 3 – Regional Geology Map
Figure 4 – Regional Fault Map
Figure 5 – Regional Seismicity Map
Figure 6 – Liquefaction Susceptibility Map
Figure 7 – Geologic Cross-Section A-A' and B-B'
Figure 8 – Ground Water Contours

APPENDICES

Appendix I Figures
Appendix II Field Exploration
Appendix III Laboratory Test Procedures And Test Results
Appendix IV Liquefaction Results
Appendix V Site-Specific Ground Motion Hazard Analysis Results
Appendix VI Infiltration Test Results
Appendix VII Historic Seismic Events

1. INTRODUCTION

1.1 Site Location and Description

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

1.2 Proposed Development

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

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

1.3 Purpose and Scope

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

Our scope of work included the following tasks:

- **Background Review** - A background review of readily available, relevant, local and regional geology maps, geohazard maps, geotechnical reports, and literature pertinent to the proposed improvements was performed.

- **Pre-Field Investigation Activities** - Prior to our drilling activities, we conducted a site reconnaissance to locate proposed boring locations for access and for coordination with Underground Service Alert (USA).

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

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

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

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

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

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

2. GEOLOGIC AND GEOTECHNICAL FINDINGS

2.1 Regional Geology

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

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

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

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

2.2 Subsurface Conditions

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

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

2.2.1 Expansion Potential

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

2.2.2 Corrosivity Potential

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

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

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

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

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

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

Table 1 – Corrosion Results (UHIG, 2015)

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

2.2.3 Excavatability

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

2.3 Groundwater

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

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

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

3. FAULTING, SEISMICITY AND SEISMIC HAZARDS

3.1 Faulting and Primary Seismic Hazards

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

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

Table 2 – Characteristics and Estimated Earthquakes for Regional Faults

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

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

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

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

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

3.1.1 Regional Seismicity

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

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

- On 7/21/1952 at 11:52:14, a magnitude 7.7 (7.7 UK, Class: Major, Intensity: VIII - XII) earthquake occurred 88.2 miles away from the city center, causing $50,000,000 total damage on 6/28/1992 at 11:57:34, a magnitude 7.6 (6.2 MB, 7.6 MS, 7.3 MW, Depth: 0.7 mi) earthquake occurred 99.1 miles away from Compton center, causing 3 deaths (1 shaking death, 2 other deaths) and 400 injuries, causing $100,000,000 total damage and $40,000,000 insured losses.

- On 10/16/1999 at 09:46:44, a magnitude 7.4 (6.3 MB, 7.4 MS, 7.2 MW, 7.3 ML) earthquake occurred 111.0 miles away from the city center.

- On 11/4/1927 at 13:51:53, a magnitude 7.5 (7.5 UK) earthquake occurred 174.9 miles away from the city center.

- On 1/17/1994 at 12:30:55, a magnitude 6.8 (6.4 MB, 6.8 MS, 6.7 MW, Depth: 11.4 mi, Class: Strong, Intensity: VII - IX) earthquake occurred 26.9 miles away from Compton center, causing 60 deaths (60 shaking deaths) and 7,000 injuries.

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

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

3.2 Secondary Seismic Hazards

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

3.2.1 Liquefaction

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

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

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

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

3.2.2 Seismically Induced Settlement

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

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

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

3.2.3 Earthquake-Induced Lateral Displacement

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

3.2.4 Surface Manifestations of Liquefaction

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

3.2.5 Seismically Induced Landslide

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

3.2.6 Hydro-Collapsible Soils

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

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

3.2.7 Other Hazards

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

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

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

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

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

4. CONCLUSIONS AND RECOMMENDATIONS

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

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

4.1 Earthwork

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

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

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

4.1.1 Site Preparation

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

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

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

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

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

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

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

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

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

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

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

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

4.1.3 Fill Placement and Compaction

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

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

4.1.4 Fill Materials

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

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

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

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

4.2 CBC Seismic Design Parameters

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

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

Table 3 – California Building Code Seismic Design Parameters

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

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

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

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

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

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

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

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

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

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

4.3 Soil Treatment

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

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

4.3.1 Deep Soil Mixing, Preliminary Recommendations

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

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

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

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

4.3.2 Settlement of the Treated soil

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

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

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

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

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

4.4 Minor Footings

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

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

4.5 Resistance to Lateral Loads

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

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

4.6 Slab-On-Grade

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

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

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

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

4.5.1 Exterior Concrete

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

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

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

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

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

4.8 Temporary Excavations

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

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

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

4.9 Minor Retaining Wall

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

Table 5 – Earth Pressures for Retaining Walls

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

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

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

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

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

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

4.10 Surface Drainage

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

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

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

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

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

4.11 Trench Backfill

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

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

4.12 Preliminary Pavement Section

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

4.12.1 Asphalt Concrete (AC) Pavement

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

Table 6 – Asphalt Pavement Sections

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

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

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

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

4.12.2 Portland Cement Concrete Pavement

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

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

Table 7 – Vehicular PCC Pavement Sections

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

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

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

- Layout joints should form square panels. When this is not practical, rectangular panels can be used if the long dimension is no more than 1.5 times the short one.
- Control joints should have a depth of at least 1/4 the slab thickness, e.g., 1 inch for a 4-inch thick slab.
- Pavement section design assumes that proper maintenance such as sealing and repair of localized distress will be performed on a periodic basis.
- The recommendations for PCC provided in this section should be verified and confirmed if necessary, at the time of construction.
- The upper 12 inches of subgrade should be compacted to at least 95% relative compaction (ASTM D1557)

4.13 General Note for Concrete and Rebar Recommendation

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

4.14 Percolation Test

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

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

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

Table 8 – Deep Borehole Percolation Rate Test Results

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

Table 9 – Shallow Borehole Percolation Rate Test Results

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

Table 10 – Reduction Factors

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

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

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

4.15 Construction Observation and Testing

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

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

4.16 Limitations

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

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

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

5. REFERENCES

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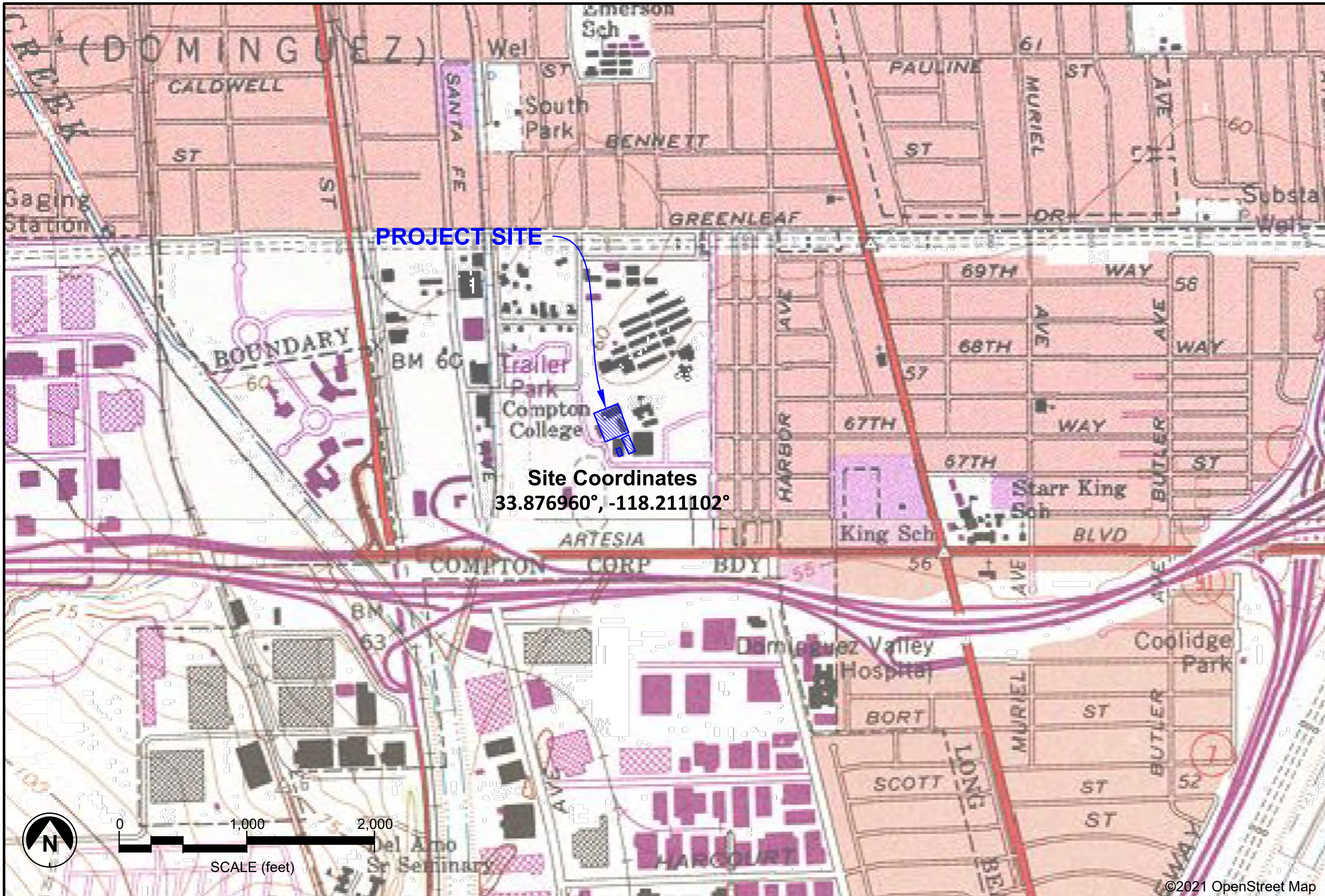
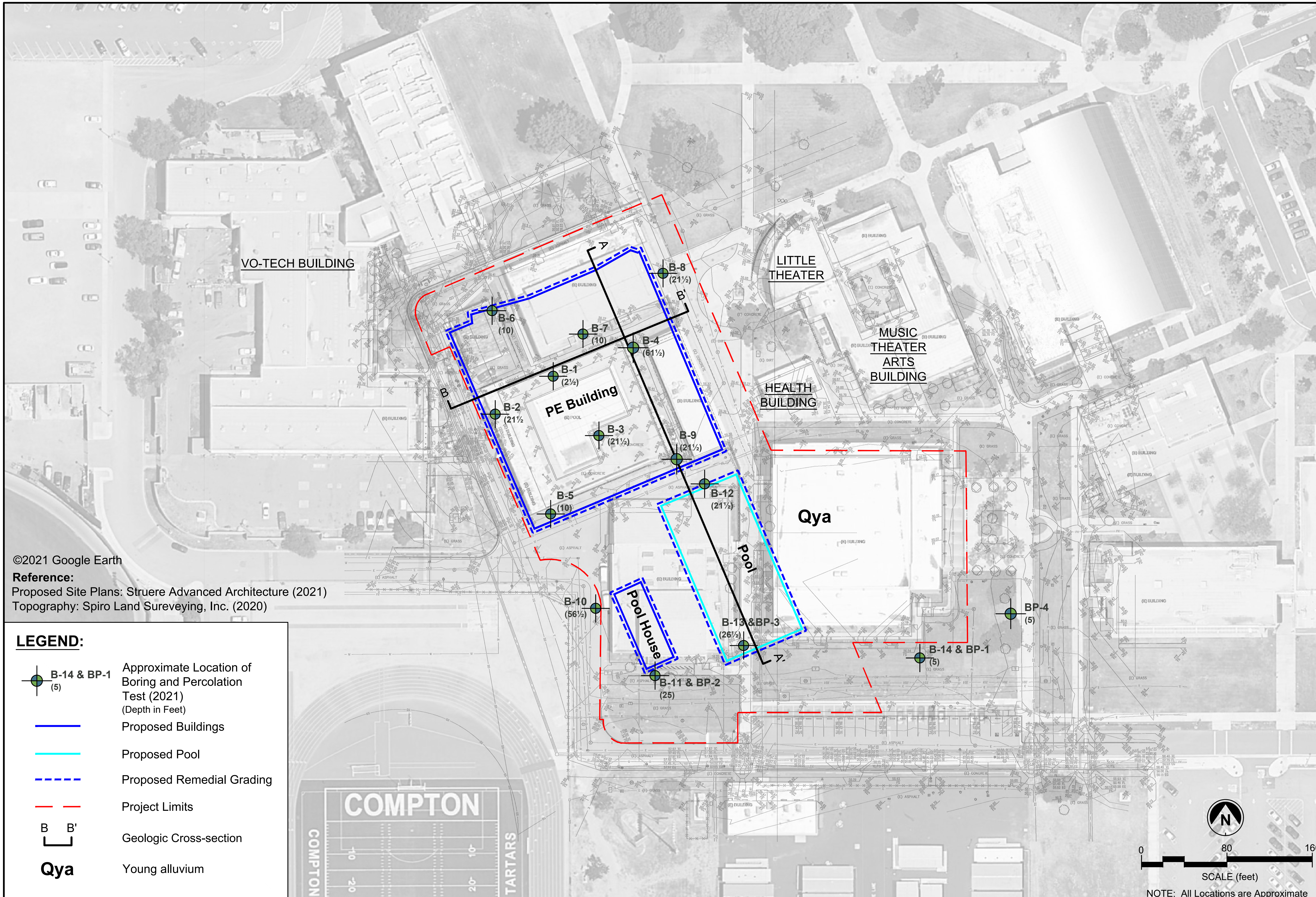
APPENDIX I
FIGURES



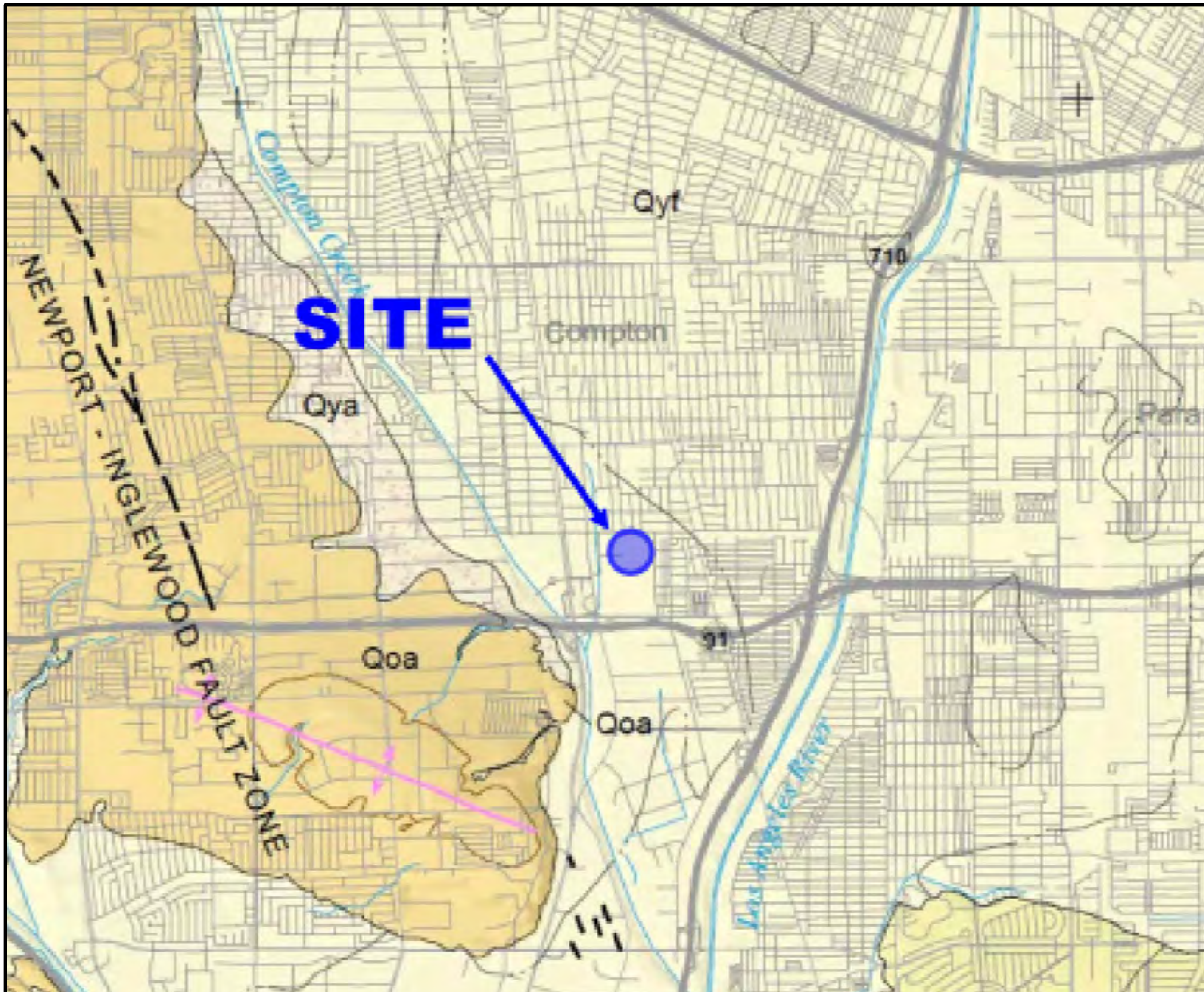
SITE VICINITY MAP Physical Education Complex Replacement Compton, California	Date: April, 2021 By: ACF Job No.: 10-57575PW	Figure: 1



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

GEOTECHNICAL MAP
Physical Education Complex Replacement
Compton, California

Figure: 2



EXPLANATION:

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

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

Qoa Old alluvial flood-plain deposits, undivided

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

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

70 U D

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

N

0 6,000 12,000

SCALE (feet)

NOTE: All locations are approximate.

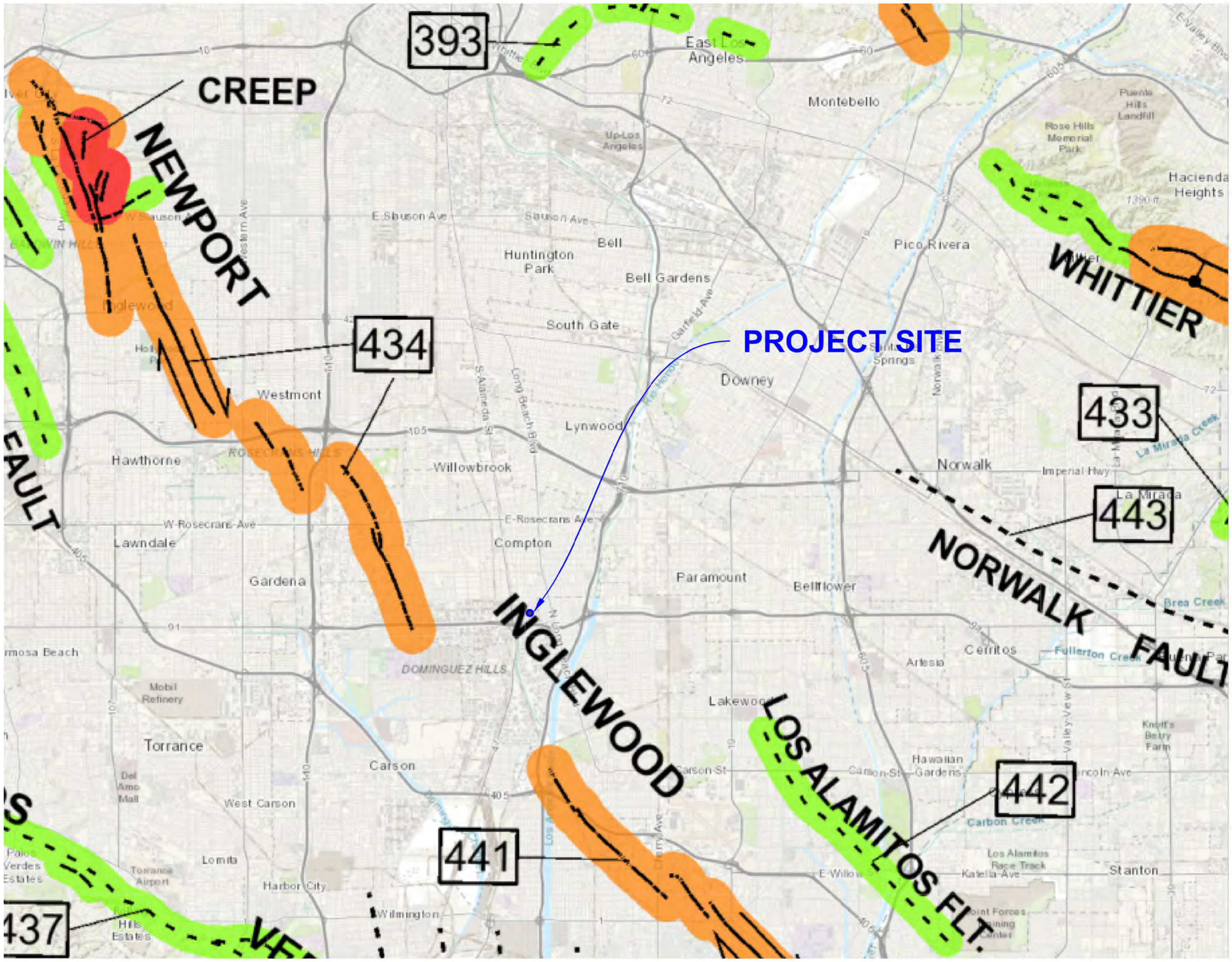
Reference:
CGS (200) Geologic Compilation of Quaternary Surficial Deposits in Southerm California Onshore Portion of the Long Beac 30'60 Quadrangle; CGS Special Report 217, Plate 8.

ATLAS

REGIONAL GEOLOGY MAP
Physical Education Complex Replacement
Compton, California

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

Figure: 3



EXPLANATION:

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

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

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

Quaternary fault (age undifferentiated).

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

Low angle fault (barbs on upper plate).

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

N

0 15,000 30,000

SCALE (feet)

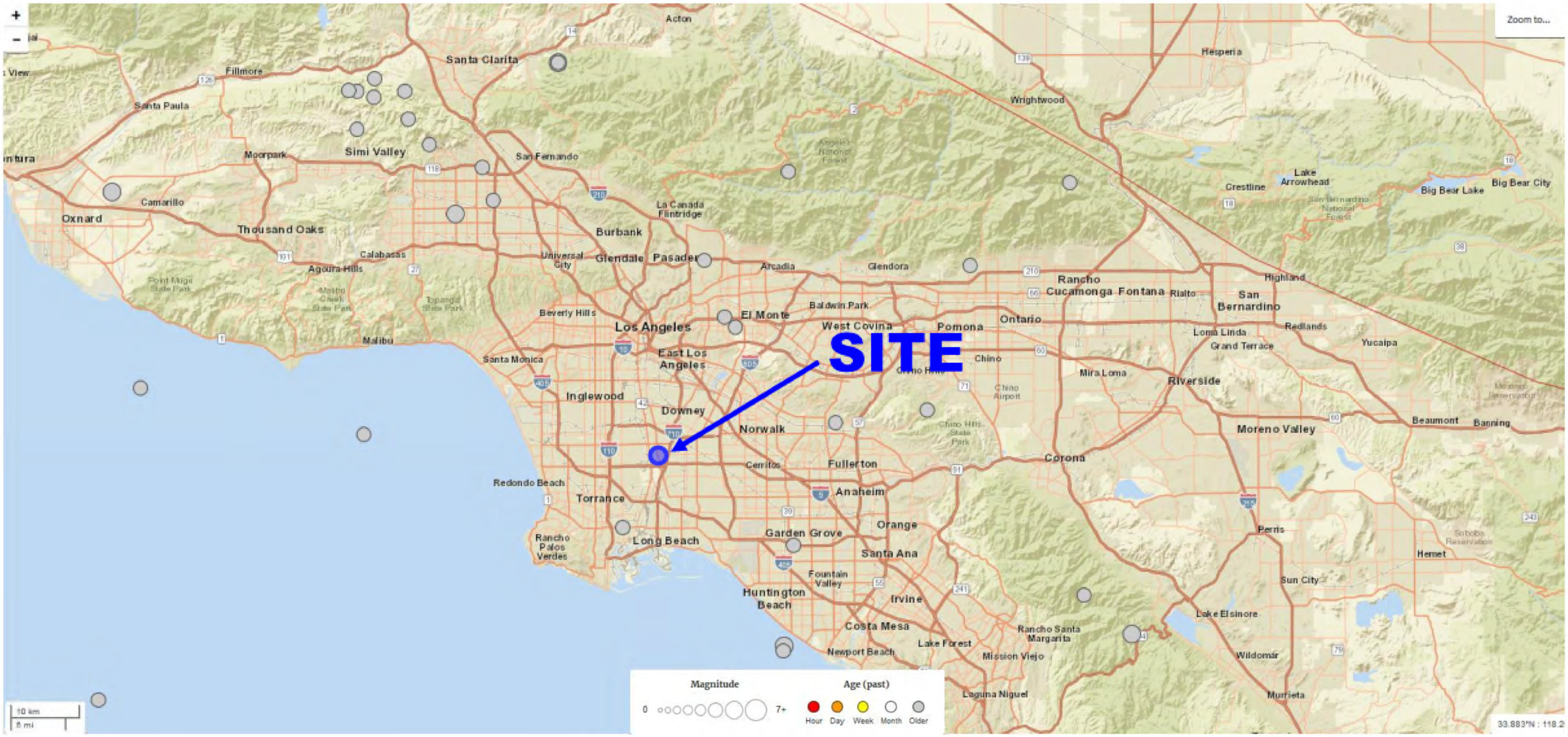
Reference:
Jennings, C.W., Bryant W.A., Esri, HERE, Garmin, FAO, NOAA, USGS, EPA, Fault Activity Map of California (2010), maps.conservation.ca.gov/cgs/fam; Accessed January, 2021

ATLAS

REGIONAL FAULT MAP
Physical Education Complex Replacement
Compton, California

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

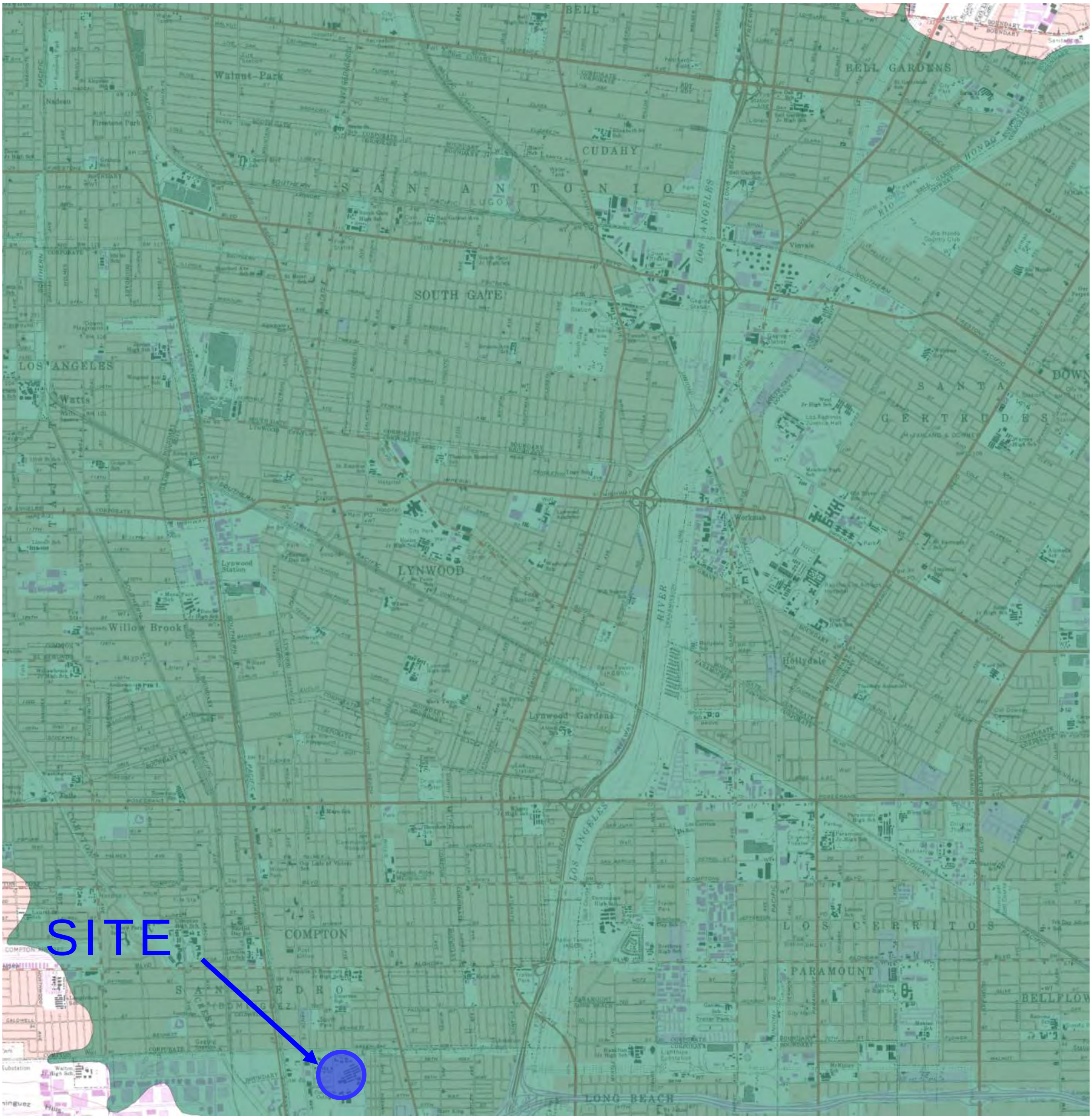
Figure: 4



Location of Historic Earthquake Epicenter (M_w>5)

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

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



LIQUEFACTION

Liquefaction

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

Earthquake-Induced Landslides

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

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

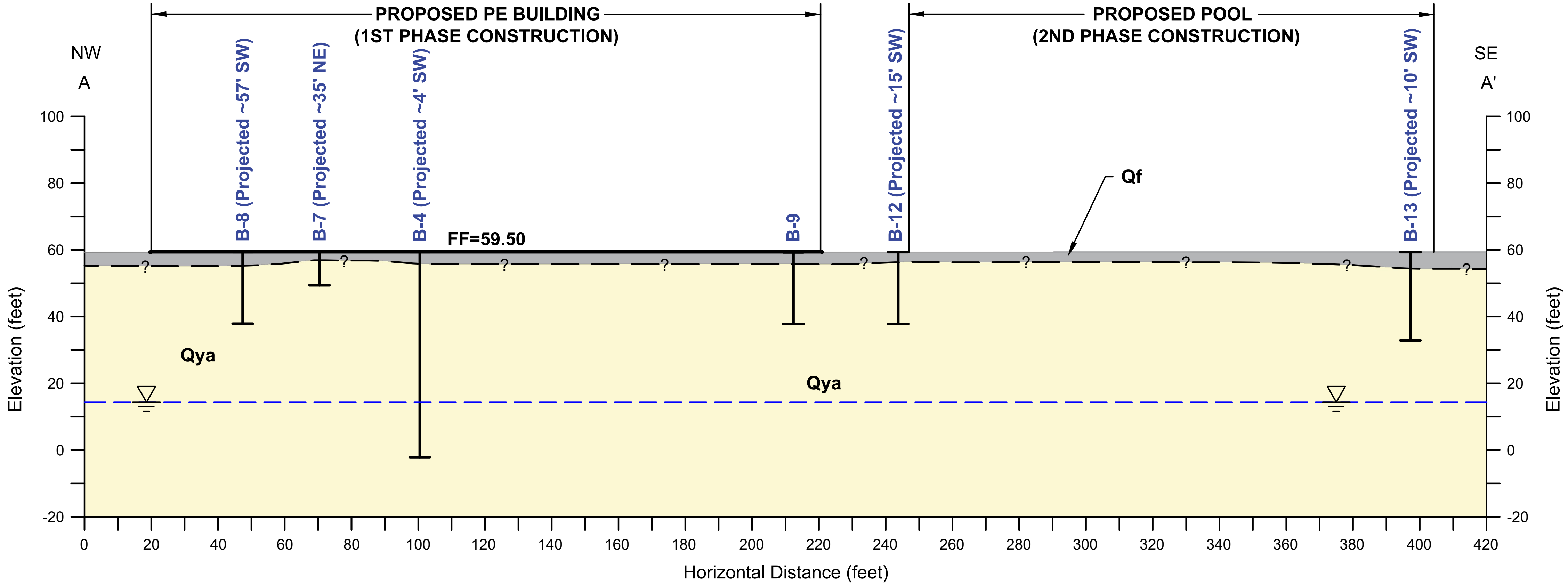
ATLAS

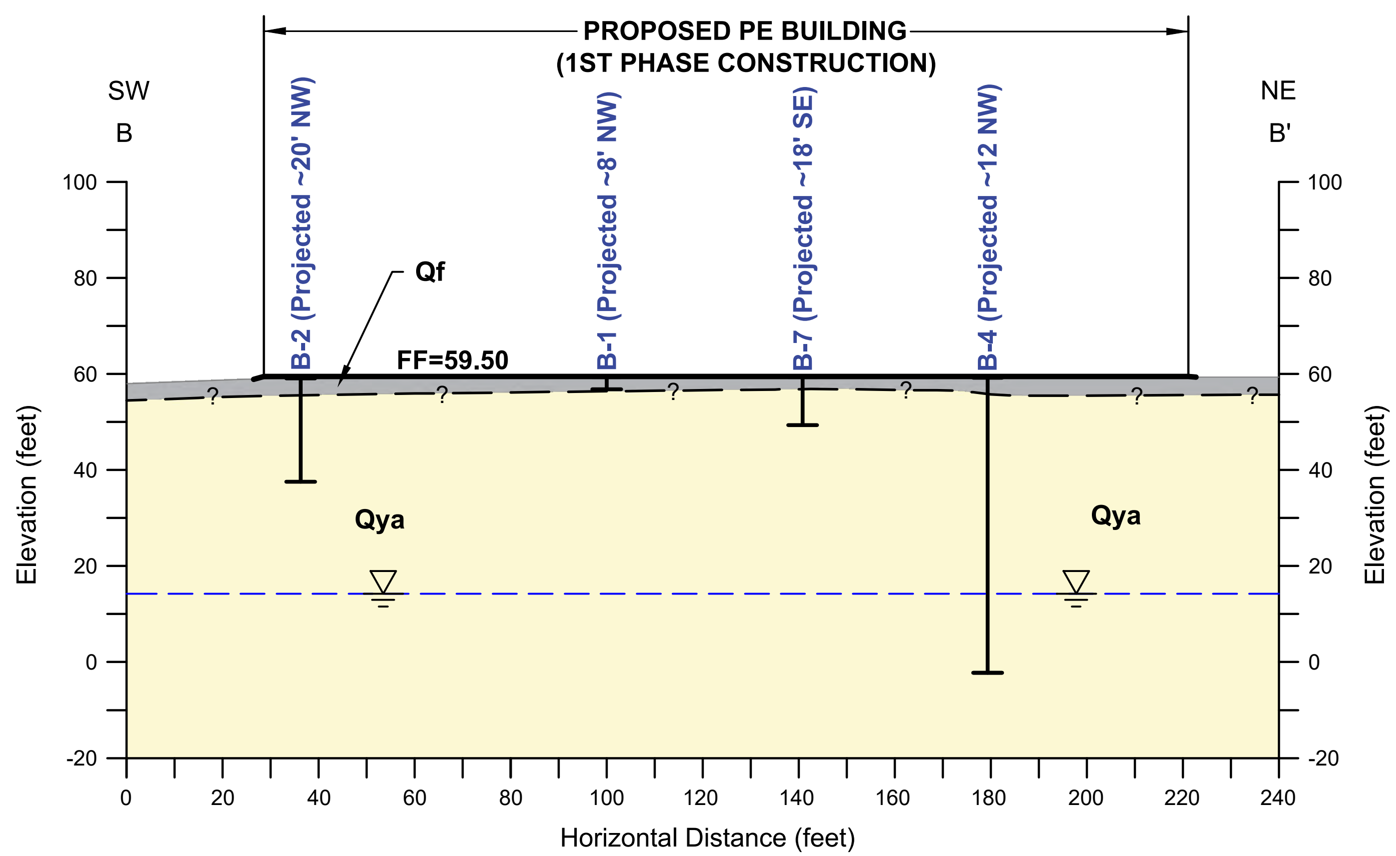
Figure 6 – Liquefaction Susceptibility Map

Physical Education Complex Replacement
Compton, California

N

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





LEGEND:

B-13 Location of Boring

Qf Fill

Qya Young Alluvium

Groundwater

Geologic Contact, Queried Where Uncertain

FF = Finish Floor

SCALE (feet) 0 40 80

NOTE: All locations and depths are approximate.

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

GEOLOGIC CROSS-SECTIONS
Physical Education Complex Replacement
Compton, California

ATLAS

Figure:
7

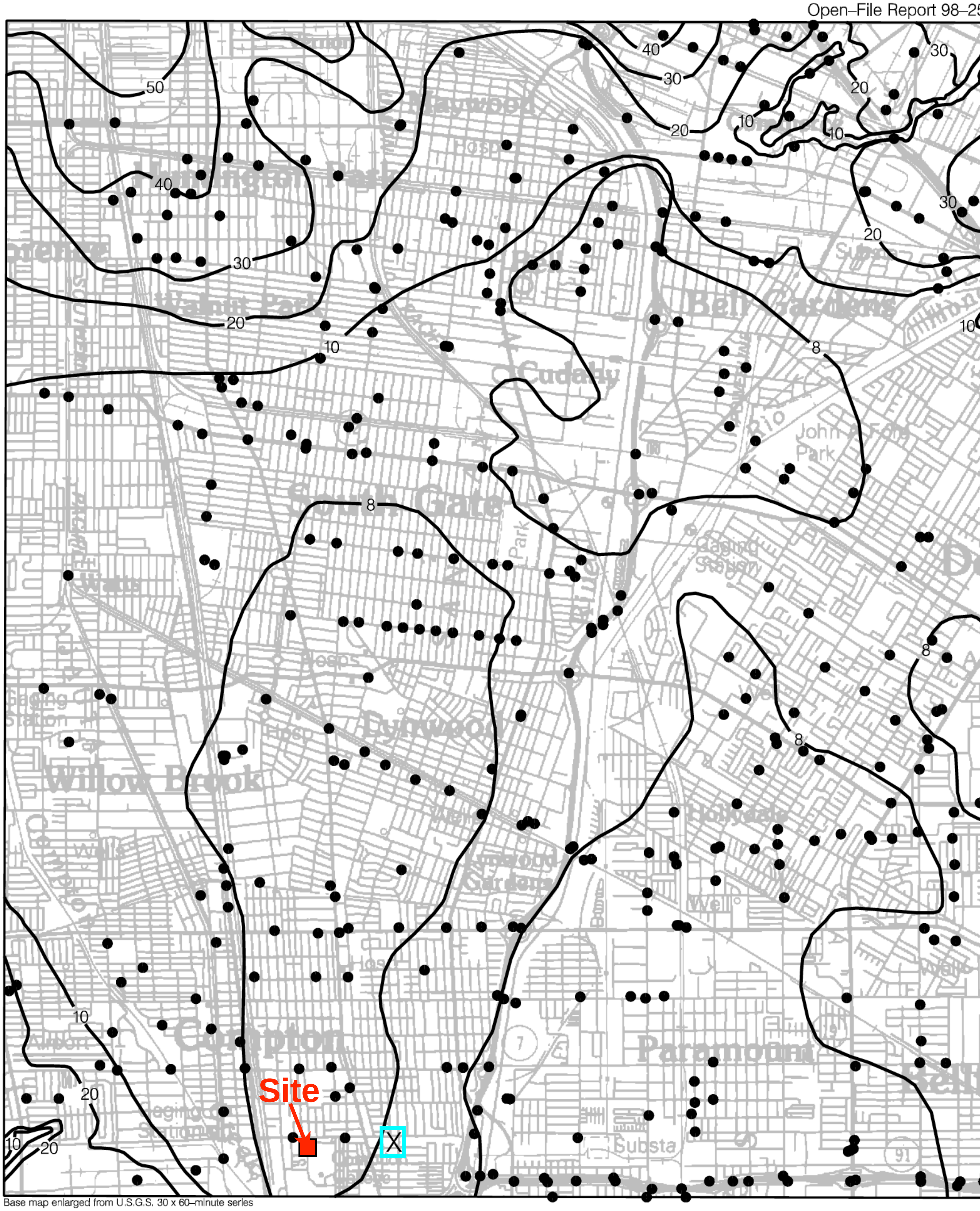


Plate 1.2 Historically Highest Ground Water Contours and Borehole Log Data Locations, South Gate Quadrangle.

● Borehole Site

30 Depth to ground water in feet

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

ONE MILE

SCALE

APPENDIX II
FIELD EXPLORATION

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

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

LOG OF TEST BORING

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

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

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

DRILLING EQUIPMENT	BORING DIA. (in.)	TOTAL DEPTH (ft)	GROUND ELEV. (ft)	DEPTH/ELEV. GROUND WATER (ft)
	8	2.5	0	AT TIME OF DRILLING ---
SAMPLING METHOD	NOTES			AT END OF DRILLING ---
	Hammer Efficiency = 73.9% N_{60}~$1.23N_{SPT}$			AFTER DRILLING ---

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

ATLAS

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

Figure

II-1

LOG OF TEST BORING

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

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

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

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

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

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

BORING TERMINATED AT 21.5 FEET

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

Figure II-2

LOG OF TEST BORING

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

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

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

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

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

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

BORING TERMINATED AT 21.5 FEET

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

Figure II-3

LOG OF TEST BORING

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

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

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

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

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

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

ATLAS

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

Figure II-4

LOG OF TEST BORING

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

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

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

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

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

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

ATLAS

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

Figure II-5

LOG OF TEST BORING

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

SITE	START	END	SHEET NO.
Compton, California	3/2/21	3/2/21	6

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

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

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

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

ATLAS

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

Figure II-6

LOG OF TEST BORING

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

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

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

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

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

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

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

Figure

II-7

LOG OF TEST BORING

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

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

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

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

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

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

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

Figure II-8

LOG OF TEST BORING

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

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

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

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

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

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

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

Figure II-9

LOG OF TEST BORING

ATLAS PROJECT NAME	ATLAS PROJECT NUMBER	
Compton College PE Complex Replacement	10-57575PW	B-7

SITE	START	END	SHEET NO.
Compton, California	3/2/21	3/2/21	10

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

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

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

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

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

Figure II-10

LOG OF TEST BORING

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

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

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

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

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

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

BORING TERMINATED AT 21.5 FEET

ATLAS

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

Figure

II-11

LOG OF TEST BORING

ATLAS PROJECT NAME: Compton College PE Complex Replacement		ATLAS PROJECT NUMBER: 10-57575PW		B-9
SITE: Compton, California		START: 3/2/21	END: 3/2/21	SHEET NO.: 12
DRILLING COMPANY: Baja Exploration	DRILL METHOD: Hollow Stem Auger	LOGGED BY: KBH	REVIEWED BY: MJ	
DRILLING EQUIPMENT: CME-75	BORING DIA. (in.): 8	TOTAL DEPTH (ft): 21.5	GROUND ELEV. (ft): 0	DEPTH/ELEV. GROUND WATER (ft): AT TIME OF DRILLING --- ; AT END OF DRILLING --- ; AFTER DRILLING ---
SAMPLING METHOD: 140-lb Hammer, 30-in Drop	NOTES: Hammer Efficiency = 73.9% N_{60}~$1.23N_{SPT}$			

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

BORING TERMINATED AT 21.5 FEET

ATLAS

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

Figure II-12

LOG OF TEST BORING

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

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

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

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

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

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

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

Figure II-13

LOG OF TEST BORING

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

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

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

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

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

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

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

LOG OF TEST BORING

ATLAS PROJECT NAME: Compton College PE Complex Replacement		ATLAS PROJECT NUMBER: 10-57575PW		B-10
SITE: Compton, California		START: 3/1/21	END: 3/1/21	SHEET NO.: 15
DRILLING COMPANY: Baja Exploration	DRILL METHOD: Hollow Stem Auger	LOGGED BY: KBH	REVIEWED BY: MJ	
DRILLING EQUIPMENT: CME-75	BORING DIA. (in.): 8	TOTAL DEPTH (ft): 56.5	GROUND ELEV. (ft): 0	DEPTH/ELEV. GROUND WATER (ft): AT TIME OF DRILLING 52.00 ft / Elev -52.00 ft; AT END OF DRILLING ---; AFTER DRILLING ---
SAMPLING METHOD: 140-lb Hammer, 30-in Drop	NOTES: Hammer Efficiency = 73.9% N_{60}~1.23N_{SPT}			

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

BORING TERMINATED AT 56½ FEET

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

Figure II-15

LOG OF TEST BORING

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

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

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

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

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

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

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

LOG OF TEST BORING

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

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

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

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

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

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

ATLAS

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

LOG OF TEST BORING

ATLAS PROJECT NAME	ATLAS PROJECT NUMBER	
Compton College PE Complex Replacement	10-57575PW	**B-12**

SITE	START	END	SHEET NO.
Compton, California	3/2/21	3/2/21	18

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

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

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

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

BORING TERMINATED AT 21.5 FEET

ATLAS

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

Figure

LOG OF TEST BORING

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

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

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

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

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

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

BORING TERMINATED AT 25.5 FEET

ATLAS

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

Figure II-19

LOG OF TEST BORING

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

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

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

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

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

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

ATLAS

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

Figure II-20

LOG OF TEST BORING

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

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

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

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

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

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

ATLAS

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

Figure

II-21

APPENDIX III
LABORATORY TEST PROCEDURES AND TEST RESULTS

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

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

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

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

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

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

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

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

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

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

ATTERBERG LIMITS

ASTM D4318

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

Modified Proctor

ASTM D1557

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

Percent Finer than No. 200 Sieve

ASTM D1140

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

R-VALUE

ASTM D2844

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

ATLAS	Compton College PE Complex Replacement Compton, California			
	By:	JRD	Date:	July, 2021
	Job Number:	10-57575PW	Figure:	III-1

EXPANSION INDEX

ASTM D4829

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

Classification of Expansive Soil [1]

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

1. ASTM - D4829

RESISTIVITY, pH, SOLUBLE CHLORIDE and SOLUBLE SULFATE

pH & Resistivity (Cal 643, ASTM G51)

Soluble Chlorides (Cal 422)

Soluble Sulfate (Cal 417)

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

Water-Soluble Sulfate Exposure [2]

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

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

Corrosivity Ratings Based on Soil Resistivity [3]

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

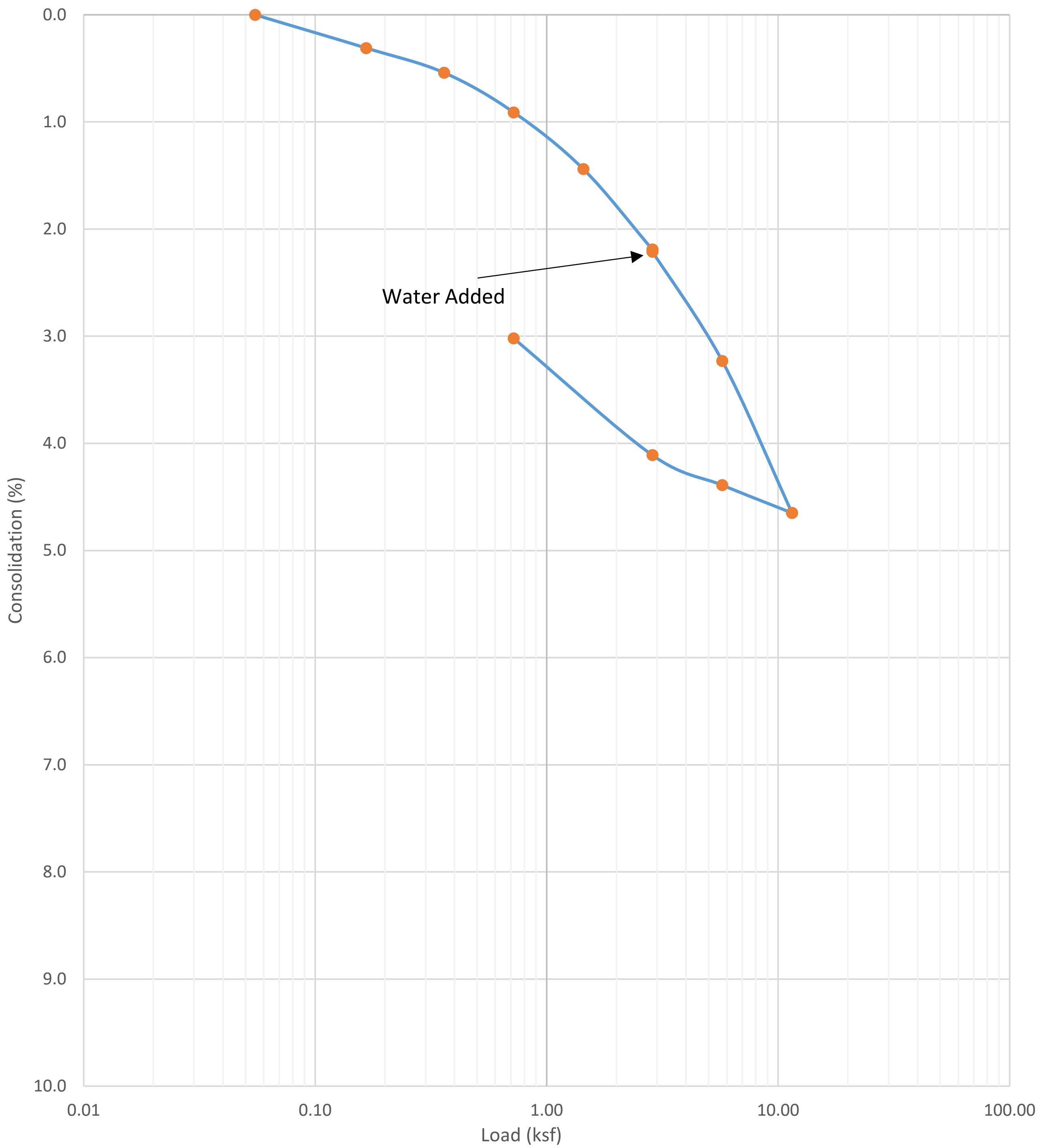
3. Roberge (2008), Corrosion Engineering, Principles and Practice

ATLAS

Compton College PE Complex Replacement Compton, California			
By:	JRD	Date:	July, 2021
Job Number:	10-57575PW	Figure:	III-2

Consolidation Test Results

ASTM D2435



Sample ID: B-8 at 6 to 6½ feet

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

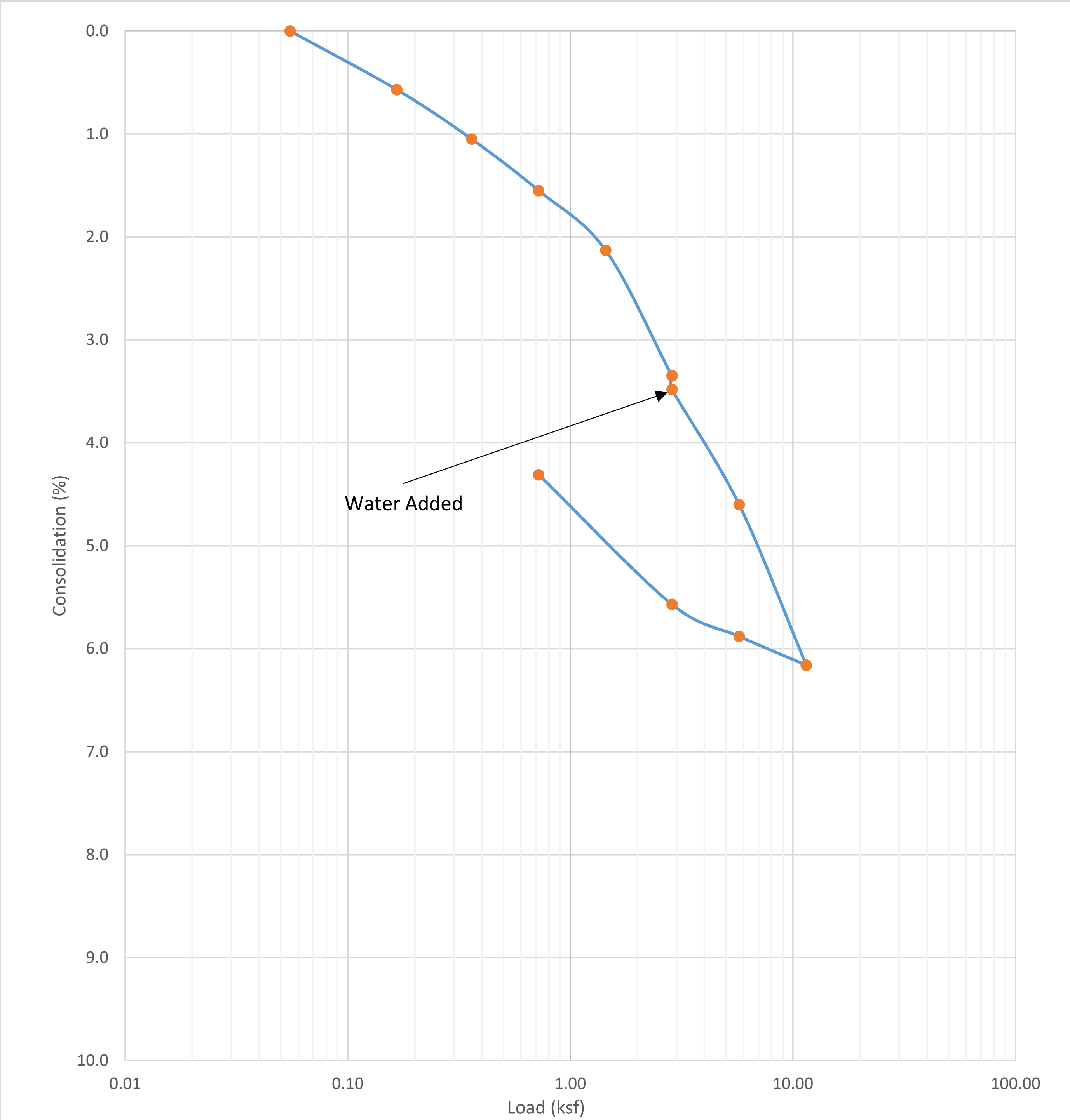
γ_d	93.9 pcf
Pre-consolidation w_c	29.5 %
Post-consolidation w_c	29.8 %

ATLAS

Compton College PE Complex Replacement Compton, California			
By:	JRD	Date:	July, 2021
Job No:	10-57575PW	Figure:	III-3

Consolidation Test Results

ASTM D2435

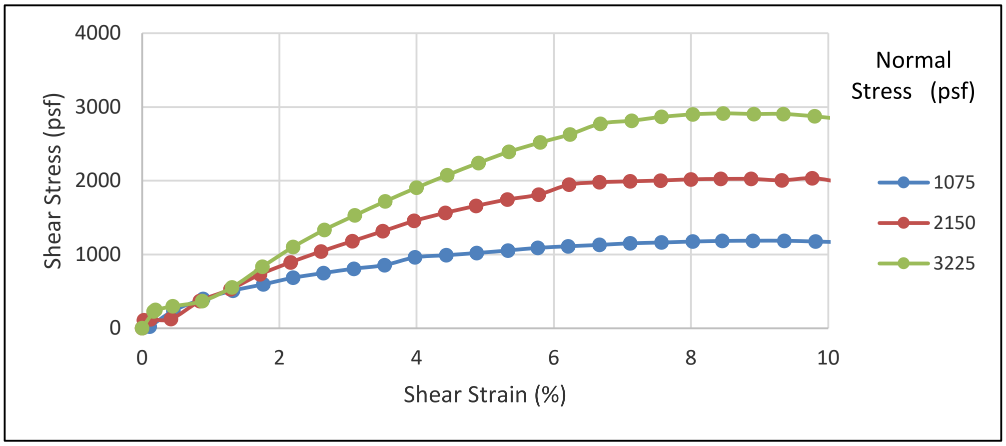


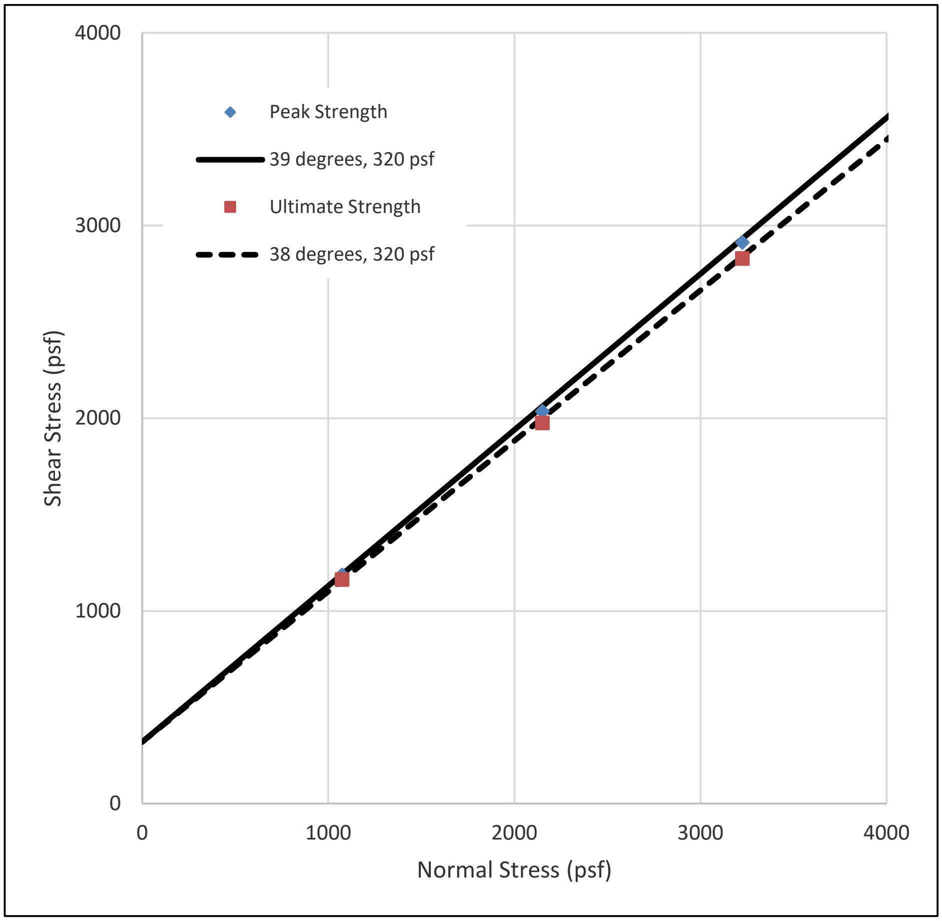
Sample ID: B-11 at 11 to 11½ Feet

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

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

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





SAMPLE ID: B-4 at 11 to 11½ feet

<u>YOUNG ALLUVIUM (Qya)</u>: SILTY SAND

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

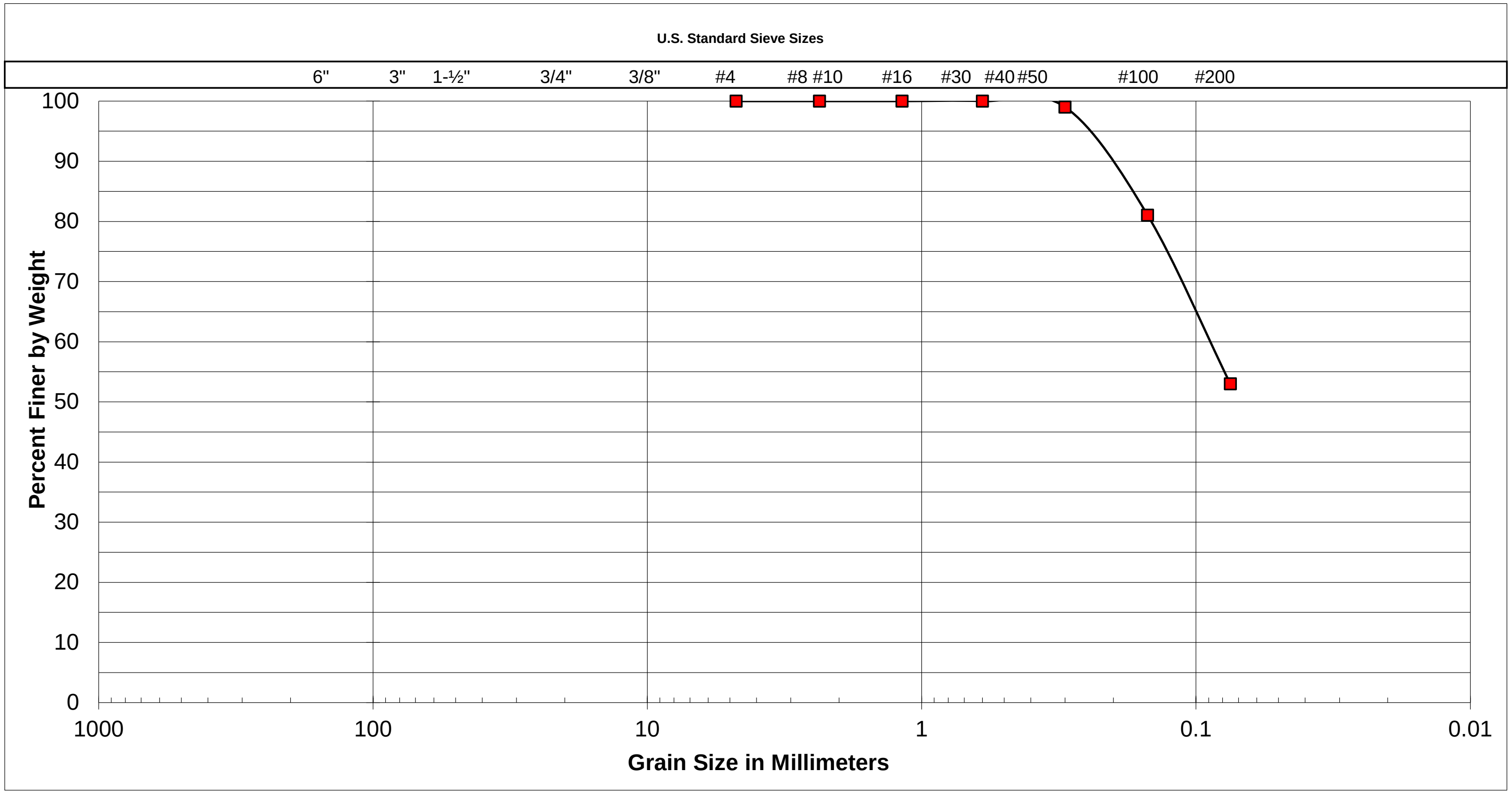
	Peak	Ultimate
Φ	39 °	38 °
c	320 psf	320 psf

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

ATLAS

Compton College PE Complex Replacement
Compton, California

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



Cobbles	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

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

UNIFIED SOIL CLASSIFICATION:	CL
DESCRIPTION	Sandy Lean Clay

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

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

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

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

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

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

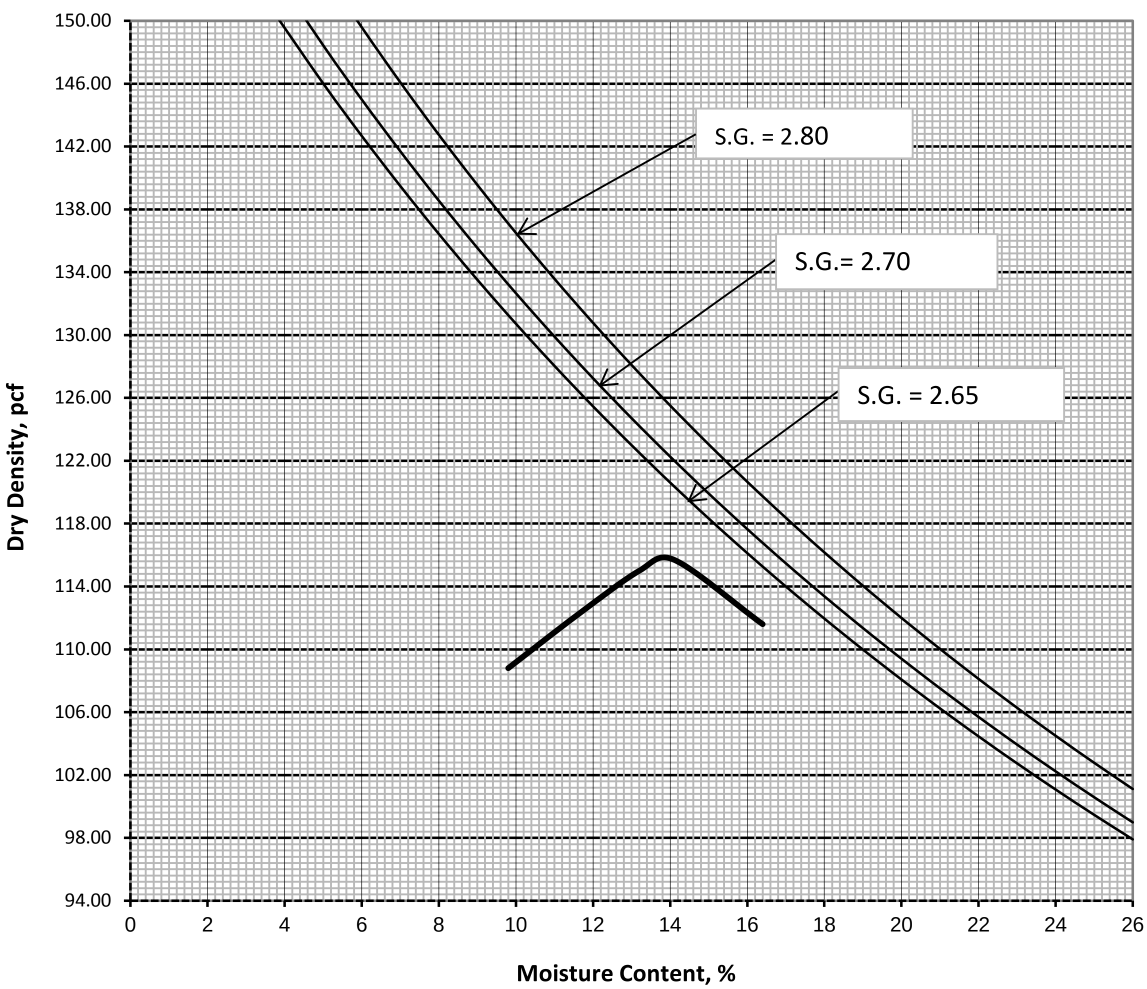
DSA File No.: NA
Dsa App No.: NA

Date: March 12, 2021

Atlas Technical Consultants Project No.: 1057575PW

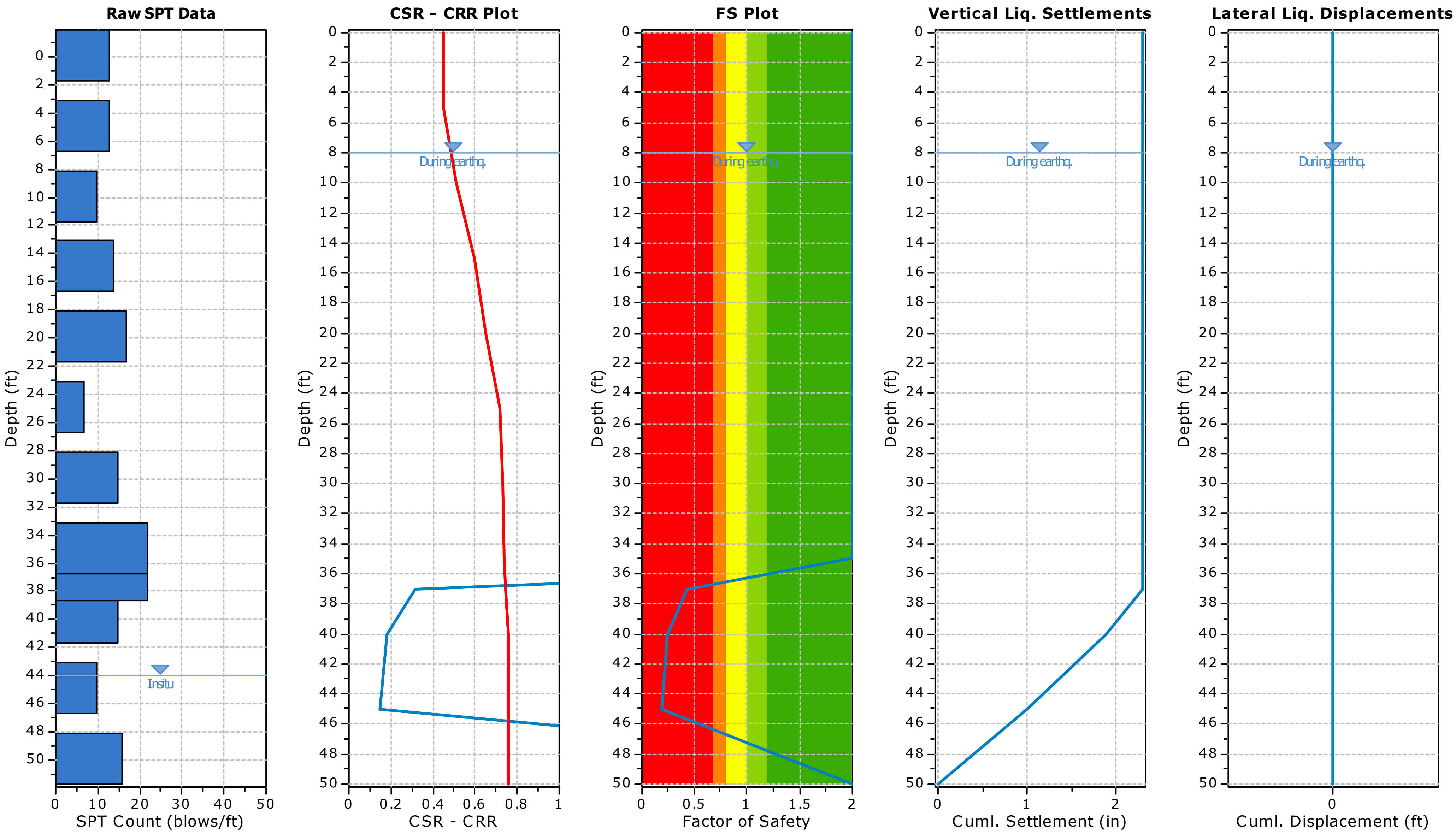
Lab Sample No.:	Sample 1		
Visual Class.:	Brown Silty fine SAND	**Test Results:**	
Sample Source:	B-3 at 0.5 - 3.5 feet	**Maximum Dry Density, pcf:**	115.7
Method of Test:	ASTM D 1557 - Method A	**Optimum Moisture Content, %:**	13.9

Maximum Density - Optimum Moisture Content, ASTM D 1557

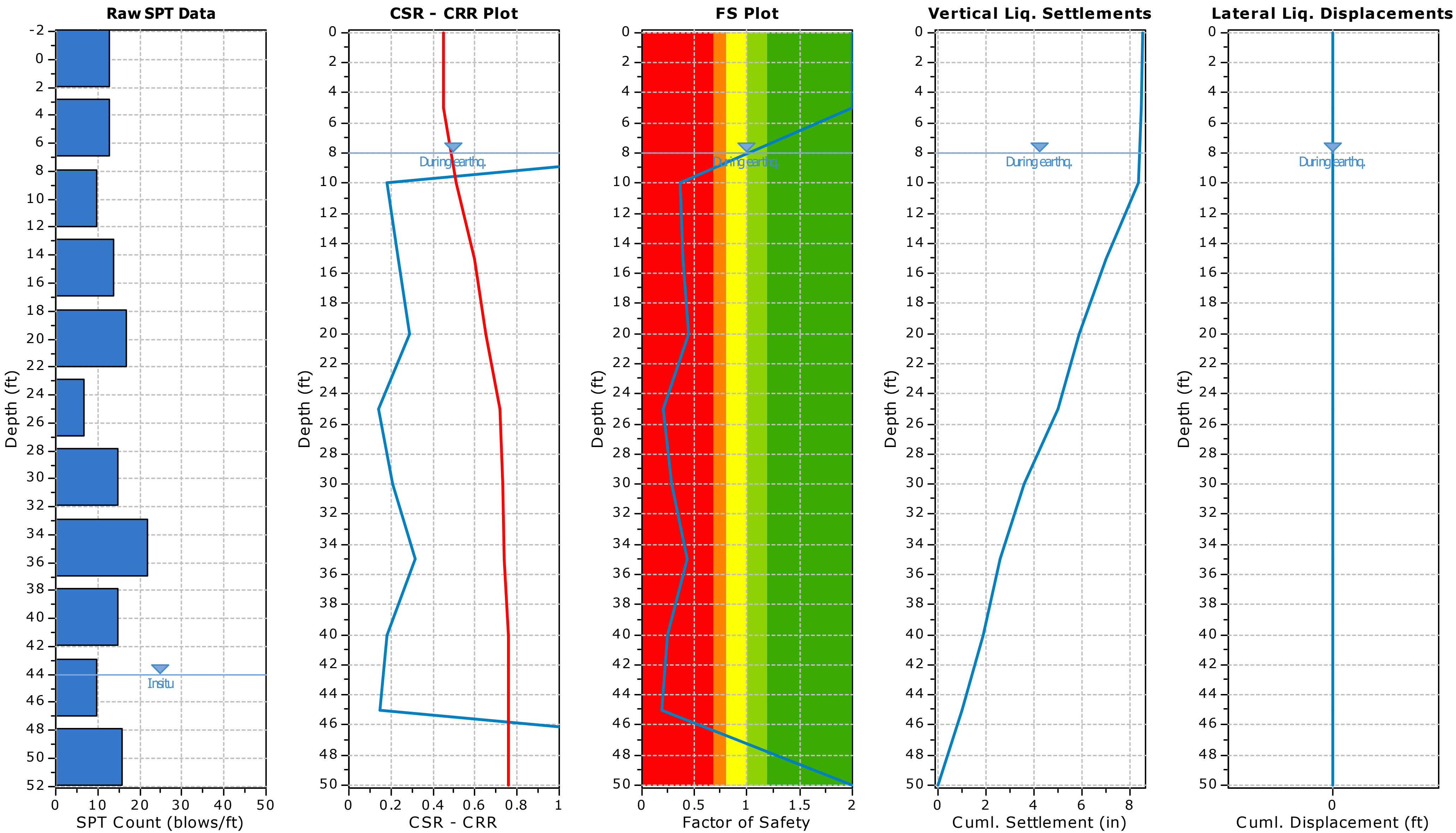


APPENDIX IV
LIQUEFACTION RESULTS

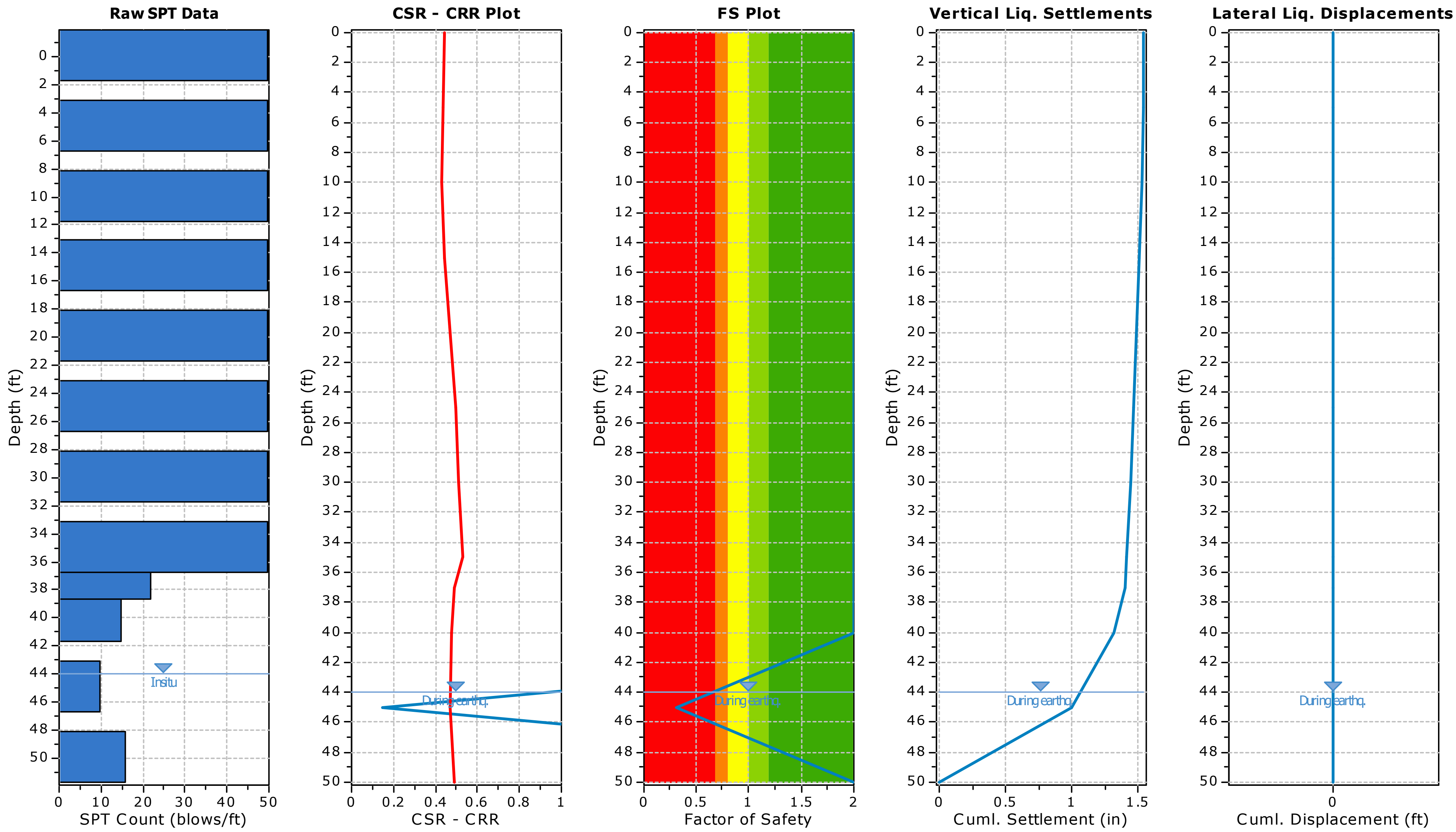
:: Overall Liquefaction Assessment Analysis Plots ::



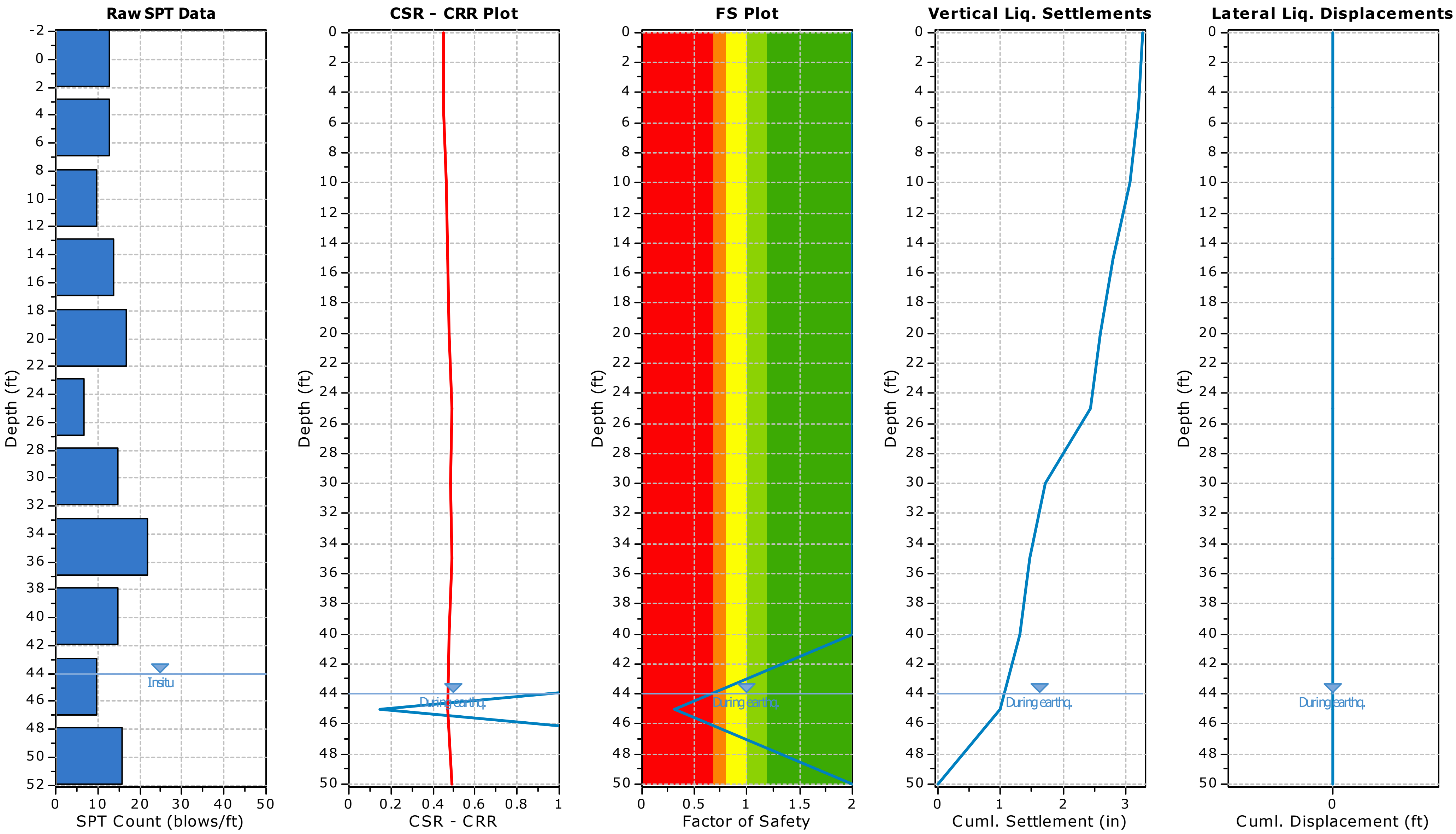
:: Overall Liquefaction Assessment Analysis Plots ::



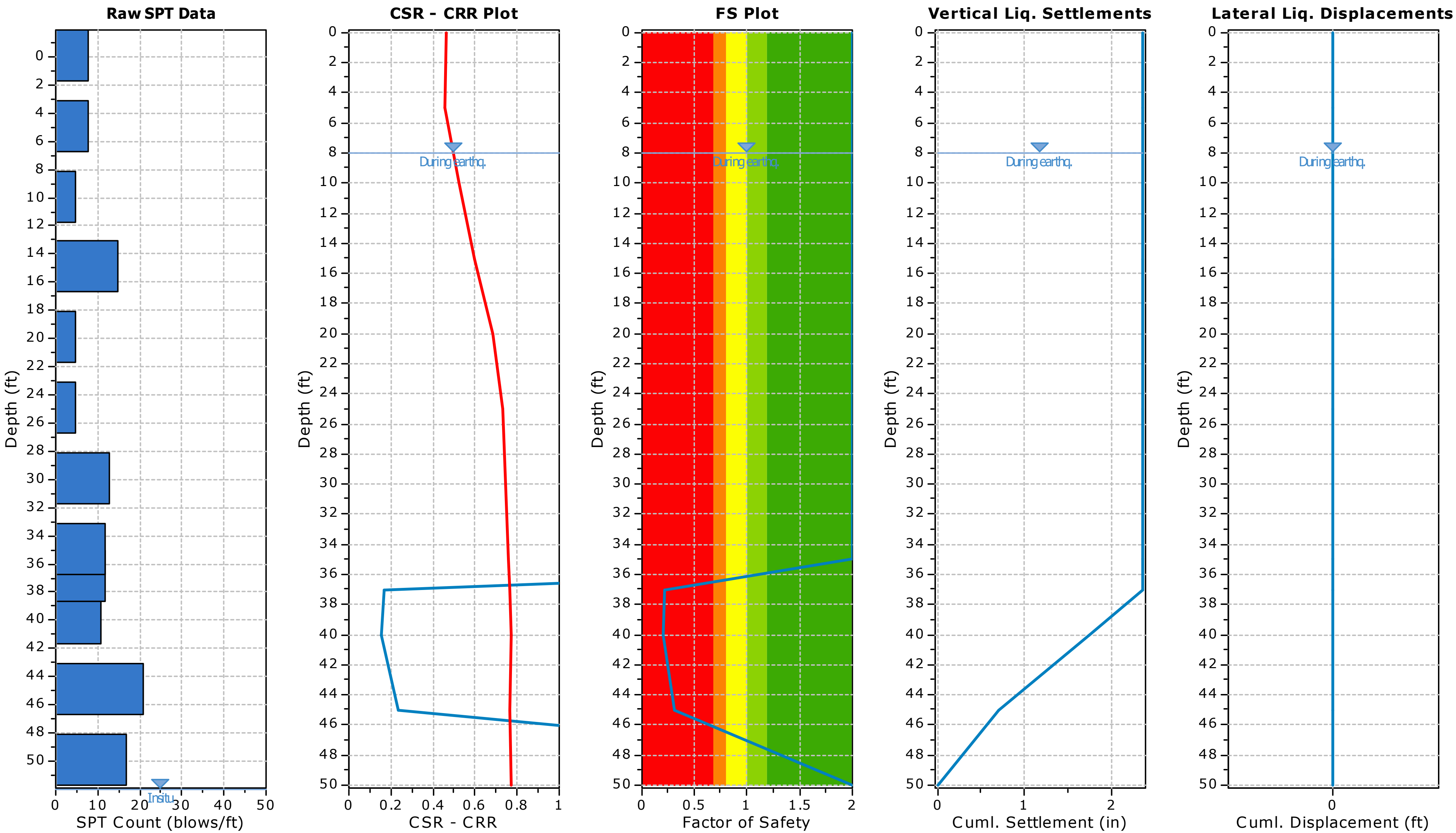
:: Overall Liquefaction Assessment Analysis Plots ::



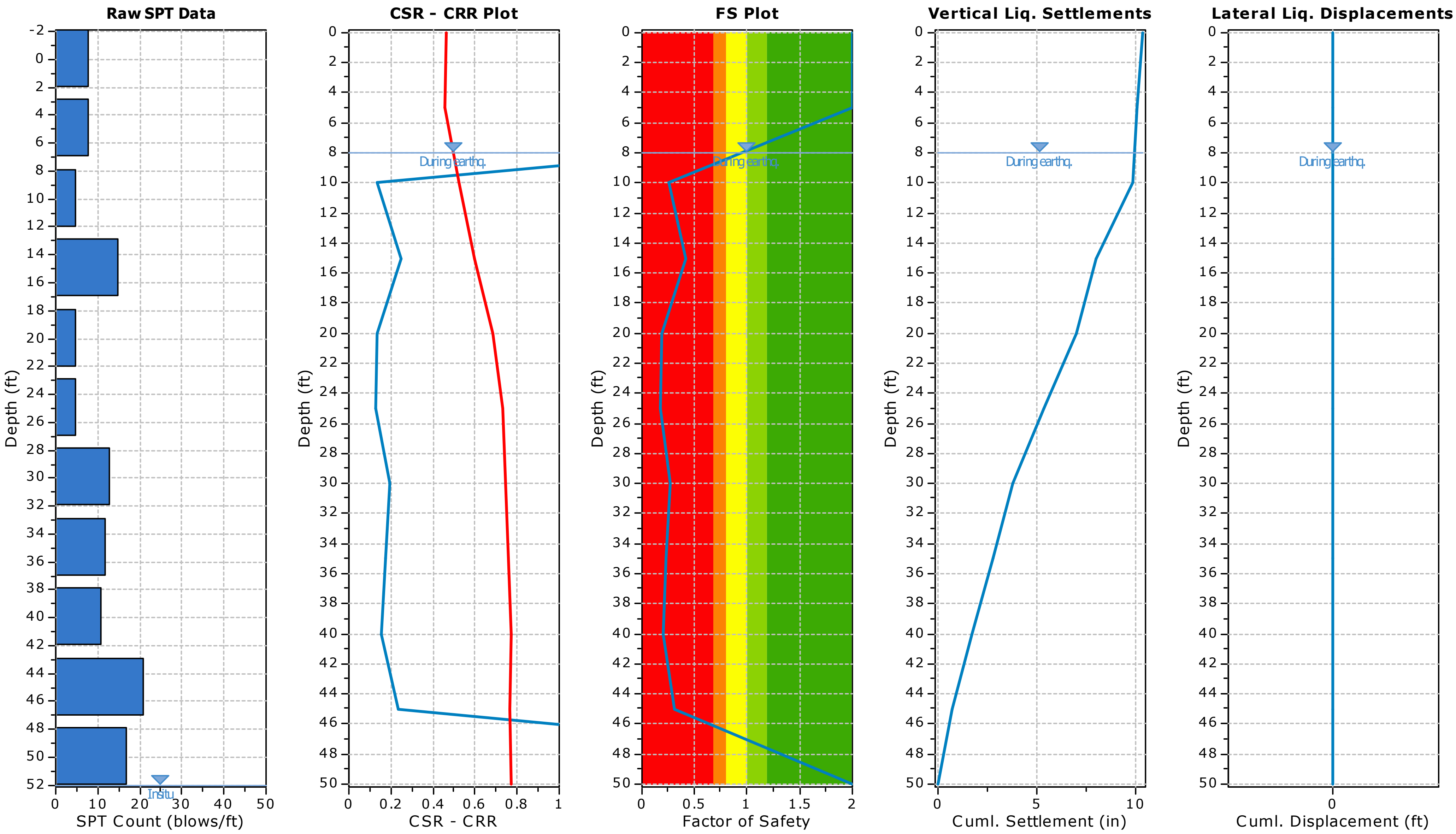
:: Overall Liquefaction Assessment Analysis Plots ::



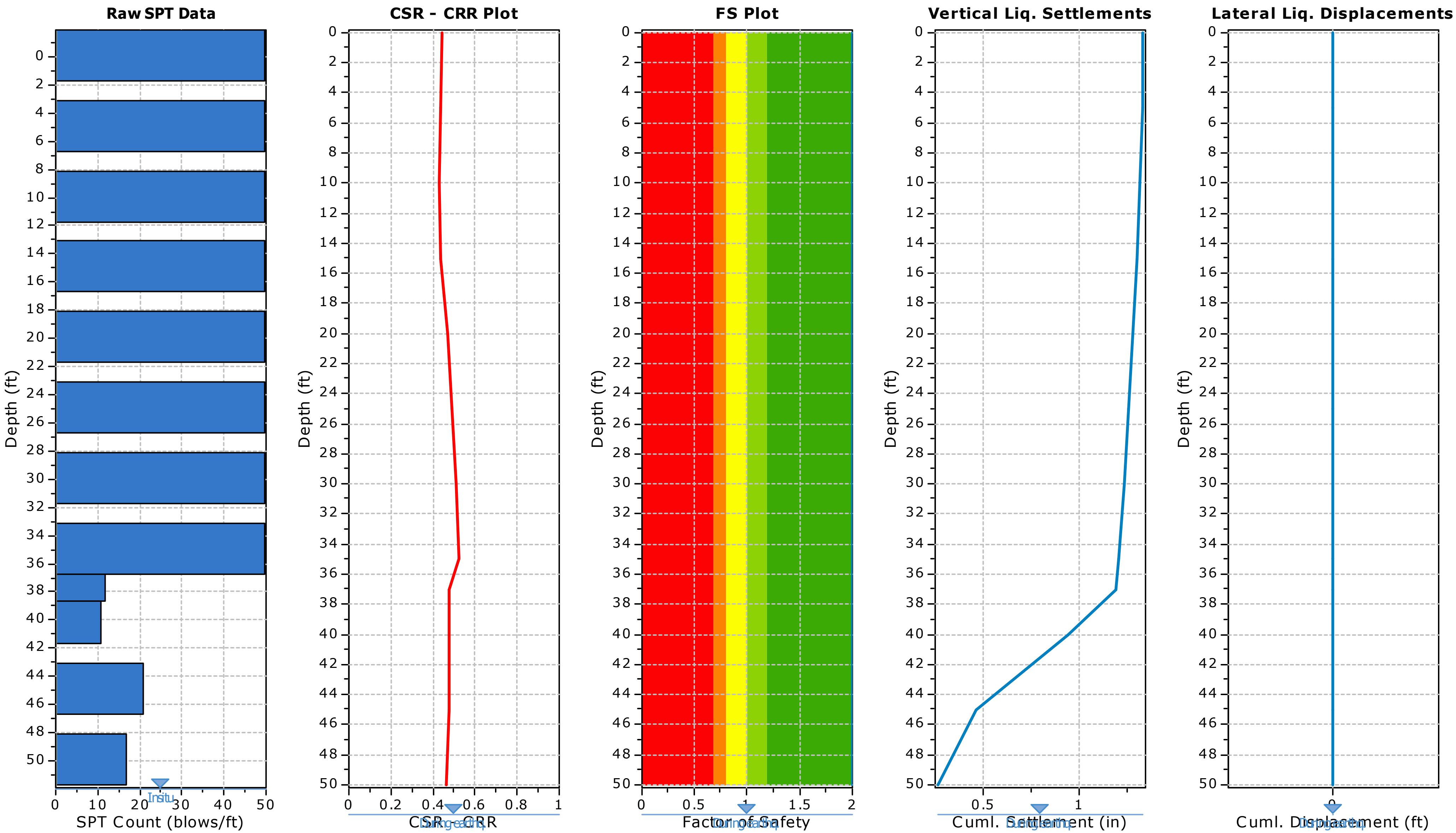
:: Overall Liquefaction Assessment Analysis Plots ::



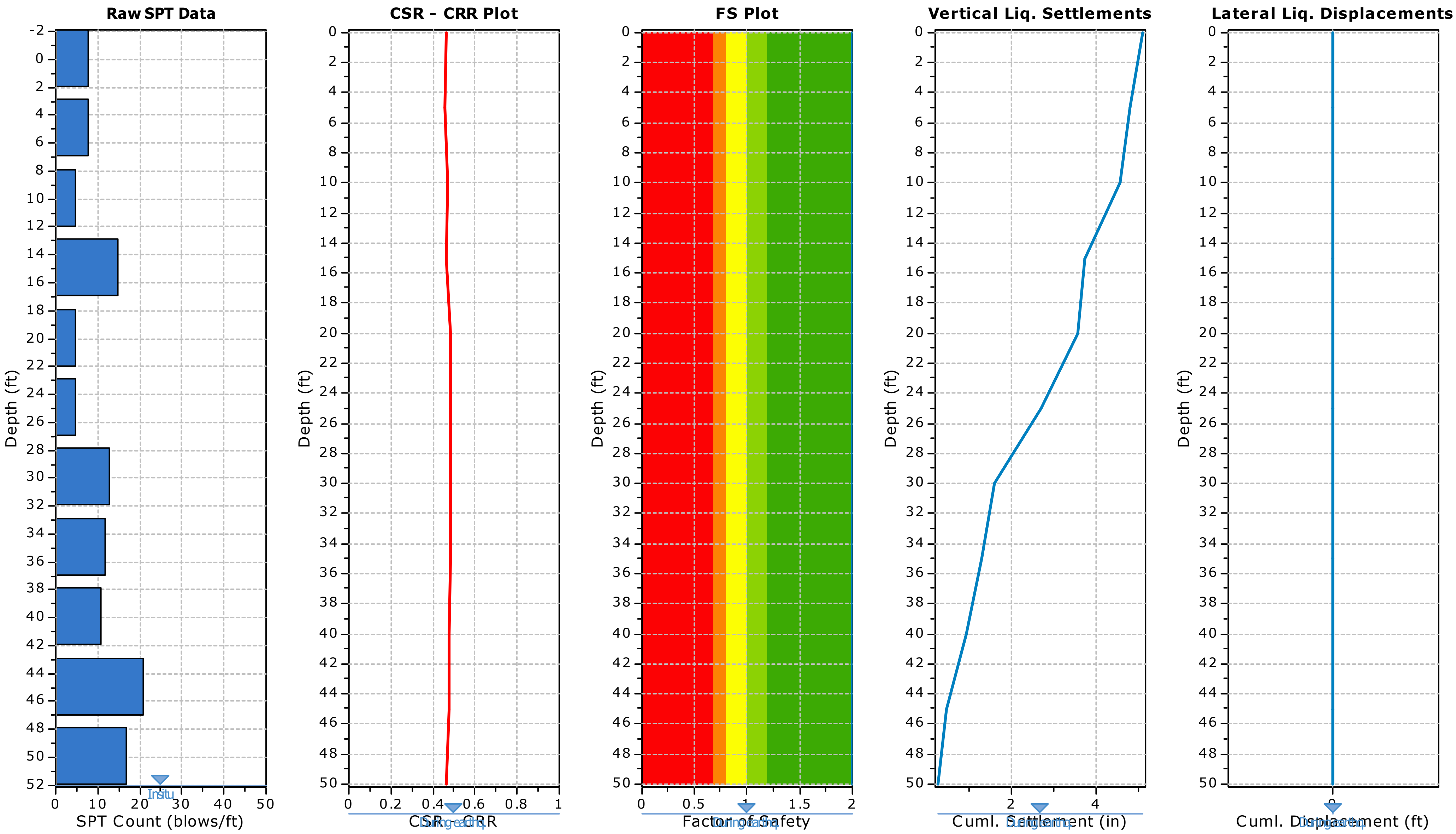
:: Overall Liquefaction Assessment Analysis Plots ::



:: Overall Liquefaction Assessment Analysis Plots ::



:: Overall Liquefaction Assessment Analysis Plots ::



Project title : Compton College 10-57575PW

Location : Compton, California

Figure IV-8

APPENDIX V
SITE-SPECIFIC GROUND MOTION HAZARD ANALYSIS RESULTS

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

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

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

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

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

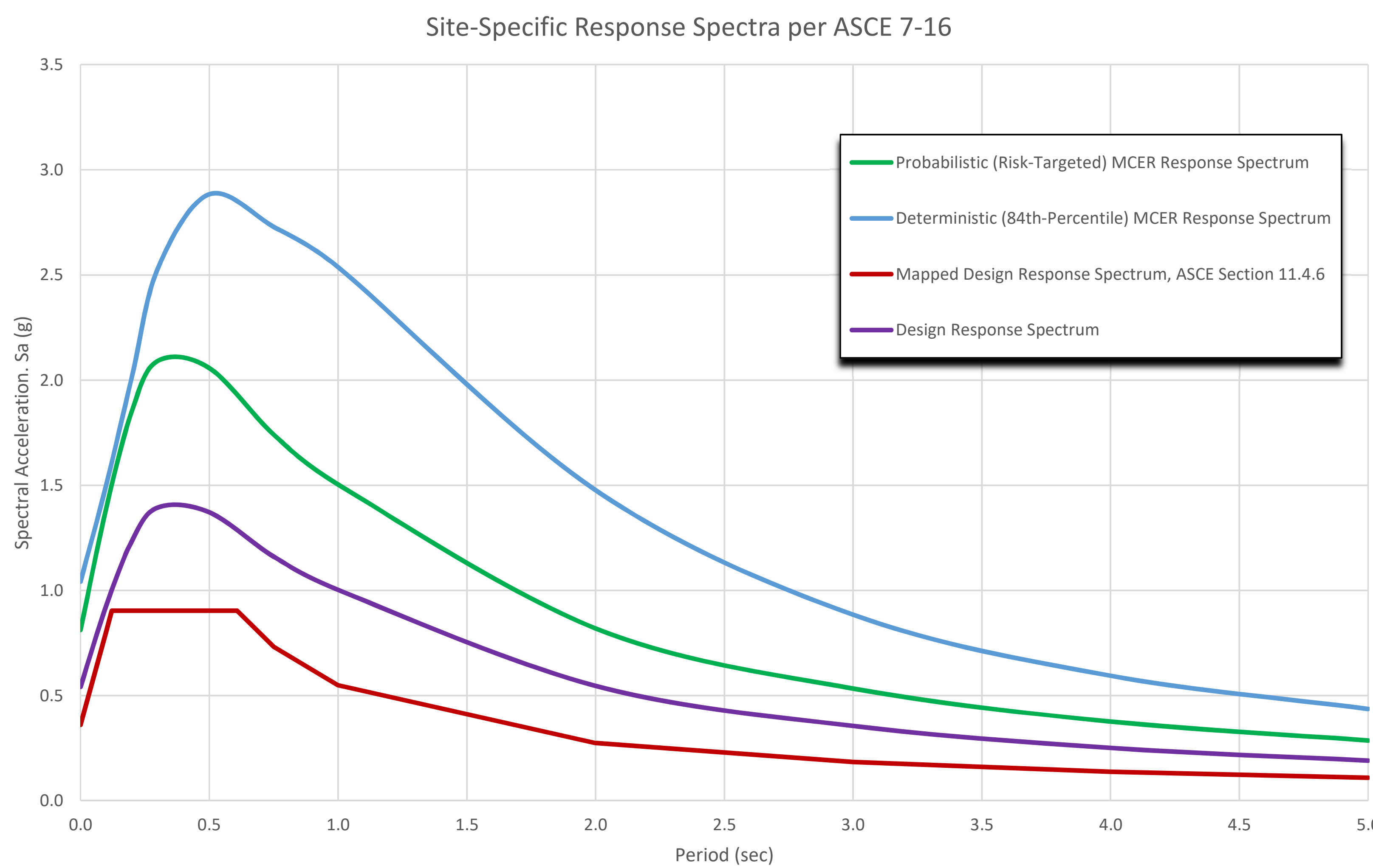
SITE-SPECIFIC DESIGN PARAMETERS

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

ATLAS

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

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



ATLAS

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

APPENDIX VI
INFILTRATION TEST RESULTS

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

Shallow Borehole Percolation Testing Field Log

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

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

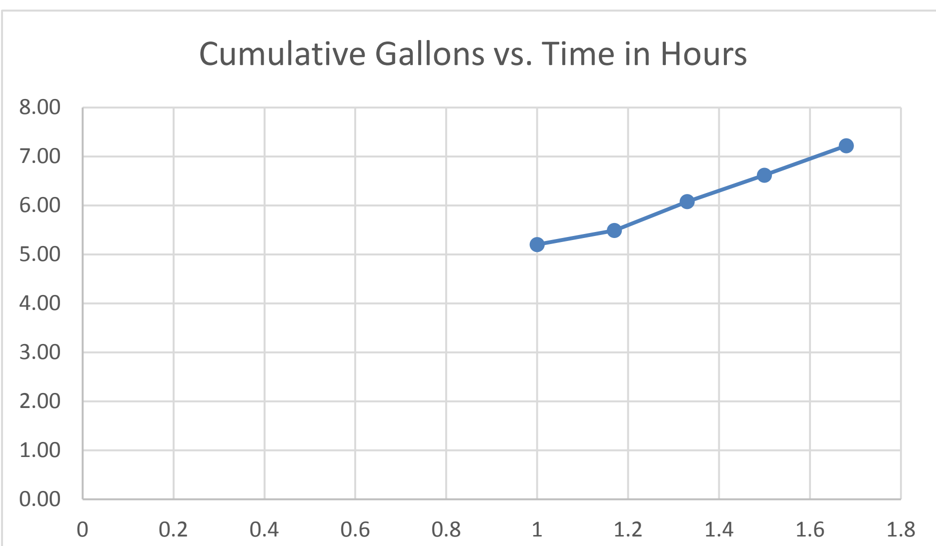
FIGURE VI-1

Shallow Borehole Percolation Testing Field Log

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

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

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



ATLAS

FIGURE VI-2

Deep Borehole Percolation Testing Field Log

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

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

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FIGURE VI-3

Deep Borehole Percolation Testing Field Log

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

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

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

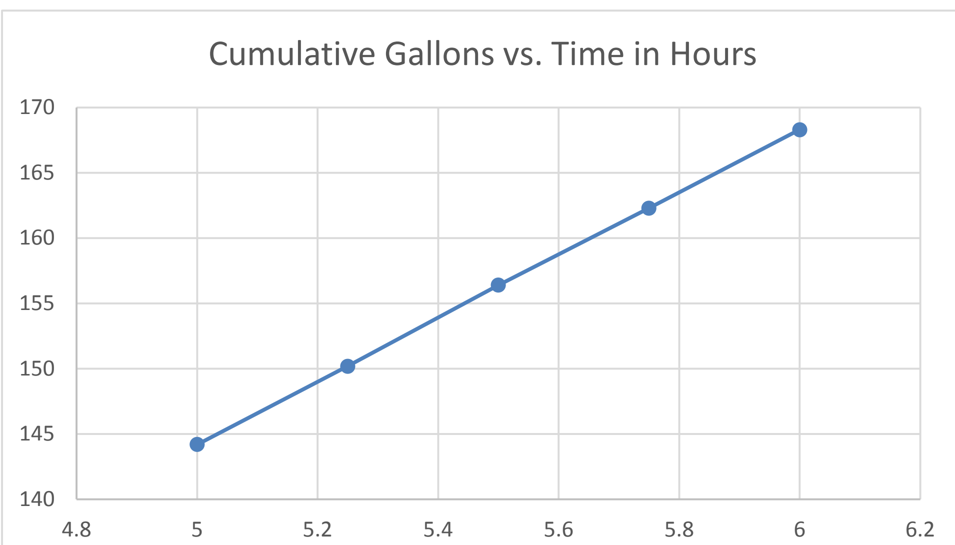

FIGURE VI-4

Deep Borehole Percolation Testing Field Log

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

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

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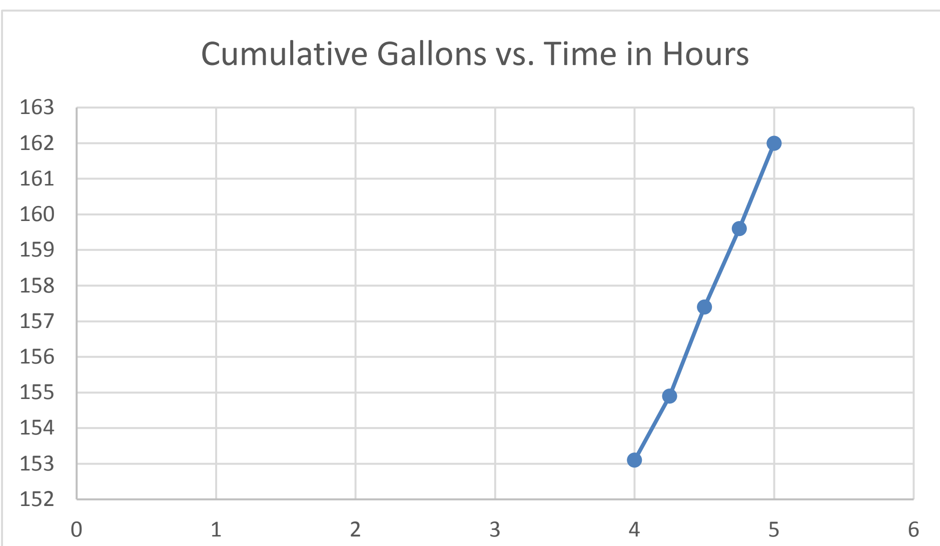
FIGURE VI-5

Deep Borehole Percolation Testing Field Log

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

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

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



ATLAS

FIGURE VI-6

Shallow Borehole Percolation Testing Field Log

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

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

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Shallow Borehole Percolation Testing Field Log

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

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

Raw Flow Rate	0.9	CF/HR
Raw Measured Rate	0.4	FT/HR
Reduction Factors		
Drywell Perc Test	2	
Site Variability	2	
Long-Term Siltation	3	
Total Reduction	12	
Design Infiltration Rat	0.03	FT/HR
	0.38	in/hr

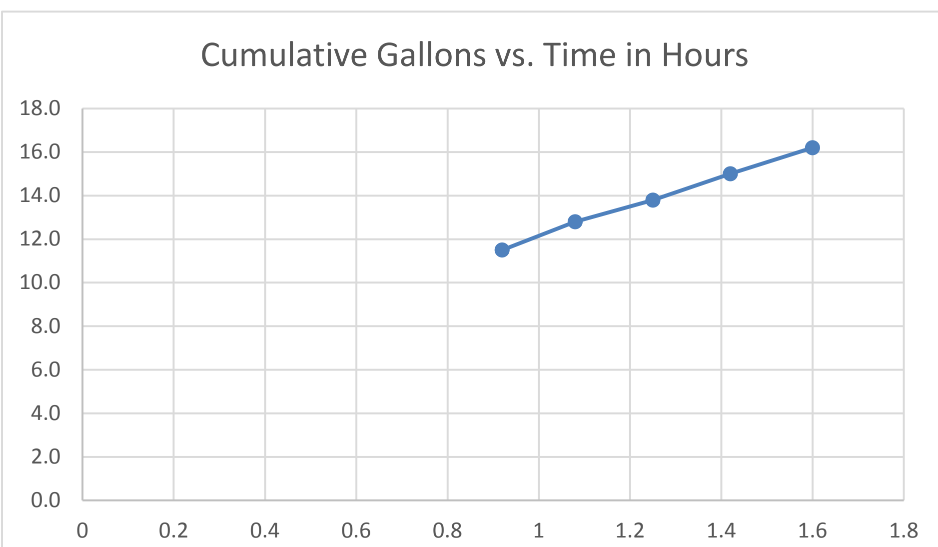


FIGURE VI-8

APPENDIX VII
HISTORIC SEISMIC EVENTS

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

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

Keller North America
17461 Derian Avenue, Suite 106
Irvine, CA 92614
Tel : 909-393-9300
Fax : 909-393-0036

Vibro Stone Columns Design
Physical Education Building of Compton Community College

1111 East Artesia Boulevard
Compton, California

Submitted to:
PCM3, Inc.
Compton CCD Office

Submitted by:
Keller North America

September 9, 2021

Keller North America

PCM3
Compton CCD Office

Attention: Ms. Sheri Phillips
Subject: Vibro Stone Column (VSC) Ground Improvement Design

Keller North America (Keller) is pleased to present the following design submittal for ground improvement for the proposed buildings at this project site. The purpose of the ground improvement program is to enhance the safety, stability, and serviceability of the proposed structures. This is accomplished by increasing the strength of the ground to the point where the ground can safely support the anticipated structures under static loads as well as during and after the design level earthquake. Additional information is provided in the attached report.

The design provided herein has been prepared for the exclusive use of Keller, with the special equipment and production procedure, for our client under the following strict limitations:

1. Only Keller may construct the work described by the design and
2. The design may not be used by others for any purpose.

Keller appreciates the opportunity to be of service. Please feel free to contact the undersigned at (909) 393-9300 with any questions, comments, or concerns.

Respectfully submitted,

REGISTERED PROFESSIONAL ENGINEER
SUNIL ARORA
C72352
EXP. 06/30/22
CIVIL
STATE OF CALIFORNIA

David Chae, Assistant Project Manager	Sunil Arora, P.E. Project Executive	Bailey Uy Engineer

1. DESIGN SUMMARY

This project site is located at 1111 East Artesia Boulevard, Compton, California. Based on our review of the provided documents, the proposed construction is a 2-story Physical Education (PE) building supported by shallow spread footings with slab-on-grade.

Keller North America (Keller) proposes installing Vibro Stone Columns (VSC) to limit the total differential settlement to 2.88 inches over a horizontal distance of 40 feet (0.006 L). These columns shall have a minimum diameter of 36 inches, spaced in a square grid pattern, 8 feet on-center, and extend to a depth of 23 feet below the existing ground surface. The working grade for Keller will be near the existing ground elevation. The densification results will be verified by liquefaction analysis based on post-treatment CPTs. Our shop drawing plans are presented in Appendix A.

2. GROUND IMPROVEMENT DESIGN BASIS

This design is based on Keller's review of the following documents and performance requirements articulated by the project structural, geotechnical, and civil engineers. Although many documents were reviewed, only those which provided information that directly affects our design are listed below:

- Geotechnical Investigation Report, Physical Education Complex Replacement, Compton Community College District, by Atlas Technical Consultants, LLC, dated July 7, 2021
- Addendum Geotechnical and Geohazard Report, Physical Education Complex Replacement, Compton Community College District, by Atlas Technical Consultants, LLC, dated September 7, 2021
- CPT Data – Compton Site, CPT-1, CPT-2, and CPT-5, dated September 3, 2021
- Preliminary Foundation Schemes, by Brandow & Johnston, Inc., dated July 13, 2021

If any of these documents are changed or altered in any way, Keller should be notified, and the design may require modifications.

2.1 Subsurface Conditions

Based on our review of the Geotechnical Investigation Report by Atlas Technical Consultants LLC (Atlas), it is Keller's understanding that the site is generally underlain by about ½ foot of grass/topsoil/surficial fill and young alluvial deposits of Holocene to late Pleistocene age. These alluvial deposits are primarily comprised of inter-layered silty sands and sandy silts. In general, the near-surface sandy soil 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. Per Atlas's Geotechnical Investigation Report, groundwater was encountered at a depth between 44 feet to 52 feet below the existing ground surface.

2.2 Design Criteria

The ground improvement design criteria have been established by the project geotechnical and structural engineers and summarized in Table 1 below. Keller has reviewed the criteria and they appear typical and reasonable for this type of project.

Table 1: Design Input and Performance Criteria

	Criteria	Reference
Groundwater Level (Static)	44’ below grade	Atlas Technical Consultants, LLC
Groundwater Level (Seismic)	8’ below grade	
PGA_M (ASCE 7-16)	0.802 g	
Mw (ASCE 7-16)	7.3	
Depth of Liquefaction Analysis	50 feet	
Allowable Bearing Pressure	4,000 psf	
Total Post-treatment Differential Settlement	≤ 2.88 inch over 40 feet (0.006*L)	Brandow & Johnston, Inc. based on Table 12.13-3 of ASCE 7-16 for Risk Category III building

3. STATIC DESIGN

3.1 Foundation Bearing Capacity

Keller has verified the soil bearing capacity (shown in Table 2) of several typical spread footings based on Preliminary Foundation Schemes by Brandow and Johnston, Inc. Conservatively, Keller used the pre-treatment soil parameters for this computation. In conclusion, the VSC treated soil will provide adequate foundation bearing capacity. Please refer to Appendix B for computation details and the corresponding geometry illustration based the provided structural drawing. The calculated factors of safety meet or exceed the generally accepted minimum factor of safety of 3.

Table 2: Factor Safety against Soil Bearing Capacity Failure

Footing	Size	Bearing Capacity	Factor of Safety
F1	4-ft x 4-ft	4,000 psf	4.2
F2	5-ft x 5-ft		4.2
F3	6-ft x 6-ft		4.2
F4	8-ft x 8-ft		4.2
F5	8-ft x 8-ft		4.3
F6	10-ft x 10-ft		4.3
F7	6-ft x 10-ft		4.3

3.2 Static Settlement Estimation

Keller has estimated the static settlement (Table 3) of the propose building using the data from

CPT-1. Keller used a total (gross) areal load of 250 psf to represent the PE building load over the footprint of 200-ft by 200-ft. Please refer to Appendix C for computation details.

Table 3: Estimated Total Static Settlement

Predicted Post-treatment CPT basis	Computed Static Settlement, inch
CPT-1	0.46

As shown in this section, the computed static performance of the proposed foundation meets the expected design criteria.

4. SEISMIC DESIGN

5.1 VSC Densification Technical Background

The installation of stone columns at this site will seek to mitigate the liquefaction potential by densification, partial replacement, and reinforcement. We are proposing the implementation of vibro stone columns by the "dry bottom feed process". The degree of densification resulting from the installation of vibro stone columns is a function of many factors, including:

- Soil type, silt, and clay content,
- Uniformity of soil gradation,
- Plasticity of the soil,
- Pre-treatment relative densities,
- Vibrator type and energy output,
- Stone shape and durability,
- Stone column area and spacing between stone columns.

Note that soils with more than about 25% fines (passing through #200 sieve) or with 5% clayey particles may NOT be densifiable. To estimate the degree of densification improvement required to meet the liquefaction-induced settlement acceptance criteria (Table 1), Keller will perform liquefaction analysis on post-treatment CPTs.

5.2 Estimation of Densification from VSC

Baez (1995) describes a procedure for the estimation of stone column parameters (column diameter and spacing between columns) required to achieve certain post improvement penetration values in sands and silty sands. Based on this procedure and Keller's proprietary data base we have determined that a 36" diameter VSC at 8' by 8' grid pattern, with an equivalent area replacement ratio of 11% is expected to meet the liquefaction mitigation performance requirements.

Based on Atlas Technical Consultants experience on the project site there may be more variation in the soil profile then what is portrayed in the CPT. Therefore, Keller is using conservative depth of treatment as provided by the project GEOR.

Keller has reviewed the SPT boring data, and the SPT-based liquefaction analysis performed by Atlas Technical Consultants, LLC. Since the resolution of layering obtained from the CPT data is deemed to be more accurate, Keller has proceeded with the CPT-based liquefaction analysis.

Keller has estimated the CPT-based post-treatment liquefaction-induced settlement using triggering method of Robertson (NCEER R&W 1998) with settlement method proposed by Zhang et al. (2002), shown in Table 4. Please refer to Appendix D for detailed computations.

Table 4: Post-Treatment Liquefaction-induced Settlement

Exploration	**Post-treatment Liquefaction Settlement (inch)**
CPT-1	1.39

5.3 VSC Densification Verification

The acceptance criteria of the stone column treatment will be based on verifying densification by means of six (6) post-treatment CPT tests performed by Atlas Technical Consultants, LLC. Please refer to Keller's shop drawing for proposed post-treatment CPT locations.

Post-treatment CPTs shall be located close to (preferably within 10 feet) the pre-treatment CPTs whenever possible, so that Ic from pre-treatment CPTs can be used for post-treatment liquefaction analysis. Ic values after stone column treatment often shift to lower values, suggesting the soil becomes coarser and less plastic. But the stone column treatment does not change the soil type and therefore the original Ic values should be used in liquefaction analyses (Nguyen et al. 2014). This can be achieved by correcting (or shifting) the post-treatment Ic back to the pre-treatment Ic.

CPTs will be performed at the center point between four adjacent stone columns. A minimum of 7 days (preferably 14 days or more if possible) shall pass after installation of stone columns before CPT testing is conducted. This will allow the dissipation of the excess pore water pressure induced by the vibrator.

The CPTs will be analyzed for liquefaction triggering and settlement using the design methods described earlier. If the initial CPTs does not meet acceptance criteria, additional CPTs may be performed later to allow for additional porewater pressure dissipation and aging. Additional CPTs may also be performed to better define the limits of any non-conforming work. If this CPT testing shows area where the post-improvement liquefaction differential settlement is not met, additional stone columns may be installed at locations to achieve the performance specification. Keller may elect to perform its own additional site exploration at any time and for any reason during the course of the project.

5. CONSTRUCTION

Method: VSC technique uses specialty purpose-built depth vibrators to densify and reinforce the soils while constructing a VSC of an average 36-inch diameter. The installation process consists of imparting energy by means of vibrations that are generated close to the tip of the vibrator and are produced by rotating eccentric weights mounted on a shaft. An electric motor turns the eccentric weights. Follower tubes are added to achieve the design depth. The follower tube has visible markings at regular increments that enable measurement of penetration and re-penetration depths. If the vibrator encounters refusal, then the ground improvement design engineer shall review this location to determine if additional work is necessary. Predrilling may be employed with a 24-inch or 30-inch diameter auger. The intent of predrilling is to loosen the soil to increase the penetration rate of vibrator. The depth of predrilling may be up to the designed tip of VSC.

Bottom Feed: For this project, Keller plans to utilize the bottom-feed method of VSC construction. The vibrator will then advance to the design depth and the vibrator is lifted in stages as the stone is fed through a side pipe and expelled at the tip of the vibrator. Installation of VSC by the bottom feed method displaces the ground. Some heave or settlement may occur across the areas worked.

Equipment: Major support equipment anticipated to be utilized for VSC construction are:

- Vibrator Hung Caterpillar 365C excavator
- Drill Rig for pre-drilling
- Generator to power the vibrator
- Air Compressor to push gravel through the follower tube
- Loader to move gravel from stockpile to skip bucket (hanging from crane)
- Keller S23 Bottom Feed Vibrator System

Following VSC installation, excess material shall be removed by others. A minimum of the top 12-24 inches disturbed soil shall be excavated with compacted engineering fill by others. Geotechnical Engineer of Record may elect to use onsite material within the treated area for scarification and re-compaction. General Contractor will perform building examinations of adjacent buildings and monitoring of adjacent buildings, as needed.

6. QA/QC

Keller will supply a full-time quality control (QC) representative during our VSC installation. The quality control representative will observe all pertinent data with respect to the installation. This information includes but is not limited to the depth, the approximate amount of stone introduced into the cavity and the amperage drawn by the vibrator during installation. Attention is required to ensure that Keller is getting adequate amperage (average peak of approximately 160A) while constructing the columns and maintenance of the average theoretical diameter of 36 inch. The average diameter of the column is calculated from the stone volume utilized for the respective column. One loader bucket holds approximately 2 cubic yards of material. Depth of the column will be checked with the markings on the vibrator.

It is common for the owner or general contractor to supply an independent inspection agency to observe our installation. The following guidelines are intended to aid any 3rd party quality assurance (QA) representatives in their inspections:

Location: Each VSC will have a designated number indicated on our shop drawings. The VSC should be located in the field by the field engineer using pin flags with numbers corresponding to those shown on the shop drawings. The center of installed VSC shall be within 6 inches of the design location.

Depth of Treatment: Markings along the shaft of the vibrator assembly indicate the depth of penetration. The drill rigs may also have depth indicators that will be verified periodically. Inspection personnel should not approach an open hole or operated machinery without first obtaining the permission of Keller's field superintendent.

Amperage: Amperage is a measure of electrical current. The amperage drawn by the vibrator during installation is a measure of the amount of compaction effort that has been applied to the stone and surrounding soil matrix. More precisely, the amperage draw is a direct measure of current required by the electric motor of the vibrator to keep the system in equilibrium. The higher the current, the more the resistance of the particles around the vibrator tip. In general, high amperage readings indicate a high degree of compaction and stiff matrix soils, while very low amperage readings indicate that the matrix soils are less dense, and a lower degree of compaction is achieved within the stone. Very high amperages should not be maintained for long periods of time, as this can cause vibrator damage.

Materials: Aggregates used for VSC construction shall consist of clean coarse aggregate conforming to the gradation specified in Table 5. Crushed concrete materials from demolition of an existing structure may be substituted with approval of the Keller ground improvement design engineer. The material shall have a minimum durability index of 40 when tested in accordance with California Test Method 229.

Table 5: Aggregate Gradation Requirement

Sieve Size	Percentage Passing
2"	100
1"	90-100
½"	5-80
No.4	0-3

7. SHOP DRAWINGS

Our shop drawing in **Appendix A** depicts our proposed soil improvement plans of VSC for the proposed Physical Education Building. An As-Built Drawing with any field changes will be provided upon completion of VSC work.

8. REFERENCES

Baez, J.I. and G.R. Martin (1993) "Advances in the Design of Vibro Systems for the Improvement of Liquefaction Resistance," Symposium of Ground Improvement, Vancouver Geotechnical Society, Vancouver, B.C.

Baez J.I. (1995) "A Design Model for the Reduction of Soil Liquefaction by Vibro-Stone Columns," Ph.D. Dissertation, University of Southern California.

Boulanger, R. W., and I. M. Idriss. "CPT and SPT based liquefaction triggering procedures." Report No. UCD/CGM.-14 1 (2014).

Cetin K.O., Bilge H.T., Wu J., Kammerer A. and Seed R.B., [2009]. "Probabilistic Models for Cyclic Straining of Saturated Clean Sands." J. Geotech. and Geoenv. Engrg., 135[3], 371-386

Martin, G.R. and M. Lew (1999) "Recommended Procedures for Implementation of DMG Special Publication 117 –Guidelines for Analyzing and Mitigating Liquefaction in California-" Southern California Earthquake Center.

Nguyen, Thang V., Lisheng Shao, James Gingery, and Peter Robertson. "Proposed modification to CPT-based liquefaction method for post-vibratory ground improvement." In *Geo-Congress 2014: Geo-characterization and Modeling for Sustainability*, pp. 1120-1132. 2014.

Rayamajhi, Deepak, Scott A. Ashford, Ross W. Boulanger, and Ahmed Elgamal. "Dense granular columns in liquefiable ground. I: shear reinforcement and cyclic stress ratio reduction." Journal of Geotechnical and Geoenvironmental Engineering 142, no. 7 (2016): 04016023.

Youd, T.L. and I.M. Idriss (1997) "Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils," NCEER Technical Publication 97-0022.

Zhang, G., P. K. Robertson, and R. WI Brachman. "Estimating liquefaction-induced ground settlements from CPT for level ground." Canadian Geotechnical Journal 39, no. 5 (2002): 1168-1180.

Appendix A
Keller Shop Drawing

COMPTON COMMUNITY COLLEGE (PHASE 1)

Underground Service Alert of Southern California

Call: TOLL FREE 800-422-4133

TWO WORKING DAYS BEFORE YOU DIG

RE-NOTIFY EVERY TWO WEEKS

TICKET # ______________

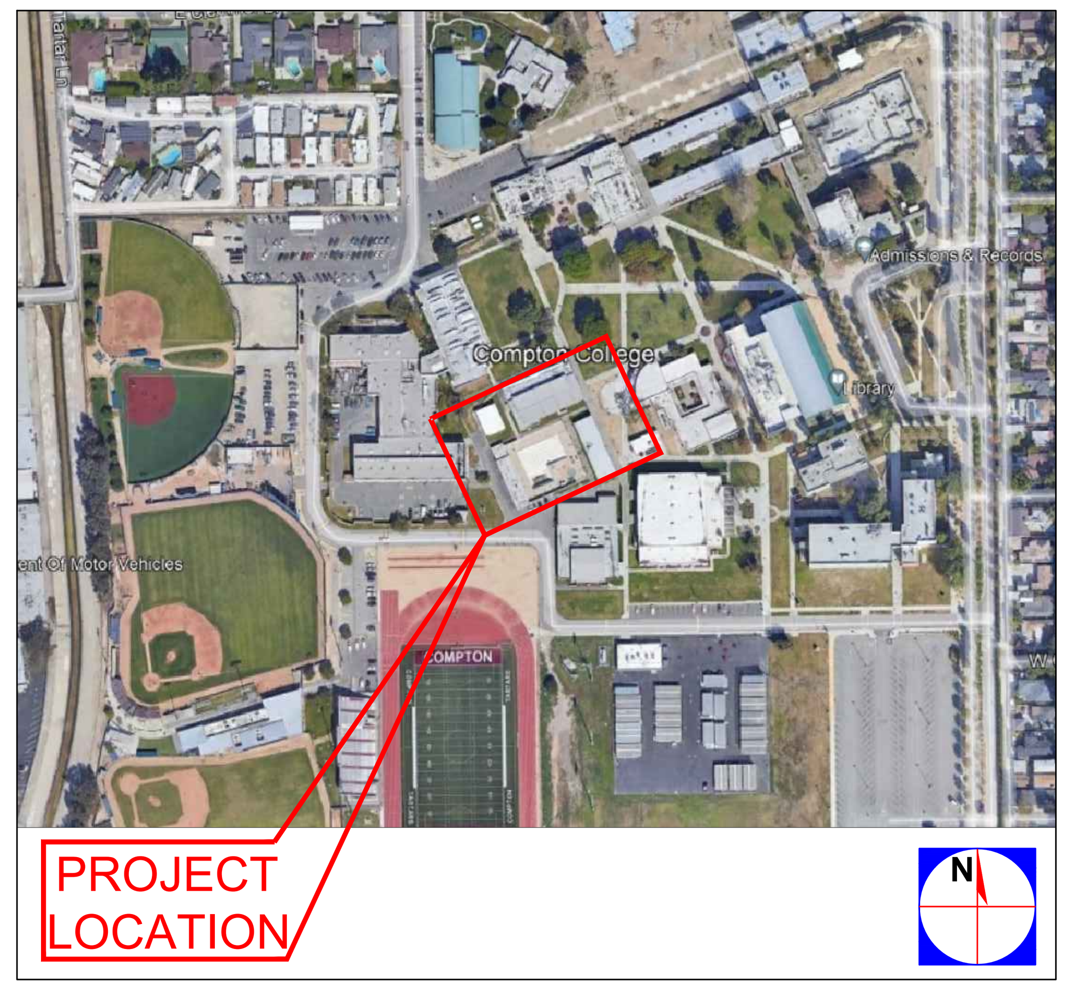
DATE CALLED: ______________

VIBRATORY STONE COLUMNS (VSC)

SHEET INDEX

KNA-1	TITLE SHEET
KNA-2	STONE COLUMN NOTES AND DETAILS
KNA-3	OVERALL GROUND IMPROVEMENT PLAN

VICINITY MAP



PROJECT ADDRESS

1111 EAST ARTESIA BOULEVARD

SACRAMENTO, CALIFORNIA

95811

GEOTECHNICAL NOTES

Commentary

Keller North America (KNA) has been contracted to design and construct Stone Columns to support the foundation of the proposed buildings. This design submittal is as on the following information:

1. Geotechnical Investigation Report, Physical Education Complex Replacement, Compton Community College District, prepared by Atlas Technical Consultants, LLC, dated July 7th, 2021.
2. Addendum Geotechnical and Geohazard Report. Physical Education Complex Replacement, Compton Community College District, prepared by Atlas Technical Consultants, LLC, dated September 7th, 2021.
3. CPT Data - Compton Site, CPT-1, CPT-2, and CPT-5, dated September 3rd, 2021.
4. Preliminary Foundation Schemes, prepared by Brandow & Johnston, Inc., dated July 13th, 2021

*If any of these documents are changed or altered in any way, KNA may need to modify our design.

KELLER

17461 DERIAN AVENUE
SUITE 106
IRVINE, CALIFORNIA
92614

WWW.KELLER-NA.COM

Engineer of Record:

REGISTERED PROFESSIONAL ENGINEER SUNIL ARORA C72352 EXP. 06/30/22 CIVIL STATE OF CALIFORNIA

TITLE SHEET

COMPTON COMMUNITY COLLEGE

1111 E. ARTESIA BLVD.
COMPTON, CALIFORNIA 90221

No.	Description	Date
A	FOR REVIEW	09/08/2021

Design by: DC | Approved by: SA

Drawn by: SPB | Checked by:

Project Number: | Date: 09/08/2021

Sheet No. KNA-1

Scale: N/A

GROUND IMPROVEMENT GENERAL NOTES:

1. Keller North America (KNA) will be provided with a stable and relative level working surface. The working surface shall be constructed and manage by others such that KNA equipment can efficiently access and travel the site. KNA is not responsible for returning the site to its original grade or condition.
2. The GENERAL CONTRACTOR shall confirm that the proposed operation does not conflict with future improvement such as structural, mechanical, plumbing, and electrical prior to installation.
3. An underground service alert must be obtained 2 days before starting work.
4. All permits shall be procured and paid for by the GENERAL CONTRACTOR.
5. KNA will provide a qualified full-time quality control (QC) representative. This representative is titled KNA superintendent, foreman, or KNA field engineer. Third party testing or inspection shall be provided by others.
6. Stone columns shall be installed to design depth or auger/vibrator refusal.
7. Locating, protecting and rerouting/removal of all utilities are the responsibility of others. KNA is not responsible for damage to existing utilities.
8. After the completion of ground improvement work, others are responsible for protection of the work. Proper site drainage to prevent ponding of water at the area of the stone columns and control coordination of earthwork activities shall be managed such that existing stone columns are not damaged.
9. The stone column locations shown on these drawings are only for ground improvement layouts. These plans should not be used for foundation layout. Refer to the “for construction” structural package for specific foundation dimensions and locations.
10. Foundations shall not be poured until a final verification and approval by the project Geotechnical Engineer of Record.
11. Alternate structural shapes, material, and details cannot be used unless reviewed and approved by the ground improvement engineer.

GROUND IMPROVEMENT SPECIFIC NOTES:

1. The Ground Improvement Engineer is the registered professional engineer whose stamp is on these drawings.
2. KNA has relied on various documents for design. Those documents are listed in the ground improvement design report submitted with these drawings. These drawings should not be separated from that design report. If any of the reference documents listed in KNA's design report change in any way, KNA shall be notified and provided the opportunity to review the proposed changes and update this design as needed.
3. KNA must be notified immediately if information included in these plans or in the ground improvement design report conflicts with the project structural or architectural drawings.
4. The stone column properties used for design are provided in the design report.
5. A licensed surveyor (provided by CONTRACTOR) will accurately stake and identify to KNA the site control points for proper stone column layout by KNA, and show on the plans before installation begin. All stone columns shall be installed within six (6) inches of the location shown on these plans, unless otherwise approved by the Ground Improvement Design Engineer. KNA retains the sole authority to modify stone column locations due to constructability and site constraints. KNA shall provide an accurate method to allow the inspector to verify the as-built location of the stone columns during construction.
6. Practical stone column refusal is defined as failure to penetrate 1 foot in 1 minute by vibrator below groundwater table.
7. The stone column design shall be verified using the methods described in the stone column verification testing notes.
8. Stone columns shall be constructed to a nominal diameter of 36 inches. Some variability of this diameter should be expected due to variations in soil density.
9. The ground improvement engineer is the registered professional engineer whose stamp is on these drawings.

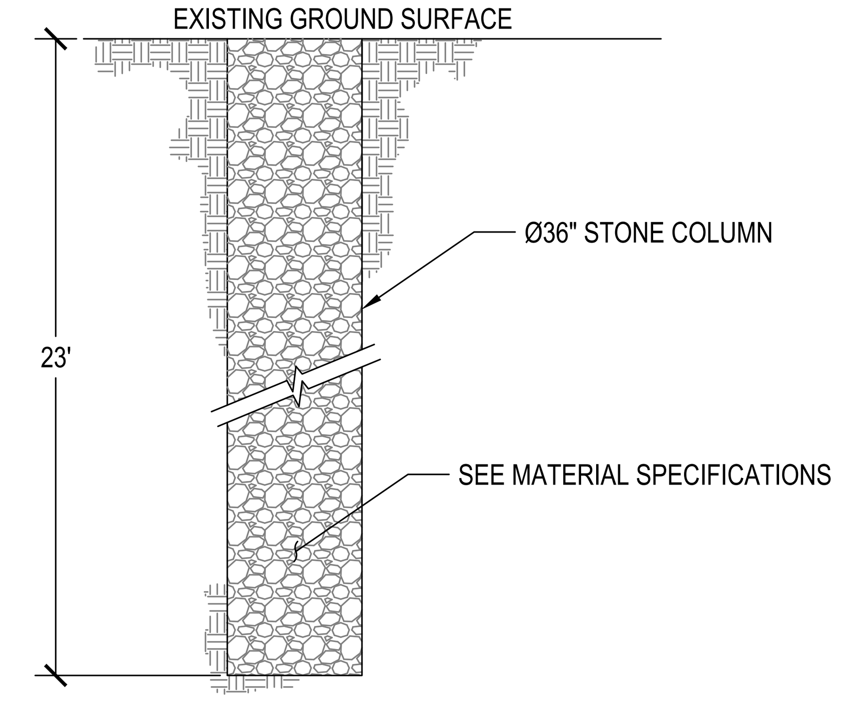
STONE COLUMN VERIFICATION NOTES:

1. The acceptance criteria of the Stone Column Treatment will be based on verifying densification by means of six (6) post-ground improvement CPT tests performed by Atlas Technical Consultants, LLC.
2. Post-improvement CPTs shall be located close to (preferably within 10 ft) the pre-construction CPTs whenever possible. CPTs will be performed at the center point between two adjacent stone columns.
3. A minimum of 7 days (preferably 14 days or more if practicable) shall pass after installation of stone columns before CPT testing is conducted. This will allow the dissipation of the excess pore water pressure induced by the vibrator.
4. The CPT's will be analyzed for liquefaction triggering and settlement using the design methods described in the stone column design report.
5. If the initial CPTs do not achieve the maximum settlement criteria, additional CPTs may be performed that allow for additional pore water pressure dissipation and aging. Additional CPTs may also be performed to better define the limits of an non-conforming work. After CPT testing is complete, additional stone columns may be installed at locations that to achieve the performance specification.
6. KNA may elect to perform its own additional site exploration at anytime and for any reason during the course of the project.

MATERIALS:

1. Aggregates used for VRSC construction shall consist of clean coarse aggregate conforming to the gradation specified below.
2. Crushed concrete materials from demolition of an existing structure maybe substituted with approval of the Keller Ground Improvement Design Engineer. The material shall have a minimum durability index of 40 when tested in accordance with California Test Method 229.

Sieve Size	Percentage Passing
2"	100
1"	90-100
1/2"	5-80
No. 4	0-3

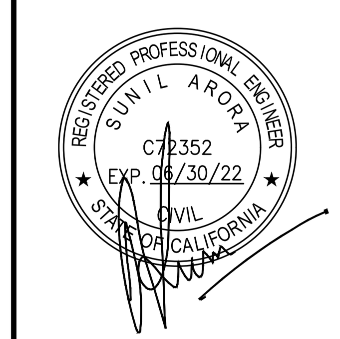


TYPICAL STONE COLUMN SECTION
NOT TO SCALE

KELLER
17461 DERIAN AVENUE
SUITE 106
IRVINE, CALIFORNIA
92614

WWW.KELLER-NA.COM

Engineer of Record:



STONE COLUMN NOTES AND DEATILS

COMPTON COMMUNITY COLLEGE

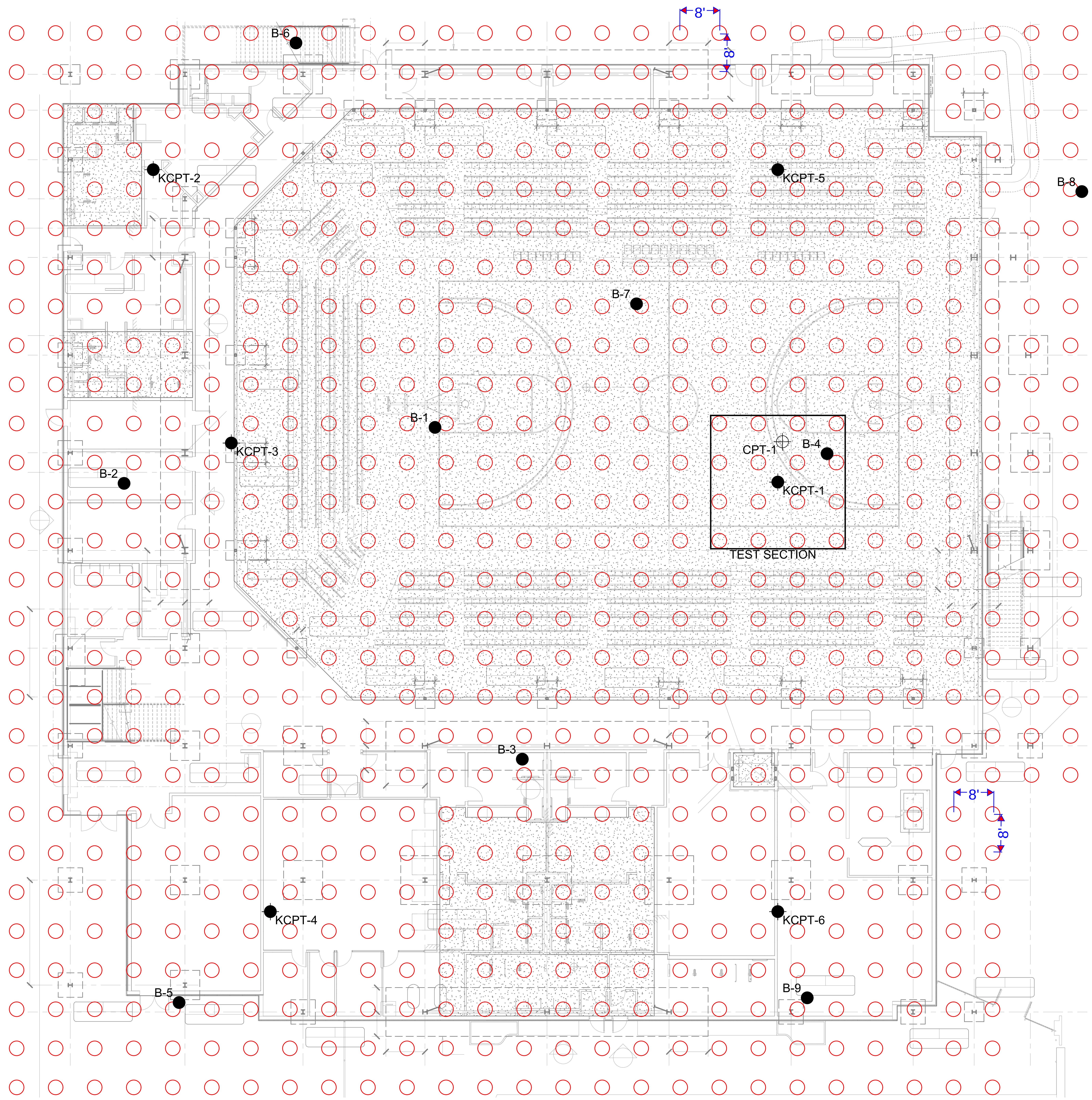
1111 E. ARTESIA BLVD.
COMPTON, CALIFORNIA 90221

No.	Description	Date
A	FOR REVIEW	09/08/2021

Design by: DC	Approved by: SA
Drawn by: SPB	Checked by:
Project Number:	Date: 09/08/2021

Sheet No.
KNA-2

Scale: N/A



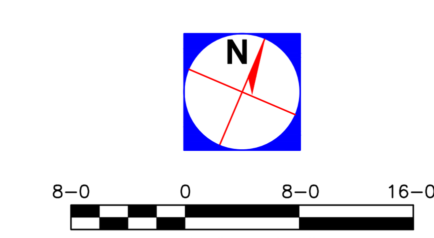
LEGEND:

Ø36" Vibratory Stone Columns (VSC).
8' x 8' spacing, Install 23' from the existing ground surface
Approximate existing ground surface elevation: 55 feet.

CPT-X Existing CPT location

B-X Existing Boring location

KCPT-X Proposed Post-Treatment CPT locations

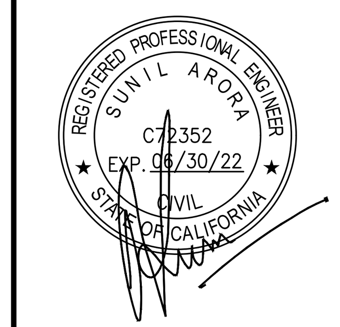


KELLER

17461 DERIAN AVENUE
SUITE 106
IRVINE, CALIFORNIA
92614

WWW.KELLER-NA.COM

Engineer of Record:



OVERALL GROUND IMPROVEMENT PLAN

COMPTON COMMUNITY COLLEGE

1111 E. ARTESIA BLVD.
COMPTON, CALIFORNIA 90221

No.	Description	Date
A	FOR REVIEW	09/08/2021

Design by: DC | Approved by: SA

Drawn by: SPB | Checked by:

Project Number: | Date: 09/08/2021

Sheet No.

KNA-3

Scale: 3/32" = 1'

Appendix B
Foundation Bearing Capacity Check

VIBRO PIER GROUP BEARING CAPACITY CALCULATION

Project name	Compton Community Coll
Project location	
Project number	OP0013298
Date	9/7/2021
Designed by	MBU
Reviewed by	

Input Parameters

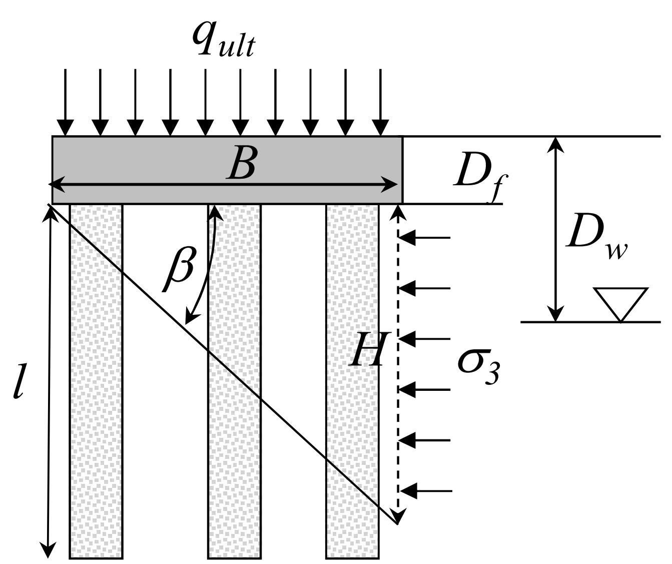
Strip or Square	-	Square	
Footing width	B	4	ft
Footing length	L	4	ft
Depth of embedment	D_f	2	ft
Area replacement ratio	ARR	0.00	
Adjacent surcharge	σ_{sur}	0	psf
Water table depth	D_w	45	ft
Moist unit weight	γ_{moist}	120	pcf
Saturated unit weight	γ_{sat}	120	pcf
Soil friction angle	ϕ_{soil}	0	°
Soil cohesion	c_{soil}	2500	psf
Stress concentration factor	n	0	
Soil elastic modulus (at H/2)	E	250,000	psf
Stone friction angle	ϕ_{stone}	45	°
Treatment Depth	l	23	ft

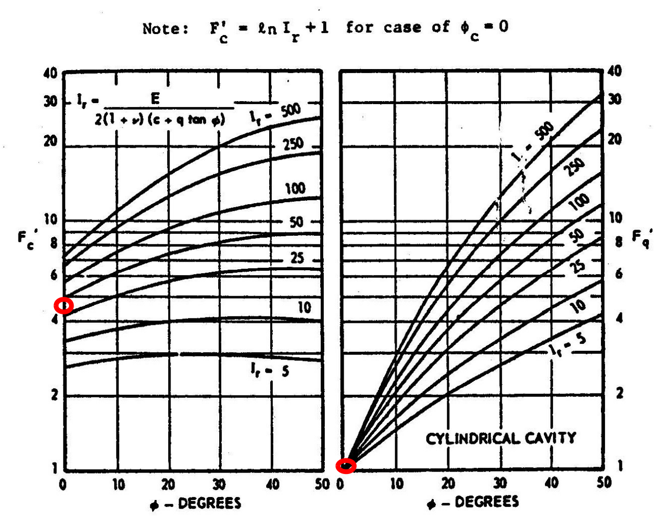
Calculated Parameters

Aggregate pier stress factor	μ_{SC}	0.0	
Soil stress factor	μ_{SOIL}	1.0	
Composite friction angle	ϕ_{avg}	0	°
Composite cohesion	c_{avg}	2500	psf
Failure plane angle	β	45	°
Vertical interface length	H	4.0	ft
Passive coefficient	K_p	1.00	
Average vertical effective stress	σ_{vo}'	480	psf
Mean normal effective stress	q	480	psf
Rigidity Index	I_R	37	
Confinement stress	σ_3	11,730	psf

Additional Input Parameters (for square footings)

Vesic cohesion factor	F_c'	4.5	
Vesic mean stress factor	F_q'	1	





From FHWA, 1983 (after Vesic)

Ultimate bearing pressure	q_{ult}	**16,730**	**psf**
Allowable bearing pressure	q_{des}	**4000**	**psf**

Factor of safety	**FS**	**4.2**

VIBRO PIER GROUP BEARING CAPACITY CALCULATION

Project name	Compton Community Coll
Project location	
Project number	OP0013298
Date	9/7/2021
Designed by	MBU
Reviewed by	

Input Parameters

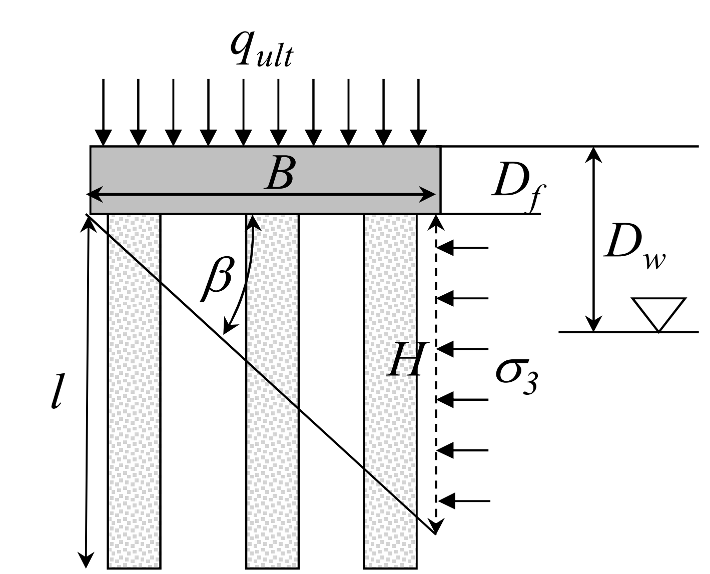
Strip or Square	-	Square	
Footing width	B	5	ft
Footing length	L	5	ft
Depth of embedment	D_f	2	ft
Area replacement ratio	ARR	0.00	
Adjacent surcharge	σ_{sur}	0	psf
Water table depth	D_w	45	ft
Moist unit weight	γ_{moist}	120	pcf
Saturated unit weight	γ_{sat}	120	pcf
Soil friction angle	ϕ_{soil}	0	°
Soil cohesion	c_{soil}	2500	psf
Stress concentration factor	n	0	
Soil elastic modulus (at H/2)	E	250,000	psf
Stone friction angle	ϕ_{stone}	45	°
Treatment Depth	l	23	ft

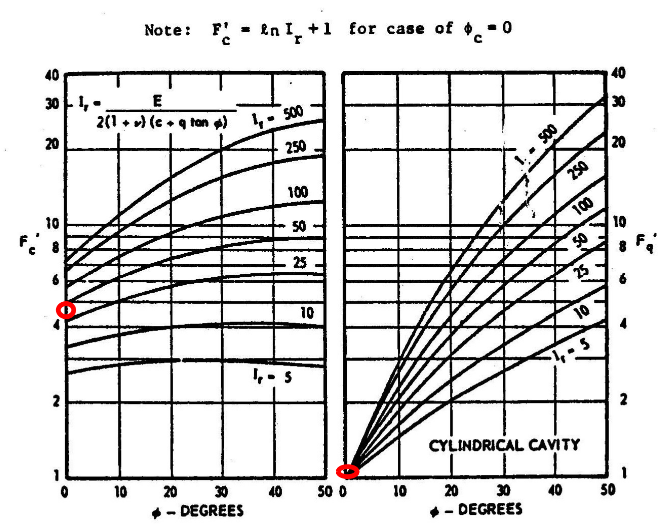
Calculated Parameters

Aggregate pier stress factor	μ_{SC}	0.0	
Soil stress factor	μ_{SOIL}	1.0	
Composite friction angle	ϕ_{avg}	0	°
Composite cohesion	c_{avg}	2500	psf
Failure plane angle	β	45	°
Vertical interface length	H	5.0	ft
Passive coefficient	K_p	1.00	
Average vertical effective stress	σ_{vo}'	540	psf
Mean normal effective stress	q	540	psf
Rigidity Index	I_R	37	
Confinement stress	σ_3	11,790	psf

Additional Input Parameters (for square footings)

Vesic cohesion factor	F_c'	4.5	
Vesic mean stress factor	F_q'	1	





From FHWA, 1983 (after Vesic)

Ultimate bearing pressure	q_{ult}	**16,790**	**psf**
Allowable bearing pressure	q_{des}	**4000**	**psf**

Factor of safety	**FS**	**4.2**

VIBRO PIER GROUP BEARING CAPACITY CALCULATION

Project name	Compton Community Coll
Project location	
Project number	OP0013298
Date	9/7/2021
Designed by	MBU
Reviewed by	

Input Parameters

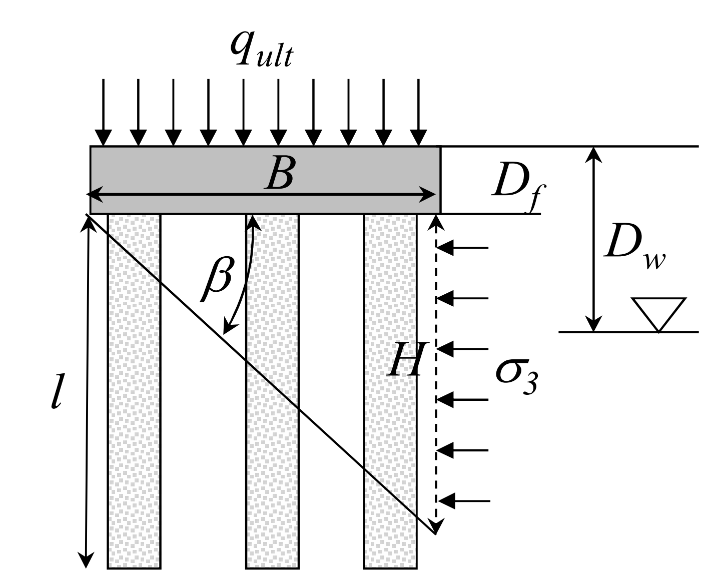
Strip or Square	-	Square	
Footing width	B	6	ft
Footing length	L	6	ft
Depth of embedment	D_f	2	ft
Area replacement ratio	ARR	0.00	
Adjacent surcharge	σ_{sur}	0	psf
Water table depth	D_w	45	ft
Moist unit weight	γ_{moist}	120	pcf
Saturated unit weight	γ_{sat}	120	pcf
Soil friction angle	ϕ_{soil}	0	°
Soil cohesion	c_{soil}	2500	psf
Stress concentration factor	n	0	
Soil elastic modulus (at H/2)	E	250,000	psf
Stone friction angle	ϕ_{stone}	45	°
Treatment Depth	l	23	ft

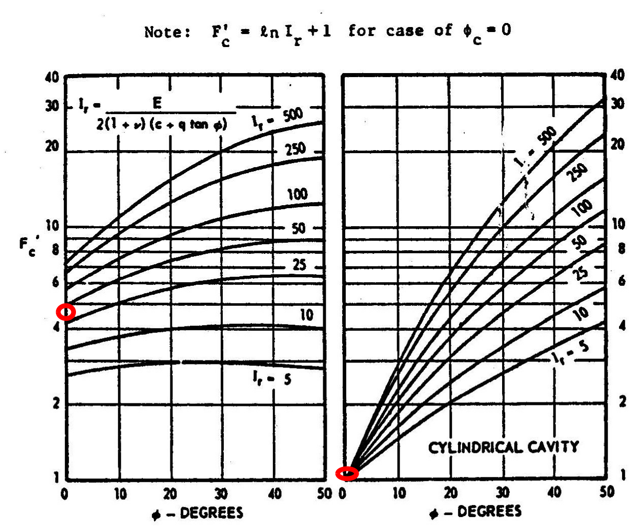
Calculated Parameters

Aggregate pier stress factor	μ_{SC}	0.0	
Soil stress factor	μ_{SOIL}	1.0	
Composite friction angle	ϕ_{avg}	0	°
Composite cohesion	c_{avg}	2500	psf
Failure plane angle	β	45	°
Vertical interface length	H	6.0	ft
Passive coefficient	K_p	1.00	
Average vertical effective stress	σ_{vo}'	600	psf
Mean normal effective stress	q	600	psf
Rigidity Index	I_R	37	
Confinement stress	σ_3	11,850	psf

Additional Input Parameters (for square footings)

Vesic cohesion factor	F_c'	4.5	
Vesic mean stress factor	F_q'	1	





From FHWA, 1983 (after Vesic)

Ultimate bearing pressure	q_{ult}	**16,850**	**psf**	**Factor of safety**	**FS**	**4.2**
Allowable bearing pressure	q_{des}	**4000**	**psf**			

VIBRO PIER GROUP BEARING CAPACITY CALCULATION

Project name	Compton Community Coll
Project location	
Project number	OP0013298
Date	9/7/2021
Designed by	MBU
Reviewed by	

Input Parameters

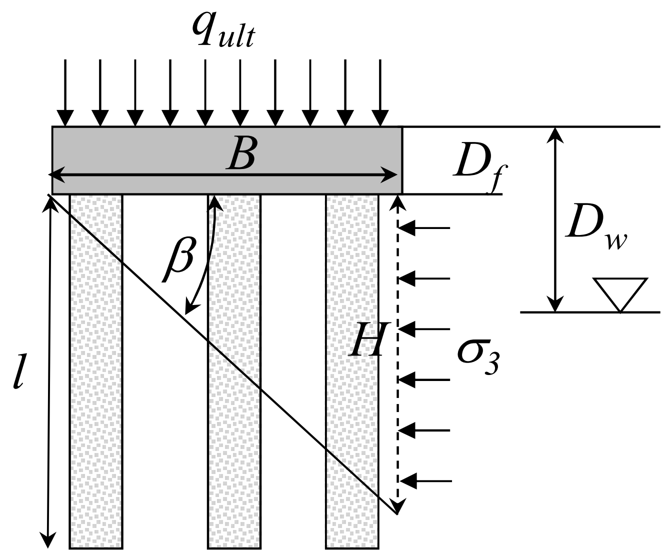
Strip or Square	-	Square	
Footing width	B	8	ft
Footing length	L	8	ft
Depth of embedment	D_f	2	ft
Area replacement ratio	ARR	0.00	
Adjacent surcharge	σ_{sur}	0	psf
Water table depth	D_w	45	ft
Moist unit weight	γ_{moist}	120	pcf
Saturated unit weight	γ_{sat}	120	pcf
Soil friction angle	ϕ_{soil}	0	°
Soil cohesion	c_{soil}	2500	psf
Stress concentration factor	n	0	
Soil elastic modulus (at H/2)	E	250,000	psf
Stone friction angle	ϕ_{stone}	45	°
Treatment Depth	l	23	ft

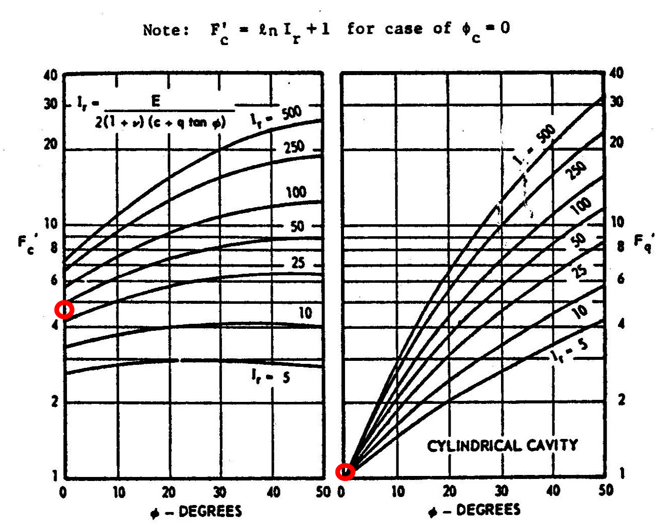
Calculated Parameters

Aggregate pier stress factor	μ_{SC}	0.0	
Soil stress factor	μ_{SOIL}	1.0	
Composite friction angle	ϕ_{avg}	0	°
Composite cohesion	c_{avg}	2500	psf
Failure plane angle	β	45	°
Vertical interface length	H	8.0	ft
Passive coefficient	K_p	1.00	
Average vertical effective stress	σ_{vo}'	720	psf
Mean normal effective stress	q	720	psf
Rigidity Index	I_R	37	
Confinement stress	σ_3	11,970	psf

Additional Input Parameters (for square footings)

Vesic cohesion factor	F_c'	4.5	
Vesic mean stress factor	F_q'	1	





From FHWA, 1983 (after Vesic)

Ultimate bearing pressure	q_{ult}	**16,970**	**psf**
Allowable bearing pressure	q_{des}	**4000**	**psf**

Factor of safety	**FS**	**4.2**

VIBRO PIER GROUP BEARING CAPACITY CALCULATION

Project name	Compton Community Coll
Project location	
Project number	OP0013298
Date	9/7/2021
Designed by	MBU
Reviewed by	

Input Parameters

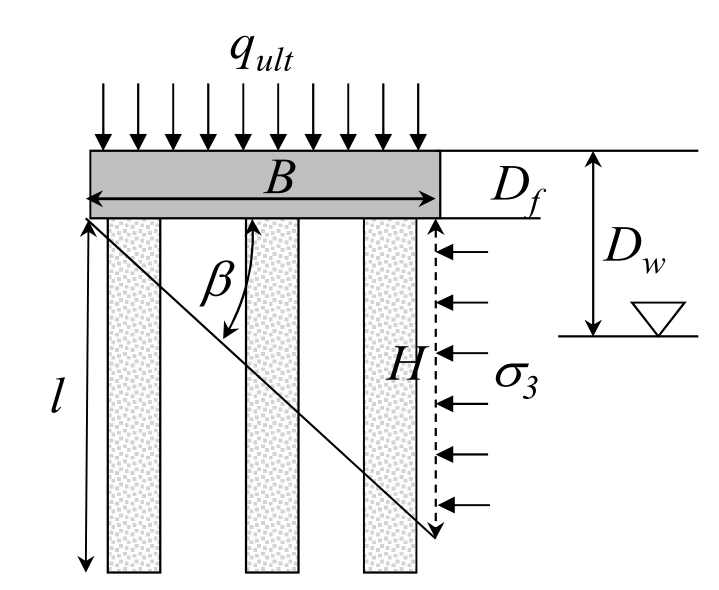
Strip or Square	-	Square	
Footing width	B	8	ft
Footing length	L	8	ft
Depth of embedment	D_f	4.5	ft
Area replacement ratio	ARR	0.00	
Adjacent surcharge	σ_{sur}	0	psf
Water table depth	D_w	45	ft
Moist unit weight	γ_{moist}	120	pcf
Saturated unit weight	γ_{sat}	120	pcf
Soil friction angle	ϕ_{soil}	0	°
Soil cohesion	c_{soil}	2500	psf
Stress concentration factor	n	0	
Soil elastic modulus (at H/2)	E	250,000	psf
Stone friction angle	ϕ_{stone}	45	°
Treatment Depth	l	23	ft

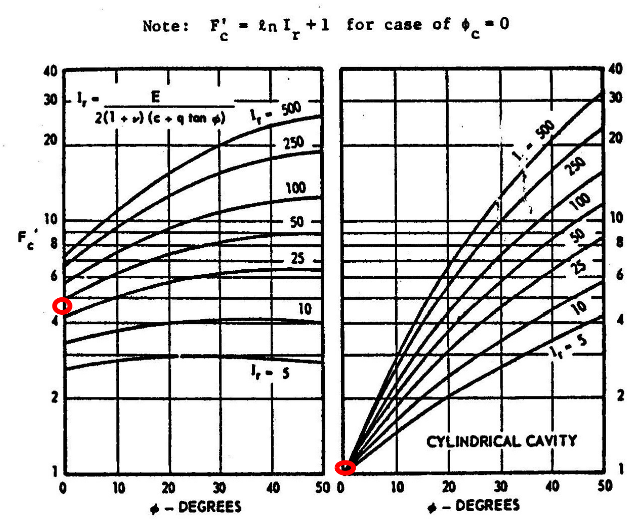
Calculated Parameters

Aggregate pier stress factor	μ_{SC}	0.0	
Soil stress factor	μ_{SOIL}	1.0	
Composite friction angle	ϕ_{avg}	0	°
Composite cohesion	c_{avg}	2500	psf
Failure plane angle	β	45	°
Vertical interface length	H	8.0	ft
Passive coefficient	K_p	1.00	
Average vertical effective stress	σ_{vo}'	1020	psf
Mean normal effective stress	q	1020	psf
Rigidity Index	I_R	37	
Confinement stress	σ_3	12,270	psf

Additional Input Parameters (for square footings)

Vesic cohesion factor	F_c'	4.5	
Vesic mean stress factor	F_q'	1	





From FHWA, 1983 (after Vesic)

Ultimate bearing pressure	q_{ult}	**17,270**	**psf**
Allowable bearing pressure	q_{des}	**4000**	**psf**

Factor of safety	**FS**	**4.3**

VIBRO PIER GROUP BEARING CAPACITY CALCULATION

Project name	Compton Community Coll
Project location	
Project number	OP0013298
Date	9/7/2021
Designed by	MBU
Reviewed by	

Input Parameters

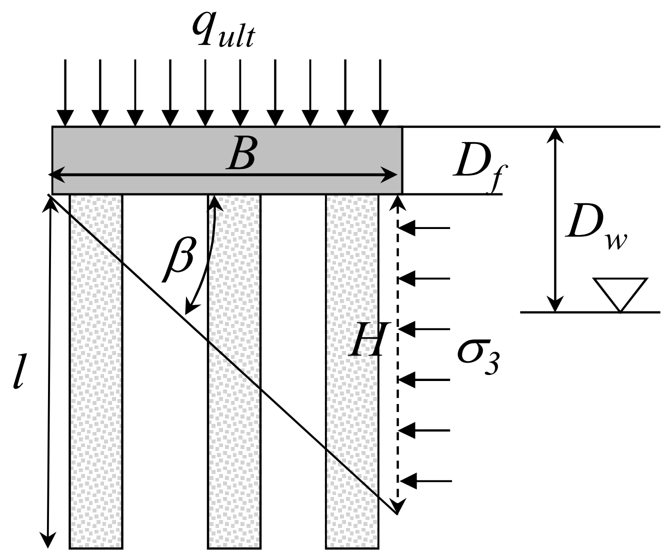
Strip or Square	-	Square	
Footing width	B	10	ft
Footing length	L	10	ft
Depth of embedment	D_f	4	ft
Area replacement ratio	ARR	0.00	
Adjacent surcharge	σ_{sur}	0	psf
Water table depth	D_w	45	ft
Moist unit weight	γ_{moist}	120	pcf
Saturated unit weight	γ_{sat}	120	pcf
Soil friction angle	ϕ_{soil}	0	°
Soil cohesion	c_{soil}	2500	psf
Stress concentration factor	n	0	
Soil elastic modulus (at H/2)	E	250,000	psf
Stone friction angle	ϕ_{stone}	45	°
Treatment Depth	l	23	ft

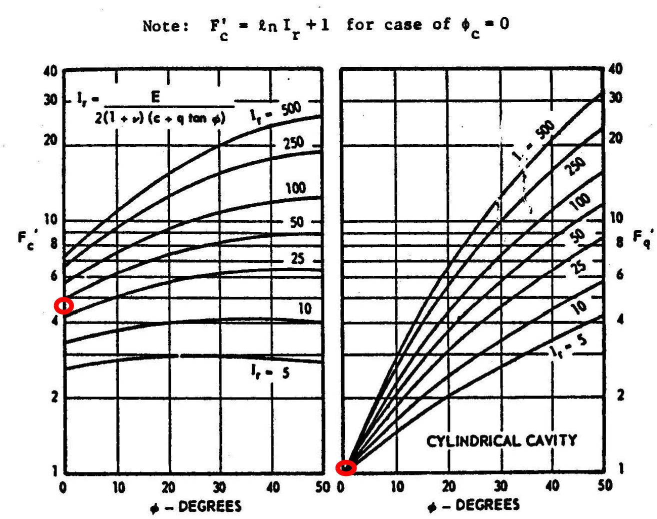
Calculated Parameters

Aggregate pier stress factor	μ_{SC}	0.0	
Soil stress factor	μ_{SOIL}	1.0	
Composite friction angle	ϕ_{avg}	0	°
Composite cohesion	c_{avg}	2500	psf
Failure plane angle	β	45	°
Vertical interface length	H	10.0	ft
Passive coefficient	K_p	1.00	
Average vertical effective stress	σ_{vo}'	1080	psf
Mean normal effective stress	q	1080	psf
Rigidity Index	I_R	37	
Confinement stress	σ_3	12,330	psf

Additional Input Parameters (for square footings)

Vesic cohesion factor	F_c'	4.5	
Vesic mean stress factor	F_q'	1	





From FHWA, 1983 (after Vesic)

Ultimate bearing pressure	q_{ult}	**17,330**	**psf**
Allowable bearing pressure	q_{des}	**4000**	**psf**

Factor of safety	**FS**	**4.3**

VIBRO PIER GROUP BEARING CAPACITY CALCULATION

Project name	Compton Community Coll
Project location	
Project number	OP0013298
Date	9/7/2021
Designed by	MBU
Reviewed by	

Input Parameters

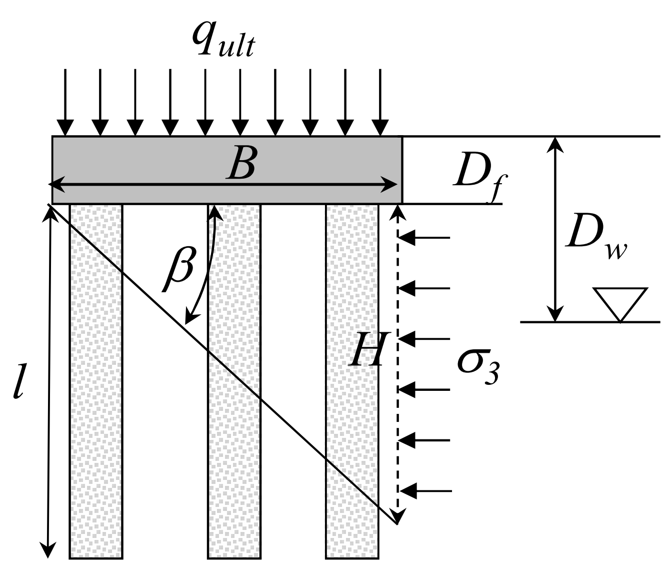
Strip or Square	-	Square	
Footing width	B	6	ft
Footing length	L	10	ft
Depth of embedment	D_f	4.6	ft
Area replacement ratio	ARR	0.00	
Adjacent surcharge	σ_{sur}	0	psf
Water table depth	D_w	45	ft
Moist unit weight	γ_{moist}	120	pcf
Saturated unit weight	γ_{sat}	120	pcf
Soil friction angle	ϕ_{soil}	0	°
Soil cohesion	c_{soil}	2500	psf
Stress concentration factor	n	0	
Soil elastic modulus (at H/2)	E	250,000	psf
Stone friction angle	ϕ_{stone}	45	°
Treatment Depth	l	23	ft

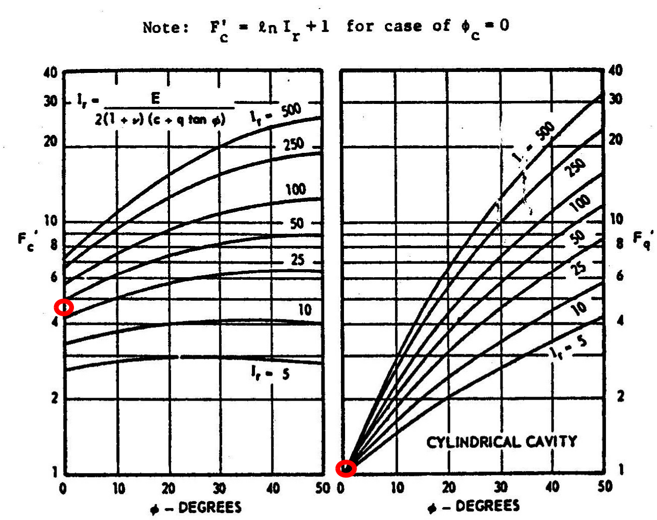
Calculated Parameters

Aggregate pier stress factor	μ_{SC}	0.0	
Soil stress factor	μ_{SOIL}	1.0	
Composite friction angle	ϕ_{avg}	0	°
Composite cohesion	c_{avg}	2500	psf
Failure plane angle	β	45	°
Vertical interface length	H	6.0	ft
Passive coefficient	K_p	1.00	
Average vertical effective stress	σ_{vo}'	912	psf
Mean normal effective stress	q	912	psf
Rigidity Index	I_R	37	
Confinement stress	σ_3	12,162	psf

Additional Input Parameters (for square footings)

Vesic cohesion factor	F_c'	4.5	
Vesic mean stress factor	F_q'	1	

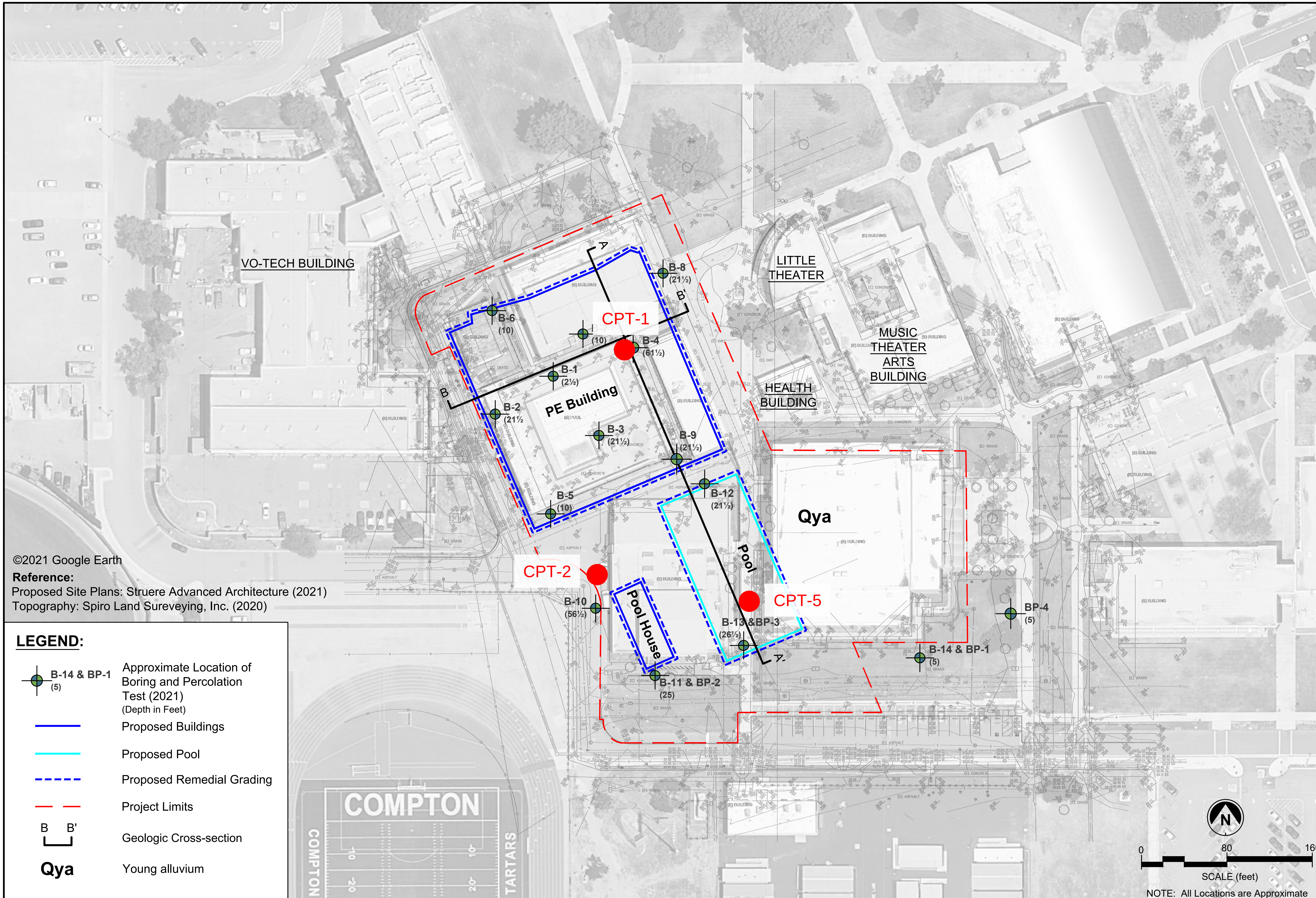




From FHWA, 1983 (after Vesic)

Ultimate bearing pressure	q_{ult}	**17,162**	**psf**
Allowable bearing pressure	q_{des}	**4000**	**psf**
Factor of safety	**FS**	**4.3**	

Appendix C
Static Settlement based on Predicted Post-treatment CPTs



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

GEOTECHNICAL MAP
Physical Education Complex Replacement
Compton, California

ATLAS

Figure: 2

CPT Correlations and Static Analyses

Project: OP0013298 - Compton Community College

CPT ID: CPT-1 (PE Building)

KELLER

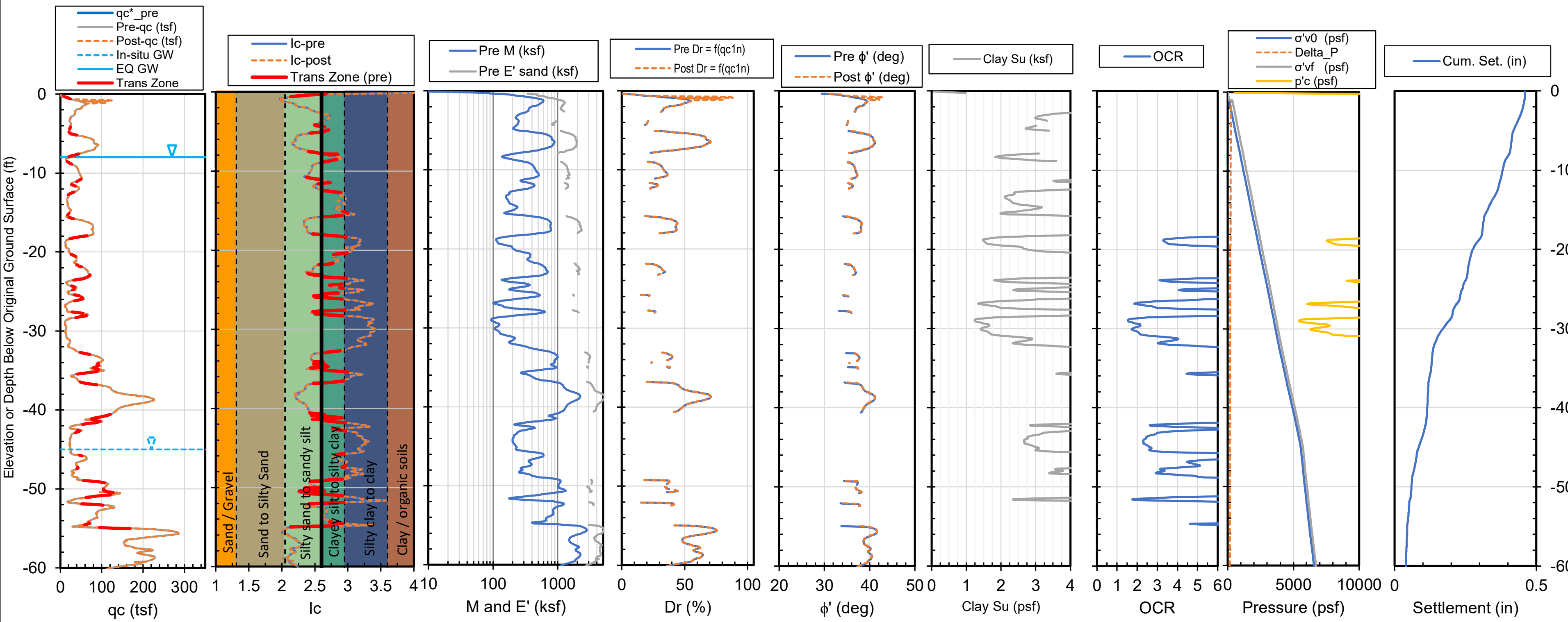
by: Bailey Uy

Date: 09.07.21

Static Settlement Calculations (No Ground Improvement)

SETTLEMENT FOR AERIAL LOAD

Total Arial Load For Settlement:		
SPREAD FOOTING W/ 2:1 RULE		
Footing Load, q:	250	psf
Footing Width, B:	200	ft
Footing Length, L:	200	ft
Footing Depth below exist. Grade, D:	1	ft
Static Settlement (no ground improvement):	0.46	in
Depth of VP for subgrade modulus calc:	35	ft
Soil Subgrade Modulus:	5 pci	



Appendix D
Pre- and Post-treatment Liquefaction Analysis

Liquefaction Analysis and Stone Column Mitigation

Project: OP0013298 - Compton Community College

CPT ID: CPT-1 (PE Building)   Surface Elev.: 0.0 ft (use 0 ft to plot depth instead of elevation)

by: Bailey Uy

Date: 09.07.21

Perform Ground Improvement Analysis?: **Stone Columns**

LIQUEFACTION ANALYSIS PARAMETERS

Triggering Method =	Robertson (NCEER R&W 1998)	
Vol. Settlement Method =	Zhang et al. (2002)	
Depth of GW During CPT =	45.00	ft
Depth of GW During Earthquake =	8.00	ft
Depth of Fill =	0.00	ft
Unit Weight of fill =	120	pcf
PGApre =	0.802	g
Mw =	7.30	
Ic Threshhold =	2.6	
Use Kσ ? =	Yes	

ADVANCED LIQUEFACTION PARAMETERS

Use Ic Transition Zones?	Yes			
Transition zone (dIc / dz) =	0.7	Ic/ft	Manual Trans Zones?:	No
Cliq TZ dIc =	0.04		Use Manual Thin Layer Cor.? =	No
Min. Trans. Zone Points:	4			
Ic_min and max =	1.9	3		

STONE COLUMN DESIGN PARAMETERS

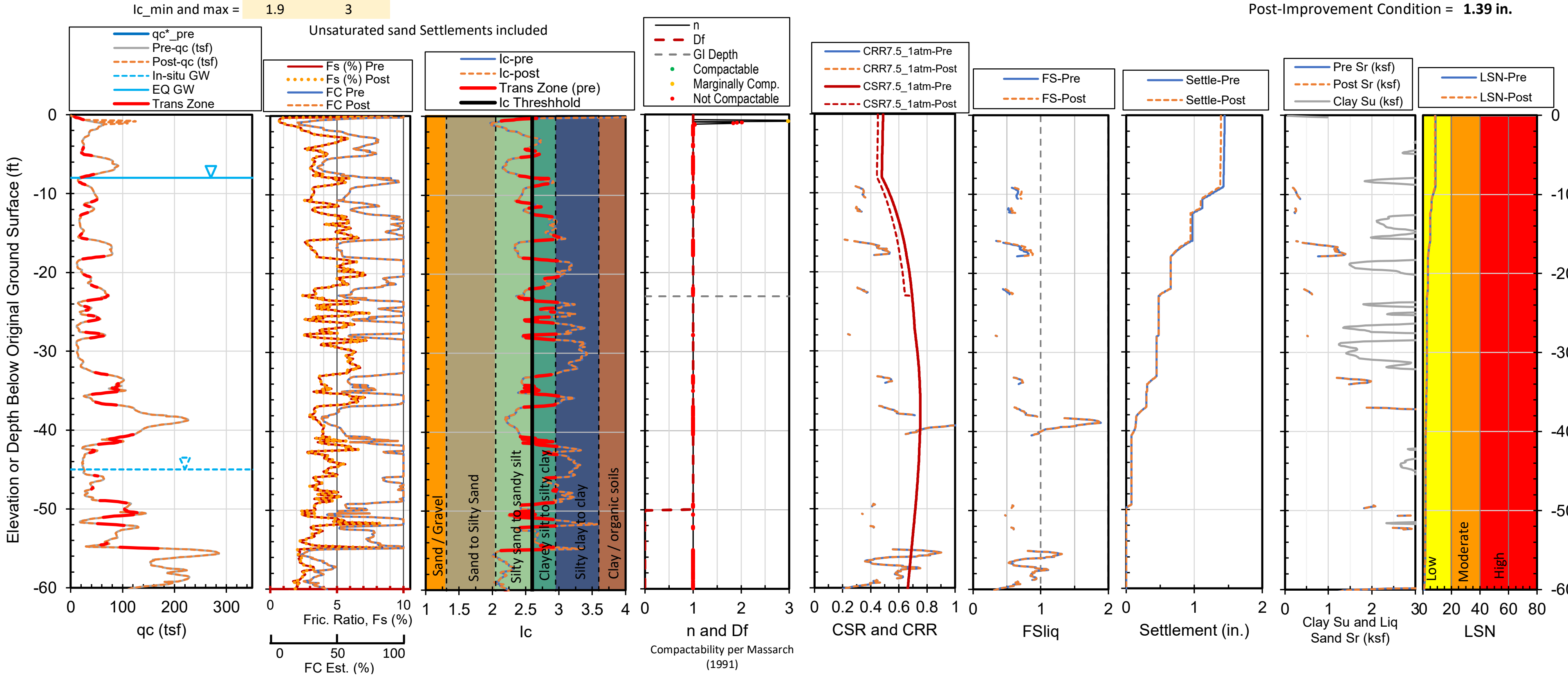
Depth Below Existing Grade =	23	ft	
Stone Column Diameter, D =	36	inch	
Stone Column spacing, S =	8	ft	
Square or Triangular Layout =	Square		
ARR =	11.0	%	
Baez for HBI =	HBI		
HBI Baez Scaling Factor (BSF):	Single	1	
Post Ic Shift	Use pre		
Gr =	6		
Rrd = PGApost/PGApre =	0.928		
PGApost in Impr. Zone =	0.744	g	

Volumetric Settlement Results:

Existing (Pre-Treatment) Condition = **1.45 in.**

Post-Improvement Condition = **1.39 in.**

Unsaturated sand Settlements included



Keller North America
17461 Derian Avenue, Suite 106
Irvine, CA 92614
Tel : 909-393-9300
Fax : 909-393-0036

Revised 9/10/2021

Deep Soil Mixing Design
Pool and Pool Building of Compton Community College

1111 East Artesia Boulevard
Compton, California

Submitted to:
PCM3, Inc.
Compton CCD Office

Submitted by:
Keller North America

September 9, 2021

Keller North America

PCM3
Compton CCD Office

Attention: Ms. Sheri Phillips
Subject: Deep Soil Mixing Design
Compton Community College Pool House and Swimming Pool

Keller North America (Keller) is pleased to present the following design submittal for ground improvement for the proposed buildings at this project site. The purpose of the ground improvement program is to enhance the safety, stability, and serviceability of the proposed structures. This is accomplished by increasing the strength of the ground to the point where the ground can safely support the anticipated structures under static loads as well as during and after the design level earthquake. Additional information is provided in the attached report.

The design provided herein has been prepared for the exclusive use of Keller, with the special equipment and production procedure, for our client under the following strict limitations:

1. Only Keller may construct the work described by the design and
2. The design may not be used by others for any purpose.

Keller appreciates the opportunity to be of service. Please feel free to contact the undersigned at (909) 393-9300 with any questions, comments, or concerns.

Respectfully submitted,

REGISTERED PROFESSIONAL ENGINEER
SUNIL ARORA
C72352
EXP. 06/30/22
CIVIL
STATE OF CALIFORNIA

David Chae,	Sunil Arora, P.E.	Bailey Uy
Assistant Project Manager	Project Executive	Engineer

1. DESIGN SUMMARY

This project site is located at 1111 East Artesia Boulevard, Compton, California. Keller proposes Deep Soil Mixing (DSM) as the ground improvement element to provide sufficient foundation support to the proposed Pool Building and swimming pool. The current site is approximately at an elevation of 55 feet. Table 1 summarizes the design depth of DSM for each proposed structure.

Based on Atlas Technical Consultants experience on the project site there may be more variation in the soil profile then what is portrayed in the CPT. Therefore, Keller is using conservative depth of treatment as provided by the project GEOR.

Table 1: Design Depth of DSM

Area	Approximate Existing Site Elevation, ft	Approximate Tip Elevation of DSM, ft	Approximate Depth (ft)
Swimming Pool	55	6	49
Pool Building	55	36	19

Deep Soil Mixing (DSM)

DSM construction involves using high torque equipment to mechanically mix grout with native soils to create a nearly homogeneous mixture of weak concrete called soilcrete. DSM is a top-down construction technique. As the mixing tool is advanced into the soil, grout slurry is pumped through the hollow stem of the shaft and injected into the soil at the tip and through the tool. The auger flights and mixing blades on the tool blend the soil with grout in a pug-mill fashion. When the design depth is reached, the tool is withdrawn to the surface. Left behind are stabilized soil mixed columns. Often predrilling can be used to simplify the disposal of construction spoils and waste soil. Depending on project requirements DSM can be used to improve ~10% to ~90% of the soil in each area.

The target average 28-days unconfined compressive strength is 150 psi. Keller plans to use 200 kg/m^3 at the beginning of the mixing operation and observe the wet soilcrete strength development to adjust cement dosage accordingly.

Figure 1: Construction of DSM

2. GROUND IMPROVEMENT DESIGN BASIS

This design is based on Keller's understanding of the following project documents and performance requirements articulated by the project structural engineer and geotechnical engineer. Although many documents were reviewed, only those which provided information that directly affects our design are listed below.

- Geotechnical Investigation Report, Physical Education Complex Replacement, Compton Community College District, by Atlas Technical Consultants, LLC, dated July 7, 2021
- Addendum Geotechnical and Geohazard Report, Physical Education Complex Replacement, Compton Community College District, by Atlas Technical Consultants, LLC, dated September 7, 2021
- CPT Data – Compton Site, CPT-1, CPT-2, and CPT-5, dated September 3, 2021
- Preliminary Foundation Schemes, by Brandow & Johnston, Inc., dated July 13, 2021

If any of these documents are changed or altered in any way, Keller should be notified, and the design may require modifications.

2.1 Subsurface Conditions

Based on our review of the Geotechnical Investigation Report by Atlas Technical Consultants LLC (Atlas), it is Keller's understanding that the site is generally underlain by about ½ foot of grass/topsoil/surficial fill and young alluvial deposits of Holocene to late Pleistocene age. These alluvial deposits are primarily comprised of inter-layered silty sands and sandy silts. In general, the near-surface sandy soil 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. Per Atlas's Geotechnical Investigation Report, groundwater was encountered at a depth between 44 feet to 52 feet below the existing ground surface.

2.2 Design and Performance Requirements

The ground improvement design criteria have been established by the project geotechnical and structural engineers and summarized in Table 1 below. Keller has reviewed the criteria and they appear typical and reasonable for this type of project.

Table 2: Design and Performance Criteria

	Criteria	Reference
Groundwater Level (Static)	44’ below grade	Atlas Technical Consultants, LLC
Groundwater Level (Seismic)	8’ below grade	
PGA_M (ASCE 7-16)	0.802 g	
M_w (ASCE 7-16)	7.3	
Depth of Liquefaction Analysis	50 feet	
Post-treatment Liquefaction-induced Differential Settlement for Swimming Pool	≤ 0.5 inch over 154 feet	
Post-treatment Liquefaction-induced Differential Settlement for Pool Building	≤ 2.4 inch over 40 feet (0.005*L)	Brandow & Johnston, Inc. based on Table 12.13-3 of ASCE 7-16 for Risk Category II building

3. DSM DESIGN

The target average 28-days unconfined compressive strength (UCS) of the DSM is 150 psi.

3.1 Foundation Bearing Capacity of Pool Building

The minimum required area replacement ratio (A_r) of DSM is 30%, per Atlas’ Addendum Geotechnical and Geohazard Report. Per Keller’s shop drawing as seen in Appendix A, the actual area replacement ratio, A_r, of the proposed Pool Building is 50%. Keller checks the bearing capacity of DSM columns against crushing under seismic condition as follows:

Working Pressure (p) = 8,000 psf/A_r = 16,000 psf
Factor Safety (FS) = UCS/p = 150 psi / 16,000 psf = 1.3

3.2 Seismic Design of DSM for Swimming Pool and Pool Building

The design of soil mixing cells to mitigate liquefaction-induced settlement relies on the reinforcement effects, as published by Nguyen, et al. (2013). The minimum design A_r of the DSM over the proposed pool building and swimming pool is approximately 30%.

Nguyen (2013) suggested incorporating, R_{rd}, the ratio of shear stress reduction for improved and unimproved case when analyzing post construction liquefaction potential to account for the shear reinforcement effect of DSM Grid. With the $A_r = 30\%$ and the soilcrete to soil shear modulus ratio of $G_r = 30$, the calculated shear stress reduction factor yields $R_{rd} = 0.229$.

Therefore, the post-treatment PGA can be computed as $PGA_{post} = PGA_{pre} \times R_{rd}$. Here in this chapter, the key computation equations are listed.

R_{rd} is given by the following equation:

$$R_{rd} = \min\left\{\frac{1}{G_r \cdot \left[A_r \cdot C_G \cdot \gamma_r + \frac{1}{G_r} \cdot (1 - A_r)\right]}, 1\right\}$$

where, G_r = average stiffness ratio, A_r = area replacement ratio

C_G = equivalent shear factor computed as the shear stiffness of the DSM grid system:

$$C_G = 1 - 0.5\sqrt{1 - A_r}$$

γ_r = shear strain ratio between DSM and soil:

$$\gamma_r = \left[1 - (1 - A_r)^{1.3} \cdot \left(\frac{G_r - 1}{185}\right)^{0.4}\right] \cdot \min\left(\frac{H}{S}, 1\right)$$

Based on the Geotechnical Investigation Report by Atlas Technical Consultants LLC., dated July 7th, 2021, the ground motion input used in Keller's post-treatment liquefaction-induced settlement analysis is:

- $M_w = 7.3$
- $PGA_{post} = 0.229 \times 0.802g = 0.184g$ (within the treatment length of DSM)

The post-treatment liquefaction-induced settlement analysis is included in Appendix B of this submittal. Table 3 below summarizes the computed results for each structure of this project:

Table 3: Pre- and Post-treatment Liquefaction-Induced Settlement Analysis

Area		**Pre-treatment Liquefaction-Induced Settlement (inch)**	**Post-treatment Liquefaction-Induced Settlement (inch)**
Pool Building	CPT-2	2.59	0.68
Swimming Pool	CPT-5	2.79	0.09

4. DSM CONSTRUCTION

4.1 Layout

Keller will provide an AutoCAD shop drawing for each DSM column coordinate overlaid on the site Civil drawing. Keller understands that the general contractor will be responsible and use a licensed surveyor to provide Keller with controlled points and survey benchmarks before installation and will prepare as-built drawings after completion. DSM columns will be installed within 6 inches of the design locations as shown in the Keller shop drawing.

4.2 Sequence of Work

Once a stable working platform has been established as shown in Keller shop drawing. DSM columns will be constructed.

4.3 Predrill

To minimize the mixing tool damage and maintaining soil mixing quality, Keller may pre-dill holes or excavate for better mixing quality. The holes will be filled with soilcrete up to the working elevation during the mixing stage.

4.4 Soil Mixing

In general, soil mixing operation parameters, such as mixing shaft speed, penetration rate, batching grout specific gravity (sg), and pumping rate will be determined based on our lab mixing result and our experience and will be fine-tuned at the beginning of mixing column production. The design cement content in place (cement weight/[soil volume + grout volume]) will start from approximately 200 kg/m^3 with grout slurry specific gravity (sg) of 1.45. Keller engineers may adjust the cement content and grout sg based on the field sample strength development.

4.4.1 Vertical Alignment

Vertical alignment of the mix tool stroke will be controlled by the drill rig operator. Two measurements of verticality will be monitored. These are the fore-aft and left-right vertical mast positions. Verticality will be measured by a level as measured on the mixing tool prior to penetration. Intermittent measurements will be made as may be necessary during mixing operations.

4.4.2 Mixing Shaft Speed

The mixing shaft speed which is anticipated to be ranging between 20-50 RPM and shall be adjusted to accommodate a constant rate of mixing shaft penetration based on the degree of drilling difficulty. The mixing shaft speed can be adjusted according to drilling difficulty. The mixing shaft speed can be adjusted to aid mixing of the soil column when needed or to assist penetration in hard drilling. Mixing shaft speed will be recorded.

4.4.3 Penetration Rate

In order to ensure adequate mixing, the penetration rate of the mixing shaft shall be maintained at about 1.0 to 3.0 feet/minute during penetration. The penetration rate and maximum depth of each stroke shall be recorded by Keller’s data acquisition system.

4.4.4 Grout Take

The grout slurry flow per vertical foot of the column will be adjusted to the requirements of the design mix. Progressive cavity pumps will be used to transfer the grout from the mixing plant to the mixing rig. Flow monitoring devices will be installed in the grout line to detect any line blockage and monitor flow, total injected grout per column and grout pressure. These parameters will be recorded.

Inevitably some variations of the grout take will occasionally occur due to field conditions. It is anticipated that a grout flow rate between 50 to 250 GPM will be used during penetration. Keller's Data Acquisition System (DAQ) can automatically adjust the grout flow rate as a function of the penetration rate and maintain the pre-set cement dosage prescribed by the design engineer.

4.4.5 Withdrawal Rate

The mixing shaft will be withdrawn at a rate of 6 to 12 feet per minute.

4.4.6 Obstruction/ Mixing Shaft Refusal

Keller will use a data acquisition system to monitor the mixing shaft penetration and the shaft rotation resistance in terms of the hydraulic pressure. The DAQ system will calculate and plot the Drilling Index as a function of depth, a mixing parameter to detect penetration resistance and refusal depth. Keller will set up the penetration criteria based on the site measurement. In case of underground obstruction, such as abandoned footings, piles, utilities, etc., the general contractor will be responsible to remove obstructions and backfilled with sandy soil prior to Keller mixing operation.

4.5 Material

Cement: Cement will be furnished by Keller and conform to ASTM C150 "Standard Specification for Portland Cement," Type II/V or equivalent. The cement will be adequately protected from moisture and contamination while in transit to and in storage at the job site. Reclaimed cement or cement containing lumps or deleterious matter will not be used.

Water: Water for the slurry will be fresh, free of deleterious substances that adversely affect the strength and mixing properties of the slurry, furnished by others.

4.6 Equipment

4.6.1 Batching Equipment

The batch plant shall consist of in-line eductor (jet valve) mixers. Dry materials shall be stored in tankers and/or silos and fed to the mixers for shearing and circulation. The resulting grout slurry will be transferred to a surge tank for continuous agitation and to supply the in-situ soil mixing rig. Grout slurry quality will be assured by frequent testing prior to injection into the soil.

4.6.2 Mixing Equipment

Single shaft mixing equipment that mechanically mixes the soil and cement slurry for the full dimensions of the column will be used for the Work. We anticipate using hydraulic drill rigs for the soil mixing operations. This rig is capable of up to > 150,000 ft-lbs. of torque at > 20 rpm. The working shaft rate of rotation ranges between 20 and 60 rpm. The mixing shaft will have mixing augers and/or blades (paddles) configured in such a manner so that they are capable of thoroughly blending the in-situ soils and cement slurry. The power source for driving the mixing shafts will be sufficient to maintain the required mix tool (shaft) rotation speed in revolutions per minute and penetration/ withdrawal rates from the ground surface to the maximum depth required. The design target Blade Rotation Number (BRN), defined as the number of blades cut in each 1.0-meter soil) will be at least 300.

The DSM equipment will be equipped with devices to assure vertical alignment in two planes (90 degrees in plan from each other): fore-aft and left-right. The DSM equipment will be equipped with a real-time display of depth, rotation speed, grout flow rate; grout specific gravity, cumulative grout injected, and grout pressure for each soil mix column. The cement will be mixed with water within the jet valve to create a 1.45 sg mix +/- 0.1. Note that sg can be changed by Keller based on UCS data and field conditions. No mixing operation will be allowed if the DAQ system not functioning.

4.6.3 Pumping Equipment

Grout slurry will be supplied to the drill using large size Moyno pumps. These pumps will be sized and powered so that design volumes and pressures can be maintained up to 1,000 feet away from the batching facility. It is anticipated that a continuous grout slurry flow of 150 gallons per minute at 100 psi to the drill rig will be necessary

4.6.4 Equipment Location

The batching and pumping facility will be set up central to both in situ soil mixing areas. This will eliminate the need to move the plant once it is established.

5. QA/QC

Following the installation of DSM columns, verification testing will include:

- Unconfined compressive test on wet soils mixed samples
- Unconfined compressive test on cored samples
- Review of production DAQ logs

5.1 Wet Soils Mixed Samples

Wet Soil mix samples will be retrieved and cast into molds for one column per rig/shift, at one random depth, typically near the end of each shift. Samples will be retrieved using an in situ wet sampler immediately after column construction and shall consist of no fewer than 8 specimens. Soil clods greater than 10% of the mold diameter will be screened off. Appropriate curing techniques shall be implemented until testing based on ASTM D 1632.

Unconfined compression testing shall be performed by an approved laboratory in pairs of specimens at 7 days. If the 7-days specimens do not reach the desired strength according to the lab test curve, another pair of specimens will be tested at 14 days, 28 days, and if needed at 56 days. All specimens at 28 days and available 56-days of age will be tested and used in the statistical calculation. The Unconfined Compressive Strength (UCS) shall be determined by ASTM D1633 “Standard Test Methods for Compressive Strength of Molded Soil-Cement Cylinders”. Sulfur caps shall be required in the UCS tests to minimize the end effects on the test specimen. The advantage of the wet sampling is that Keller can get an early trend of the soilcrete strength development without waiting to the end of the project for coring and can make early decisions in the field program to add additional soil mixing column s if necessary.

If wet grab strengths at 7 days of age are greater than the average required unconfined compressive strength, additional tests may be omitted. Wet grab samples will be kept on-site (approximately 3 days) for an initial set before being shipped to the lab.

5.2 Core Samples

Keller will core 2% of the DSM columns.

All core locations shall be randomly selected, to collect core samples for unconfined compression testing. Coring will start after the soilcrete has gained adequate strength and verified by the strength development from the wet sample tests. The double-tubes coring method, with the utilization of vibrators to assist the core to depth, can be used instead of the conventional coring technique. At minimum three (3) samples from each core will be extracted. Keller anticipates 4 specimens trimmed from each core hole to be tested by ASTM D1633.

Uniformity of mixing shall be evaluated by the geotechnical engineer of record (GEOR) based on the continuous core samples recovered. The continuous core holes shall extend the entire depth of the DSM column. Estimated recovery of 80 percent for each 5-foot-long segment of a boring and at least 90 percent when averaged over all core runs within a single boring shall be achieved. The lumps of unimproved soils shall not exceed 20 percent of the total volume of any 5-foot core segment from a boring. If the core recovery below the anticipated value due to the gravel particles in the soilcrete matrix, Keller shall be allowed to utilize a downhole camera or other approved methods to verify the core hole.

Keller will calculate the average 28-day UCS value from all core samples and wet grab samples. No more than 10 percent of all specimens tested shall exhibit an unconfined compressive strength of less than 75 psi at 28 days. A ceiling, the not-to-exceed value of four times the average unconfined compressive strength (i.e., 600 psi) shall be used for individual specimens in calculating the average strength achieved in the field from each coring and wet sample and for the entire project.

If the acceptance criteria are not achieved in a designated area, Keller may be given the opportunity to conduct additional UCS test on soilcrete specimens on 56 days of age, site exploration, coring, sampling, downhole imaging, and strength testing from the additional cured specimen to better define the average design strength at Keller's preference and expense. If a designated area is rejected, Keller shall submit a Remixing or Mitigation plan.

At the end of the project, to not unnecessary delay subsequent activities by waiting for a 28-day test result, correction of early strength gain will be used to approve the DSM work. However, this correlation will not relieve the contractor of the responsibility to achieve average 28-days strength of 150 psi. Based on FHWA (2013) guidelines, the following UCS aging factor correlations will be applied to this job:

- 28: 3-day, 1.72
- 28: 7-day, 1.35
- 28: 14-day, 1.15

A site-specific correlation between 3-days and 28-days strength may be used to supersede this correlation if in the opinion of the Engineer the site-specific correlation is more appropriate.

5.3 Production DAQ Logs

During the soil mixing production, Keller will review the wet soilcrete strength development as well as production column mixing logs and may add additional soil mixing columns if the soilcrete strength is below the target average UCS values as listed above.

6. SHOP DRAWINGS

Our shop drawing in **Appendix A** depicts our proposed soil improvement plans of DSM for the proposed pool building and swimming pool. An As-Built Drawing with any field changes will be provided upon completion of DSM work.

7. REFERENCES

Bruce, Mary Ellen C., Ryan R. Berg, James G. Collin, George M. Filz, Masaaki Terashi, David S. Yang, and Sa Geotechnical. Federal Highway Administration design manual: Deep mixing for embankment and foundation support. No. FHWA-HRT-13-046. The United States. Federal Highway Administration. Offices of Research & Development, 2013.

Filz, G. M., and E. Templeton. "Design guide for levee and floodwall stability using deep-mixed shear walls.”." New Orleans District and Hurricane Protection Office, US Army Corps of Engineers, Final Report Contract W912P8-07 (2011): 0031.

Nguyen, T.V., Rayamajhi, D., Boulanger, R.W., Ashford, S.A., Lu, J., Elgamal, A. and Shao, L. (2013). "Design of DSM grids for liquefaction remediation." Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 139(11), 1923-1933

Appendix A
Deep Soil Mixing Design Shop Drawing

COMPTON COMMUNITY COLLEGE (PHASE 2)

DEEP SOIL MIXING

Underground Service Alert of Southern California

Call: TOLL FREE 800-422-4133

TWO WORKING DAYS BEFORE YOU DIG

RE-NOTIFY EVERY TWO WEEKS

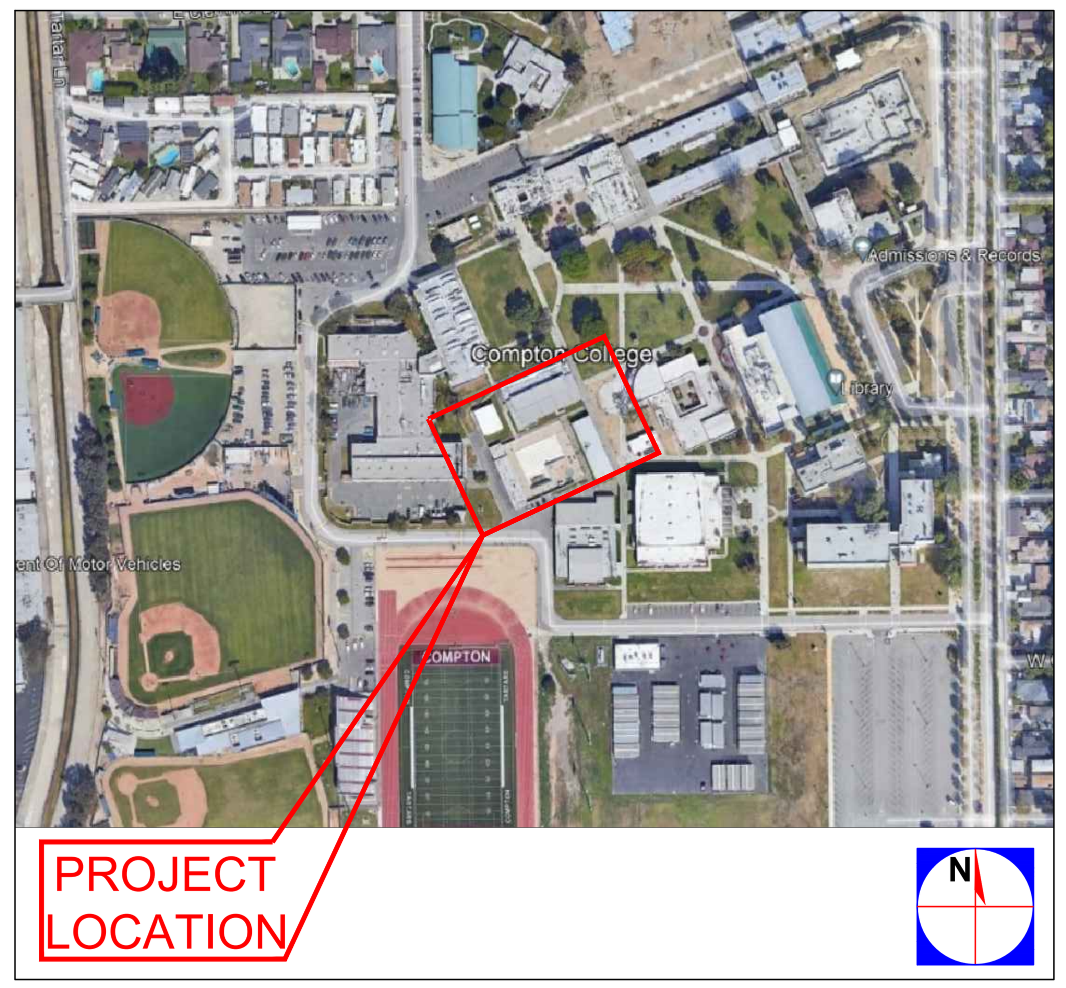
TICKET # ____________

DATE CALLED: ____________

SHEET INDEX

KNA-1	TITLE SHEET
KNA-2	STONE COLUMN NOTES AND DETAILS
KNA-3	OVERALL GROUND IMPROVEMENT PLAN

VICINITY MAP



PROJECT ADDRESS

1111 EAST ARTESIA BOULEVARD

SACRAMENTO, CALIFORNIA

95811

GEOTECHNICAL NOTES

Commentary

Keller North America (KNA) has been contracted to design and construct Stone Columns to support the foundation of the proposed buildings. This design submittal is as on the following information:

1. Geotechnical Investigation Report, Physical Education Complex Replacement, Compton Community College District, prepared by Atlas Technical Consultants, LLC, dated July 7th, 2021.
2. Addendum Geotechnical and Geohazard Report. Physical Education Complex Replacement, Compton Community College District, prepared by Atlas Technical Consultants, LLC, dated September 7th, 2021.
3. CPT Data - Compton Site, CPT-1, CPT-2, and CPT-5, dated September 3rd, 2021.
4. Preliminary Foundation Schemes, prepared by Brandow & Johnston, Inc., dated July 13th, 2021

*If any of these documents are changed or altered in any way, KNA may need to modify our design.

KELLER

17461 DERIAN AVENUE
SUITE 106
IRVINE, CALIFORNIA
92614

WWW.KELLER-NA.COM

Engineer of Record:

REGISTERED PROFESSIONAL ENGINEER SUNIL ARORA C72352 EXP. 06/30/22 CIVIL STATE OF CALIFORNIA

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TITLE SHEET

COMPTON COMMUNITY COLLEGE

1111 E. ARTESIA BLVD.
COMPTON, CALIFORNIA 90221

No.	Description	Date
A	FOR REVIEW	09/08/2021

Design by: DC — Approved by: SA

Drawn by: SPB — Checked by:

Project Number: — Date: 09/08/2021

Sheet No.

KNA-1

Scale: N/A

1" 0" 1/2" 1"

GROUND IMPROVEMENT GENERAL NOTES:

1. OTHERS are to provide a dry, stable, and relative level working platform. It is KNA North America's (KNA) understanding that the working grade will be near existing grade of Elv. 55. The working surface shall be constructed and managed by others such that KNA's equipment can safely track and efficiently work under its own weight without the need for steel plates or crane mats.
2. The GENERAL CONTRACTOR shall confirm that the proposed operation does not conflict with future improvement such as structural, mechanical, plumbing, and electrical prior to DSM installation.
3. An underground service alert must be obtained 2 days before starting work.
4. All permits shall be procured and paid for by the OWNER, other than transportation permits required for KNA's mobilization and demobilization.
5. All encroachment permits within the public right of way and letters of permission from private owners must be obtained by the OWNER.
6. KNA will provide a qualified full-time quality control (QC) representative. This representative is either KNA's Superintendent/Foreman/or Field Engineer. Third party testing and/or inspection shall be provided by OTHERS.
7. Locating, protecting and rerouting/removal of all utilities are the responsibility of OTHERS. KNA is not responsible for damage to existing utilities.
8. After the completion of Ground Improvement work, OTHERS are responsible for the protection of DSM columns. Proper site drainage to prevent ponding of water at the area of the soil-mixed columns and control coordination of earthwork activities shall be managed such that existing soil-mixed columns are not damaged.
9. The DSM locations shown on the approved construction drawings are only for Ground Improvement layouts. These plans should not be used for foundation layout.
10. All post-improvement testing including frequency and criteria for soil-mixed columns are noted on the plans and design submittal.
11. Foundations shall not be poured until approved by the project Geotechnical Engineer of Record.
12. Alternate structural shapes, material, and details cannot be used unless reviewed and approved by the ground improvement engineer.

DSM VERIFICATION NOTES:

1. The acceptance of the work shall be based on demonstrating that the in-place mixing of grout with the treatment soils has achieved the average design strength requirements. Soilcrete strengths shall be determined statistically by wet (grab) sample and core samples. Confirmation sample collection and testing will be conducted by KNA. Samples shall be collected by KNA using wet sampling and continuous core sampling techniques described below. Test shall be performed at the frequencies described below.
2. Wet Soil mix samples will be retrieved and cast into molds for one column per rig/shift, at one random depth, typically near the end of each shift. Samples will be retrieved using an in situ wet sampler immediately after column construction and shall consist of no fewer than 8 specimens. These samples shall be tested in pairs: two at seven (7) days, two at fourteen days (14), two at twenty eight (28) days and two at fifty six (56) days if necessary. Soil clods greater than 10% of the mold diameter will be screened off. Appropriate curing techniques shall be implemented until testing based on ASTM D 1632.
3. Unconfined compression testing shall be performed by an approved laboratory working directly for the OWNER. Samples shall be tested in pairs starting at 7-days. If the 7-day specimens do not reach the desired strength according to the lab test curve, another pair of specimens will be tested at 14 days, 28 days, and if needed at 56 days. All specimens at 28 days and available 56-days of age will be tested and used in the statistical calculation.
4. If wet grab strengths at 7 days of age are greater than the average required unconfined compressive strength, additional tests may be omitted. Web grab samples will be kept on-site (approximately 3 days) for an intial set before being shipped to the lab.
5. The Unconfined Compressive Strength (UCS) shall be determined by ASTM D1633 "Standard Test Methods for Compressive Strength of Molded Soil-Cement Cylinders". Sulfur caps shall be required in the UCS tests to minimize the end effects on the test specimen. The advantage of the wet sampling is that KNA can get an early trend of the soilcrete strength development without waiting to the end of the project for coring and can make early decisions in the field program to add additional soil mixing columns if necessary.
6. KNA will core 1% of the production DSM columns.
7. All core locations shall be randomly selected. Coring will start after the soilcrete has gained adequate strength (100psi+) and verified by the strength development from the wet sample tests.
8. The double-tubes coring method, with the utilization of vibrators to assist the core to depth, can be used instead of the conventional coring technique. At minimum three (3) samples from each core will be extracted. KNA anticipates 4 specimens trimmed from each core hole and tested by ASTM D1633.
9. KNA will calculate the average 28-day UCS value from all core samples and wet grab samples. The target average 28 days UCS value shall be 150 psi or greater.
10. No more than 10 percent of all specimens tested shall exhibit an unconfined compressive strength of less than 75 psi at 28 day of age. A ceiling, the not-to-exceed value of four times the average unconfined compressive strength (i.e., 600 psi) shall be used for individual specimens in calculating the average strength achieved in the field from each coring and wet sample and for the entire project.
11. If the acceptance criteria is not achieved in a designated area, KNA may be given the opportunity to conduct additional UCS test on soilcrete specimens on 56 days of age, site exploration, coring, sampling, downhole imaging, and strength testing from the additional cured specimen to better define the average design strength at KNA's preference and expense. If a designated area is rejected, KNA shall submit a Remixing or Mitigation plan.
12. Uniformity of mixing shall be evaluated by the Geotechnical Engineer of Record (GEOR) based on the continuous core samples recovered. The continuous core holes shall extend the entire depth of the DSM column. Estimated recovery of 80 percent for each 5-foot-long segment of a boring and at least 90 percent when averaged over all core runs within a single boring shall be achieved. The lumps of unimproved soils shall not exceed 20 percent of the total volume of any 5-foot core segment from a boring. If the core recovery below the anticipated value due to the gravel particles in the soilcrete matrix, KNA shall be allowed to utilize a downhole camera or other approved methods to verify the core hole. This may include additional cores in the same column.
13. At the end of the project, to not unnecessary delay subsequent activities by waiting for 28 days test result, a correction of early strength gain will be used to approve the soil-mixed column work. However, this correlation will not relieve the contractor of the responsibility to achieve average 28 days strength. Based on FHWA (2013) guidelines, the following UCS aging factor correlations will be applied to this job:
    a. 28: 3 day, 1.72
    b. 28: 7 day, 1.35
    c. 28: 14 day, 1.15
14. A site-specific correlation between 3 days and 28 days strength may be used to supersede this correlation if in the opinion of the Engineer, the site-specific correlation is more appropriate.

DSM CONSTRUCTION:

1. OWNER will provide to KNA, at least four (4) control points. KNA will provide an AutoCAD Shop Drawing for all DSM columns overlaid on the site Civil drawing and stake all DSM locations.
2. DSM columns will be installed within 6 inches of the design locations as shown in the KNA shop drawing. Construction tolerances:
    a. Plan location ±6 inches
    b. Verticality ±1% of plumb
3. KNA retains the sole authority to modify DSM column locations due to constructability and/or site constraints. KNA will prepare as-built drawings after completion.
4. Once a stable working platform has been established as shown in KNA Shop Drawing. DSM columns will be constructed sequentially based on a pattern dictated in the Field. KNA requires access to all DSM locations at all times to maximize efficiency.
5. To minimize the mixing tool damage and maintaining soil mixing quality, KNA may pre-dill holes or excavate for better mixing quality. The holes will be filled with soilcrete up to the working elevation during the mixing stage.
6. In general, soil mixing operation parameters, such as mixing shaft speed, penetration rate, batching grout specific gravity, and pumping rate will be determined based on our lab mixing results and our experience and will be fine-tuned at the beginning of mixing column production. The design cement content in place (cement weight/[soil volume + grout volume]) will start from approximately 200 kg/m^3 with grout slurry specific gravity of 1.45. KNA's Engineers may adjust the cement content and specific gravity based on the field sample strength development.
7. Vertical alignment of the mix tool stroke will be controlled by the drill rig operator. Two measurements of verticality will be monitored. These are the fore-aft and left-right vertical mast positions. Verticality will be measured by a level as measured on the mixing tool prior to penetration. Intermittent measurements will be made as may be necessary during mixing operations.
8. The mixing shaft speed which is anticipated to be ranging between 20-50 RPM and shall be adjusted to accommodate a constant rate of mixing shaft penetration based on the degree of drilling difficulty. The mixing shaft speed can be adjusted according to drilling difficulty. The mixing shaft speed can be adjusted to aid mixing of the soil column when needed or to assist penetration in hard drilling. Mixing shaft speed will be recorded.
9. In order to ensure adequate mixing, the penetration rate of the mixing shaft shall be maintained at about 1.0 to 3.0 feet/minute during penetration but will vary based on actual site conditions. The penetration rate and maximum depth of each stroke shall be recorded by KNA's data acquisition system (DAQ).
10. The grout slurry flow per vertical foot of the column will be adjusted to the requirements of the design mix. Progressive cavity pumps will be used to transfer the grout from the mixing plant to the mixing rig. Flow monitoring devices will be installed in the grout line to detect any line blockage and monitor flow, total injected grout per column and grout pressure. These parameters will be recorded.
11. Inevitably some variations of the grout take will occasionally occur due to field conditions. It is anticipated that a grout flow rate between 50 to 250 GPM will be used during penetration. KNA's Data Acquisition System (DAQ) can automatically adjust the grout flow rate as a function of the penetration rate and maintain the pre-set cement dosage prescribed by the design engineer.
12. The mixing shaft will be withdrawn at a rate of 6 to 12 feet per minute during the re-stroke operation and complete removal of the mixing shaft from the ground thus mixed.
13. KNA will use a data acquisition system to monitor the mixing shaft penetration and the shaft rotation resistance in terms of the hydraulic pressure. The DAQ system will calculate and plot the Drilling Index as a function of depth, a mixing parameter to detect penetration resistance and refusal depth. KNA will set up the penetration criteria based on the site measurement. In case of underground obstruction, such as abandoned footings, piles, utilities, etc., the general contractor will be responsible to remove obstructions and backfilled with sandy soil prior KNA mixing operation.
14. Cement will be furnished by KNA and conform to ASTM C150 "Standard Specification for Portland Cement," Type II/V or equivalent. The cement will be adequately protected from moisture and contamination while in transit to and in storage at the job site. Reclaimed cement or cement containing lumps or deleterious matter will not be used.
15. Water for the slurry will be fresh, free of deleterious substances that adversely affect the strength and mixing properties of the slurry, furnished by the OTHERS.
16. The batch plant shall consist of in-line eductor (jet valve) mixers. Dry materials shall be stored in tankers and/or silos and fed to the mixers for shearing and circulation. The resulting grout slurry will be transferred to a surge tank for continuous agitation and to supply the in-situ soil mixing rig. Grout slurry quality will be assured by frequent testing prior to injection into the soil.
17. Single shaft mixing equipment that mechanically mixes the soil and cement slurry for the full dimensions of the column will be used for the work. We anticipate using hydraulic drill rigs for the soil mixing operations. This rig is capable of up to > 150,000 ft-lbs. of torque at > 20 rpm. The working shaft rate of rotation ranges between 20 and 50 rpm. The mixing shaft will have mixing augers and/or blades (paddles) configured in such a manner so that they are capable of thoroughly blending the in-situ soils and cement slurry. The power source for driving the mixing shafts will be sufficient to maintain the required mix tool (shaft) rotation speed in revolutions per minute and penetration/ withdrawal rates from the ground surface to the maximum depth required. The design target Blade Rotation Number (BRN, defined as the number of blades cut in each 1.0-meter soil) will be at least 300.
18. The DSM equipment will be equipped with devices to assure vertical alignment in two planes (90 degrees in plan from each other): fore-aft and left-right. The DSM equipment will be equipped with a real-time display of depth, rotation speed, grout flow rate; grout specific gravity, cumulative grout injected, and grout pressure for each soil mix column. The cement will be mixed with water within the jet valve to create a 1.45 specific gravity mix +/- .1. No mixing operation will be only allowed if the DAQ system not functioning.
19. Grout slurry will be supplied to the drill using large size Moyno pumps. These pumps will be sized and powered so that design volumes and pressures can be maintained up to 1,000 ft away from the batching facility. It is anticipated that a continuous grout slurry flow of 150 gallons per minute at 100 psi to the drill rig will be necessary.
20. The batching and pumping facility will be set up at a central location to areas all structures. This will eliminate the need to move the plant once it is established.

KELLER

17461 DERIAN AVENUE
SUITE 106
IRVINE, CALIFORNIA
92614

WWW.KELLER-NA.COM

Engineer of Record:

REGISTERED PROFESSIONAL ENGINEER
SUNIL ARORA
C72352
EXP. 09/30/22
CIVIL
STATE OF CALIFORNIA

STONE COLUMN NOTES AND DEATILS

COMPTON COMMUNITY COLLEGE

1111 E. ARTESIA BLVD.
COMPTON, CALIFORNIA 90221

No.	Description	Date
A	FOR REVIEW	09/08/2021

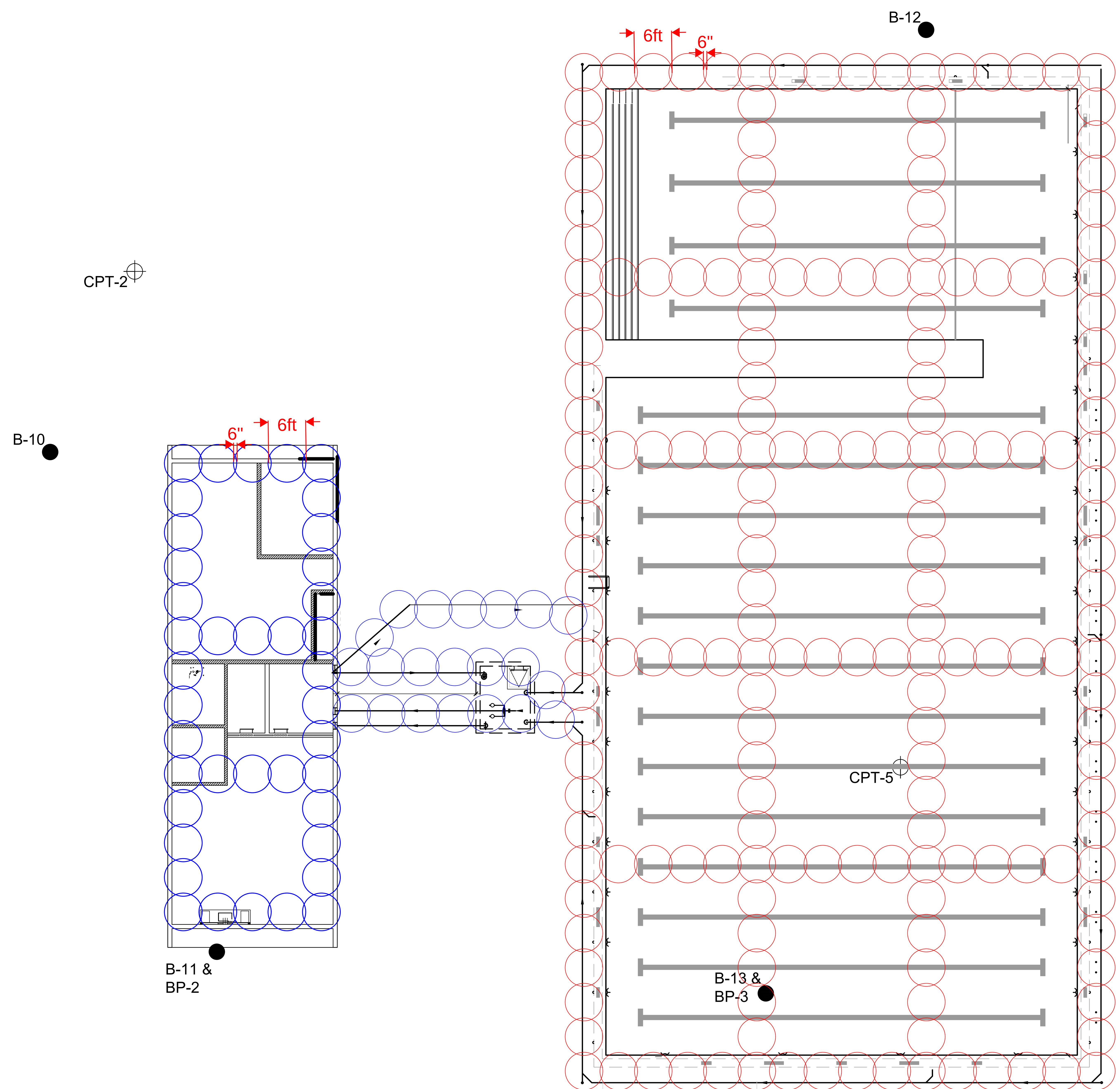
Design by: DC
Approved by: SA
Drawn by: SPB
Checked by:

Project Number:
Date: 09/08/2021

Sheet No.
KNA-2

Scale: N/A

1" 0" 1/2" 1"



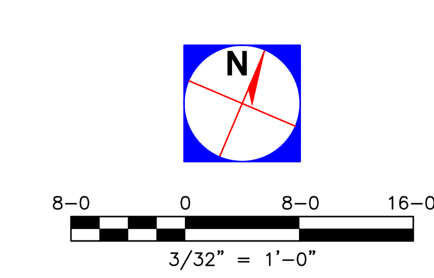
LEGEND:

Ø6' DEEP SOIL MIX COLUMN (DSM).
Install 49' from the existing ground surface.
Approximate existing ground surface elevation: 55 feet.

Ø6' DEEP SOIL MIX COLUMN (DSM).
Install 19' from the existing ground surface.
Approximate existing ground surface elevation: 55 feet.

CPT-X Existing CPT location

B-X Existing Boring location



KELLER
17461 DERIAN AVENUE
SUITE 106
IRVINE, CALIFORNIA
92614

WWW.KELLER-NA.COM

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C72352
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OVERALL GROUND IMPROVEMENT PLAN

COMPTON COMMUNITY COLLEGE

1111 E. ARTESIA BLVD.
COMPTON, CALIFORNIA 90221

No.	Description	Date
A	FOR REVIEW	09/08/2021

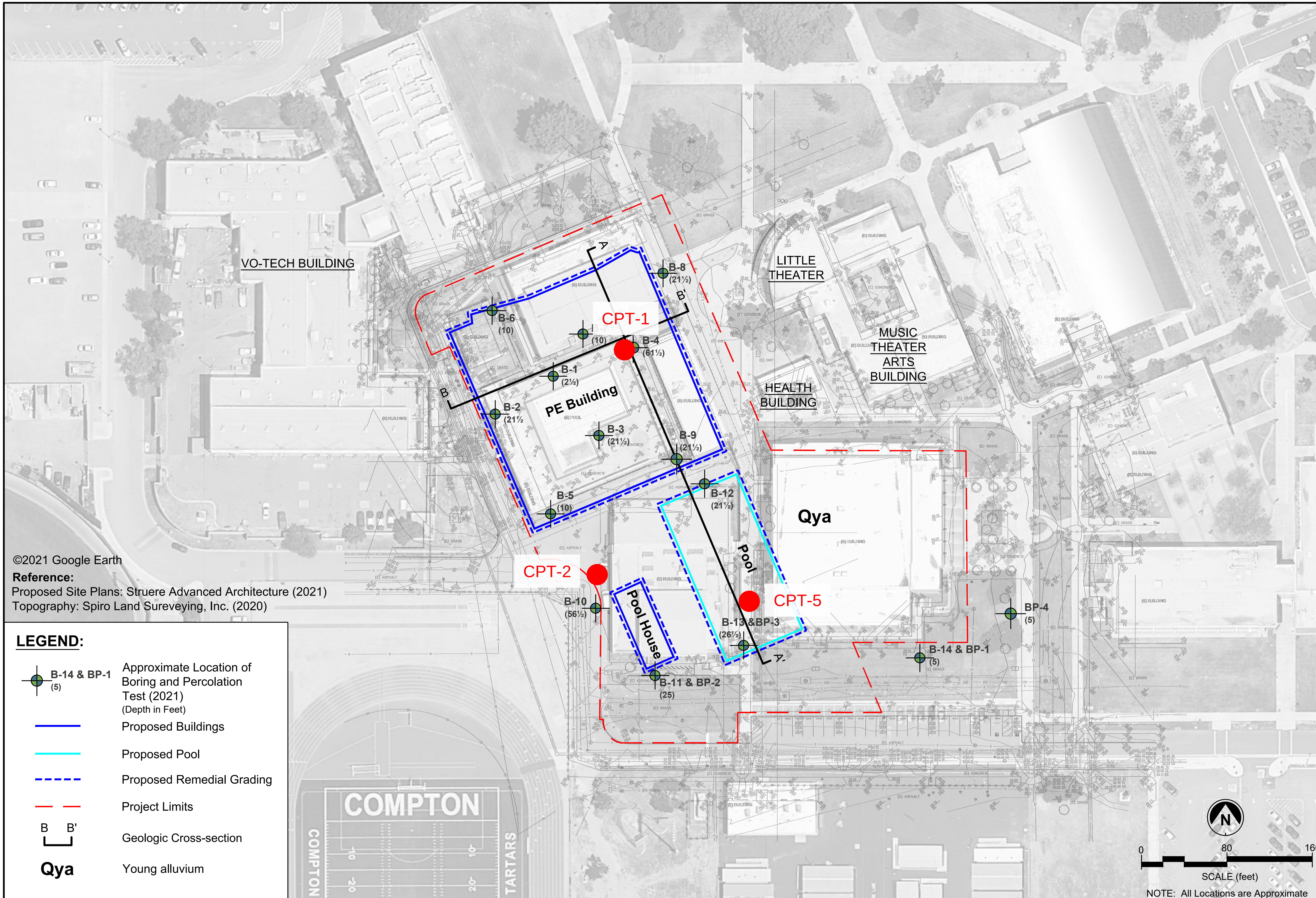
Design by: DC
Approved by: SA
Drawn by: SPB
Checked by:

Project Number:
Date: 09/08/2021

Sheet No.
KNA-3

Scale: 3/32" = 1'

Appendix B
Pre- and Post-treatment Liquefaction-induced Settlement Computation



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

GEOTECHNICAL MAP
Physical Education Complex Replacement
Compton, California

ATLAS

Figure: 2

Liquefaction Analysis and Deep Soil Mixing Mitigation

Project: OP0013298 - Compton Community College

CPT ID: CPT-2 (Pool Building) Surface Elev.: 0.0 ft (use 0 ft to plot depth instead of elevation)

KELLER

by: Bailey Uy
Date: 09.07.21

Perform Ground Improvement Analysis?: **Deep Soil Mixing**

LIQUEFACTION ANALYSIS PARAMETERS

Triggering Method =	Robertson (NCEER R&W 1998)	
Vol. Settlement Method =	Zhang et al. (2002)	
Depth of GW During CPT =	44.00	ft
Depth of GW During Earthquake =	8.00	ft
Depth of Fill =	0.00	ft
Unit Weight of fill =	120	pcf
PGApre =	0.802	g
Mw =	7.30	
Ic Threshhold =	2.6	
Use Kσ ? =	Yes	

ADVANCED LIQUEFACTION PARAMETERS

Use Ic Transition Zones?	Yes			
Transition zone (dIc / dz) =	0.65	Ic/ft	Manual Trans Zones?:	No
Cliq TZ dIc =	0.03		Use Manual Thin Layer Cor.? =	No
Min. Trans. Zone Points:	4			
Ic_min and max =	1.9	3		

DSM GRID DESIGN PARAMETERS

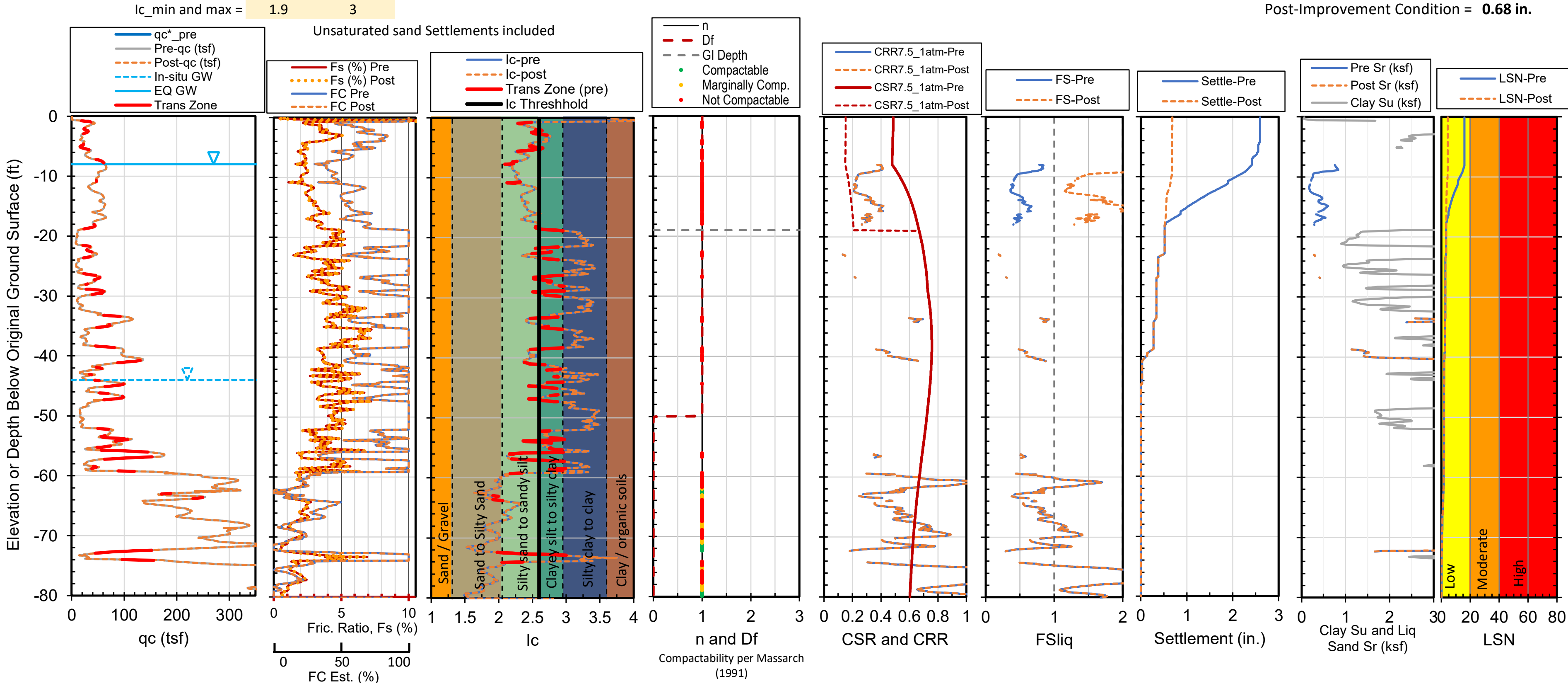
Depth Below Existing Grade =	19	ft
ARR =	30	%
S =	28	ft
Gr =	30	
Rrd = PGApost/PGApre =	0.314	
PGApost in Impr. Zone =	0.252	g

Volumetric Settlement Results:

Existing (Pre-Treatment) Condition = **2.59 in.**

Post-Improvement Condition = **0.68 in.**

Unsaturated sand Settlements included



Liquefaction Analysis and Deep Soil Mixing Mitigation

KELLER

Project: OP0013298 - Compton Community College

CPT ID: CPT-5 (Swimming Pool)   Surface Elev.: 0.0 ft   (use 0 ft to plot depth instead of elevation)

by: Bailey Uy

Date: 09.07.21

Perform Ground Improvement Analysis?: **Deep Soil Mixing**

LIQUEFACTION ANALYSIS PARAMETERS

Triggering Method =	Robertson (NCEER R&W 1998)	
Vol. Settlement Method =	Zhang et al. (2002)	
Depth of GW During CPT =	44.00	ft
Depth of GW During Earthquake =	8.00	ft
Depth of Fill =	0.00	ft
Unit Weight of fill =	120	pcf
PGApre =	0.802	g
Mw =	7.30	
Ic Threshhold =	2.6	
Use Kσ ? =	Yes	

ADVANCED LIQUEFACTION PARAMETERS

Use Ic Transition Zones?	Yes			
Transition zone (dIc / dz) =	0.65	Ic/ft	Manual Trans Zones?:	No
Cliq TZ dIc =	0.04		Use Manual Thin Layer Cor.? =	No
Min. Trans. Zone Points:	4			
Ic_min and max =	1.9	3		

DSM GRID DESIGN PARAMETERS

Depth Below Existing Grade =	49	ft
ARR =	30	%
S =	28	ft
Gr =	30	
Rrd = PGApost/PGApre =	0.229	
PGApost in Impr. Zone =	0.184	g

Volumetric Settlement Results:

Existing (Pre-Treatment) Condition = **2.79 in.**

Post-Improvement Condition = **0.09 in.**

Unsaturated sand Settlements included