Appendix F

Geotechnical Investigation

GEOTECHNICAL INVESTIGATION

Capitol Annex Project
1315 10th Street
Sacramento, California

PREPARED FOR:

DGS / RESD / PMDB
707 3RD STREET, 4TH FLOOR, MS-504
WEST SACRAMENTO, CALIFORNIA 95605

PREPARED BY:

GEOCON CONSULTANTS, INC.
3160 GOLD VALLEY DRIVE, SUITE 800
RANCHO CORDOVA, CALIFORNIA 95742

GEOCON

GEOCON PROJECT NO. S1748-05-01 **AUGUST 2019**

Project No. S1748-05-01
August 13, 2019

VIA ELECTRONIC MAIL

Joel Griffith
Capital Outlay Program Manager
DGS / RESD / PMDB
707 3rd Street, 4th Floor, MS-504
West Sacramento, California 95605

Subject: GEOTECHNICAL INVESTIGATION
CAPITOL ANNEX PROJECT
SACRAMENTO, CALIFORNIA

Mr. Griffith:

In accordance with your approval of our proposal (Geocon Proposal No. LS-19-37B dated March 12, 2019) we have prepared this geotechnical investigation report for the subject project. The project consists of a new visitor center, parking structure, and annex building at the California State Capitol grounds in Downtown Sacramento, California.

The accompanying report presents our findings, conclusions, and recommendations pertaining to the geotechnical aspects of designing and constructing the project as presently proposed. This report provides a seismic hazards evaluation and design-level geotechnical recommendations for the proposed improvements. In our opinion, no adverse geotechnical or geologic conditions are present that would preclude development at the site provided the recommendations of this report are incorporated into design and construction of the project. The primary geotechnical constraints identified at the site include existing fill, soft compressible silt/clay, and shallow groundwater. These constraints are discussed and mitigation alternatives are provided in this report.

Please contact us if you have any questions concerning the contents of this report. We look forward to reviewing the project plans as they develop further, providing engineering consultation as needed and performing geotechnical observation and testing services during construction.

Sincerely,

GEOCON CONSULTANTS, INC.

REGISTERED PROFESSIONAL ENGINEER
JEREMY J. ZORNE
No. 2636
GEOTECHNICAL
STATE OF CALIFORNIA

Jeremy J. Zorne, PE, GE
Senior Engineer

Sean M. Dixon, PG
Senior Project Geologist

Ronald E. Loutzenhiser, PE, GE
Senior Engineer (Quality Assurance Reviewer)

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GEOTECHNICAL INVESTIGATION

1.0 INTRODUCTION AND SCOPE OF SERVICES

This report presents the results of our geotechnical investigation for the proposed visitor center, parking structure, and annex building at the California State Capitol grounds in Downtown Sacramento, California. The approximate site location is shown on the Vicinity Map, Figure 1.

The purpose of our investigation was to explore and evaluate subsurface soil, geologic, and seismic conditions at the site and provide conclusions and design-level recommendations for the project as presently proposed.

To prepare this report, we performed the following scope of services:

- Reviewed area geologic maps, previous geotechnical investigation reports for nearby projects, and other technical literature pertaining to the site and vicinity (see References in Section 9.0 of this report).
- Performed a site reconnaissance to review project limits, determine exploration equipment access, and mark out exploratory excavation locations.
- Coordinated with the Department of General Services (DGS), California Highway Patrol (CHP), and Capitol grounds staff for our field investigative activities.
- Notified subscribing utility companies via Underground Service Alert (USA) a minimum of two working days (as required by law) prior to performing exploratory excavations at the site.
- Paid required fees and obtained a subsurface exploration permit from the Sacramento County Environmental Management Department (SCEMD).
- Performed six cone penetrometer (CPT) soundings (C1 through C6) to refusal depths ranging from approximately 29 to 51 feet. CPT soundings were performed in general accordance with American Society for Testing and Materials (ASTM) Procedure D5778.
- Performed seismic shear wave velocity measurements at approximate 5-foot intervals in CPT1, CPT3, and CPT4.
- Performed six exploratory borings utilizing a truck-mounted drill rig equipped with hollow-stem auger and mud rotary drilling equipment to depths ranging from approximately 30 to 71 feet.
- Obtained disturbed and undisturbed soil samples from the borings using Standard Penetration Test (SPT) and California-modified driven split-spoon samplers.
- Logged the exploratory boring in accordance with ASTM D2487 which is based on the Unified Soil Classification System (USCS).
- Completed two borings (B1 and B3) as temporary piezometers using PVC casing and a flush-mount, traffic rated well cover for long-term groundwater level measurements.
- Containerized the drill cuttings in sealed 55-gallon steel drums and temporarily stored onsite pending characterization. Subsequently disposed the drums at a licensed waste acceptance facility.
- Upon completion, backfilled the borings and CPT soundings with neat cement grout in accordance with SCEMD permit requirements.
- Performed laboratory tests on selected samples to evaluate pertinent geotechnical parameters.

- Performed engineering analysis to evaluate seismic design parameters, liquefaction potential, foundation design parameters, and earthwork recommendations.
- Prepared this report summarizing our findings, conclusions, and recommendations regarding the geotechnical aspects of designing and constructing the project as presently proposed.

Approximate locations of subsurface explorations are shown on the Site Plan, Figure 2. Details of our field exploration program including exploration logs are presented in Appendix A. Details of our laboratory testing program and test results are summarized in Appendix B. Results of our liquefaction analysis are presented in Appendix C.

2.0 SITE AND PROJECT DESCRIPTION

The California State Capitol grounds comprises approximately 38 acres and is bounded by 10th Street on the west, L Street on the north, 15th Street on the east, and N Street on the south. The State Capitol building and existing Annex occupies the western portion of the grounds and fronts 10th Street. The site configuration is shown on the Site Plan, Figure 2.

Based on site-specific topography provided by Morrow Surveying (*ALTA/NSPS Land Title Survey*, July 10, 2019), the elevation of 10th, 11th, N, and L Streets adjacent to the site ranges from approximately 17 to 19 feet. The elevation of the first level of the Capitol and Annex buildings (top of the front steps) is approximately 30 feet. The elevation of the basement level of the existing Capitol and Annex Buildings is approximately 14 feet. The elevations presented on the ALTA/NSPS Land Title Survey are based on the National Geodetic Vertical Datum of 1929 (NGVD29). Unless otherwise stated, all elevations stated in this report are referenced to this datum.

The project consists of constructing: (1) a new underground Visitor Center, (2) a new underground parking structure, and (3) and a new five-story Capitol Annex Building. The approximate project layout is shown on the Site Plan, Figure 2.

The Visitor Center and Parking Structure will be one-level (+/- 15 feet) below grade and will include cast-in-place concrete walls and top decks. The Annex Building will be a five-level (basement plus four levels above), structural steel-framed building. The basement will extend approximately 15 feet below grade. The Annex Building will have a total height less than 100 feet and will not be subject to the performance-based seismic design. Structural loading was not provided to us for review. However, we anticipate moderate to heavy structural loading for the Annex Building and relatively lighter loading for the subterranean Visitor Center and Parking Structure. Therefore, we anticipate that the Annex Building will be supported on deep foundations and the Visitor Center and Parking Structure will be supported on conventional shallow foundations or a mat foundation.

Other proposed improvements will likely include new underground utility infrastructure and street-level improvements such as sidewalks and minor landscaping. Grading plans were not available as of the date of this report. We anticipate that the majority of grading and earthwork will involve mass excavation to attain design grades within the building areas. We do not anticipate any significant fill placement to raise grades around the project area.

3.0 SOIL AND GEOLOGIC CONDITIONS

We identified soil and geologic conditions by observing exploratory borings, in-situ CPT soundings, and reviewing the referenced geologic literature (Section 9.0). The soil descriptions provided in this report include the Unified Soil Classification System (USCS) symbol where applicable. Regional and local geology is shown on the Geologic Map, Figure 3. General subsurface profiles through the site are presented as Cross-Sections A-A' and B-B', Figures 4 and 5, respectively.

3.1 Regional and Local Geology

The site is located within the Great Valley Geomorphic Province of California, more commonly referred to as the Central Valley. The Central Valley is a broad depression bounded by the Sierra Nevada mountain range to the east and the Coast Ranges to the west. The valley has been filled with a thick sequence of sediments derived from weathering of the adjacent mountain ranges resulting in a stratigraphic section of Cretaceous, Tertiary, and Quaternary deposits.

The site is located near the southern end of the Sacramento Valley, approximately one mile east of the Sacramento River and approximately one mile south of the American River. Published geologic mapping depicts the site vicinity underlain by Quaternary-age, Holocene alluvial deposits (map symbol Qha), which generally consists of interbedded mixtures of alluvial sand, silt, clay, and gravel (California Geological Survey [CGS], 2011). Regional and local geology is shown on the Geologic Map, Figure 3.

3.2 Fill

We encountered fill within each boring from the ground surface to depths ranging from 3½ to 11 feet. The fill generally consisted of soft to medium stiff, sandy silt (ML) and loose to medium dense silty sand (SM) with some occasional gravel and broken brick fragments. We expect that the fill material will be removed by excavation of the proposed basement level of the Annex and below-grade Visitor Center/Parking Structure. This material will be retained at the perimeters of the proposed structures with temporary construction shoring and permanent basement/retaining walls.

3.3 Alluvium (Recent and Older)

Below the fill, we encountered alluvial soils extending beyond our maximum depth of exploration of 71 feet. The alluvium can be subdivided into two distinct units: "recent" alluvium and "older" alluvium as described herein and shown on Cross-Sections A-A' and B-B', Figures 4, and 5. The depths/elevations described below are approximate and assume an average site elevation of 25 feet.

- <u>0 to 30 feet (elevation +25 to - feet):</u> "recent" alluvium consisting of layers of soft to medium-stiff silt (ML) and lean clay (CL) interbedded with loose to medium dense silty and poorly graded sand. This material has high in-situ moisture content (approximately 10% to 40%). This material is generally suitable for support of light to moderate foundation loading (such as the subterranean Visitor Center and Parking Structure) but is not suitable for the heavier foundation loading anticipated for the Annex Building. Therefore, deep foundations will be required for support of this structure.
- <u>30 feet and deeper (elevation -5 and lower):</u> "older" alluvium consisting of interbedded of very stiff to hard (cemented) clay (CL), dense to very dense gravel (GP), and medium dense to dense clayey sand. This material is considered to be a suitable bearing layer for the deep foundations required for the Annex Building.

The "older alluvium" is the bearing layers for most deep foundation systems in downtown Sacramento. The composition of the older alluvium is variable; where some areas contain a heavy concentration of large gravel and small cobble, and other areas contain very little, if any, gravel. The depth to the top of this layer at the site ranges from approximately 25 to 40 feet, with an average of approximately 30 feet.

Soil and geologic conditions described in the previous paragraphs and shown on Cross-Sections A-A' and B-B' (Figures 4 and 5), are generalized. The exploration logs included in Appendix A detail soil type, color, moisture, consistency, and USCS classification of the soils encountered at specific locations and elevations.

4.0 GROUNDWATER

Table 4.0A summarizes the depth to groundwater/seepage encountered in our exploratory borings.

TABLE 4.0A
DEPTH TO GROUNDWATER / SEEPAGE IN BORINGS

Boring ID	Date	Depth to Groundwater/Seepage at Time of Drilling (feet)
B1	6/10/2019	19
B2	6/10/2019	19
B3	6/11/2019	15
B4	6/11/2019	21
B5	6/12/2019	19
B6	6/12/2019	17

Depth to groundwater / seepage was measured in each boring with a cloth tape during and/or immediately after drilling. The accuracy of measurements should be considered to be within one foot. We note that the State Capitol grounds are irrigated and we understand that breaks/leaks in irrigation lines are common. Therefore, some of the groundwater/seepage observed in the borings may be attributed to leaking irrigation lines and not representative of static groundwater conditions.

To better assess static groundwater conditions, we completed Borings B1 and B3 as temporary piezometers for long-term groundwater elevation monitoring. The borings were completed with 2-inch-diameter PVC casing with a 10-foot-long, 0.020-inch screened interval between 20 and 30 feet and a sand filter extending up to 10 feet. Groundwater elevation measurements to date are presented in Table 4.0B.

TABLE 4.0B
GROUNDWATER ELEVATION SUMMARY (PIEZOMETERS)

Piezometer	Date	Depth to Groundwater (feet[1,2])	Top of Casing Elevation[3]	Groundwater Elevation (feet NGVD29)
B1	6/10/2019	19.0	19.6	+0.6
	6/25/2019	17.0		+2.6
	7/08/2019	15.6		+4.0
	7/11/2019	15.7		+3.9
B3	6/11/2019	19.0	19.5	+0.5
	6/25/2019	17.0		+2.5
	7/08/2019	15.6		+3.9
	7/11/2019	15.6		+3.9

Notes:

1. *Depth below top of casing.*
2. *Depth to groundwater was measured with an electronic well-sounder (accuracy within 0.1 foot).*
3. *Top of Casing Elevation per Morrow Surveying (2019), NGVD29 datum*

Review of the *Spring 2007 Sacramento County Groundwater Elevation Map* (County of Sacramento, Water Resources Division, April 2007) indicates that the average springtime (seasonal high) groundwater elevation in the site vicinity is approximately +0 feet mean sea level (MSL), which corresponds to approximately 20 to 30 feet below existing grades at the site, depending on location. Based on our experience in Downtown Sacramento, reported elevations based on "MSL" are generally within one-half foot of reported elevations based on NGVD29. For the purposes of this report, we assume that the elevations based on both datums (MSL and NGVD29) are roughly equal.

We reviewed available groundwater elevation monitoring for nearby groundwater monitoring wells available on the State Water Resources Control Board GeoTracker website (https://geotracker.waterboards.ca.gov/). Depth to groundwater measurements in three shallow groundwater monitoring wells associated with the Hyatt Regency located on the north side of L Street, approximately 300 feet north of the site. Groundwater elevation information was available for the 3-year period between January 2000 through July 2003. Groundwater elevation measured during this period fluctuated between a low of -2.7 feet to a high of approximately +2.6 feet.

In addition, we reviewed historic groundwater elevation information for some of the existing Railyards "South Plume" groundwater monitoring wells located near the site (Wells SPW-20 and WCC-71). Groundwater elevation information was available for the 11-year period between March 2006 and 2017.

Figure 6 shows plots of the various groundwater elevation measurements from the referenced nearby monitoring wells, SP wells, and onsite piezometers (B1 and B3). The date range roughly covers 19 years between 2000 and 2019. As shown In Figure 6, the groundwater elevation at the site fluctuates seasonally and is generally highest during spring and early summer. The average (mean) groundwater elevation at the site is approximately +2 feet and the seasonal high groundwater elevation often approaches +7 feet. We recommend that the project designer consider using one of the two following groundwater elevations for design:

- Option 1 – average groundwater elevation plus one standard deviation: +4.5 feet
- Option 2 – average groundwater elevation plus two standard deviations: +6.5 feet

Since the historic groundwater elevation data presented herein covers a relatively short date range and some of the wells are located some distance from the site, Option 2 may be a more appropriate (conservative) assumption for project design. In any event, hydrostatic pressure will need to be considered in project design of subterranean structures.

Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, and other factors. Groundwater depth at the site is likely influenced by the level of water in the nearby Sacramento and American Rivers.

5.0 GEOLOGIC HAZARDS

5.1 Mapped Geologic Hazard Zones

The site is not located in any currently established official geologic hazard zones (e.g. liquefaction, active faulting, landslides) established by CGS or the City of Sacramento Specific Plan.

5.2 Faulting / Ground Rupture

Based on our research, analyses, and observations, the site is not located on any known "active" earthquake fault trace. In addition, the site is not located within an Alquist-Priolo Earthquake Fault Zone. Therefore, we consider the potential for ground rupture due to onsite active faulting to be low.

To determine the distance of known active faults within 50 miles of the site, we used the 2013 *Caltrans Fault Database* KML overlay file for Google Earth. Principal references used within the 2013 Caltrans Fault Database are the *Fault Activity Map of California* (Jennings and Bryant 2010) , *Working Group on California Earthquake Predictions* (WGCEP), and *Uniform California Earthquake Rupture Forecast Version 3*. Results are summarized in Table 5.2.

TABLE 5.2
REGIONAL FAULT SUMMARY

Fault Name	Approximate Distance from Site (miles)	Maximum Earthquake Magnitude, M_w
Foothills Fault System	22	6.5
Great Valley, Segment 4	28	6.6
Great Valley, Segment 3	28	6.8
Great Valley, Segment 5	30	6.5
Hunting Creek – Berryessa	39	6.9
Concord – Green Valley	39	6.9
Great Valley, Segment 6	41	6.7
West Napa	49	6.5
Greenville	49	6.9

5.3 Historical Earthquakes and Ground Shaking

The Sacramento region of Northern California has a history of relatively low seismicity in comparison to more active seismic regions such as the Bay Area or Southern California. The two most commonly referenced earthquakes that resulted in some reported building damage in Downtown Sacramento are the Winters and Vacaville events in 1892. There are no reported occurrences of seismic-related ground failure in the Sacramento region due to earthquakes.

We used the United States Geological Survey (USGS) *Unified Hazard Tool* (https://earthquake.usgs.gov/hazards/interactive/) to determine the deaggregated seismic source parameters including controlling magnitude and fault distance. The USGS estimated modal magnitude is 6.6 and the estimated Peak Ground Acceleration (PGA) for the Maximum Considered Earthquake (MCE) with a 2,475-year return period is 0.35g.

5.4 Liquefaction

Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary loss of shear strength due to pore pressure buildup under the cyclic shear stresses associated with intense earthquakes. Primary factors that trigger liquefaction are: moderate to strong ground shaking (seismic source), relatively clean, loose granular soils (primarily poorly graded sands and silty sands), and saturated soil conditions (shallow groundwater).

The site is not located in a currently established State of California Seismic Hazard Zone for liquefaction. In addition, we are not aware of any reported historical instances of liquefaction in the greater Sacramento area. However, soil and groundwater conditions exist at the site that may be susceptible to seismic-induced liquefaction under the design-level seismic event.

We analyzed liquefaction potential in general accordance with the *Recommended Procedures for Implementation of DMG Special Publication 117A, Guidelines for Analyzing and Mitigating Liquefaction in California"* (Southern California Earthquake Center, 2002). We used the computer software program CLiq (Version 1.7, Geologmiski) and the in-situ soil parameters measured in our CPT soundings. The software utilizes the 1998 *National Center for Earthquake Engineering Research* (NCEER) method of analysis which was developed with the broad consensus of national geotechnical and earthquake engineering experts. We used a groundwater depth of 15 feet below existing grade, an earthquake moment magnitude of 6.6 (USGS deaggregation), and a MCE (2,475-year return interval event) PGA of 0.31g (PGA_M adjusted for Site Class per American Society of Civil Engineers [ASCE] 7-10, Eq. 11.8-1).

Based on the results of our analyses, there is the potential for liquefaction at the site within apparently discontinuous, relatively thin sandy soil layers generally present between depths of approximately 15 to 40 feet. Consequences of liquefaction may include ground surface settlement, ground loss (sand boils), and lateral slope displacements (lateral spreading). Dynamic settlement of the soils that experience liquefaction may occur after earthquake shaking has ceased. We estimated potential dynamic settlements of liquefied soil layers using the computer program CLiq (Version 1.7, Geologmiski) and the in-situ soil parameters measured in the CPT soundings. The results of the analysis indicate relatively small total liquefaction settlements ranging from less than 0.1 inches to approximately 0.4 inches. Given the apparently discontinuous and relatively thin liquefiable layers, the incremental and total settlements are considered negligible and need not be considered for structural design.

The Liquefaction Potential Index (LPI) is an index used to assess liquefaction hazard of surficial geologic units. LPI was originally developed in Japan to estimate the potential of liquefaction to cause foundation damage at a site (Iwasaki 1978). The index assumes that the severity of liquefaction is proportional to the:

1. thickness of the liquefied layer;
2. proximity of the liquefied layer to the surface; and
3. amount by which the factor safety (FS) is less than 1.0, where FS is the ratio of the liquefaction resistance to the load imposed by the earthquake.

Researchers (Toprak and Holzer) correlated surface manifestations of liquefaction with LPI for the 1989 Loma Prieta, California, earthquake and concluded that sand boils and lateral spreading occur primarily where LPI > 5 and >12, respectively. The estimated LPI at this site ranges from approximately 0 to 1.5. Based on this criteria, there is low potential for both sand boils (ground loss) and lateral spreading at the site due to a liquefaction event.

Given the above, the potential for seismic-induced liquefaction is considered to be low and no special design measures with respect to liquefaction are considered necessary for the project. Details of our liquefaction analysis are presented in Appendix C.

5.5 Expansive Soil

Laboratory testing (Appendix B) for the fine-grained soils at the site indicate relatively low plasticity and low expansion potential. Mitigation and/or special design considerations with respect to expansive soil is not necessary for the project.

5.6 Soil Corrosion Screening

We performed a soil corrosion potential screening by conducting laboratory testing on a representative near-surface soil sample. The laboratory test results and published screening levels are presented in Appendix B.

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 General Discussion

6.1.1 No soil or geologic conditions were encountered during our investigation that would preclude the proposed development as presently planned, provided the recommendations contained in this report are incorporated into design and construction of the project.

6.1.2 Based on the anticipated structural loading and the subsurface conditions at the site, the subterranean Visitor Center and Parking Structure may be supported on shallow foundations such as reinforced concrete mat foundations. Since these foundations must resist some hydrostatic pressure from groundwater, we do not recommend using conventional spread footings with concrete slabs-on-grade. Given the higher anticipated structural loading, the Annex Building will require deep foundations support. At this time, structural design loading is not available. Therefore, we are providing general recommendations for both mat foundations and deep foundations. Once design loads are established, we should review and provide revised recommendations as necessary.

6.1.3 The excavation for subterranean structures may likely extend near or below the groundwater elevation, depending on time of year. We expect that exposed subgrade soil in the mass excavation will be wet and unstable. Stabilization measures will likely be necessary, depending on the degree of instability. Specific recommendations are provided herein.

6.1.4 Construction dewatering measures may be required to control groundwater seepage during excavation and construction. Based on our experience, proper and sufficient dewatering is of extreme importance to the performance of the excavation shoring and maintaining subgrade stability for construction of the foundation. Therefore, a properly-designed, installed and operated dewatering system is considered essential for the project. Recommendations for excavation bottom stabilization and temporary construction dewatering are provided herein.

6.1.5 Groundwater can seasonally rise as high as approximately +7 feet and must be considered in the project design. Waterproofing of subterranean walls will be required. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method that would provide protection to subterranean walls, floor slabs and foundations. In addition, an experienced waterproofing inspector should be retained to check proper installation of the system during construction.

6.1.6 Improvements which are not supported by the building foundations, such as walkways, paving, and utilities, may experience post-construction settlement due to variable-consistency undocumented fill and underlying soft soils. The project team should consider the flexibility of the products, pavements, and improvements being installed. For example, using interlocking pavers which typically conform to differential settlement and are easily repaired, may be preferred over rigid concrete flatwork. Utilities traversing through existing site soils should use flexible connections in order to minimize the damage to underground installations caused by potential soil movements.

6.1.7 We should be retained to review the project plans as they develop further, provide engineering consultation as-needed, and perform geotechnical observation and testing services during construction. The recommendations contained in this report are preliminary until verified during construction by representatives of our firm.

6.2 Seismic Site Class / Seismic Design Criteria

6.2.1 Seismic design of structures should be performed in accordance with the provisions of the 2016 California Building Code (CBC) which is based on the 2015 International Building Code (IBC) and the ASCE publication: *Minimum Design Loads for Buildings and Other Structures* (ASCE 7-10).

6.2.2 In accordance with the 2016 CBC, we evaluated seismic Site Class in accordance with Chapter 20 of ASCE 7-10. As discussed in Section 5.4 of this report, there is a low potential for liquefaction and associated ground failure or collapse, therefore Site Class "F" does not apply. Based on the subsurface conditions encountered in our borings and CPT soundings, the three required criteria for Site Class "E" (Section 20.3.2 of ASCE 7-10) do not apply. Therefore, we evaluated seismic Site Class on the basis of in-situ shear wave velocity measured in our CPT soundings. The estimated average shear wave velocity (per ASCE 7-10, Eq. 20.4-1) is approximately 760 feet/sec. Therefore, the site is classified as Site Class "D" per Table 20.3-1 of ASCE 7-10.

6.2.3 Seismic design of the structures should be performed in accordance with the provisions of the 2016 California Building Code (CBC) which is based on the American Society of Civil Engineers (ASCE) publication: *Minimum Design Loads for Buildings and Other Structures* (ASCE 7-10). We used the Structural Engineers Association of California (SEAOC) and Office of Statewide Health Planning and Development (OSHPD) web application *Seismic Design Maps* (https://seismicmaps.org/) to evaluate site-specific seismic design parameters in accordance with the 2016 CBC/ASCE 7-10. We assumed a seismic Risk Design Category II (per 2016 CBC Table 1604.5) for the project. Results are summarized in Table 6.2.3. The values presented are for the risk-targeted maximum considered earthquake (MCE_R).

TABLE 6.2.3
2016 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2016 CBC / ASCE 7-10 Reference
Site Class	D	Section 1613.3.2/ Table 20.3-1
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	0.677g	Figure 1613.3.1(1) / Figure 22-1
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.294g	Figure 1613.3.1(2) / Figure 22-2
Site Coefficient, F_A	1.258	Table 1613.3.3(1) / Table 11.4-1
Site Coefficient, F_V	1.812	Table 1613.3.3(2) / Table 11.4-2
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	0.852g	Eq. 16-37 / Eq. 11.4-1
Site Class Modified MCE_R Spectral Response Acceleration (1 sec), S_{M1}	0.532g	Eq. 16-38 / Eq. 11.4-2
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.568g	Eq. 16-39 / Eq. 11.4-3
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.355g	Eq. 16-40 / Eq. 11.4-4

6.2.4 Table 6.2.3 presents additional seismic design parameters for projects with Seismic Design Categories of D through F in accordance with ASCE 7-10 for the mapped maximum considered geometric mean (MCE_G).

TABLE 6.2.3
2016 CBC SITE ACCELERATION DESIGN PARAMETERS

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.231g	Figure 22-7
Site Coefficient, F_{PGA}	1.339	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.309g	Section 11.8.3 (Eq. 11.8-1)

6.2.5 Conformance to the criteria presented in Tables 6.2.2 and 6.2.3 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a maximum level earthquake occurs. The primary goal of seismic design is to protect life and not to avoid structural damage, since such design may be economically prohibitive.

6.3 Soil and Excavation Characteristics

6.3.1 In our opinion, site soils can be excavated with light to moderate effort using conventional heavy duty grading equipment. Slumping and caving should be expected in un-shored excavations, especially where saturated or granular soils are encountered.

6.3.2 Optimum moisture content for the silty soil expected to be encountered in excavations at the site ranges between approximately 11% to 15%. Measured in-situ moisture content for soil encountered in our borings ranged from approximately 10% to 40% which is significantly above optimum. Therefore, significant instability should be expected in project excavations and exposed subgrades as well as difficulty achieving compaction when excavated, onsite soil is placed as backfill.

6.3.3 Contractors should be aware of the high in-situ moisture content, moisture sensitivity, and potential compaction/workability difficulties. The contractor should expect that, at a minimum, aerating/drying soils will be required to achieve proper compaction. If aerating/drying the soils is too slow based on weather conditions at the time, chemical treatment may be an alternative. We should evaluate unstable soil conditions in the field at the time of construction and determine the type, level, and extent of mitigation alternatives as necessary.

6.3.4 Project excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the surcharging object. Penetrations below this 1:1 projection will require special excavation measures such as shoring.

6.3.5 Temporary excavations must meet Cal/OSHA requirements as appropriate. Excavation sloping, benching, shoring, the use of excavation shields, and the placement of trench spoils should conform to the latest applicable Cal/OSHA standards. The contractor should have a Cal/OSHA-approved "competent person" onsite during excavation to evaluate trench conditions and to make appropriate recommendations where necessary. It is the contractor's responsibility to provide sufficient and safe excavation support as well as protecting nearby utilities, structures, and other improvements, which may be damaged by earth movements.

6.3.6 The excavation support recommendations provided by Cal/OSHA are generally geared towards protecting human life and not necessarily towards preventing damage to nearby structures or surface improvements. The contractor should be responsible for using the proper active shoring systems or underpinning measures to prevent damage to any structure or improvements located near excavations.

6.3.7 Permanent cut and fill slopes (if any) should be constructed no steeper than 2H:1V (horizontal to vertical). To mitigate potential erosion, slopes should be vegetated as soon as possible and surface drainage should be directed away from the tops of slopes.

6.4 Groundwater and Construction Dewatering

6.4.1 The groundwater elevations measured in nearby groundwater monitoring wells over a 19-year time period fluctuates seasonally between about elevation -3 and +7 feet (see Figure 6). Based on this information we recommend assuming a seasonal high groundwater elevation of +6.5 feet which is based on the annual average groundwater elevation plus two standard deviations.

6.4.2 This groundwater elevation is higher than the subterranean structure bottom elevations. The structures will likely include elevator pits and utility excavations that may extend below this depth. To increase excavation bottom stability, construction dewatering should lower groundwater levels to at least <u>3 feet below planned excavation depths</u>. Based on our experience, proper and sufficient dewatering is of extreme importance to the performance of the excavation shoring and maintaining subgrade stability for construction of foundations. Therefore, a properly-designed, installed and operated dewatering system is considered essential for the project. Based on our experience in the area, conventional pumping dewatering wells tend to work better than low-pressure vacuum wellpoint systems. Key factors that may influence dewatering system performance include the number of wells, the depth and positioning of the wells, the screen intervals of the wells, and the pumping rates. Different combinations of these variables can be used to successfully dewater the site.

6.4.3 Special care must be taken to reduce the removal of fines from the granular layers during dewatering. A properly designed and installed filter pack in the dewatering wells will reduce the removal of fines; however, monitoring the discharge effluent for fines should be performed. The dewatering system should be operated until sufficient structure weight and/or uplift capacity is available to resist the hydrostatic uplift forces on the bottom of the foundation (as determined by the structural engineer).

6.4.4 Based on the size of the structures, perimeter wells only may not sufficiently dewater the central portion of the excavations and some interior wells may also be needed. A gravel working pad (as discussed in Section 6.5) can also be used as a temporary drainage blanket in addition to dewatering wells. Perforated drain pipes may be placed in the gravel to collect and convey water to sumps for removal. The sumps and collector pipes should be decommissioned once they are no longer needed as they may interfere with subsequent foundation construction.

6.4.5 In general, site dewatering the site should be as localized as possible. Widespread dewatering could result in subsidence of the area around the site due to increases in effective stress in the soil as the groundwater level is lowered. Nearby buildings, streets, and other improvements should be monitored for vertical movement and groundwater levels outside of the site should be monitored while dewatering is in progress. Should excessive settlement or groundwater drawdown be measured, a contingency plan, such as halting dewatering or groundwater recharge (if allowed) should be immediately implemented. If proposed, a recharge program should be included with the dewatering plan.

6.4.6 The dewatering system should be designed and implemented by an experienced dewatering contractor with local experience. Geocon should review the dewatering system proposed by the contractor prior to installation.

6.5 Excavation Bottom/Subgrade Stabilization Measures

6.5.1 Due to high in-situ moisture content and the presence of groundwater, significant instability should be expected in excavation bottoms/exposed subgrade areas. We expect that soil in the structure excavation bottoms will be nearly saturated even after dewatering. Stabilization measures will likely be necessary in order to provide access for construction equipment. The use of low contact-pressure tracked equipment should be considered to reduce disturbance and deterioration of the exposed subgrades. Final excavation should be made with excavators with smooth buckets.

6.5.2 Since we do not know the magnitude and extent of possible soft or unstable areas, our field representative should provide mitigation recommendations in the field at the time of construction. For planning purposes, we are providing two stabilization alternatives for consideration: (1) over-excavation and placement of a gravel mat (layer) over a durable geosynthetic fabric and (2) chemical treatment with high-calcium quicklime or Portland cement. The appropriateness and effectiveness of these alternatives will depend on the severity of the instability at the time of construction. We note that Option 1 may be preferred where mat foundations are used and Option 2 may be preferred where deep foundations are used.

6.5.3 **Option 1:** Stabilization may be accomplished by placing at least 18 inches of open-graded, angular ¾-inch gravel over a stabilization geotextile fabric (Mirafi 500X or equivalent). This procedure should be conducted in sections until the entire excavation bottom has been blanketed by fabric and gravel. Adjacent edges of fabric should be overlapped at least 2 feet or as recommended by the manufacturer. In order to reduce disturbance to the soft subgrade, we recommend using low-contact pressure, tracked equipment to perform the gravel spreading operations. Heavy equipment may operate on the completed gravel mat. The gravel should be compacted/consolidated to a dense state utilizing track equipment or a drum roller. If used as a drainage blanket, collector pipes and sumps placed in these materials for dewatering will need to be decommissioned once they are no longer needed.

6.5.4 **<u>Option 2:</u>** Based on the experience of the contractor and equipment being utilized, it may be possible to create a stable excavation bottom by blending high-calcium quicklime or Portland cement into the wet soils exposed in the excavation bottom. We anticipate that a minimum 18-inch-thick lift of chemical-treated soil will be required. We anticipate lime or cement content required for stabilization will be approximately 4% to 6% by dry weight. However, laboratory analyses should be performed to confirm which chemical reagent to use and that this percentage is effective.

6.5.5 Once the chemical-treated soil has been mixed, re-mixed (if using lime), compacted, and allowed to cure for a minimum of two days, we recommend placing at least 4 to 6 inches of crushed aggregate or AB over the treated soil to protect/enhance the durability of the section. The stabilized soil is essentially a "crust" that will bridge over the underlying soft wet soils and heavy construction equipment could damage the existing bridging capacity of the soil crust resulting in pumping, instability, and deterioration. The aggregate surfacing should enhance the durability of the section.

6.5.6 Chemical-treated soil has high alkalinity (pH typically greater than 12) and is severely detrimental to landscaping. If used, lime-treatment should be limited to areas below the proposed building and pavement areas. **The designers of subsurface improvements in lime-treated areas should be informed of the chemical characteristics of lime-treated soil.**

6.6 Grading and Earthwork

6.6.1 Excavated near-surface soils generated from cut operations at the site are suitable for use as engineered fill in structural areas provided they do not contain deleterious matter, organic material, or rocks larger than 6 inches in maximum dimension. Existing undocumented fill at the site likely contains some various debris (commonly brick, concrete rubble, trash, wood, etc.) that will require removal. Also, as previously discussed, excavated soils will have high in-situ moisture content and significant aerating/drying and/or chemical treatment will be required to achieve proper compaction.

6.6.2 Import fill material should be primarily granular with a "low" expansion potential (Expansion Index less than 50), a Plasticity Index less than 15, be free of organic material and construction debris, and not contain rock larger than 6 inches in greatest dimension.

6.6.3 Environmental characteristics and corrosion potential of import soil materials should also be considered. Proposed import materials should be sampled, tested, and approved by Geocon prior to its transportation to the site.

6.6.4 Earthwork operations should be observed and fills tested for recommended compaction and moisture content by a representative of Geocon.

6.6.5 References to relative compaction and optimum moisture content in this report are based on the latest American Society for Testing and Materials (ASTM) D1557 Test Procedure. Structural building pad areas should be considered as areas extending a minimum of 5 feet horizontally beyond the outside dimensions of buildings, including footings and overhangs carrying structural loads.

6.6.6 Prior to earthwork operations, a pre-construction conference with representatives of the client, grading contractor, and Geocon should be held at the site. Site preparation, soil handling and/or the grading plans should be discussed at the pre-construction conference.

6.6.7 Site preparation should begin with complete removal of existing pavements, concrete flatwork, underground utilities, and debris, as necessary. Any encountered deleterious debris such as wood, brick, trash, etc. should be excavated and removed from the site.

6.6.8 Once the excavation bottoms have been established, Geocon should observe the exposed conditions and coordinate with the grading and dewatering contractors to evaluate the appropriate bottom stabilization alternatives as discussed in Section 6.5.

6.6.9 Any areas to receive fill and/or pavements/flatwork should be scarified at least 12 inches, uniformly moisture-conditioned to near optimum moisture content and compacted to at least 90% relative compaction. Scarification and re-compaction operations should be performed in the presence of our representative in order to verify stability and/or identify loose/soft zones that may require further removals.

6.6.10 Engineered fill and excavation backfill should be compacted in horizontal lifts not exceeding 8 inches (loose thickness) and brought to final design elevations. Each lift should be moisture-conditioned to near optimum moisture content, and compacted to at least 90% relative compaction.

6.6.11 The top 6 inches of final vehicular pavement subgrade, whether completed at-grade, by excavation, or by filling, should be uniformly moisture-conditioned to near optimum moisture content and compacted to at least 95% relative compaction. Final pavement subgrade should be finished to a smooth, unyielding surface. We further recommend proof-rolling the subgrade with a loaded water truck (or similar equipment with high contact pressure) to verify the stability of the subgrade prior to placing AB.

6.6.12 Underground utility trenches should be backfilled with properly compacted material. Pipe bedding, shading, and trench backfill should conform to the requirements of the appropriate utility authority. Soil excavated from trenches should be adequate for use as general backfill above shading provided it does not contain deleterious matter, vegetation, or rock/cementations larger than 6 inches in maximum dimension. Trench backfill should be placed in loose lifts not exceeding 6 inches. Lifts should be compacted to a minimum of 90% relative compaction near optimum moisture content. Compaction should be performed by mechanical means only; jetting of trench backfill should not be allowed.

6.7 Mat Foundations (Visitor Center and Parking Structure)

6.7.1 Mat foundations consist of thick, relatively rigid reinforced concrete mats that distribute building loads across the entire footprint of the structure. Mat foundation systems allow the structures to settle with the ground and should have sufficient rigidity to allow the structure to move as a single unit, thus reducing the potential for damaging differential settlement. Mat foundations also provide increased resistance against hydrostatic uplift pressures.

6.7.2 The excavation for the below-grade structures will result in a decrease in overburden pressure under the proposed building footprints, on the order of 1,200 pounds per square foot (psf) to 1,500 psf. In consideration of this unloading, the allowable applied mat foundation loading (based on 1 inch of total settlement) is approximately 2,000 psf. If larger settlement is tolerable (say 2 inches), this allowable bearing value may be increased. Once design loads are established, we should review and perform additional settlement analyses to further evaluate the feasibility of using a mat foundation. If feasible, we will provide appropriate subgrade modulus values and other design parameters for mat foundation design.

6.8 Deep Foundations

The Annex Building may be supported on deep foundations bearing within the older alluvium at and below elevation -10 feet. A discussion of deep foundation types, project delivery methods, axial capacity estimates, and installation and load testing program recommendations are provided herein.

6.8.1 Deep Foundation Types and Project Delivery Method

6.8.1.1 There is a wide variety of deep foundation “pile” types available and each pile type behaves differently depending on installation and construction methods. Each pile type has specific advantages and disadvantages with respect to structural capacity, constructability, production rates, cost, and a host of other factors. The two major pile types include (1) manufactured “fixed-length” piles, such as pre-cast concrete, steel, or timber piles, and (2) drilled, cast-in-place piles. Each of these pile types includes “displacement” and “non-displacement” versions. Displacement piles move the soil laterally during installation (i.e. does not excavate or remove the soil) while non-displacement piles either cut through the soil (in the case of driven piles) or removes the soil (in the case of drilled piles). Displacement piles typically develop higher axial and lateral capacities due to the densification achieved as a result of soil displacement.

6.8.1.2 Due to the density variations within the older alluvium at the site, the use of fixed-length impact-driven piles can be problematic due to early refusal and/or deeper penetration; both of which may require post-installation modifications to the pile such as cutting or splicing, which can add significant cost. In addition, pile driving noise and vibrations may be undesirable for the project and adjacent improvements. Therefore, we do not recommend the use of fixed-length, impact driven piles for the project.

6.8.1.3 Due to the relatively high density and presence of cemented soil layers within the older alluvium, these materials are not “displaceable” and the use of drilled-displacement piles (such as drilled pipe piles and displacement auger cast piles) would require pre-drilling, which essentially negates any benefits derived from using displacement piles. Therefore, we do not recommend using drilled displacement piles.

6.8.1.4 Based on the subsurface conditions at the site and our experience on nearby, similar projects, we recommend using continuous flight auger (CFA) piles, also known as auger cast piles. CFA piles are installed using a plugged continuous flight auger that is advanced into ground. Once the desired depth is reached, the plug is removed and high-strength grout is pumped under pressure as the auger is withdrawn. As the auger is withdrawn, soil is removed from the hole on the auger flights and replaced with grout. After the auger is removed, the required steel reinforcement is then “wet-set” into the pile to complete the installation. This pile type produces approximately 100% to 120% of the theoretical hole volume of spoils.

6.8.1.5 From a geotechnical perspective, site soils are well-suited for CFA piles up to 24 inches in diameter; however, these piles will generate spoils that will require removal and offsite disposal. CFA piles are typically designed and installed by specialty geotechnical contractors because constructability, installation production, performance, and capacity will vary depending on the contractor’s equipment, experience, skill, materials, and installation procedures. **We strongly recommend performing a comprehensive pile installation and load testing program to evaluate constructability as well as capacity. Recommendations are provided in Section 6.8.4**.

6.8.1.6 The specialty foundation contractor should prepare a complete design-build submittal with design details, calculations, estimated capacities, installation procedures, proposed load testing procedures, acceptance criteria, and quality control procedures. Geocon should perform a geotechnical review of the design-build submittal.

6.8.2 Axial Capacity

6.8.2.1 Deep foundation systems will generate vertical load-carrying capacity from a combination of side friction in and end-bearing in the older alluvium. Table 6.8.2 presents estimated pile lengths and estimated axial capacities for 16-, 18-, and 24-inch CFA piles. We note that other pile diameters may also be used. **Pile lengths presented in Table 6.8.2.1 are <u>estimates only</u> based on empirical design procedures and local experience. These estimates are intended for planning purposes only as pile capacity can vary due to contractor equipment, installation procedures, and experience. Actual pile lengths should be determined by the design-build contractor and confirmed based on a pile load testing program.** Geocon should review, and if necessary, can assist the design-build contractor in developing a suitable testing program; preliminary recommendations are provided in Section 6.8.4.

TABLE 6.8.2.1
CFA PILES – ESTIMATED AXIAL CAPACITY

Pile Type	Approximate Pile Embedment into Older Alluvium (feet)[1]	Approximate Pile Length (feet)[2]	Estimated Allowable Axial Compression Capacity[2]
16-inch CFA Piles	15	30	100
	25	40	150
	35	50	225
18-inch CFA Piles	15	30	125
	25	40	175
	35	50	250
24-inch CFA Piles	15	30	175
	25	40	275
	35	50	375

<u>Notes:</u>
1. *Assumes Older Alluvium is approximately 15 feet below bottom of Pile Cap.*
2. *Pile Length below Pile Cap*
3. *Dead + Live Loading Conditions (minimum FS = 2.0)*

6.8.2.2 Allowable tension capacity is approximately 50% of the compression capacity. Tension piles should be properly reinforced to transfer uplift forces to the pile tip. We note that deeper embedment into the older alluvium may be required to achieve the allowable compression or tension capacities depending on the pile type and construction methods.

6.8.2.3 If pile spacing is at least 3 times the maximum dimension of the pile, a reduction in axial capacity for group effects is not considered necessary. Geocon should be contacted for review if piles are spaced closer than 3 times the maximum dimension of the pile.

6.8.3 Lateral Resistance

6.8.3.1 Lateral analysis of the deep foundations may be performed by the pile designer using the computer program LPILE or similar lateral analysis software. Table 6.8.3.1 summarizes our recommended soil parameters for use in LPILE.

TABLE 6.8.3.1
RECOMMENDED SOIL PARAMETERS FOR LPILE ANALYSIS

Soil Layer	Soil Model	Elevation (feet)		Effective Unit Weight (pcf)	Undrained Cohesion (psf)	Friction Angle (deg)
		Top	Bottom			
1	Cemented Silt (c-ϕ soil)	+5	-10	58	500	29
2	Sand (Reese)	-10	---	68	0	34

Notes: *Use LPILE default values for ε_{50} (clay) and subgrade modulus K (sand)*
pcf= pounds per cubic foot, psf = pounds per square foot, deg = degrees

6.8.3.2 Additional resistance to lateral loading may be provided by passive pressure acting on the sides of the piles caps and grade beams (if any). An allowable passive resistance of 190 pounds per cubic foot (pcf) equivalent fluid pressure may be used. Achieving full passive resistance requires movement. Assuming pile caps are about 3 feet thick, 100% passive pressure mobilization requires about 1 inch of lateral movement and 50% passive pressure mobilization requires ¼ inch of movement. Intermediate values may be approximated by linear interpretation. Frictional resistance along the bottom of the pile caps and grade beams should be neglected.

6.8.4 Installation and Load Test Program

6.8.4.1 We recommend performing a comprehensive pile installation and load testing program to evaluate constructability as well as capacity. The purposes of the test program will be to verify installation conditions, production rates, and axial capacity. A representative of Geocon should be present to observe test pile installation and load testing. The information obtained from the pile load testing should be used to evaluate the need to modify pile lengths to achieve design capacities, as well as develop installation criteria that can be used during construction of production piles.

6.8.4.2 At a minimum, we recommend installing at least two pre-production test piles of each diameter and type, equally spaced across the site. The test piles should be tested in compression and tension. The project structural engineer should evaluate the need for lateral load testing. The sacrificial test piles should be instrumented with strain gauges at various locations along the length of the pile and at the pile toe to provide the load distribution along the length of the pile to aid in evaluating pile capacity.

6.8.4.3 Static compression load tests should be performed in accordance with ASTM D1143 and static uplift (tension) tests should be performed in accordance with ASTM D3689. Test piles should loaded to at least 200% of the allowable design load for compression and 150% of the design uplift load for tension, assuming the design tension loads are due to wind or seismic loading per Section 1810.3.3.1.5 of the 2016 CBC. If the design tension loads are not due to wind or seismic, the load tests should be performed to 200% of the design tension load.

6.8.4.4 Prior to performing static load testing of the CFA piles, we recommend Thermal Integrity Profile (TIP) testing in accordance with ASTM D7949. The purpose of the TIP testing will be to verify the pile integrity, physical properties of the constructed pile (cross-sectional area, length, continuity and presence of cracks, cold-joints, necking or bulging) and to establish a correlation between the static load test results and TIP testing results. As a quality assurance measure during construction, TIP testing should be performed on 10% of the production piles. The frequency of TIP testing may be increased by the geotechnical engineer if installation difficulties are encountered. If the results of TIP testing indicate questionable pile properties, additional high-strain dynamic testing (ASTM D4945) may be required to verify production pile capacity.

6.8.4.5 At the completion of pile testing and prior to construction of production piles, the design-build foundation contractor should provide a written report of the load testing program prepared by a California registered design professional. This report should contain load test interpretations and evaluations in accordance with the applicable sections of the 2016 CBC. The contractor should also prepare a report after the completion of the production piles presenting information of the construction procedures, tip elevations, grout volumes/pressures, and a statement that the foundations can accept the design loading conditions.

6.9 Ancillary Structure Foundations

6.9.1 Foundations for lightly loaded ancillary structures not structurally connected to the structure, such as planter/landscape walls (up to 6 feet high), monuments, trash enclosures, or similar structures, may be supported on conventional foundations bearing on a minimum of 24 inches of newly placed engineered fill placed in accordance with the recommendations of this report.

6.9.2 Shallow foundations may be designed for an allowable bearing capacity of 1,500 pounds per square foot (psf), and should be a minimum of 12 inches wide and embedded at least 18 inches below the lowest adjacent grade. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

6.9.3 Allowable passive pressure used to resist lateral movement of footings may be assumed to be equal to a fluid weighing 300 pcf. The allowable coefficient of friction to resist sliding of footings is 0.30 for concrete against soil. Combined passive resistance and friction may be utilized for footing design provided that the frictional resistance is reduced by 50%.

6.9.4 Continuous footings should be reinforced with at least four No. 4 reinforcement bars, two each placed near the top and bottom of the footing to allow footings to span isolated soil irregularities. The reinforcement recommended above is for soil characteristics only and is not intended to replace reinforcement required for structural considerations. The project structural engineer should evaluate the need for additional reinforcement.

6.9.5 Light poles and similar structures may be supported on straight-shaft, CIDH concrete piers which may be designed using formulae from the 2016 CBC. An allowable lateral soil-bearing pressure (CBC parameters S_1 in equation 18-1 and S_3 in equations 18-2 and 18-3) of 150 psf per foot of depth may be used. If ½-inch deflection at the ground surface is acceptable, this value may be doubled.

6.9.6 The bottom of the pier excavation should be cleaned of loose cuttings prior to the placement of steel and concrete. Experience indicates that backspinning the auger does not remove loose material, and a flat cleanout plate is necessary. Concrete should be placed within the excavation as soon as possible after the auger/cleanout plate is withdrawn to reduce the potential for caving.

6.9.7 If seepage or groundwater is encountered, water should be pumped from the pier excavation prior to placement of concrete. Concrete should be placed by tremie methods from the bottom of the hole keeping the tremie pipe below the surface of the concrete at all times. Concrete should have a minimum 28-day design strength of 3,000 psi.

6.9.8 A Geocon representative should observe foundation excavations prior to placing reinforcing steel or concrete to observe that the exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.

6.10 Waterproofing / Concrete Moisture Protection Considerations

6.10.1 The seasonal high groundwater elevation at the site may approach +7 feet. Waterproofing of subterranean walls will be required. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method which would provide protection to subterranean walls, floor slabs and foundations. In addition, an experienced waterproofing inspector should be retained to check proper installation of the system during construction.

6.10.2 Migration of moisture through concrete slabs-on-grade or moisture otherwise released from slabs is not a geotechnical issue. However, for the convenience of the owner and design team, we are providing the following general suggestions for consideration by the owner, architect, structural engineer, and contractor. The suggested procedures may reduce the potential for moisture-related floor covering failures on concrete slabs-on-grade, but moisture problems may still occur even if the procedures are followed. If more detailed recommendations are desired, we recommend consulting a specialist in this field.

6.10.3 In areas where waterproofing materials are not present, a minimum 10-mil-thick vapor barrier meeting ASTM E1745-97 Class C requirements may be placed directly below the slab, without a sand cushion. To reduce the potential for punctures, a higher quality vapor barrier (15 mil, Class A or B) may be used. The vapor barrier, if used, should extend to the edges of the slab, and should be sealed at all seams and penetrations.

6.10.4 For interior slabs-on-grade located near street level (if any), at least 4 inches of ½ or ¾ inch crushed rock, with no more than 5 percent passing the No. 200 sieve, may be placed below the vapor barrier to serve as a capillary break.

6.10.5 The concrete water/cement ratio should be as low as possible. The water/cement ratio should not exceed 0.45 for concrete placed directly on the vapor barrier. Midrange plasticizers could be used to facilitate concrete placement and workability.

6.10.6 Proper finishing, curing, and moisture vapor emission testing should be performed in accordance with the latest guidelines provided by the ACI, PCA, and ASTM.

6.11 Permanent Retaining Walls

6.11.1 Design of permanent retaining walls and buried structures may be based on the lateral earth pressures (equivalent fluid pressure) summarized in Table 6.11.

TABLE 6.11.1
RECOMMENDED LATERAL EARTH PRESSURES

Condition[1]	Equivalent Fluid Density
Active (Above Groundwater)	45 pcf
Active (Below Groundwater)	85 pcf
At-Rest (Above Groundwater)	65 pcf
At-Rest (Below Groundwater)	95 pcf
Passive (Above Groundwater)	300 pcf
Passive (Below Groundwater)	190 pcf
Seismic Earth Pressure[2]	10 pcf

1. *Recommended design groundwater elevation +6.5 feet.*
2. *Applicable for walls that support more than 6 feet of backfill in accordance with Section 1803.5.12 of the 2016 CBC. Conventional triangular distribution. Should be combined with ACTIVE lateral earth pressure for seismic case analysis.*

6.11.2 The soil pressures above assume that the backfill material within an area bounded by the wall and a 1:1 plane extending upward from the base of the wall will be composed of the existing onsite soils.

6.11.3 Additional active pressure should be added for surcharge conditions due to vehicular traffic or adjacent structures.

6.11.4 In addition to the recommended earth pressure, the upper 10 feet of subterranean walls adjacent to streets should be designed to resist a uniform lateral pressure of 100 psf, acting as a result of an assumed 300 psf surcharge behind the walls due to normal street traffic. If the traffic is kept back at least 10 feet from the subterranean walls, the traffic surcharge may be neglected.

6.11.5 If not designed for hydrostatic conditions, retaining walls should be provided with a drainage system and waterproofed as required by the project architect. Positive drainage for retaining walls should consist of a vertical layer of permeable material positioned between the retaining wall and the soil backfill. The permeable material may be composed of a composite drainage geosynthetic or a natural permeable material such as crushed gravel at least 12 inches thick and capped with at least 12 inches of native soil. A geosynthetic filter fabric should be placed between the gravel and the soil backfill. Provisions for removal of collected water should be provided for either system by installing a perforated drainage pipe along the bottom of the permeable material which leads to suitable drainage facilities.

6.11.6 Moisture affecting below-grade walls is a common post-construction complaint. Poorly applied or omitted waterproofing can lead to efflorescence or standing water. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.

6.12 Temporary Shoring Design Recommendations

6.12.1 The construction of the subterranean structures will require excavations of about 15 to 20 feet below existing adjacent grades. In addition, the shoring design should also take into account any over-excavation required for bottom soil stabilization (gravel layer), pile cap excavations, and utility excavations. During mass excavation, shoring will be required to laterally restrain the sides of the excavation and limit the movement of adjacent improvements, such as public streets and sidewalks and adjacent buildings.

6.12.2 Typically, continuous excavation shoring consists of a system of soldier piles and wood or concrete lagging or interlocking steel sheet piles. If excavation depths exceed 15 feet, lateral bracing utilizing drilled tie-back anchors, raker braces, or other lateral restraint may be necessary. Internal braces may be required if there are obstructions precluding use of tiebacks or if extending tiebacks beyond property lines is not permitted.

6.12.3 For a soldier pile and lagging system, steel soldier piles would be placed in predrilled holes and backfilled with lean and/or full-strength concrete prior to site excavation. Wood or concrete lagging would be placed between the soldier piles as the excavation proceeds. Drilling of the holes for the soldier piles may require casing and/or the use of drilling mud to reduce the potential for caving in the sand and gravel layers. Because of the presence of adjacent structures that may be susceptible to vibration induced damage, we do not recommend the use of vibratory or impact hammers to install soldier piles. The shoring system and adjacent improvements should be monitored for movements throughout the excavation until the street-level slab is cast.

6.12.4 Lateral movements associated with the soldier pile and lagging system may adversely affect adjacent structures and improvements. The shoring designer should evaluate lateral movements and impacts to adjacent structures. If these movements are not tolerable, then a stiffer shoring system may be required, such as a soil-cement mix wall or concrete diaphragm walls. These systems are more rigid than a conventional soldier pile and lagging system and consequently, will deflect less.

6.12.5 As a starting point of preliminary design, the lateral earth pressures presented herein for permanent retaining structures may be used for temporary shoring design. Please note that these parameters may be conservative for temporary shoring design. We should be contacted once the details of the proposed shoring system are known so we can provide specific geotechnical recommendations.

6.13 Underpinning

6.13.1 Depending on the adjacency and elevation of the new Annex Building to the existing Capitol Building, underpinning of the Capitol Building may be required during demolition and construction of the Annex Building. Once the foundation details and elevations of the new building are known, we should be consulted to evaluate appropriate underpinning recommendations, if needed.

6.14 Elevator Pit Design

6.14.1 Below-grade elevator pit slabs and retaining walls should be designed by the project structural engineer. As a minimum, the pit slab-on-grade should be at least 5 inches thick and reinforced with No. 4 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Elevator pit walls may be designed using the lateral earth pressures provided in Section 6.11.

6.14.2 We recommend that the elevator pit walls and slab be waterproofed to prevent excessive moisture inside of the elevator pit. Waterproofing design and installation is not the responsibility of the geotechnical engineer.

6.14.3 If a plunger-type elevator piston is installed for this project, a deep drilled excavation will be required. It is important to verify that the drilled excavation is not situated immediately adjacent to an existing pile foundation, especially if the drilling is performed after the foundation is in place.

6.14.4 Casing will be required if caving is experienced in the drilled excavation, especially if the excavation is conducted below the groundwater level. The contractor should be prepared to use casing and should have it readily available at the commencement of drilling activities. The contractor should also be prepared to mitigate buoyant forces since the casing will be below the groundwater level. Continuous observation of the drilling and installation of the elevator piston by Geocon is highly recommended.

6.14.5 The annular space between the piston casing and drilled excavation wall should be filled with a minimum of 1½-sack slurry pumped from the bottom up. As an alternative, pea gravel may be utilized. The use of soil to backfill the annular space is not acceptable.

6.15 Concrete Sidewalks and Flatwork

6.15.1 Sidewalk, curb, and gutter within City right-of-way should be designed and constructed in accordance with the latest City of Sacramento standards and details as applicable. We note that the City of Sacramento requires at least 6 inches of compacted Class 2 AB below sidewalks.

6.15.2 Exterior concrete flatwork, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint.

6.15.3 Crack control joints should be spaced at intervals not greater than 8 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. The project structural engineer should design construction joints as necessary.

6.15.4 The recommendations of this report are intended to reduce the potential for cracking of slabs. However, even with the incorporation of these recommendations, concrete flatwork may exhibit some cracking due to soil movement and/or concrete shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

6.16 Rigid Concrete Pavement

6.16.1 Rigid concrete pavement may be used in vehicular traffic areas, such as loading and parking areas. Based on the soil conditions encountered at the site, concrete pavement should consist of at least 6 inches PCC overlying at least 12 inches of Class 2 AB meeting the requirements of Section 26 of Caltrans' *Standard Specifications*, unless specifically designed by the project structural engineer.

6.16.2 Subgrade soils should be prepared in accordance with the recommendations of the geotechnical report. Class 2 AB and subgrade should be compacted to at least 95% relative compaction near optimum moisture content. Subgrade should be proof-rolled with a loaded water truck to verify stability.

6.16.3 Concrete should have a minimum 28-day compressive strength of 3,500 psi. Adequate construction and crack control joints should be used to control cracking inherent in concrete construction. It would be advantageous to provide minimal reinforcement, such as No. 3 steel bars placed 18 inches on center in both horizontal directions to help control cracking. Consideration should be given to providing maximum control joint spacing of 12 feet in both directions for a 6-inch-thick slab. Adequate dowels should also be used at joints to facilitate load transfer and reduce vertical offset. In addition, the recommendations in Section 6.17.3 pertaining to deepened curbs, moisture cut-offs, and subsurface drainage applies to concrete pavements, sidewalks and flatwork, as well as asphalt pavements.

6.16.4 In general, we recommend that concrete pavements be designed, constructed and maintained in accordance with industry standards such as those provided by the American Concrete Pavement Association.

6.17 Site Drainage

6.17.1 Proper site drainage is critical to reduce the potential for differential soil movement, soil expansion, erosion and subsurface seepage. Under no circumstances should water be allowed to pond adjacent to building foundations. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with the 2016 CBC or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices.

6.17.2 Underground utilities should be leak free. Utility and irrigation lines should be checked periodically for leaks, and detected leaks should be repaired promptly. Detrimental soil movement could occur if water is allowed to infiltrate the soil for prolonged periods of time.

6.17.3 Landscaping planters adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. We recommend that area drains to collect excess irrigation water and transmit it to drainage structures or impervious above-grade planter boxes be used. In addition, where landscaping is planned adjacent to the pavement, we recommend construction of a cutoff wall (deepened concrete curb, plastic root barrier, or similar cutoff) along the edge of the pavement that extends at least 4 inches into the soil subgrade below the bottom of the base material.

6.17.4 We recommend that roof drains be connected to water-tight drainage piping connected to the storm drain system. However, we understand that Leadership in Engineering and Environmental Design (LEED) requests disconnecting the roof drains to help obtain certification. At a minimum, the water from the roof drains should be directed away from buildings. Consideration should be given to draining roofs to lined planter boxes or placing liners below the proposed landscape areas to prevent infiltration of the water. Geocon can be contacted for additional recommendations.

6.17.5 Experience has shown that even with these provisions, subsurface seepage may develop in areas where no such water conditions existed prior to site development. This is particularly true where a substantial increase in surface water infiltration has resulted from an increase in landscape irrigation.

7.0 FURTHER GEOTECHNICAL SERVICES

7.1 Plan and Specification Review

7.1.1 We should review the improvement plans and specifications prior to final design submittal to assess whether our recommendations have been properly implemented and evaluate if additional analysis and/or recommendations are required.

7.2 Testing and Observation Services

7.2.1 The recommendations provided in this report are based on the assumption that we will continue as Geotechnical Engineer of Record throughout the construction phase and provide geotechnical testing and observation services. It is important to maintain continuity of geotechnical interpretation and confirm that field conditions encountered are similar to those anticipated during design. If we are not retained for these services, we cannot assume any responsibility for other’s interpretation of our recommendations, and therefore the future performance of the project.

8.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, we should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous materials or environmental contamination was not part of our scope of services.

This report is issued with the understanding that it is the responsibility of the owner or their representative to ensure that the information and recommendations contained herein are brought to the attention of the design team for the project and incorporated into the plans and specifications, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

The recommendations contained in this report are preliminary until verified during construction by representatives of our firm. Changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. Additionally, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated partially or wholly by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices used in the site area at this time. No warranty is provided, express or implied.

9.0 REFERENCES

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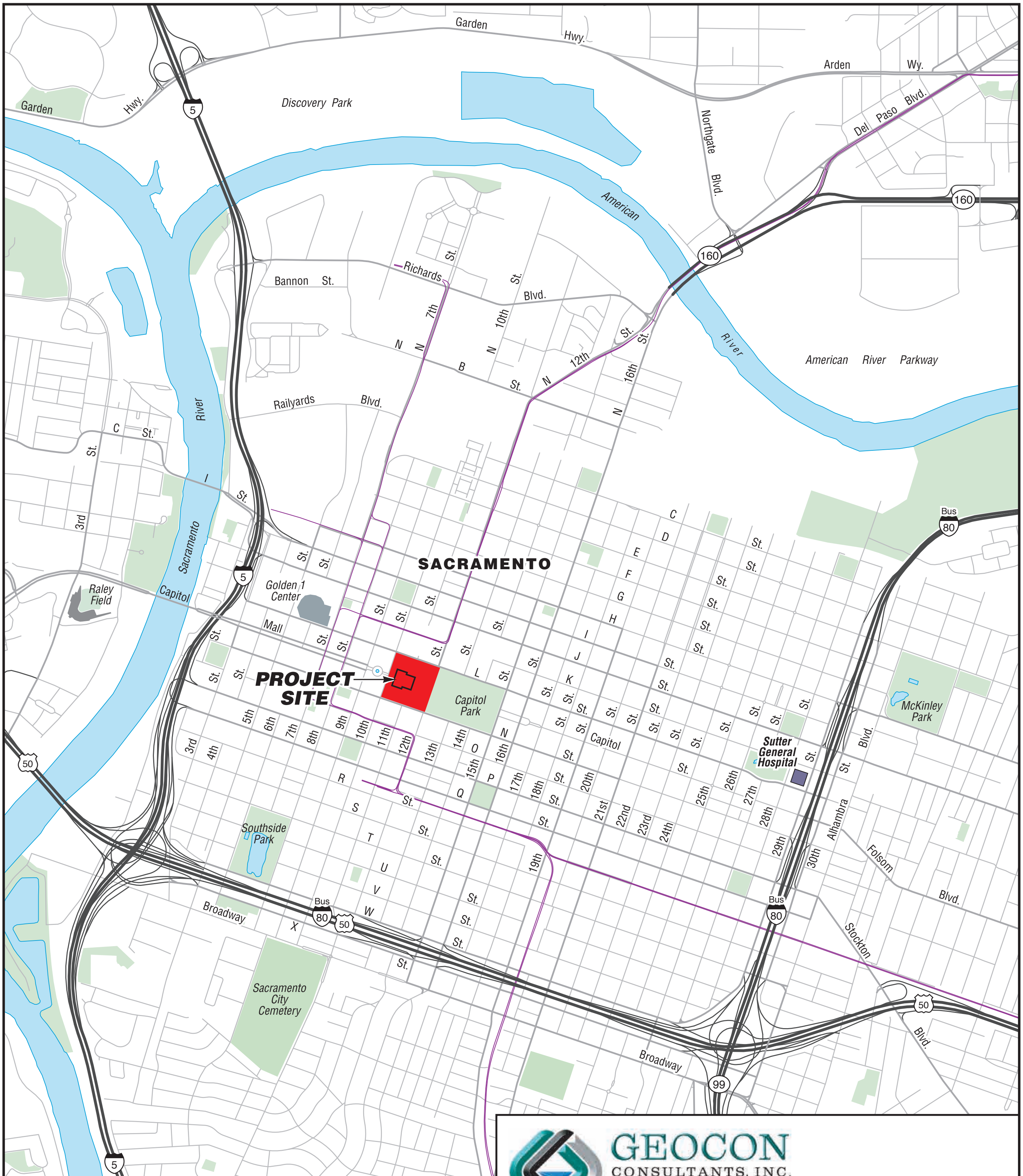

0 1/2
Scale in Miles

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3160 GOLD VALLEY DR – SUITE 800 – RANCHO CORDOVA, CA 95742
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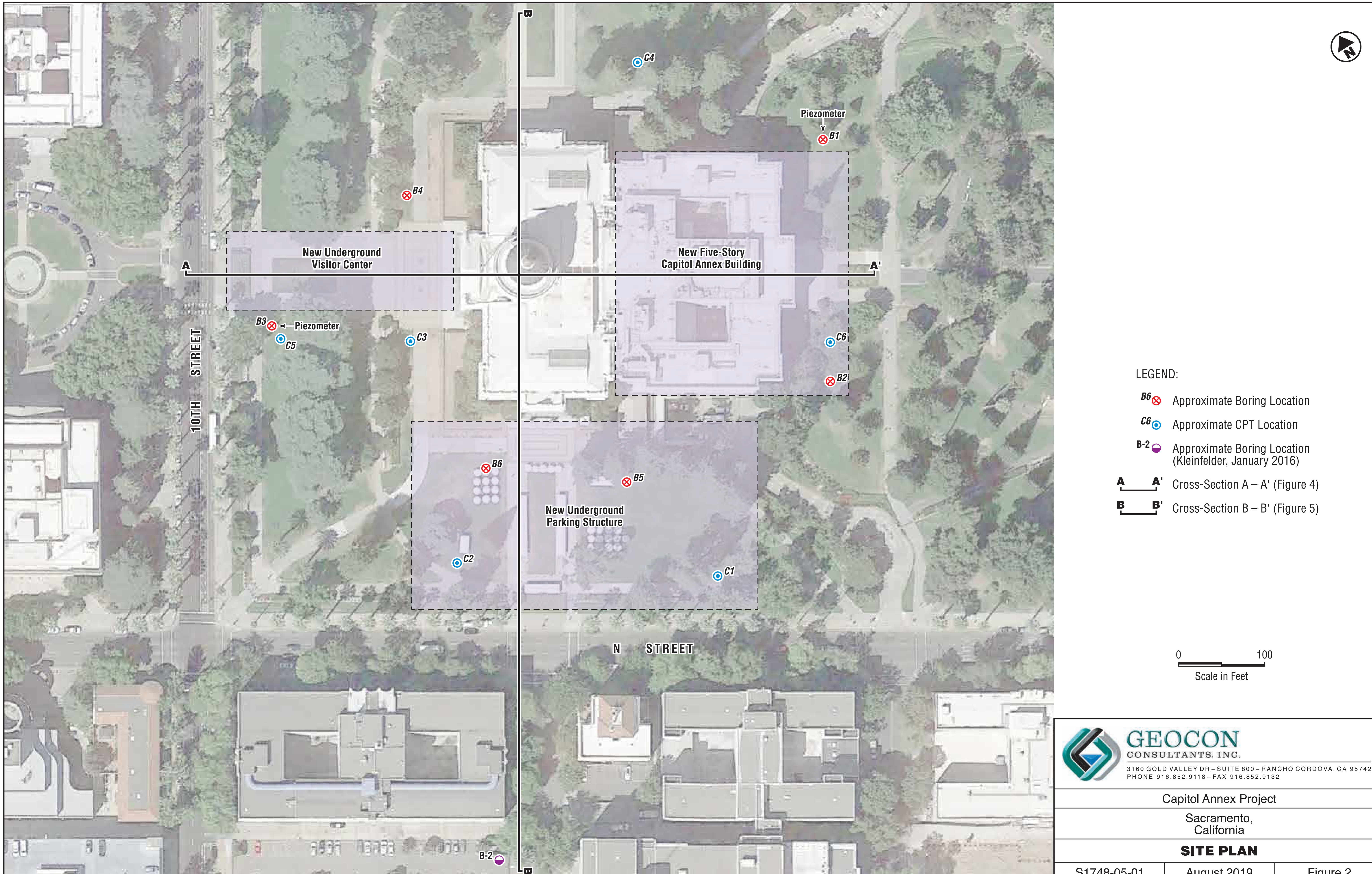
Capitol Annex Project

Sacramento,
California

VICINITY MAP

S1748-05-01	August 2019	Figure 1



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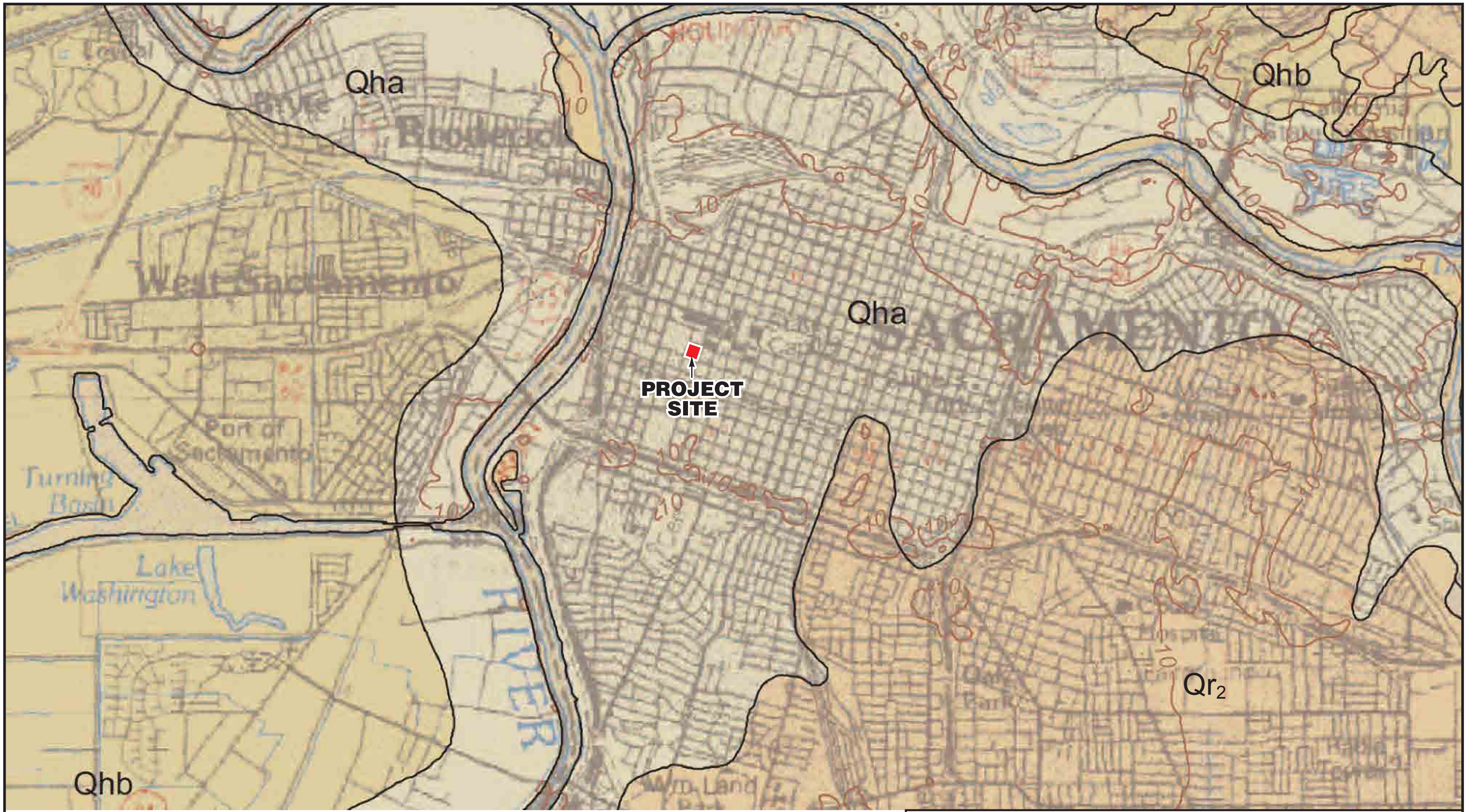
Sacramento, California		
SITE PLAN		
S1748-05-01	August 2019	Figure 2



Ref: Preliminary Geologic Map of the Sacramento 30' x 60' Quadrangle, California, 2011, CGS

Qha	Holocene Alluvium
Qhb	Holocene Basin Deposits
Qr_2	Riverbank Formation – Middle Unit

N

0 1
Scale in Miles

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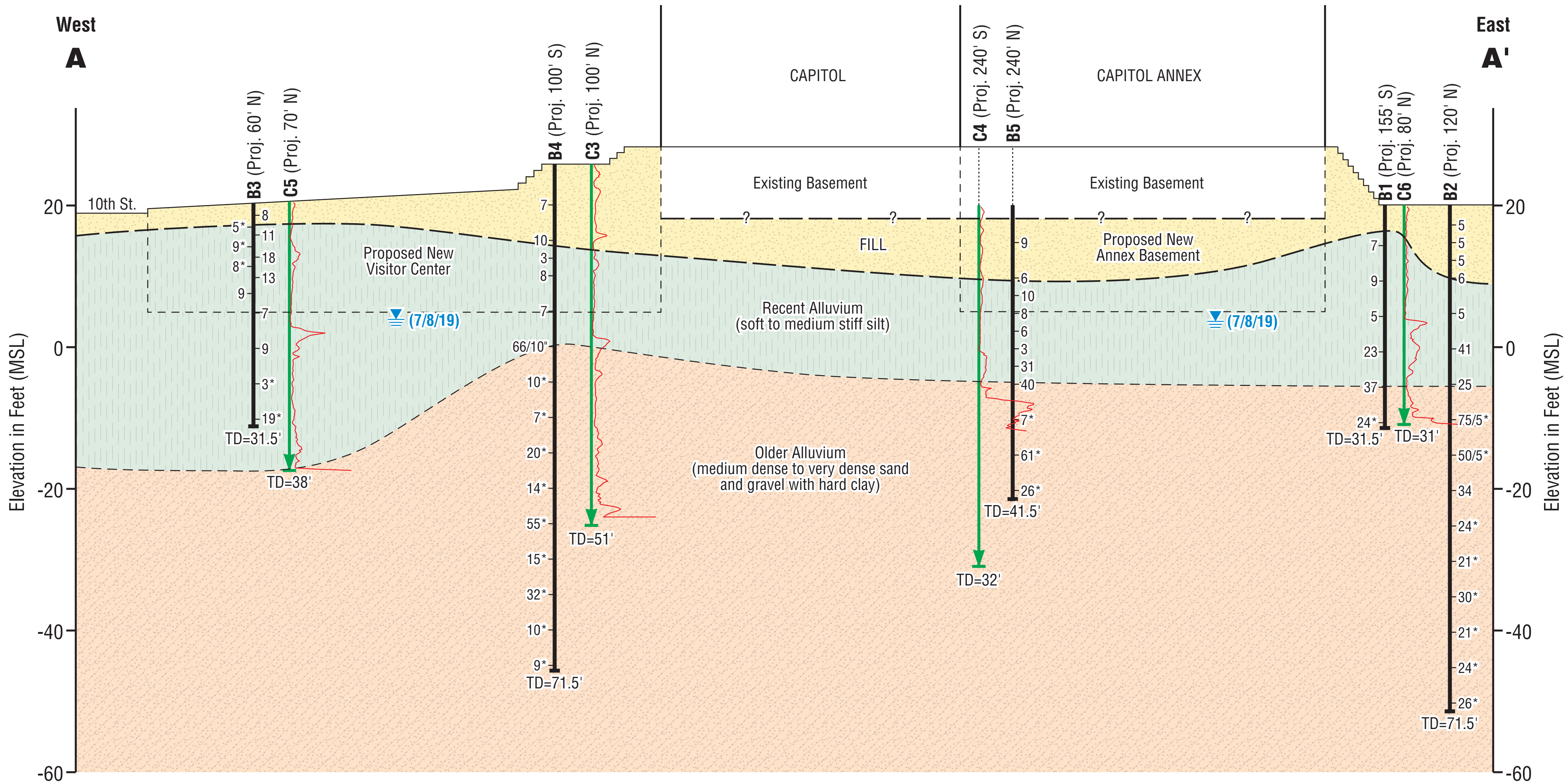
GEOLOGIC MAP

S1748-05-01	August 2019	Figure 3



B2 Boring Location

9 Blow Counts (California Modified Sampler/*SPT Sampler)

TD=Total Depth (Ft.)

Geologic Contact

Lithologic Contact

(7/8/19) Groundwater (date)

C1 CPT Location

Tip Resistance (tsf)

TD=Total Depth (Ft.)

This figure depicts generalized subsurface conditions inferred from our explorations and is intended for use as an aid for design. Actual subsurface conditions, including groundwater depths/elevations, may vary. Please refer to the Boring Logs (Appendix A) for detailed subsurface conditions encountered at each boring location.

Vertical Scale in Feet: 0, 20

Note: 4X Vertical Exaggeration

Horizontal Scale in Feet: 0, 80

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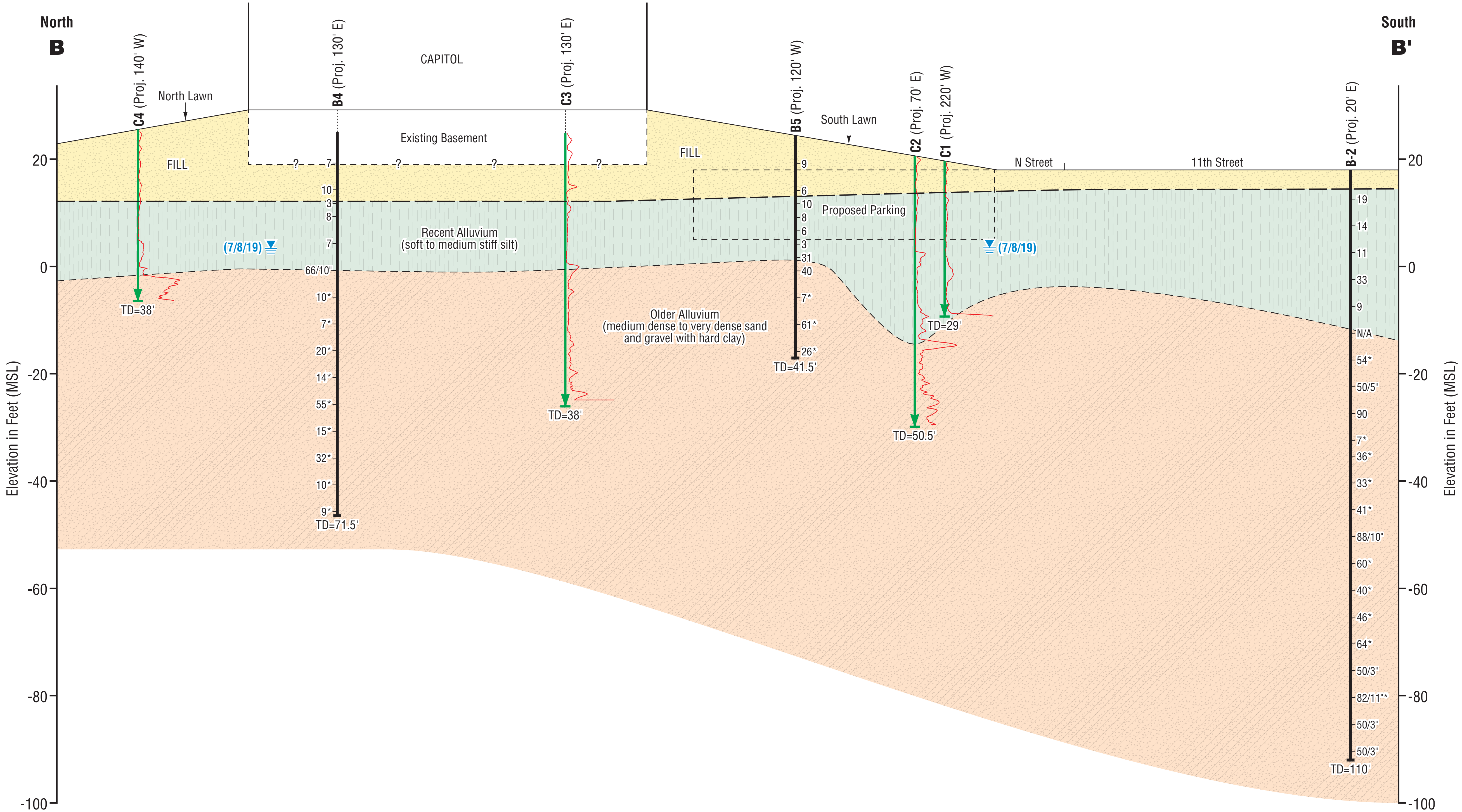
CROSS-SECTION A – A'

S1748-05-01	August 2019	Figure 4



B2 Boring Location

9 Blow Counts (California Modified Sampler/*SPT Sampler)

TD=Total Depth (Ft.)

Geologic Contact

Lithologic Contact

(7/8/19) Groundwater (date)

C1 CPT Location

Tip Resistance (tsf)

TD=Total Depth (Ft.)

This figure depicts generalized subsurface conditions inferred from our explorations and is intended for use as an aid for design. Actual subsurface conditions, including groundwater depths/elevations, may vary. Please refer to the Boring Logs (Appendix A) for detailed subsurface conditions encountered at each boring location.

Vertical Scale in Feet: 0, 20

Note: 4X Vertical Exaggeration

Horizontal Scale in Feet: 0, 80

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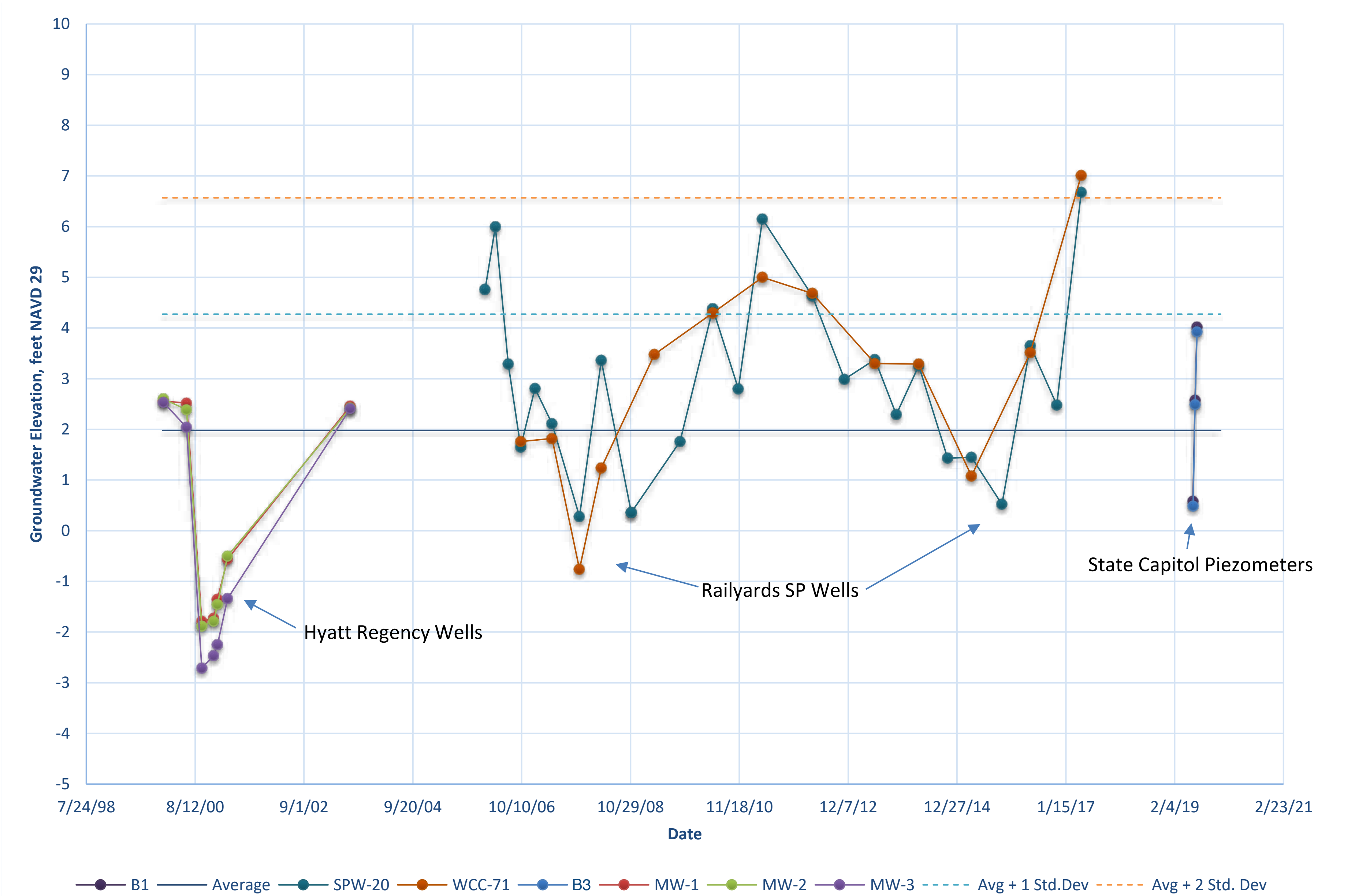
CROSS-SECTION B – B'

S1748-05-01	August 2019	Figure 5



GEOCON CONSULTANTS, INC. 3160 GOLD VALLEY DR – SUITE 800 – RANCHO CORDOVA, CA 95742 PHONE 916.852.9118 – FAX 916.852.9132	**Historic Groundwater Elevation**	Capitol Annex Project Sacramento, California	
		S1748-05-01	August 2019	Figure 6

APPENDIX A

APPENDIX A

FIELD EXPLORATION

Our geotechnical field exploration was performed on June 10 through June 12, 2019, and consisted of advancing six exploratory borings (B1 through B6) and six cone penetration test (CPT) soundings (CPT1 through CPT6) at the approximate locations shown on the Site Plan, Figure 2.

Exploratory boring was performed using a truck-mounted CME-55 drill rig equipped with 8-inch outside-diameter hollow-stem augers and mud-rotary drilling equipment. Sampling was accomplished using a 140-pound, automatic hammer with a 30-inch drop. Samples were obtained with a 3-inch OD, split-spoon (California Modified) sampler and a 2-inch OD Standard Penetration Test (SPT) sampler. The number of blows required to drive the samplers the last 12 inches (or portion thereof) of the 18-inch sampling interval were recorded on the boring logs.

The CPT soundings were performed using 20-ton truck-mounted CPT rig. CPT parameters, including tip resistance (q_c), sleeve friction (f_s) and dynamic pore pressure (U), were measured at approximate 2-inch intervals as the cone advanced. Soil behavior types were determined using correlations based on the comprehensive review by Lunne, Robertson and Powell (1997).

Subsurface conditions encountered in the exploratory borings were visually examined, classified and logged in general accordance with the American Society for Testing and Materials (ASTM) Practice for Description and Identification of Soils (Visual-Manual Procedure D2488-90). This system uses the Unified Soil Classification System (USCS) for soil designations. The logs depict soil and geologic conditions encountered and depths at which samples were obtained. The logs also include our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the field logs were revised based on subsequent laboratory testing. Logs of the explorations are presented herein.

UNIFIED SOIL CLASSIFICATION

MAJOR DIVISIONS				TYPICAL NAMES
COARSE-GRAINED SOILS MORE THAN HALF IS COARSER THAN NO. 200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO.4 SIEVE SIZE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL GRADED GRAVELS WITH OR WITHOUT SAND, LITTLE OR NO FINES
			GP	POORLY GRADED GRAVELS WITH OR WITHOUT SAND, LITTLE OR NO FINES
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, SILTY GRAVELS WITH SAND
			GC	CLAYEY GRAVELS, CLAYEY GRAVELS WITH SAND
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO.4 SIEVE SIZE	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL GRADED SANDS WITH OR WITHOUT GRAVEL, LITTLE OR NO FINES
			SP	POORLY GRADED SANDS WITH OR WITHOUT GRAVEL, LITTLE OR NO FINES
		SANDS WITH OVER 12% FINES	SM	SILTY SANDS WITH OR WITHOUT GRAVEL
			SC	CLAYEY SANDS WITH OR WITHOUT GRAVEL
FINE-GRAINED SOILS MORE THAN HALF IS FINER THAN NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50% OR LESS		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTS WITH SANDS AND GRAVELS
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, CLAYS WITH SANDS AND GRAVELS, LEAN CLAYS
			OL	ORGANIC SILTS OR CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50%		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OR CLAYS OF MEDIUM TO HIGH PLASTICITY
	HIGHLY ORGANIC SOILS		PT	PEAT AND OTHER HIGHLY ORGANIC SOILS

BORING/TRENCH LOG LEGEND

- No Recovery
- Shelby Tube Sample
- Bulk Sample
- SPT Sample
- Modified California Sample
- Groundwater Level (At Completion)
- Groundwater Level (Seepage)

PENETRATION RESISTANCE

SAND AND GRAVEL			SILT AND CLAY			
RELATIVE DENSITY	BLOWS PER FOOT (SPT)*	BLOWS PER FOOT (MOD-CAL)*	CONSISTENCY	BLOWS PER FOOT (SPT)*	BLOWS PER FOOT (MOD-CAL)*	COMPRESSIVE STRENGTH (tsf)
VERY LOOSE	0 - 4	0 - 6	VERY SOFT	0 - 2	0 - 3	0 - 0.25
LOOSE	5 - 10	7 - 16	SOFT	3 - 4	4 - 6	0.25 - 0.50
MEDIUM DENSE	11 - 30	17 - 48	MEDIUM STIFF	5 - 8	7 - 13	0.50 - 1.0
DENSE	31 - 50	49 - 79	STIFF	9 - 15	14 - 24	1.0 - 2.0
VERY DENSE	OVER 50	OVER 79	VERY STIFF	16 - 30	25 - 48	2.0 - 4.0
			HARD	OVER 30	OVER 48	OVER 4.0

*NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE LAST 12 INCHES OF AN 18-INCH DRIVE

MOISTURE DESCRIPTIONS

FIELD TEST	APPROX. DEGREE OF SATURATION, S (%)	DESCRIPTION
NO INDICATION OF MOISTURE; DRY TO THE TOUCH	$S<25$	DRY
SLIGHT INDICATION OF MOISTURE	$25\leq S<50$	DAMP
INDICATION OF MOISTURE; NO VISIBLE WATER	$50\leq S<75$	MOIST
MINOR VISIBLE FREE WATER	$75\leq S<100$	WET
VISIBLE FREE WATER	100	SATURATED

QUANTITY DESCRIPTIONS

APPROX. ESTIMATED PERCENT	DESCRIPTION
<5%	TRACE
5 - 10%	FEW
11 - 25%	LITTLE
26 - 50%	SOME
>50%	MOSTLY

GRAVEL/COBBLE/BOULDER DESCRIPTIONS

CRITERIA	DESCRIPTION
PASS THROUGH A 3-INCH SIEVE AND BE RETAINED ON A NO. 4 SIEVE (#4 TO 3")	GRAVEL
PASS A 12-INCH SQUARE OPENING AND BE RETAINED ON A 3-INCH SIEVE (3"-12")	COBBLE
WILL NOT PASS A 12-INCH SQUARE OPENING (>12")	BOULDER

LABORATORY TEST KEY

- CP – COMPACTION CURVE (ASTM D1557)
- CR – CORROSION ANALYSIS (CTM 422, 643, 417)
- DS – DIRECT SHEAR (ASTM D3080)
- EI – EXPANSION INDEX (ASTM D4829)
- GSA – GRAIN SIZE ANALYSIS (ASTM D422)
- MC – MOISTURE CONTENT (ASTM D2216)
- PI – PLASTICITY INDEX (ASTM D4318)
- R – R-VALUE (CTM 301)
- SE – SAND EQUIVALENT (CTM 217)
- TXCU – CONSOLIDATED UNDRAINED TRIAXIAL (ASTM D4767)
- TXUU – UNCONSOLIDATED UNDRAINED TRIAXIAL (ASTM D2850)
- UC – UNCONFINED COMPRESSIVE STRENGTH (ASTM D2166)

BEDDING SPACING DESCRIPTIONS

THICKNESS/SPACING	DESCRIPTOR
GREATER THAN 10 FEET	MASSIVE
3 TO 10 FEET	VERY THICKLY BEDDED
1 TO 3 FEET	THICKLY BEDDED
3 5/8-INCH TO 1 FOOT	MODERATELY BEDDED
1 1/4-INCH TO 3 5/8-INCH	THINLY BEDDED
3/8-INCH TO 1 1/4-INCH	VERY THINLY BEDDED
LESS THAN 3/8-INCH	LAMINATED

STRUCTURE DESCRIPTIONS

CRITERIA	DESCRIPTION
ALTERNATING LAYERS OF VARYING MATERIAL OR COLOR WITH LAYERS AT LEAST 1/4-INCH THICK	STRATIFIED
ALTERNATING LAYERS OF VARYING MATERIAL OR COLOR WITH LAYERS LESS THAN 1/4-INCH THICK	LAMINATED
BREAKS ALONG DEFINITE PLANES OF FRACTURE WITH LITTLE RESISTANCE TO FRACTURING	FISSURED
FRACTURE PLANES APPEAR POLISHED OR GLOSSY, SOMETIMES STRIATED	SLICKENSIDED
COHESIVE SOIL THAT CAN BE BROKEN DOWN INTO SMALLER ANGULAR LUMPS WHICH RESIST FURTHER BREAKDOWN	BLOCKY
INCLUSION OF SMALL POCKETS OF DIFFERENT SOIL, SUCH AS SMALL LENSES OF SAND SCATTERED THROUGH A MASS OF CLAY	LENSED
SAME COLOR AND MATERIAL THROUGHOUT	HOMOGENOUS

CEMENTATION/INDURATION DESCRIPTIONS

FIELD TEST	DESCRIPTION
CRUMBLES OR BREAKS WITH HANDLING OR LITTLE FINGER PRESSURE	WEAKLY CEMENTED/INDURATED
CRUMBLES OR BREAKS WITH CONSIDERABLE FINGER PRESSURE	MODERATELY CEMENTED/INDURATED
WILL NOT CRUMBLE OR BREAK WITH FINGER PRESSURE	STRONGLY CEMENTED/INDURATED

IGNEOUS/METAMORPHIC ROCK STRENGTH DESCRIPTIONS

FIELD TEST	DESCRIPTION
MATERIAL CRUMBLES WITH BARE HAND	WEAK
MATERIAL CRUMBLES UNDER BLOWS FROM GEOLOGY HAMMER	MODERATELY WEAK
1/8-INCH INDENTATIONS WITH SHARP END FROM GEOLOGY HAMMER	MODERATELY STRONG
HAND-HELD SPECIMEN CAN BE BROKEN WITH **ONE** BLOW FROM GEOLOGY HAMMER	STRONG
HAND-HELD SPECIMEN CAN BE BROKEN WITH **COUPLE** BLOWS FROM GEOLOGY HAMMER	VERY STRONG
HAND-HELD SPECIMEN CAN BE BROKEN WITH **MANY** BLOWS FROM GEOLOGY HAMMER	EXTREMELY STRONG

IGNEOUS/METAMORPHIC ROCK WEATHERING DESCRIPTIONS

DEGREE OF DECOMPOSITION	FIELD RECOGNITION	ENGINEERING PROPERTIES
SOIL	DISCOLORED, CHANGED TO SOIL, FABRIC DESTROYED	EASY TO DIG
COMPLETELY WEATHERED	DISCOLORED, CHANGED TO SOIL, FABRIC MAINLY PRESERVED	EXCAVATED BY HAND OR RIPPING (Saprolite)
HIGHLY WEATHERED	DISCOLORED, HIGHLY FRACTURED, FABRIC ALTERED AROUND FRACTURES	EXCAVATED BY HAND OR RIPPING, WITH SLIGHT DIFFICULTY
MODERATELY WEATHERED	DISCOLORED, FRACTURES, INTACT ROCK-NOTICEABLY WEAKER THAN FRESH ROCK	EXCAVATED WITH DIFFICULTY WITHOUT EXPLOSIVES
SLIGHTLY WEATHERED	MAY BE DISCOLORED, SOME FRACTURES, INTACT ROCK-NOT NOTICEABLY WEAKER THAN FRESH ROCK	REQUIRES EXPLOSIVES FOR EXCAVATION, WITH PERMEABLE JOINTS AND FRACTURES
FRESH	NO DISCOLORATION, OR LOSS OF STRENGTH	REQUIRES EXPLOSIVES

IGNEOUS/METAMORPHIC ROCK JOINT/FRACTURE DESCRIPTIONS

FIELD TEST	DESCRIPTION
NO OBSERVED FRACTURES	UNFRACTURED/UNJOINTED
MAJORITY OF JOINTS/FRACTURES SPACED AT 1 TO 3 FOOT INTERVALS	SLIGHTLY FRACTURED/JOINTED
MAJORITY OF JOINTS/FRACTURES SPACED AT 4-INCH TO 1 FOOT INTERVALS	MODERATELY FRACTURED/JOINTED
MAJORITY OF JOINTS/FRACTURES SPACED AT 1-INCH TO 4-INCH INTERVALS WITH SCATTERED FRAGMENTED INTERVALS	INTENSELY FRACTURED/JOINTED
MAJORITY OF JOINTS/FRACTURES SPACED AT LESS THAN 1-INCH INTERVALS; MOSTLY RECOVERED AS CHIPS AND FRAGMENTS	VERY INTENSELY FRACTURED/JOINTED

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3160 GOLD VALLEY DR – SUITE 800 – RANCHO CORDOVA, CA 95742
PHONE 916.852.9118 – FAX 916.852.9132

KEY TO LOGS

Figure A1

PROJECT NO. **S1748-05-01** PROJECT NAME **Capitol Annex**

BORING B1

ELEV. (MSL.) **22 Feet** DATE COMPLETED **6/10/2019**

ENG./GEO. **Sean Dixon** DRILLER **V&W Drilling**

EQUIPMENT **Truck-mounted CME55 w/ 8" HSA** HAMMER TYPE **Automatic | 140lb**

DEPTH IN FEET	SAMPLE INTERVAL & RECOVERY	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	ADDITIONAL TESTS
0		ML	**FILL** Soft, moist, dark brown, SILT				
1	B1-Bulk						CP, EI, CR
2							
3							
4		ML	**ALLUVIUM** Medium stiff, moist, brown, SILT with sand				
5							
6	B1-5.5 B1-6.0			7	96.7	26.0	PI
7							
8							
9							
10							
11	B1-10.5 B1-11.0			9	92.5	27.6	TXCU
12							
13							
14							
15							
16	B1-15.5 B1-16.0		- moist to wet	5	102.2	21.9	
17							
18							
19							
20							
21	B1-20.5 B1-21.0			23	113.3	20.3 19.4	GSA
22							
23							
24		SP	Medium dense, wet, brown and gray, Poorly graded SAND				

Figure A2, Log of Boring, page 1 of 2

IN PROGRESS S1748-05-01 CAPITOL ANNEX.GPJ 07/15/19

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SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

BORING B1

ELEV. (MSL.) **22 Feet** DATE COMPLETED **6/10/2019**

ENG./GEO. **Sean Dixon** DRILLER **V&W Drilling**

EQUIPMENT **Truck-mounted CME55 w/ 8" HSA** HAMMER TYPE **Automatic | 140lb**

DEPTH IN FEET	SAMPLE INTERVAL & RECOVERY	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	ADDITIONAL TESTS
25	NR				with gravel, medium to coarse sand, fine gravel	37			
26					- no recovery, 1 foot of heave				
27									
28									
29									
30	B1-30.0					24			
31									

BORING TERMINATED AT 31.5 FEET
GROUNDWATER ENCOUNTERED AT 19 FEET
SET PIEZOMETER TO 30 FEET: 10 FEET BLANK, 20 FEET SCREEN

Figure A3, Log of Boring, page 2 of 2

GEOCON

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

BORING B2

ELEV. (MSL.) **21 Feet** DATE COMPLETED **6/10/2019**
ENG./GEO. **Sean Dixon** DRILLER **V&W Drilling**
EQUIPMENT **Truck-mounted CME55 w/ Mud Rotary** HAMMER TYPE **Automatic | 140lb**

DEPTH IN FEET	SAMPLE INTERVAL & RECOVERY	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	ADDITIONAL TESTS
0		SM	**FILL** Loose, moist, light brown, Silty SAND				
3	B2-3.0 B2-3.5			5	93.4	23.3	
4.5		CL	Soft, moist, light brown with dark brown, Lean CLAY				
5.5	B2-5.5 B2-6.0			5	85.8	28.9	
8	B2-8.0 B2-8.5	ML	**ALLUVIUM** Soft, wet, brown, Sandy SILT, fine sand	5	93.5	30.1	
10.5	B2-10.5 B2-11.0			6			
15.5	B2-15.5 B2-16.0			5	101.2	24.7	
19			(groundwater)				
20.5	B2-20.5 B2-21.0		- very stiff, wet	41	107.6	21.1	

Figure A4, Log of Boring, page 1 of 3

IN PROGRESS S1748-05-01 CAPITOL ANNEX.GPJ 07/15/19

GEOCON

SAMPLE SYMBOLS
... SAMPLING UNSUCCESSFUL
... DISTURBED OR BAG SAMPLE
... STANDARD PENETRATION TEST
... CHUNK SAMPLE
... DRIVE SAMPLE (UNDISTURBED)
... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

BORING B2

ELEV. (MSL.) **21 Feet** DATE COMPLETED **6/10/2019**
ENG./GEO. **Sean Dixon** DRILLER **V&W Drilling**
EQUIPMENT **Truck-mounted CME55 w/ Mud Rotary** HAMMER TYPE **Automatic | 140lb**

DEPTH IN FEET	SAMPLE INTERVAL & RECOVERY	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	ADDITIONAL TESTS
25	B2-25.5								
26	B2-26					25			
27									
28									
29				GM	Very dense, wet, brown and gray, Silty GRAVEL with sand				
30	B2-30					75/5"		11.3	GSA
31									
32									
33									
34									
35	B2-35					50/5"			
36									
37				CL	Very stiff, wet, light brown, Lean CLAY				
38									
39									
40									
41	B2-41.0					34			
42									
43				SP	Medium dense, wet, light brown, Poorly graded SAND, medium sand				
44									
45	B2-45.0					24			
46									
47									
48									
49									

Figure A5, Log of Boring, page 2 of 3

GEOCON

SAMPLE SYMBOLS
... SAMPLING UNSUCCESSFUL
... DISTURBED OR BAG SAMPLE
... STANDARD PENETRATION TEST
... CHUNK SAMPLE
... DRIVE SAMPLE (UNDISTURBED)
... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. **S1748-05-01** PROJECT NAME **Capitol Annex**

BORING B2

ELEV. (MSL.) **21 Feet**
DATE COMPLETED **6/10/2019**
ENG./GEO. **Sean Dixon**
DRILLER **V&W Drilling**
EQUIPMENT **Truck-mounted CME55 w/ Mud Rotary**
HAMMER TYPE **Automatic | 140lb**

DEPTH IN FEET	SAMPLE INTERVAL & RECOVERY	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	ADDITIONAL TESTS
50	B2-50.0				- fine sand	21			
51									
52									
53									
54				SP-SC	Medium dense, wet, light brown, Poorly graded SAND with clay				
55	B2-55.0					30			
56									
57									
58									
59									
60	B2-60.0					21			
61									
62									
63									
64									
65	B2-65.0				- 1 inch thick clay lense	24			
66				CL	Hard, wet, light brown, Lean CLAY				
67									
68									
69									
70	B2-70.0					26			
71									

BORING TERMINATED AT 71.5 FEET
GROUNDWATER ENCOUNTERED AT 19 FEET
BACKFILLED WITH NEAT CEMENT GROUT

Figure A6, Log of Boring, page 3 of 3

IN PROGRESS S1748-05-01 CAPITOL ANNEX.GPJ 07/15/19

GEOCON

SAMPLE SYMBOLS
... SAMPLING UNSUCCESSFUL
... DISTURBED OR BAG SAMPLE
... STANDARD PENETRATION TEST
... CHUNK SAMPLE
... DRIVE SAMPLE (UNDISTURBED)
... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. **S1748-05-01** PROJECT NAME **Capitol Annex**

BORING B3

ELEV. (MSL.) **21 Feet** DATE COMPLETED **6/11/2019**

ENG./GEO. **Sean Dixon** DRILLER **V&W Drilling**

EQUIPMENT **Truck-mounted CME55 w/ 8" HSA** HAMMER TYPE **Automatic | 140lb**

DEPTH IN FEET	SAMPLE INTERVAL & RECOVERY	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	ADDITIONAL TESTS
0		ML	**FILL**				
1	B3-0.5, B3-Bulk, B3-1.0, B3-1.5		Medium stiff, moist, brown, Sandy SILT, fine sand	8 5			CP, EI, CR
3		ML	**ALLUVIUM**				
4	B3-3.5, B3-4.0, B3-4.5		Medium stiff, moist, brown, Sandy SILT, fine sand - stiff	11 9			
7	B3-7.0, B3-7.5		- medium stiff	18 8			
11	B3-10.5, B3-11.0			13			
13	B3-13.0, B3-13.5			9	90.2	31.1	
15		CL-ML	Medium stiff, wet, brown, SILTY CLAY (▼ water table or seepage)				
16	B3-15.5, B3-16.0			7	102.8	23.1	PI
20	NR		- no recovery	9			
24		ML	Soft, wet, brown, SILT				

Figure A7, Log of Boring, page 1 of 2

IN PROGRESS S1748-05-01 CAPITOL ANNEX.GPJ 07/15/19

GEOCON

SAMPLE SYMBOLS: ... SAMPLING UNSUCCESSFUL; ... DISTURBED OR BAG SAMPLE; ... STANDARD PENETRATION TEST; ... CHUNK SAMPLE; ... DRIVE SAMPLE (UNDISTURBED); ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

BORING B3

ELEV. (MSL.) **21 Feet** DATE COMPLETED **6/11/2019**

ENG./GEO. **Sean Dixon** DRILLER **V&W Drilling**

EQUIPMENT **Truck-mounted CME55 w/ 8" HSA** HAMMER TYPE **Automatic | 140lb**

DEPTH IN FEET	SAMPLE INTERVAL & RECOVERY	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	ADDITIONAL TESTS
25	B3-25.0					3		35.3	
26									
27									
28									
29									
30	B3-30.0				- very stiff	19		39.7	
31									

BORING TERMINATED AT 31.5 FEET
GROUNDWATER ENCOUNTERED AT 15 FEET
SET PIEZOMETER TO 30 FEET: 10 FEET BLANK, 20 FEET SCREEN

Figure A8, Log of Boring, page 2 of 2

GEOCON

SAMPLE SYMBOLS
... SAMPLING UNSUCCESSFUL
... DISTURBED OR BAG SAMPLE
... STANDARD PENETRATION TEST
... CHUNK SAMPLE
... DRIVE SAMPLE (UNDISTURBED)
... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

BORING B4

ELEV. (MSL.) **28 Feet** DATE COMPLETED **6/11/2019**

ENG./GEO. **Sean Dixon** DRILLER **V&W Drilling**

EQUIPMENT **Truck-mounted CME55 w/ Mud Rotary** HAMMER TYPE **Automatic | 140lb**

DEPTH IN FEET	SAMPLE INTERVAL & RECOVERY	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	ADDITIONAL TESTS
0	B4-Bulk			CL	**FILL** Medium stiff, moist, dark brown, Lean CLAY with gravel and brick				
5				SM	Loose, moist, dark brown, Silty SAND				
6	B4-6.0					7	101.6	6.9	
10							99.1	15.9	
11	B4-10.5 B4-11.0					10			
12			▼	ML	**ALLUVIUM** Soft, wet, brown, Clayey SILT				
14	B4-13.5					3			
15					- medium stiff				
16	B4-15.5 B4-16.0					8	90.9	31.1	
21	B4-20.5 B4-21.0					7	98.1	27.2	

Figure A9, Log of Boring, page 1 of 3

GEOCON

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

BORING B4

ELEV. (MSL.) **28 Feet** DATE COMPLETED **6/11/2019**
ENG./GEO. **Sean Dixon** DRILLER **V&W Drilling**
EQUIPMENT **Truck-mounted CME55 w/ Mud Rotary** HAMMER TYPE **Automatic | 140lb**

DEPTH IN FEET	SAMPLE INTERVAL & RECOVERY	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	ADDITIONAL TESTS
25	B4-25.5				- hard, cemented				
26	B4-26.0					66/10"	102.5	21.8	
27									
28									
29									
30	B4-30.0				- stiff, less cemented	10			
31									
32									
33									
34									
35	B4-35.0				- medium stiff	7			
36									
37									
38									
39									
40	B4-40.0				- very stiff	20			
41									
42									
43									
44				ML	Medium dense, wet, brown, Sandy SILT				
45	B4-45.0					14		32.4	GSA
46									
47									
48									
49				GP	Very dense, wet, brown and gray, Poorly graded GRAVEL with sand				

Figure A10, Log of Boring, page 2 of 3

GEOCON

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

BORING B4

ELEV. (MSL.) **28 Feet** DATE COMPLETED **6/11/2019**

ENG./GEO. **Sean Dixon** DRILLER **V&W Drilling**

EQUIPMENT **Truck-mounted CME55 w/ Mud Rotary** HAMMER TYPE **Automatic | 140lb**

DEPTH IN FEET	SAMPLE INTERVAL & RECOVERY	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	ADDITIONAL TESTS
50	B4-50.0					55			
51									
52									
53									
54				CL	Stiff, wet, light brown, Sandy lean CLAY				
55	B4-55.0					15			
56									
57									
58									
59									
60	B4-6.0				- hard	32		22.4	
61									
62									
63									
64									
65	B4-65.0				- soft	10			
66									
67									
68									
69									
70	B4-70.0					9			
71									

BORING TERMINATED AT 71.5 FEET
GROUNDWATER ENCOUNTERED AT 12 FEET
BACKFILLED WITH NEAT CEMENT GROUT

Figure A11, Log of Boring, page 3 of 3

GEOCON

SAMPLE SYMBOLS			
... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)	
... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE	

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. **S1748-05-01** PROJECT NAME **Capitol Annex**

BORING B5

ELEV. (MSL.) **25 Feet** DATE COMPLETED **6/12/2019**

ENG./GEO. **Sean Dixon** DRILLER **V&W Drilling**

EQUIPMENT **Truck-mounted CME55 w/ Mud Rotary** HAMMER TYPE **Automatic | 140lb**

DEPTH IN FEET	SAMPLE INTERVAL & RECOVERY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	ADDITIONAL TESTS
0			SM	**FILL** Loose, moist, brown, Silty SAND				
1								
2								
3								
4								
5	B5-5.5							
6	B5-6.0				9			DS
7								
8								
9								
10	B5-10.5		CL	**ALLUVIUM** Soft, wet, dark brown, Lean CLAY				
11	B5-11.0				6	85.9	35.5	
12								
13	B5-13.0 B5-13.5		ML	Medium stiff, moist, brown, SILT				
14					10			
15	B5-15.5			- some fine sand				
16	B5-16.0				8	96.6	27.7	
17								
18	B5-18.0 B5-18.5			- soft, wet				
19		▼			6			
20	B5-20.5			- very soft				
21	B5-21.0				3	100.8	28.6	
22								
23	B5-23.0 B5-23.5			- very stiff, cemented		96.7	28.0	
24					31			

Figure A12, Log of Boring, page 1 of 2

IN PROGRESS S1748-05-01 CAPITOL ANNEX.GPJ 07/15/19

GEOCON

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. **S1748-05-01** PROJECT NAME **Capitol Annex**

DEPTH IN FEET	SAMPLE INTERVAL & RECOVERY	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B5 ELEV. (MSL.) 25 Feet; DATE COMPLETED 6/12/2019; ENG./GEO. Sean Dixon; DRILLER V&W Drilling; EQUIPMENT Truck-mounted CME55 w/ Mud Rotary; HAMMER TYPE Automatic | 140lb — MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	ADDITIONAL TESTS
25	B5-25.5				- less cemented				
26	B5-26.0					40			
27									
28									
29									
30	B5-30.0				- medium stiff	7			
31									
32									
33									
34				GP	Very dense, wet, gray, Poorly graded GRAVEL with sand				
35	B5-35.0					61			
36									
37									
38									
39									
40	B5-40.0				- medium dense	26			
41									

BORING TERMINATED AT 41.5 FEET
GROUNDWATER ENCOUNTERED AT 19 FEET
BACKFILLED WITH NEAT CEMENT GROUT

Figure A13, Log of Boring, page 2 of 2

IN PROGRESS S1748-05-01 CAPITOL ANNEX.GPJ 07/15/19

GEOCON

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. **S1748-05-01** PROJECT NAME **Capitol Annex**

BORING B6

ELEV. (MSL.) **27 Feet** DATE COMPLETED **6/12/2019**

ENG./GEO. **Sean Dixon** DRILLER **V&W Drilling**

EQUIPMENT **Truck-mounted CME55 w/ Mud Rotary** HAMMER TYPE **Automatic | 140lb**

DEPTH IN FEET	SAMPLE INTERVAL & RECOVERY	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	ADDITIONAL TESTS
0					**TREE STUMP**				
3				ML	**FILL** Medium stiff, moist, brown, Sandy SILT, fine sand				
6	B6-5.5 B6-6.0					8	94.8	21.1	PI
10					- soft				
11	B6-11.0					5	71.5	26.6	
12				ML	**ALLUVIUM** Very soft, wet, brown, Sandy SILT, fine sand				
14	B6-13.0 B6-13.5					2			TXUU
15					- soft				
16	B6-15.5 B6-16.0					6	93.5	30.3	
17			▼						
19	B6-18.0 B6-18.5					6			
21	B6-20.5 B6-21.0					5	93.7	29.6	
24	B6-23.0 B6-23.5					5			

Figure A14, Log of Boring, page 1 of 3

IN PROGRESS S1748-05-01 CAPITOL ANNEX.GPJ 07/15/19

GEOCON

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. **S1748-05-01** PROJECT NAME **Capitol Annex**

BORING B6

ELEV. (MSL.) **27 Feet** DATE COMPLETED **6/12/2019**
ENG./GEO. **Sean Dixon** DRILLER **V&W Drilling**
EQUIPMENT **Truck-mounted CME55 w/ Mud Rotary** HAMMER TYPE **Automatic | 140lb**

DEPTH IN FEET	SAMPLE INTERVAL & RECOVERY	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	ADDITIONAL TESTS
25	B6-25.0				- medium stiff	7			
26				ML	Stiff, wet, brown, SILT				
30	B6-30.0					13			
35	B6-35.0					9			
40	B6-40.0				- very stiff, some fine sand	25			
43.5				ML	Medium dense, wet, brown, Sandy SILT, fine sand, trace gravel				
45	B6-45.0					20		34.3	GSA
48				SP	Medium dense, wet, reddish brown, Poorly graded SAND				

Figure A15, Log of Boring, page 2 of 3

GEOCON

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

BORING B6

ELEV. (MSL.)	27 Feet		DATE COMPLETED	6/12/2019
ENG./GEO.	Sean Dixon		DRILLER	V&W Drilling
EQUIPMENT	Truck-mounted CME55 w/ Mud Rotary		HAMMER TYPE	Automatic | 140lb

DEPTH IN FEET	SAMPLE INTERVAL & RECOVERY	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	ADDITIONAL TESTS
50	B6-50.0					18			
51									
52									
53									
54									
55	B6-55.0				- dense to very dense, some gravel	52			
56									

BORING TERMINATED AT 56.5 FEET
GROUNDWATER ENCOUNTERED AT 17 FEET
BACKFILLED WITH NEAT CEMENT GROUT

Figure A16, Log of Boring, page 3 of 3

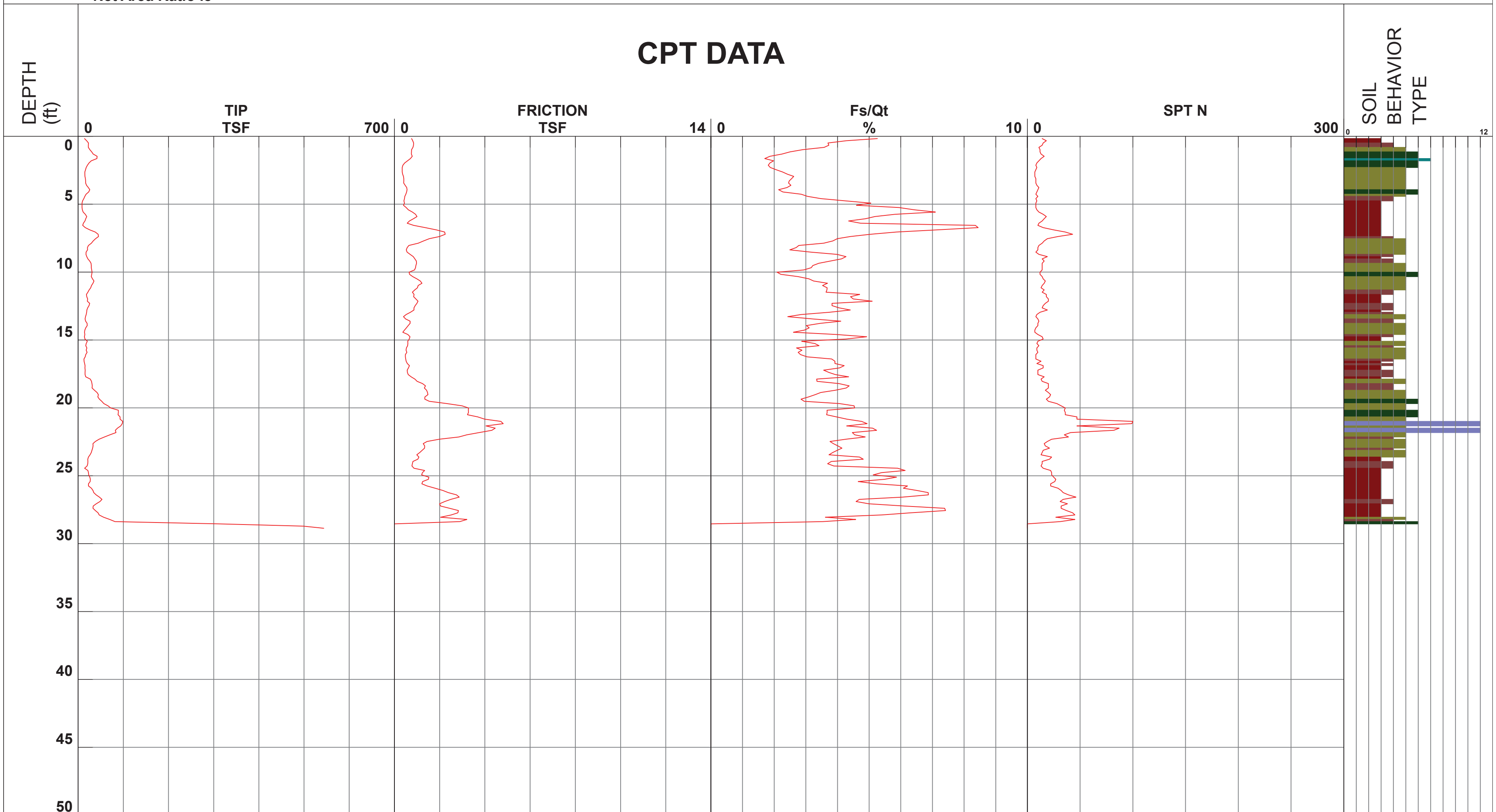
IN PROGRESS S1748-05-01 CAPITOL ANNEX.GPJ 07/15/19

GEOCON

SAMPLE SYMBOLS
... SAMPLING UNSUCCESSFUL
... DISTURBED OR BAG SAMPLE
... STANDARD PENETRATION TEST
... CHUNK SAMPLE
... DRIVE SAMPLE (UNDISTURBED)
... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

Net Area Ratio .8



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 15cm squared

S*Soil behavior type and SPT based on data from UBC-1983

GEOCON CONSULTANTS, INC.
3160 GOLD VALLEY DR – SUITE 800 – RANCHO CORDOVA, CA 95742
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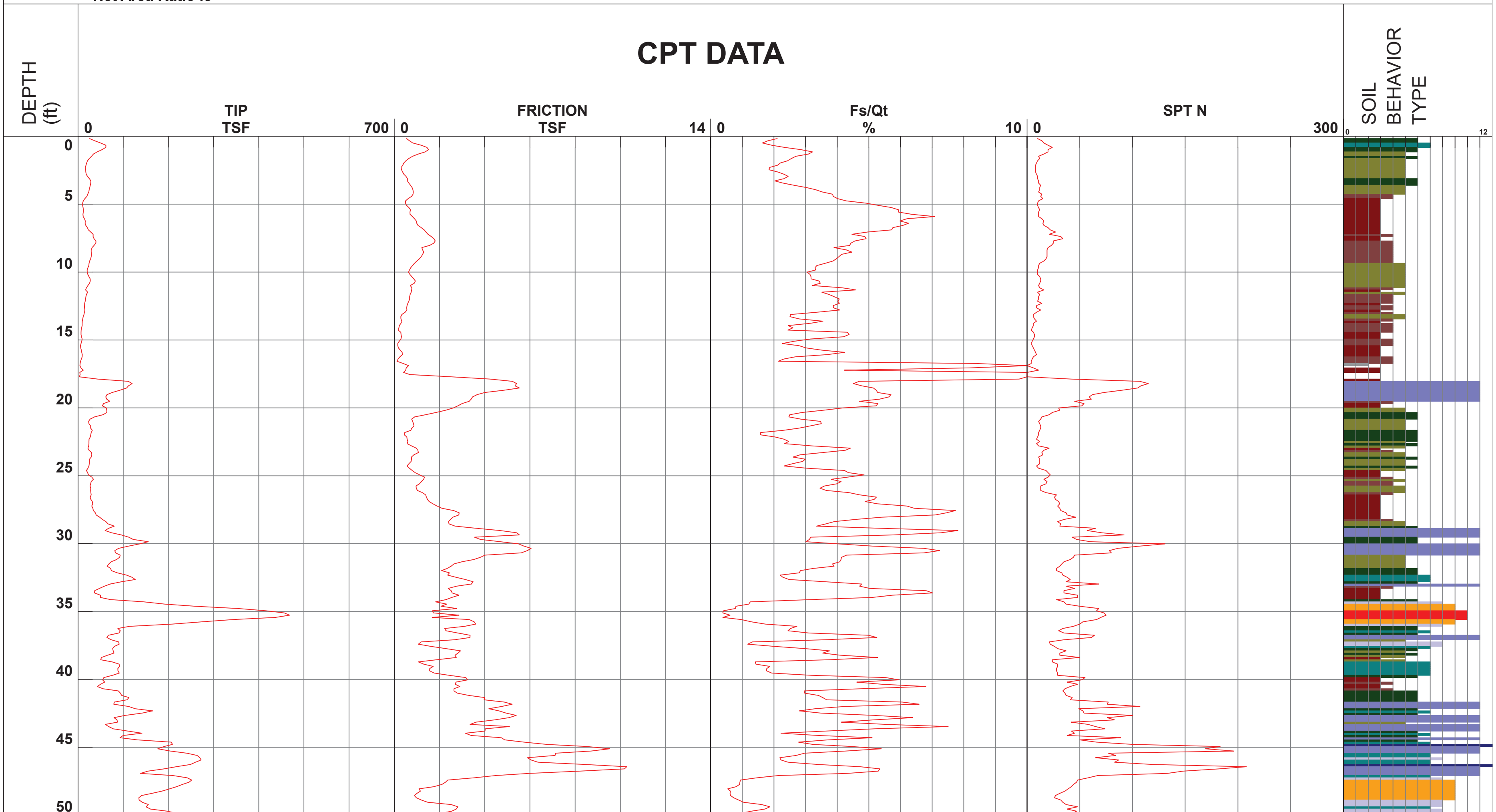
CPT1

Capitol Annex Project		
Sacramento, California		
S1748-05-01	August 2019	Figure A17

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 15cm squared

S*Soil behavior type and SPT based on data from UBC-1983

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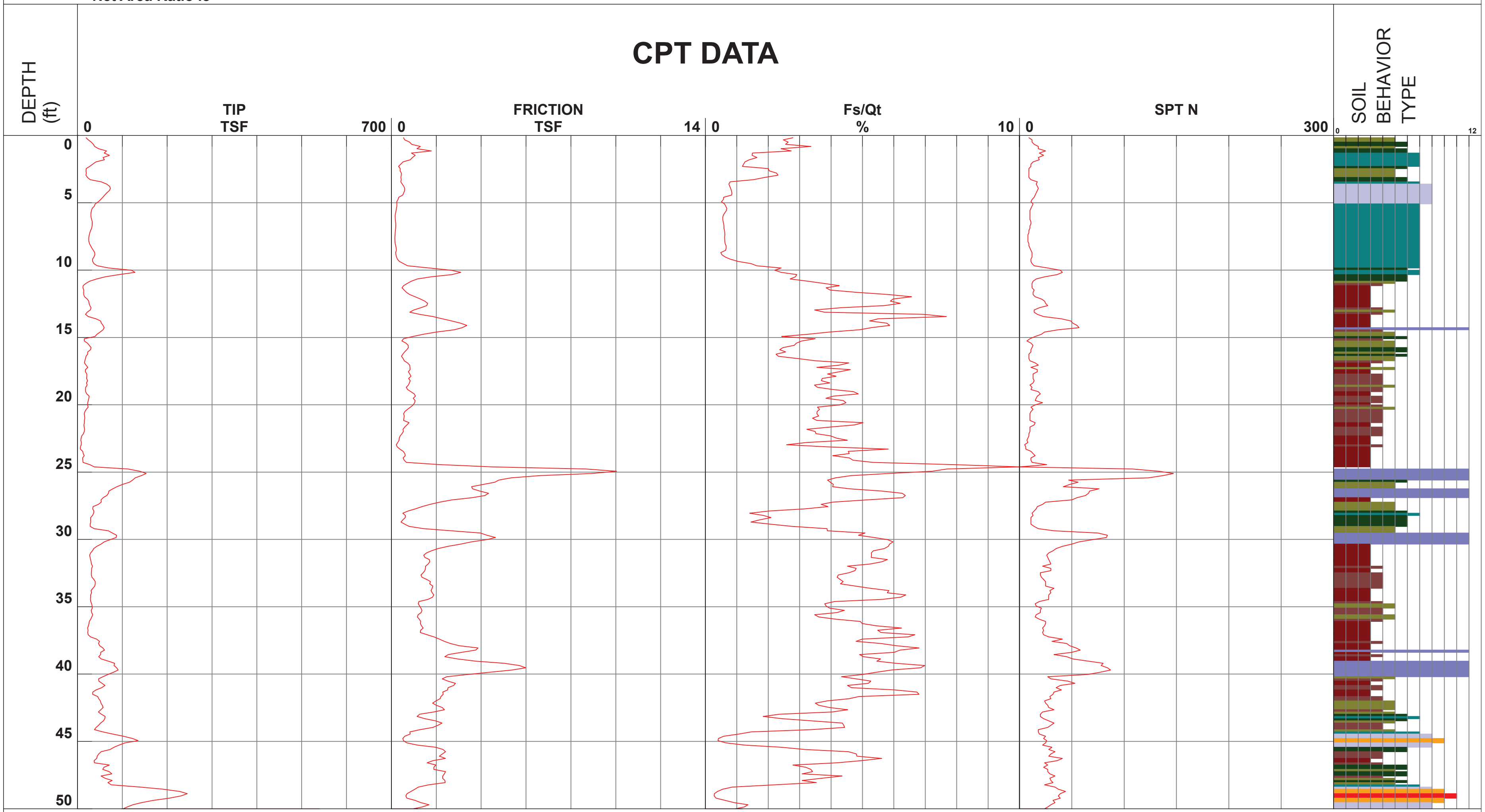
CPT2

Capitol Annex Project		
Sacramento, California		
S1748-05-01	August 2019	Figure A18

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 15cm squared

S*Soil behavior type and SPT based on data from UBC-1983

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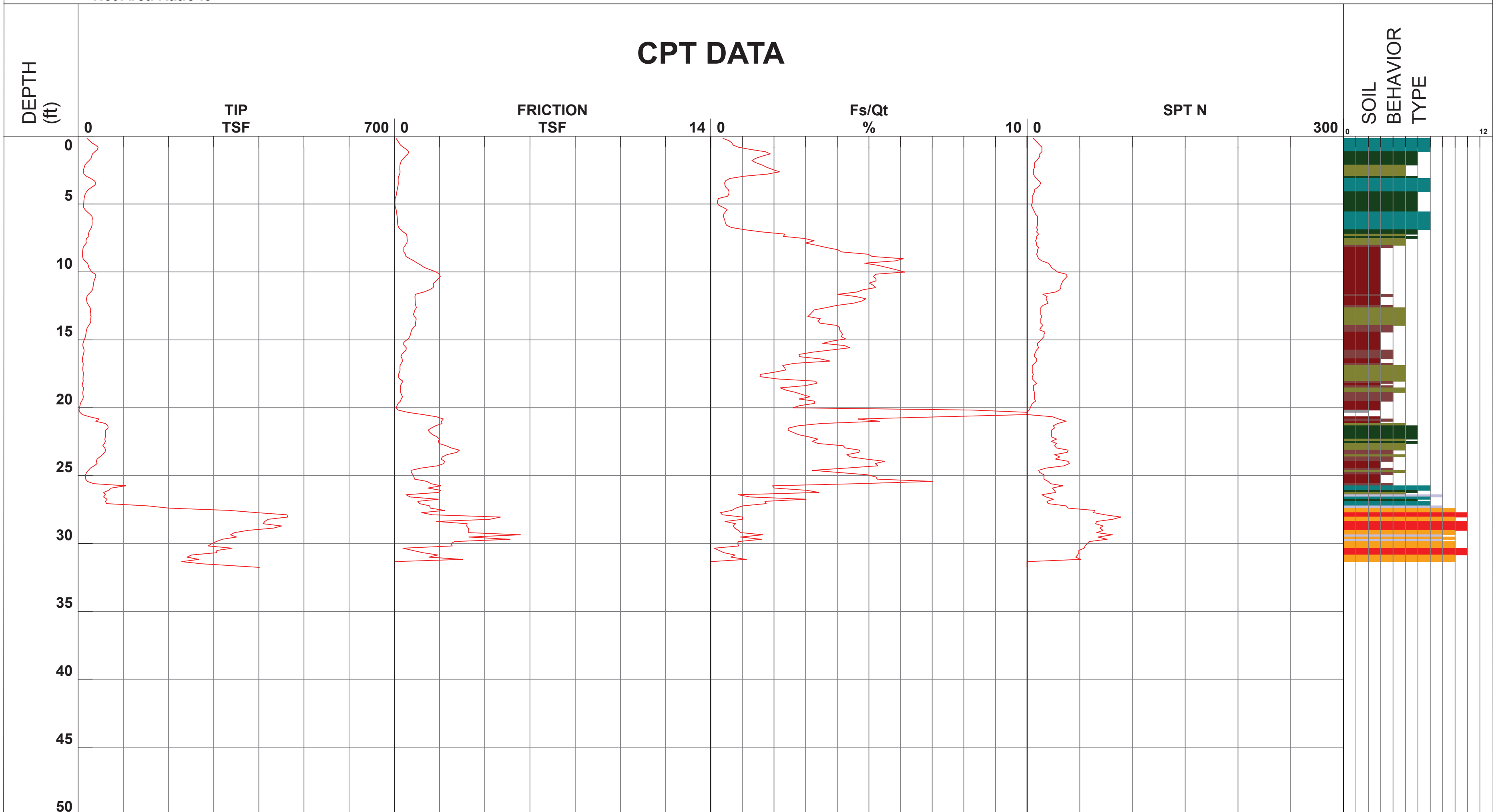
CPT3	Capitol Annex Project	
	Sacramento, California	
S1748-05-01	August 2019	Figure A19

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 15cm squared

S*Soil behavior type and SPT based on data from UBC-1983

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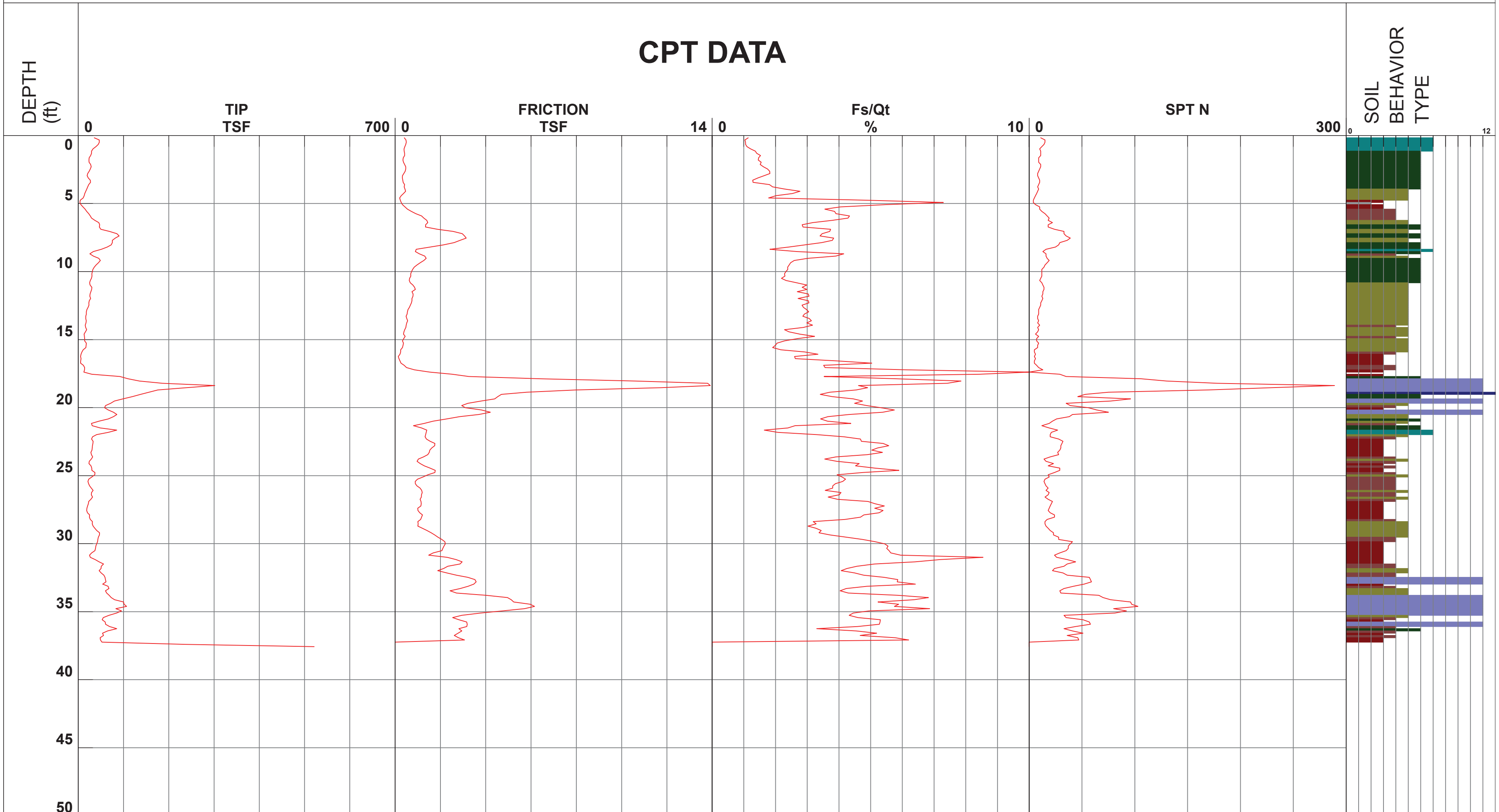
CPT4

Capitol Annex Project		
Sacramento, California		
S1748-05-01	August 2019	Figure A20

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 15cm squared

S*Soil behavior type and SPT based on data from UBC-1983

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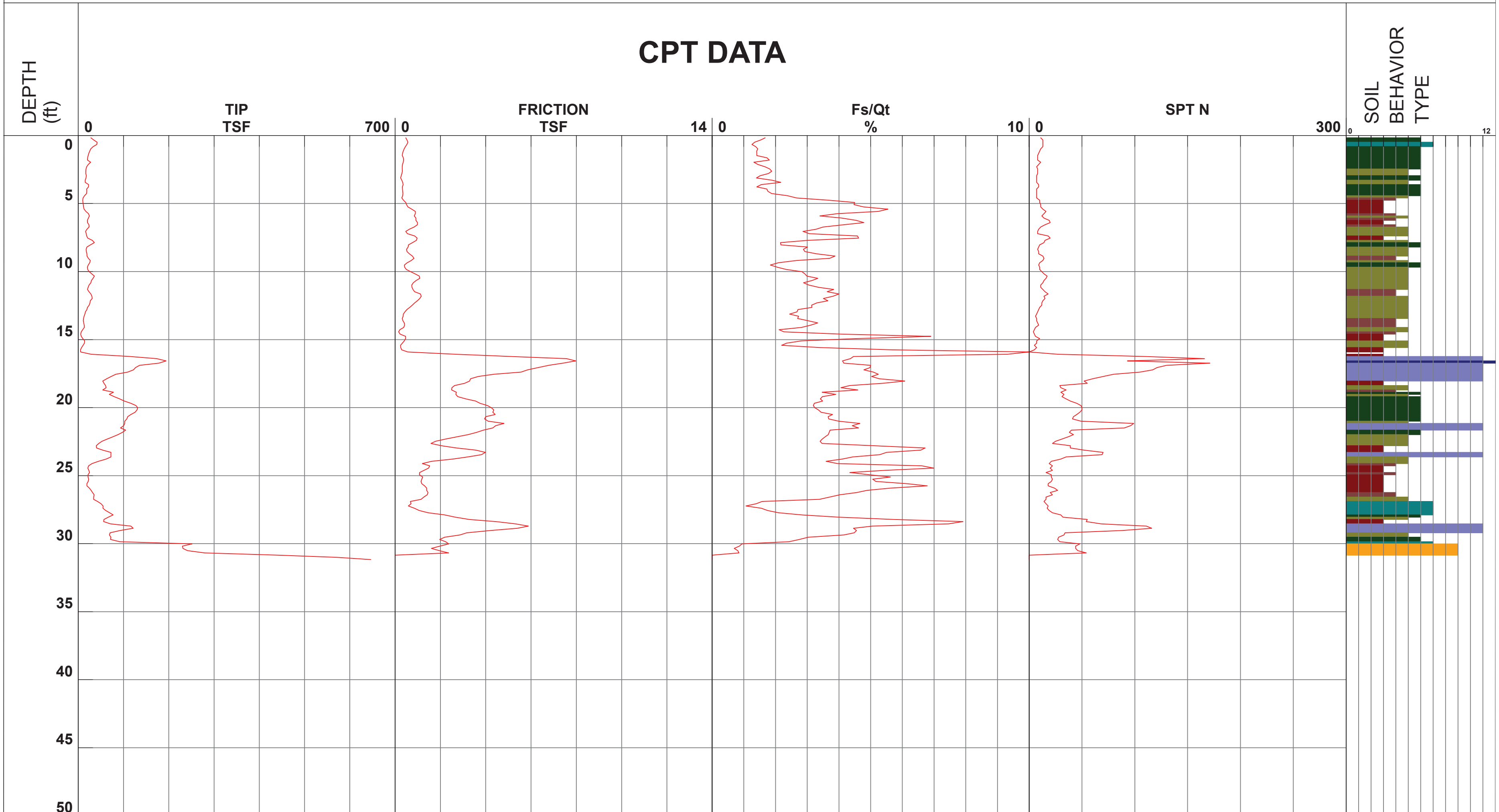
CPT5

Capitol Annex Project

Sacramento, California

S1748-05-01	August 2019	Figure A21

Net Area Ratio .8



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 15cm squared

S*Soil behavior type and SPT based on data from UBC-1983

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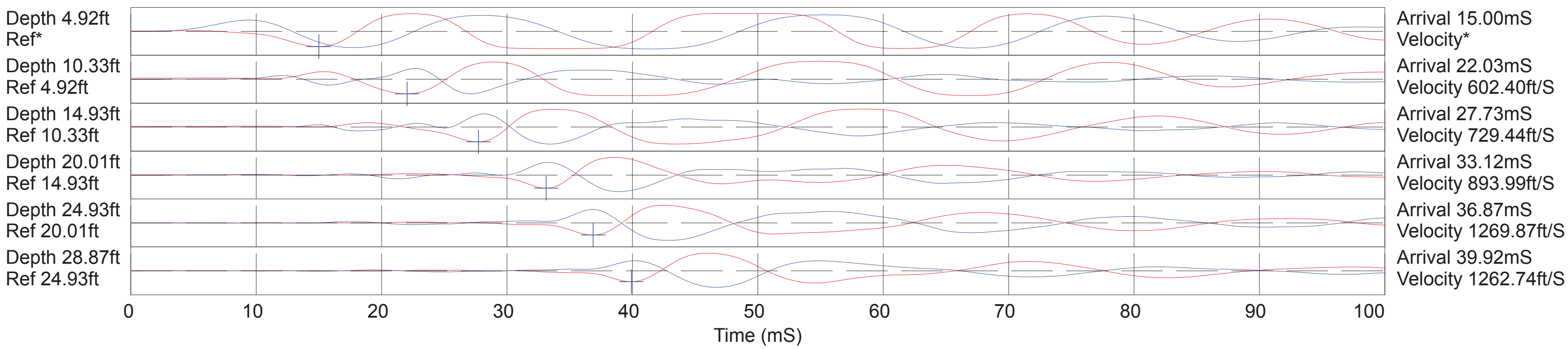
CPT6

Capitol Annex Project		
Sacramento, California		
S1748-05-01	August 2019	Figure A22



Hammer to Rod String Distance (ft): 5.83
* = Not Determined

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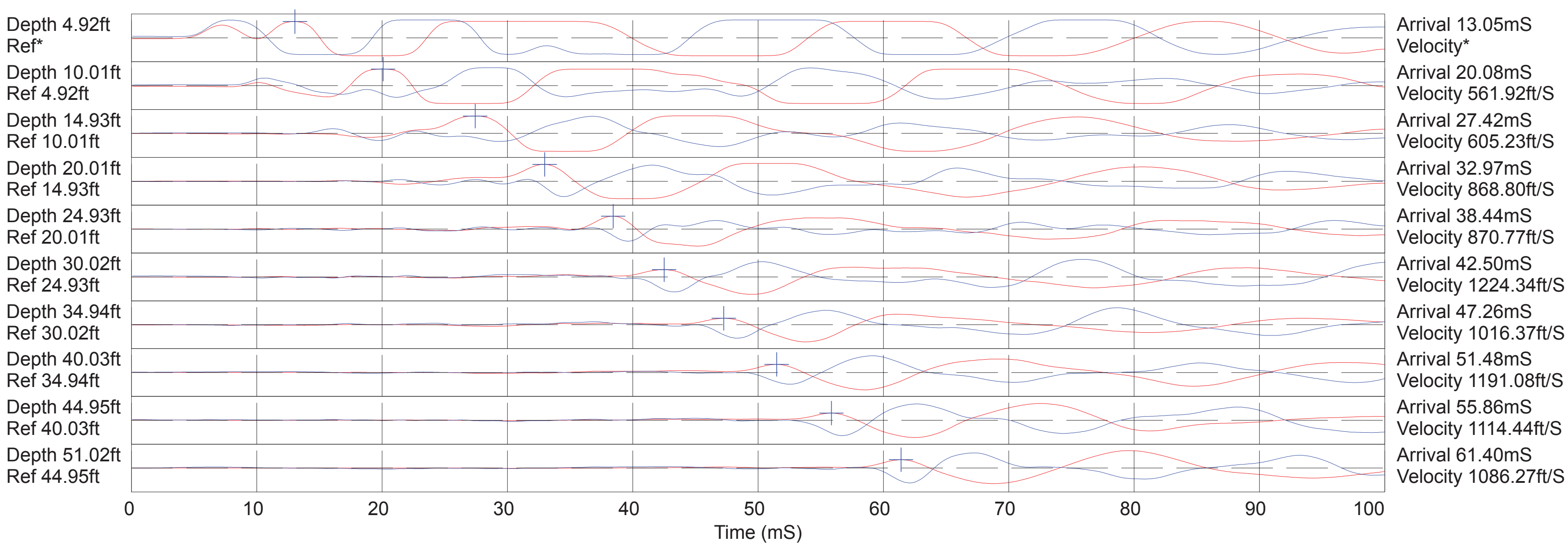
Shear Wave Velocity Plots – CPT1

Capitol Annex Project

Sacramento, California

S1748-05-01	August 2019	Figure A23



Hammer to Rod String Distance (ft): 5.83
* = Not Determined

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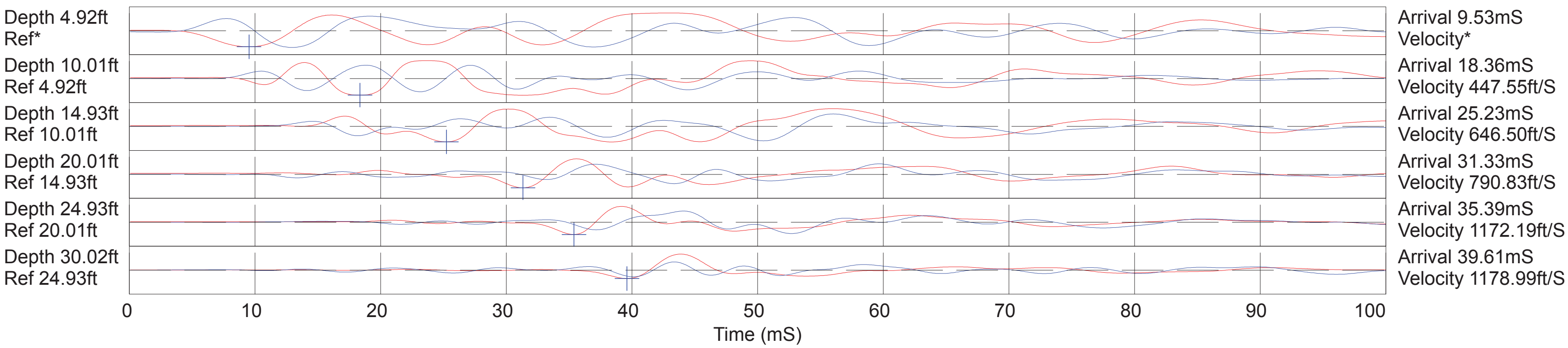
Shear Wave Velocity Plots – CPT3

Capitol Annex Project		
Sacramento, California		
S1748-05-01	August 2019	Figure A24



Hammer to Rod String Distance (ft): 5.83
* = Not Determined

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Shear Wave Velocity Plots – CPT4

Capitol Annex Project		
Sacramento, California		
S1748-05-01	August 2019	Figure A25

PLOTTED: 10/08/2018 01:20 PM BY: DRoss

Date Begin - End: 9/06/2018 - 9/07/2018	**Drilling Co.-Lic.#:** Woodward Drilling - #710079		**BORING LOG B-2**
Logged By: E. Santos	**Drill Crew:** J. Franco, R. Vega, J.C Licea, F. Carranza		
Hor.-Vert. Datum: Not Available	**Drilling Equipment:** B-57 Track Rig	**Hammer Type - Drop:** 140 lb. Auto - 30 in.	
Plunge: -90 degrees	**Drilling Method:** Hollow Stem Auger/Mud Rotary	**Hammer Efficiency:** 69.2%	
Weather: Sunny	**Auger Diameter:** 8 in./3.88 in. O.D.	**Hammer Cal. Date:** 1/03/2016	

Latitude: 38.57492°
Longitude: -121.49451°
Surface Condition: Asphalt Concrete

FIELD EXPLORATION							LABORATORY RESULTS								
Depth (feet)	Graphical Log	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Push Tube(PT)= psi Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		ASPHALT CONCRETE: about 5 inches												Hand Auger to 5 ft.
		AGGREGATE BASE: about 4 inches	1											
		Poorly Graded SAND (SP): olive to brown, dry, fine sand, withy concrete chunks, nails and debris												
		SILT (ML): low plasticity, olive to dark brown, moist, stiff												
5			2b 2a		BC=7 9 10	NR								Switch to HSA Resampled with sand catcher, 3-in. recovery
10		**Sandy SILT (ML)**: non-plastic to low plasticity, yellowish brown, moist, stiff, fine sand	3b 3a		BC=4 6 8 PP=1.0	16"								
15		**SILT (ML)**: low plasticity, yellowish brown, moist, very stiff	3c		BC=3 5 6 PP=2.5	4"								
20		**SILT with Sand (ML)**: low plasticity, yellowish brown, wet, hard, fine sand	4b 4a		BC=9 20 13 PP=>4.5	16"								
25		**Sandy SILT (ML)**: low plasticity, yellowish brown, wet, medium stiff, fine sand	5b 5a		BC=5 5 4 PP=0.5	18"								
30			6		PT=400			34.2	78.3					
		Silty SAND (SM): olive brown, wet, very dense, fine sand, non-plastic fines												

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20191381 DRAWN BY: DR CHECKED BY: ES DATE: 9/13/2018 REVISED: 10/8/2018	BORING LOG B-2 1100 O Street Office Building Sacramento, California	FIGURE A-5 PAGE: 1 of 4

gINT FILE: Klf_gint_master_2017 PROJECT NUMBER: 20191381.001A OFFICE FILTER: SACRAMENTO
gINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2017.GLB [_KLF_BORING/TEST PIT SOIL LOG]

Date Begin - End:	9/06/2018 - 9/07/2018	**Drilling Co.-Lic.#:**	Woodward Drilling - #710079		**BORING LOG B-2**
Logged By:	E. Santos	**Drill Crew:**	J. Franco, R. Vega, J.C Licea, F. Carranza		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	B-57 Track Rig	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger/Mud Rotary	**Hammer Efficiency:**	69.2%
Weather:	Sunny	**Auger Diameter:**	8 in./3.88 in. O.D.	**Hammer Cal. Date:**	1/03/2016

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 38.57492°
Longitude: -121.49451°
Surface Condition: Asphalt Concrete

Depth (feet)	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Push Tube(PT)= psi Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
	Silty SAND (SM): olive brown, wet, very dense, fine sand, fine subrounded gravel, non-plastic fines	7		BC=7 19 35	17"				85	16			
	Poorly Graded GRAVEL with Silt sand sand (GP-GM): wet, very dense, subrounded gravel up to 2 inches, fine to coarse sand												Rig chatter
40		8		BC=50/5"	NR								Stop 9/6/18 Resume 9/7/18
45		9b 9a		BC=25 45 45	12"	GP-GM			44	7.9			
	Poorly Graded SAND with Silt (SP-SM): wet, dense, fine sand												
50		10		BC=2 3 4	12"	SP-SM			90	8.5			
													Switch to Mud Rotary
		11		BC=10 17 19 PP=2.75	18"								
55	**SILT with Sand (ML)**: low plasticity, olive to olive yellow, wet, very stiff, fine sand												
	Silty SAND (SM): wet, dense, fine sand, non-plastic to low plasticity fines	12		BC=9 13 20	18"								
60													
	fine to medium sand	13		BC=15 21 20	18"								
65	**Fat CLAY (CH)**: medium to high plasticity, pale olive with black mottling, wet, hard	14		PP=>4.5									
	Sandy Fat CLAY (CH): medium plasticity, pale olive, wet, hard, fine sand	15b 15a		BC=20 38 50/4" PP=>4.5	16"								

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PROJECT NO.: 20191381	BORING LOG B-2		FIGURE
DRAWN BY: DR			
CHECKED BY: ES	1100 O Street Office Building Sacramento, California		A-5
DATE: 9/13/2018			
REVISED: 10/8/2018			PAGE: 2 of 4

PLOTTED: 10/08/2018 01:20 PM BY: DRoss

Date Begin - End:	9/06/2018 - 9/07/2018	**Drilling Co.-Lic.#:**	Woodward Drilling - #710079		**BORING LOG B-2**
Logged By:	E. Santos	**Drill Crew:**	J. Franco, R. Vega, J.C Licea, F. Carranza		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	B-57 Track Rig	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger/Mud Rotary	**Hammer Efficiency:**	69.2%
Weather:	Sunny	**Auger Diameter:**	8 in./3.88 in. O.D.	**Hammer Cal. Date:**	1/03/2016

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 38.57492°
Longitude: -121.49451°
Surface Condition: Asphalt Concrete

Depth (feet)	Graphical Log	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Push Tube(PT)= psi Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		Sandy Fat CLAY (CH): medium plasticity, pale olive, wet, hard, fine sand												
		Silty SAND (SM): olive, wet, very dense, fine sand, non-plastic fines, with mica flakes	16		BC=20 27 33	18"								
75														
		dense	17		BC=13 20 20	18"					17			
80														
			18		BC=16 19 27	17"	SM			100	12			
85														
		very dense	19		BC=18 28 36	17"								
90														
		olive	20a		BC=50 50/3"	9"								
95														
			21 22		BC=22 32 50/5" PP=>4.5	18"								
100		**SILT (ML):** low plasticity, pale olive, moist, hard												
		moderately to strongly cemented, with black mottling	23a		BC=20 50/3" PP=>4.5	9"								

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PROJECT NO.: 20191381	BORING LOG B-2		FIGURE
DRAWN BY: DR			
CHECKED BY: ES	1100 O Street Office Building Sacramento, California		A-5
DATE: 9/13/2018			
REVISED: 10/8/2018			PAGE: 3 of 4

gINT FILE: Klf_gint_master_2017 PROJECT NUMBER: 20191381.001A OFFICE FILTER: SACRAMENTO
gINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2017.GLB [_KLF_BORING/TEST PIT SOIL LOG]

PLOTTED: 10/08/2018 01:20 PM BY: DRoss

Date Begin - End:	9/06/2018 - 9/07/2018	Drilling Co.-Lic.#:	Woodward Drilling - #710079		BORING LOG B-2
Logged By:	E. Santos	Drill Crew:	J. Franco, R. Vega, J.C Licea, F. Carranza		
Hor.-Vert. Datum:	Not Available	Drilling Equipment:	B-57 Track Rig	Hammer Type - Drop:	140 lb. Auto - 30 in.
Plunge:	-90 degrees	Drilling Method:	Hollow Stem Auger/Mud Rotary	Hammer Efficiency:	69.2%
Weather:	Sunny	Auger Diameter:	8 in./3.88 in. O.D.	Hammer Cal. Date:	1/03/2016

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 38.57492°
Longitude: -121.49451°
Surface Condition: Asphalt Concrete

Depth (feet)	Graphical Log	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. / Push Tube(PT)= psi / Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		SILT (ML): low plasticity, pale olive, moist, hard												
110		**Silty SAND (SM):** olive and greenish black, wet, very dense, fine sand, non-plastic fines	24a		BC=38 50/3"	9"								

The boring was terminated at approximately 110 ft. below ground surface. The boring was backfilled with grout to 5 ft., soil cuttings to 5 inches and rapid set concrete at surface on September 10, 2018.

GROUNDWATER LEVEL INFORMATION:
Groundwater was observed at approximately 18 ft. below ground surface during drilling.
Groundwater was observed at approximately 18.5 ft. below ground surface after drilling completion.
GENERAL NOTES:

115
120
125
130
135

APPENDIX B

APPENDIX B

LABORATORY TESTING PROGRAM

Laboratory tests were performed in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. Selected soil samples were tested for their in-situ dry density and moisture content, grain size distribution, plasticity characteristics, expansion potential, shear strength parameters, and corrosion potential. The results of the laboratory tests are presented on the following tables and pages.

TABLE B1
CORROSION PARAMETER TEST RESULTS
(CALIFORNIA TEST METHODS 643, 417, AND 422)

Sample No.	Sample Depth (ft.)	pH	Minimum Resistivity (ohm-cm)	Chloride (ppm) / (%)	Sulfate (ppm) / (%)
B1-Bulk	1 – 5	6.8	3,200	92 / 0.009	30 / 0.003
B3-Bulk	1 – 10	6.9	2,100	108 / 0.011	10 / 0.001

*Caltrans considers a site corrosive to foundation elements if one or more of the following conditions exist for the representative soil samples at the site:

- The pH is equal to or less than 5.5.
- The resistivity is equal to or less than 1,000 ohm-cm.
- Chloride concentration is equal to or greater than 500 parts per million (ppm).
- Sulfate concentration is equal to or greater than 2,000 ppm.

According to the 2016 California Building Code Section 1904.1 which refers to the durability requirements of American Concrete Institute (ACI) 318 (Chapter 4), Type II cement may be used where soluble sulfate levels in soil are below 2,000 ppm.

TABLE B2
EXPANSION INDEX TEST RESULTS (ASTM D4829)

Sample No.	Sample Depth (ft.)	Moisture Content		Dry Density		Expansion Index	Expansion Potential based on Expansion Index*
		Before Test (%)	After Test (%)	Before Test (pcf)	After Test (pcf)		
B1 Bulk	1 – 5	12.2	24.7	100.1	97.3	**23**	**Low**
B3 Bulk	1 – 10	14.0	26.3	97.4	95.1	**19**	**Low**

**Expansion Potential Classification per ASTM D4829.*

Sample ID	Depth (feet)	Liquid Limit	Plastic Limit	Plasticity Index	Maximum Size (mm)	%<#200 Sieve	Water Content (%)	Dry Density (pcf)
B1-5.5	5.5	41	26	15	---			
B1-6.0	6				---		26.0	96.7
B1-11	11				---		27.6	92.5
B1-16.0	16				---		21.9	102.2
B1-20.5	20.5				---	77.8	20.3	
B1-21.0	21				---		19.4	113.3
B2-3.0	3				---		23.3	93.4
B2-6.0	6				---		28.9	85.8
B2-8.5	8.5				---		30.1	93.5
B2-16.0	16				---		24.7	101.2
B2-21.0	21				---		21.1	107.6
B2-30.0	30				---	15.5	11.3	
B3-13.0	13				---		31.1	90.2
B3-15.5	15.5	24	19	5	---			
B3-16.0	16				---		23.1	102.8
B3-25.0	25				---		35.3	
B3-30.0	30				---		39.7	
B4-6.0	6				---		6.9	101.6
B4-10.0	10				---		15.9	99.1
B4-16.0	16				---		31.1	90.9
B4-21.0	21				---		27.2	98.1
B4-26.0	26				---		21.8	102.5
B4-45.0	45				---	51.4	32.4	
B4-60.0	60				---		22.4	
B5-11.0	11				---		35.5	85.9
B5-16.0	16				---		27.7	96.6
B5-21.0	21				---		28.6	100.8
B5-23.0	23				---		28.0	96.7
B6-6.0	6	25	26	NP	---		21.1	94.8
B6-11.0	11				---		26.6	71.5
B6-16.0	16				---		30.3	93.5
B6-21.0	21				---		29.6	93.7
B6-45.0	45				---	59.1	34.3	

US LAB SUMMARY GEOTECH 2 S1748-05-01 CAPITOL ANNEX.GPJ US LAB.GDT 7/15/19

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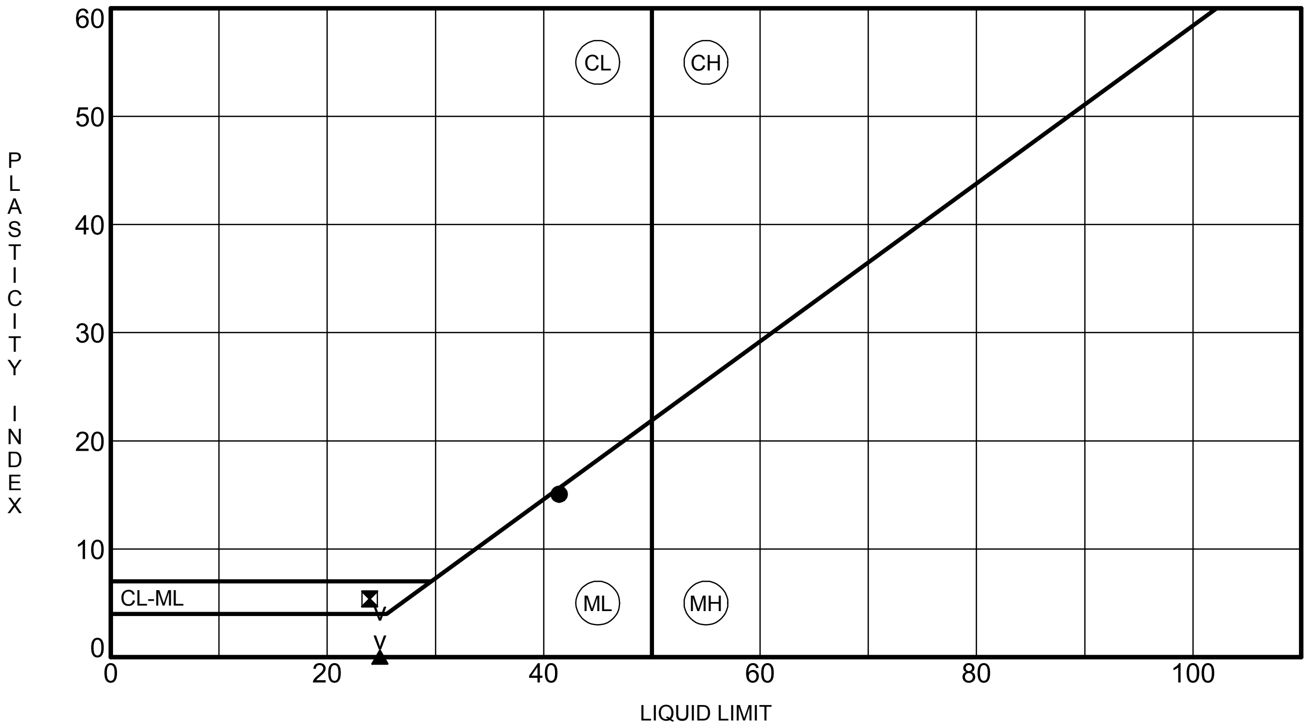
Geocon Consultants
3160 Gold Valley Drive, Suite 800
Rancho Cordova, CA 95742
Telephone: 9168529118

Summary of Laboratory Results

Project: Capitol Annex
Location: Sacramento, California
Number: S1748-05-01
Figure: B1



	Sample No.	Liquid Limit	Plastic Limit	Plasticity Index	% Pass #200 Sieve	Unified Soil Classification Description	Preparation Method
●	**B1-5.5**	**41**	**26**	**15**		**SILT with SAND (ML)**	**dry**
⊠	**B3-15.5**	**24**	**19**	**5**		**SILTY CLAY (CL-ML)**	**dry**
▲	**B6-6.0**	**25**	**26**	**NP**		**SANDY SILT (ML)**	**dry**

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Rancho Cordova, CA 95742
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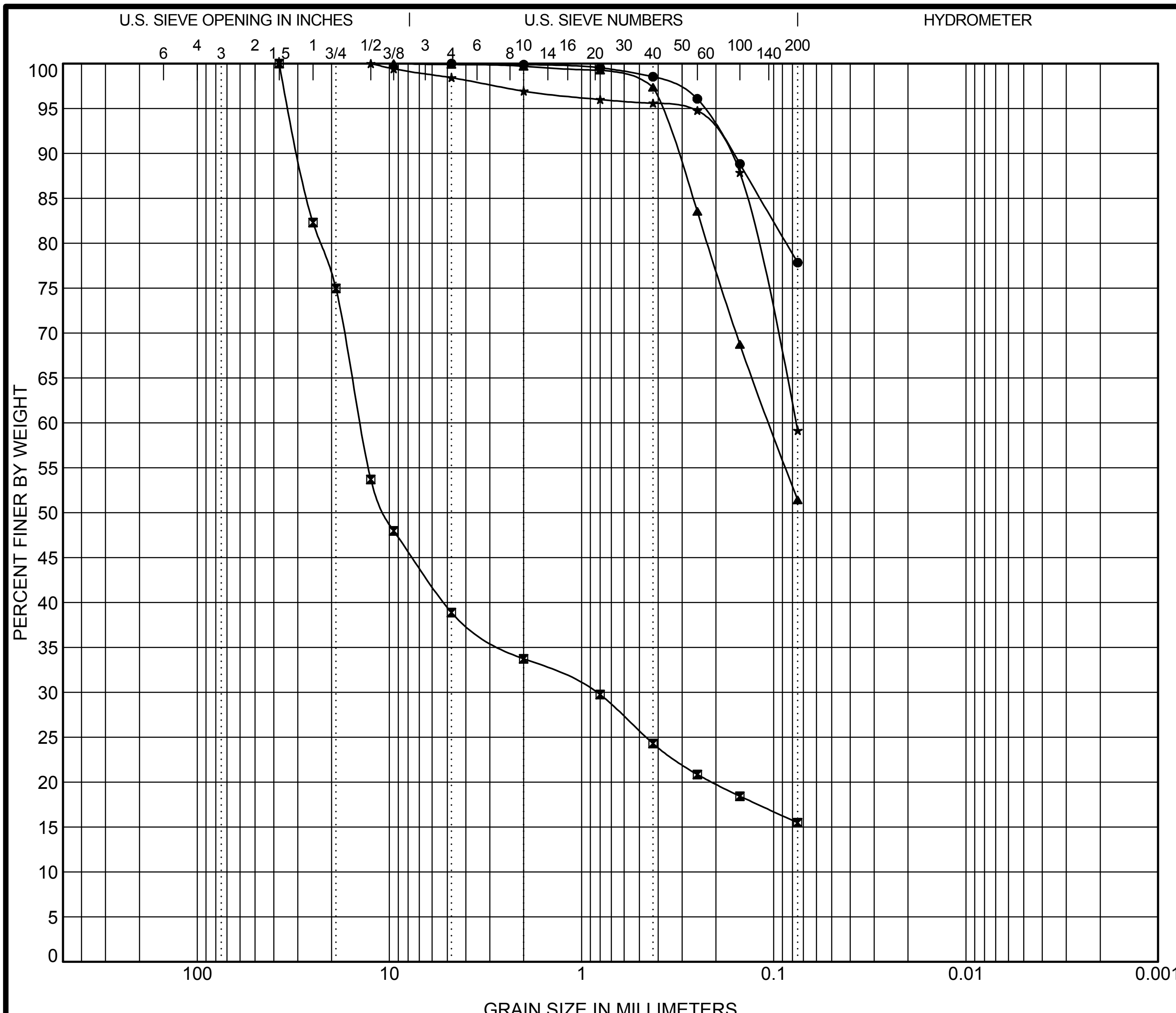
ATTERBERG LIMITS (ASTM D4318)

Project: Capitol Annex
Location: Sacramento, California
Number: S1748-05-01
Figure: B2



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample No.	Classification	LL	PL	PI	Cc	Cu
●	**B1-20.5**	**SILT with SAND (ML)**					
⧗	**B2-30.0**	**SILTY GRAVEL with SAND (GM)**					
▲	**B4-45.0**	**SANDY SILT (ML)**					
★	**B6-45.0**	**SANDY SILT (ML)**					

	Sample No.	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B1-20.5**	**4.75**				**0.0**	**22.2**	**77.8**	
⧗	**B2-30.0**	**37.5**	**14.151**	**0.848**		**61.1**	**23.4**	**15.5**	
▲	**B4-45.0**	**9.5**	**0.106**			**0.1**	**48.5**	**51.4**	
★	**B6-45.0**	**12.5**	**0.077**			**1.6**	**39.3**	**59.1**	

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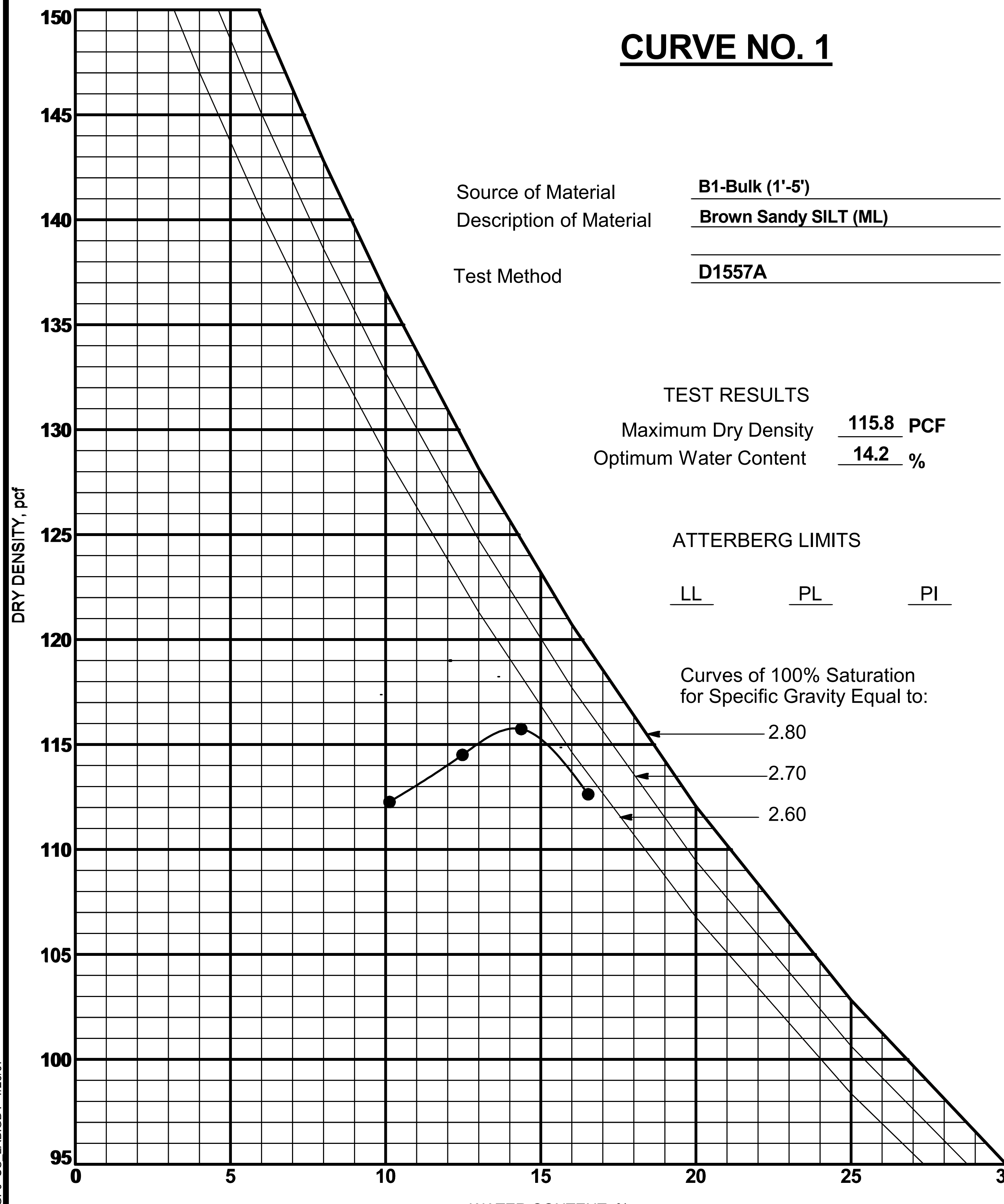
GRAIN SIZE DISTRIBUTION (ASTM D422, D6913)

Project: Capitol Annex
Location: Sacramento, California
Number: S1748-05-01
Figure: B3

GRAIN SIZE COPY 2 S1748-05-01 CAPITOL ANNEX.GPJ US_LAB.GDT 7/15/19



CURVE NO. 1

Source of Material: B1-Bulk (1'-5')

Description of Material: Brown Sandy SILT (ML)

Test Method: D1557A

TEST RESULTS

Maximum Dry Density 115.8 PCF

Optimum Water Content 14.2 %

ATTERBERG LIMITS

LL	PL	PI

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3160 Gold Valley Drive, Suite 800
Rancho Cordova, CA 95742
Telephone: (916) 852-9118
Fax: (916) 852-9132

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MOISTURE-DENSITY RELATIONSHIP

Project: Capitol Annex

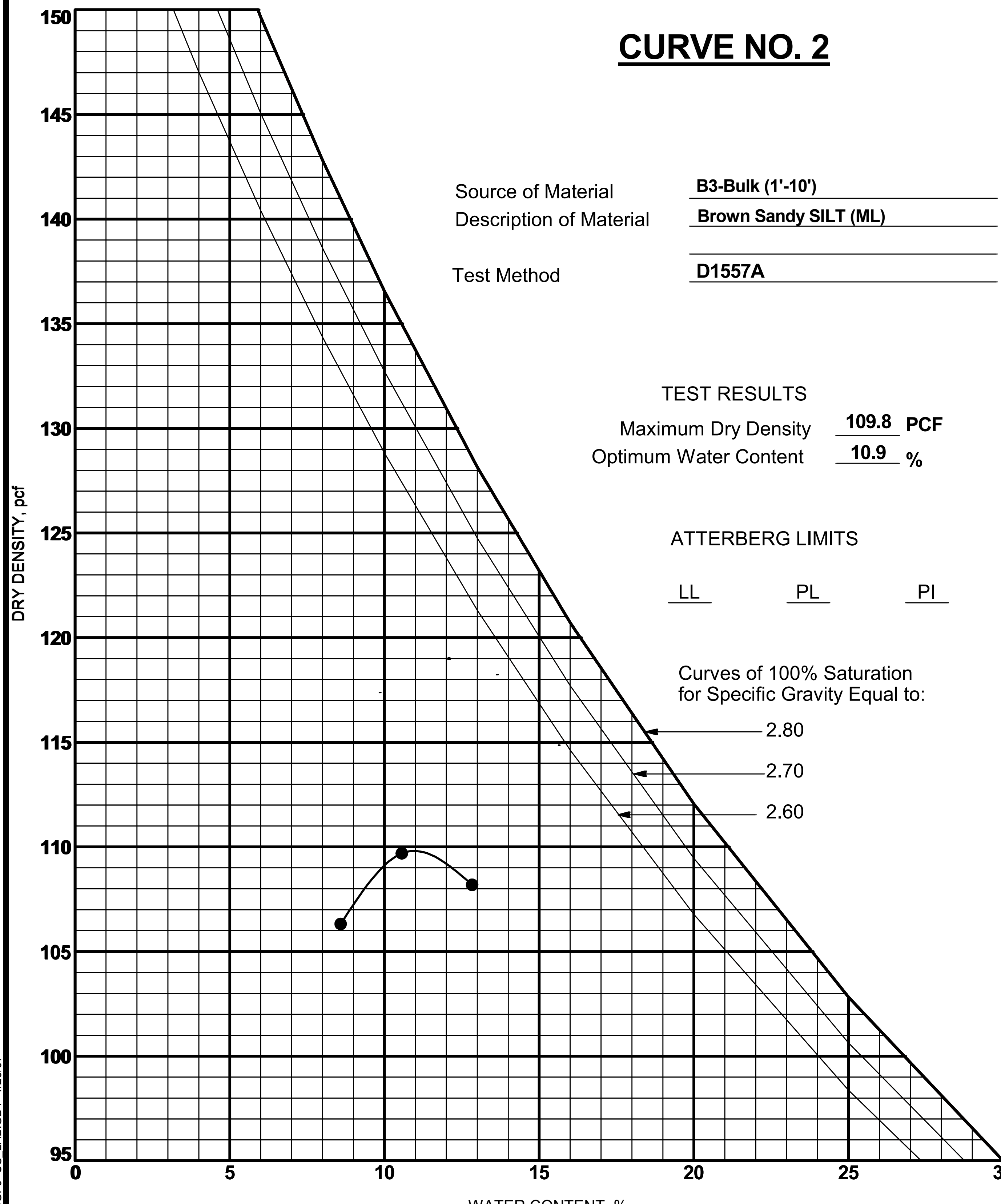
Location: Sacramento, California

Number: S1748-05-01

Figure: B4

US COMPACTION COPY 2.GPJ US LAB.GDT 1/26/07

CURVE NO. 2



Source of Material	B3-Bulk (1'-10')
Description of Material	Brown Sandy SILT (ML)
Test Method	D1557A

TEST RESULTS

Maximum Dry Density 109.8 PCF

Optimum Water Content 10.9 %

ATTERBERG LIMITS

LL	PL	PI

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MOISTURE-DENSITY RELATIONSHIP

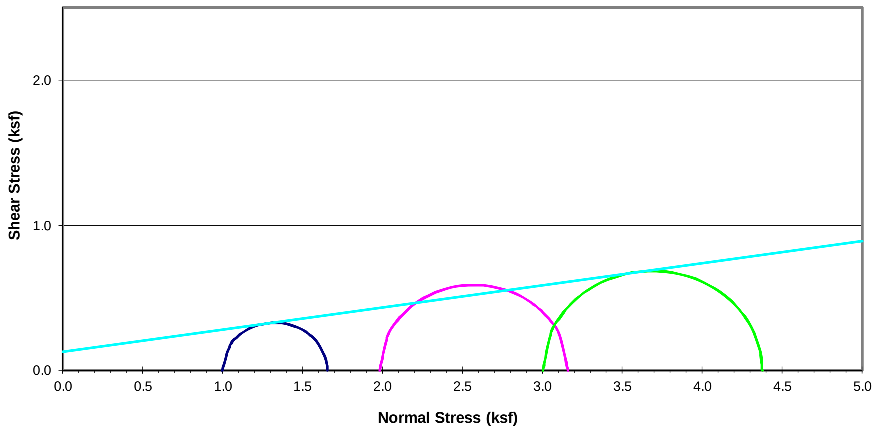
Project: Capitol Annex

Location: Sacramento, California

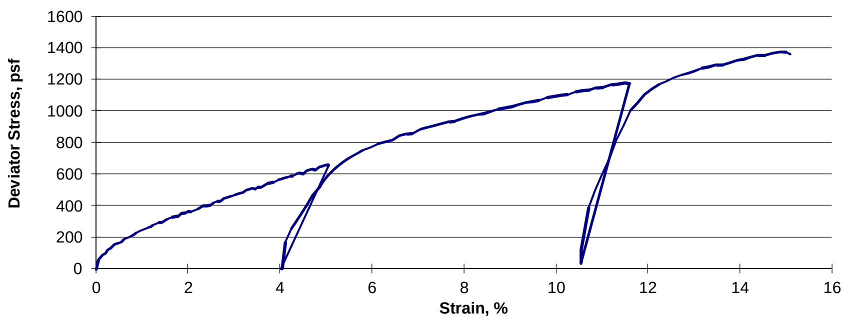
Number: S1748-05-01

Figure: B5

MOHR'S CIRCLES



STRESS-STRAIN



Failure Photo

Test Results

	ϕ, degrees	**8.7**
	c, psf	**128**

Sample Description

	Sample Number	B6-13.5
	Sample Depth (feet)	13.5
	Material Description	Dark yellowish brown Sandy SILT (ML)

Initial Conditions at Start of Stage

Sample ID (psf), minor principal stress	**1000**	**2000**	**3000**
Height (inch)	4.960	4.709	4.253
Diameter (inch)	2.371	2.433	2.488
Moisture Content (%)	32.6	32.6	32.6
Dry Density (pcf)	89.6	89.6	89.6
Saturation (%)	99.9	99.9	99.9

Shear Test Conditions

Strain Rate (%/min)	0.9938	0.9845	0.9678
Major Principal Stress at Failure (psf)	1650	3160	4380
Strain at failure (%)	5.05	11.50	14.87
Deviator Stress and Fail (psf)	660	1180	1370

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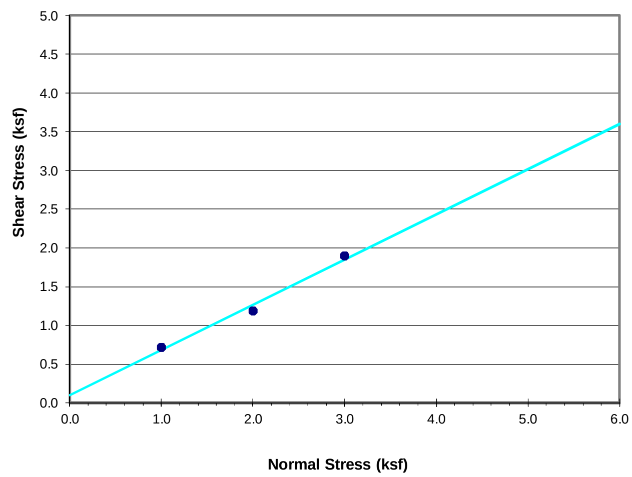
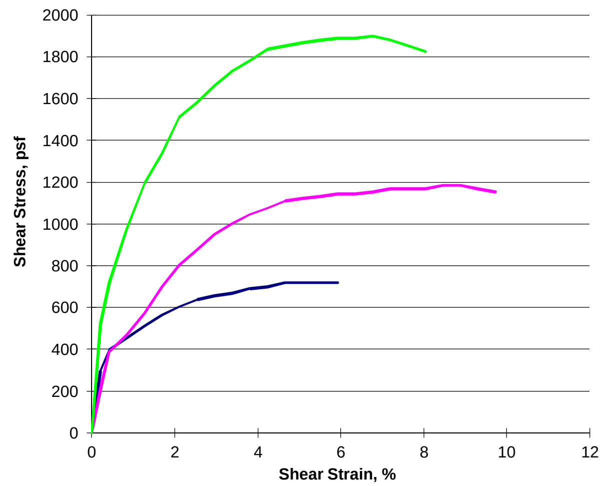
Triaxial Shear Strength - UU Test (staged)

Project: Capitol Annex
Location: Sacramento, CA
Number: S1748-05-01
Figure: B6

SHEAR STRENGTH



STRESS-STRAIN



Sample Description			
Boring Number	B5-6		
Sample Depth (feet)	6.00		
Material Description	Dark yellowish brown Silty Clayey SAND		
Initial Conditions at Start of Test			
Sample ID (psf)	**1000**	**2000**	**3000**
Height (inch)	1.00	1.00	1.00
Diameter (inch)	2.363	2.363	2.363
Moisture Content (%)	24.0	12.5	9.2
Dry Density (pcf)	86.5	90.4	92.7
Estimated Specific Gravity	2.70	2.70	2.70
Saturation (%)	68.5	39.1	30.3
Shear Test Conditions			
Strain Rate (%/min)	0.846	0.927	0.937
Major Principle Stress at Failure (psf)	719	1186	1900
Strain at Failure (%)	4.66	8.46	6.77
Test Results			
ϕ, degrees	**30.3**		
c, psf	**100**		

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Rancho Cordova, California 95742
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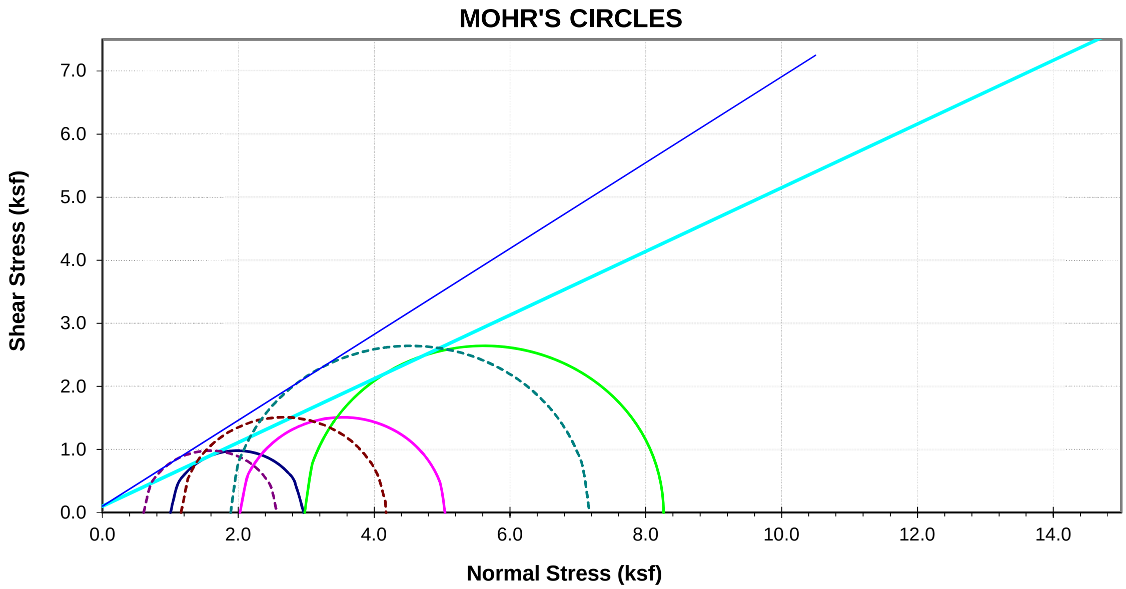
Fax: (916) 852-9132

Direct Shear Strength Test (ASTM D3080)

Project: Capitol Annex
Location: Sacramento, CA
Number: S1748-05-01
Figure: B7

Consolidated Undrained Triaxial Compression - ICU Test ASTM D4767

Boring Number	B1
Sample Number	B1-11
Sample Description	Brown Sandy SILT



Test Results, At Maximum Principal Stress Ratio		Total		Effective
Friction Angle ϕ (degrees)		**26.8**		**34.3**
cohesion (psf)		**100**		**100**
Initial Conditions at Start of Test		stage 1	stage 2	stage 3
Sample ID (psf), Initial Confining Pressure		**1000**	**2000**	**3000**
Height (inch)		4.870	4.821	4.769
Diameter (inch)		2.424	2.429	2.438
Moisture Content (%)		27.6	--	--
Dry Density (pcf)		92.5	--	--
Saturation (%)		88.7	--	--
After Saturation				
Dry Density (pcf)		92.5	--	--
After Consolidation				
Dry Density (pcf)		93.0	--	--
Shear Test Conditions				
Dry Density (pcf)		93.0	93.3	93.8
Moisture Content (%)		--	--	29.7
Saturation (%)		--	--	98.6
Strain rate (%/hr)		1.94	2.05	2.04
Cell pressure (psf)		11160	12150	13100
Initial Back Pressure (psf)		10150	10130	10130
Initial Effective Confining Pressure (psf)		1000	2030	2980
Total Major Principal Stress At Failure (psf)		2960	5040	8260
Effective Major Principal Stress At Failure (psf)		2570	4180	7170
Pore Pressure At Failure (psf)		400	870	1090
Effective Minor Principal Stress At Failure (psf)		610	1160	1890

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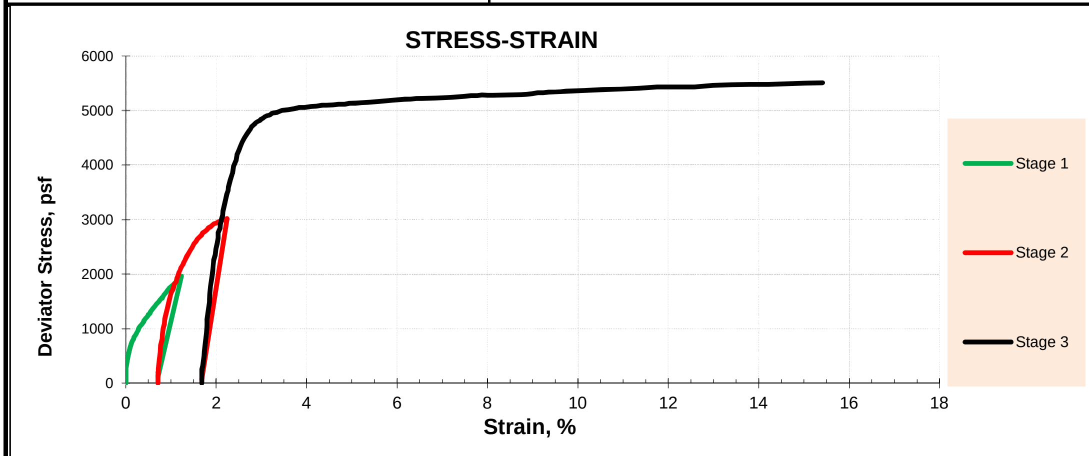
3160 Gold Valley Drive, Suite 800
Rancho Cordova, California 95742
Telephone: (916) 852-9118
Fax: (916) 852-9132

Triaxial Shear Strength - CU Test, ASTM D4767 with Pore Pressure Measurements (staged)

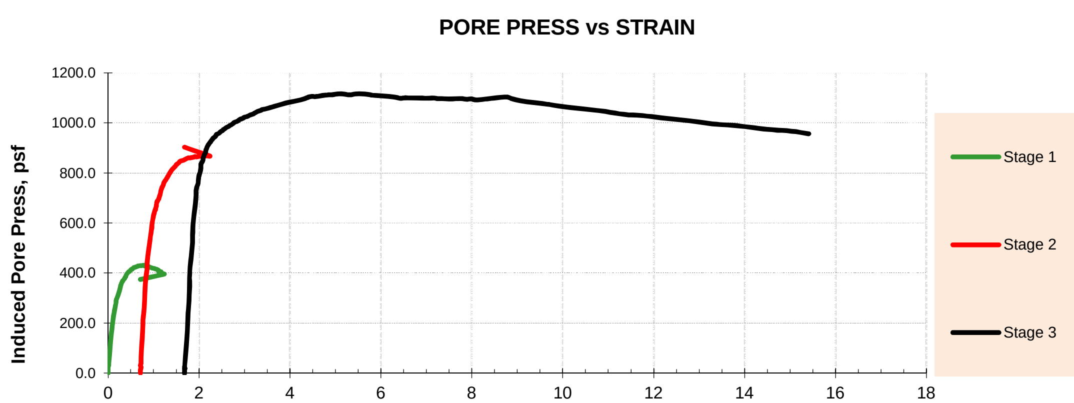
Project: Capitol Annex
Location: Sacramento, CA
Number: S1748-05-01
Figure: B8

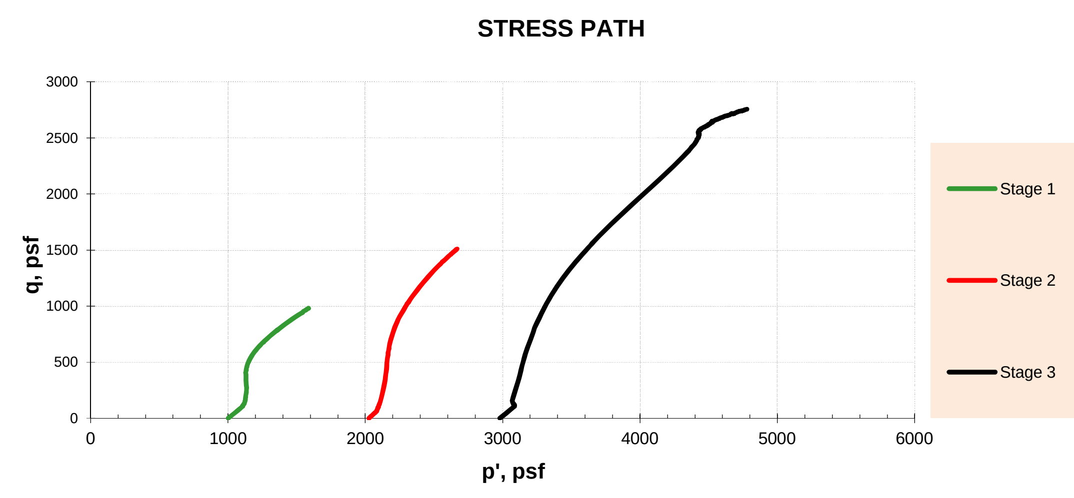
CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION - ICU TEST ASTM D4767

Boring Number	B1
Sample Number	B1-11
Sample Description	Brown Sandy SILT



After shear photo





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3160 Gold Valley Drive, Suite 800
Rancho Cordova, California 95742
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Fax: (916) 852-9132

Triaxial Shear Strength - CU Test, ASTM D4767 with pore pressure measurements

Project: Capitol Annex
Location: Sacramento, CA
Number: S1748-05-01
Figure: B8

APPENDIX C

APPENDIX C

LIQUEFACTION ANALYSIS

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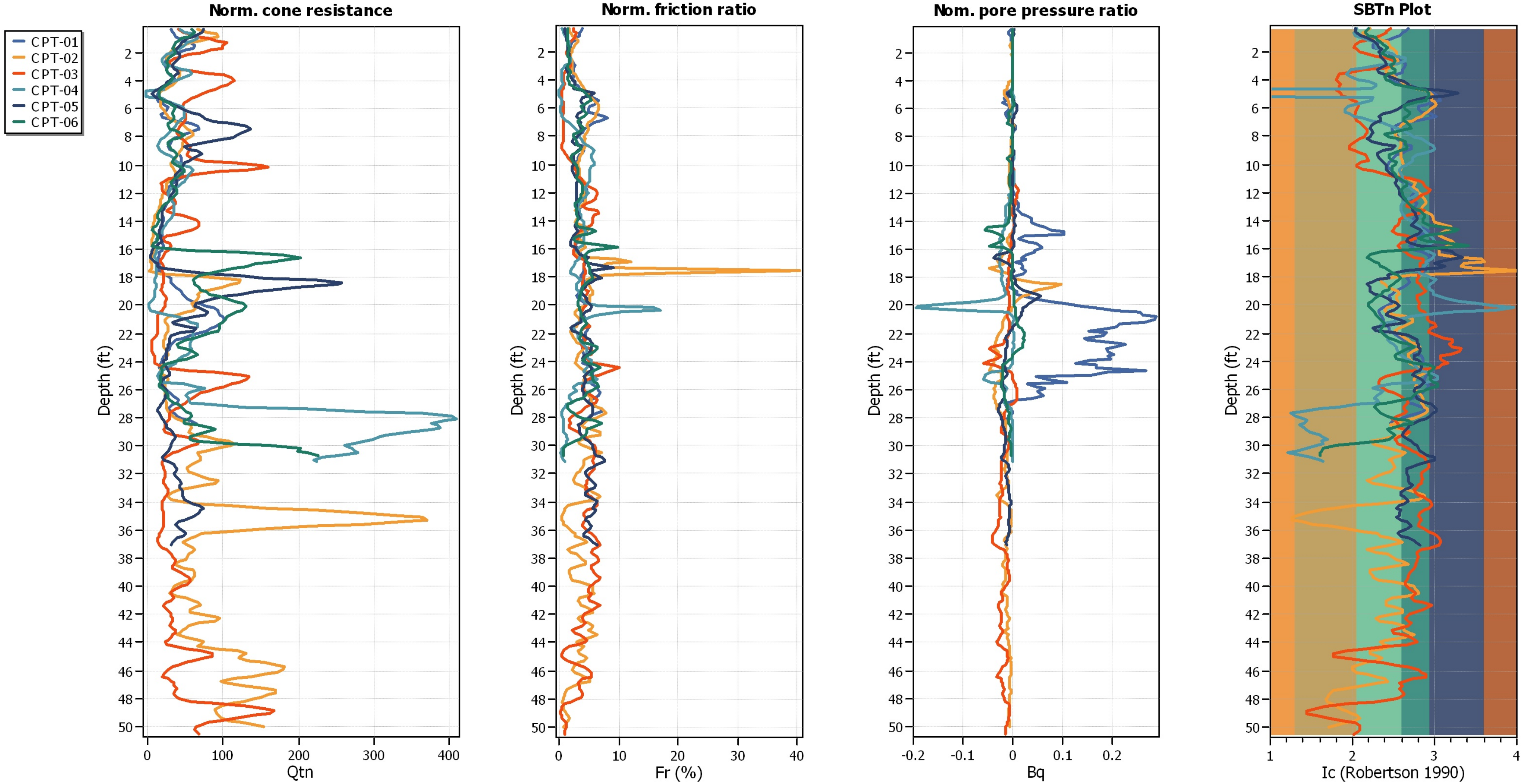
Overlay Normalized Plots



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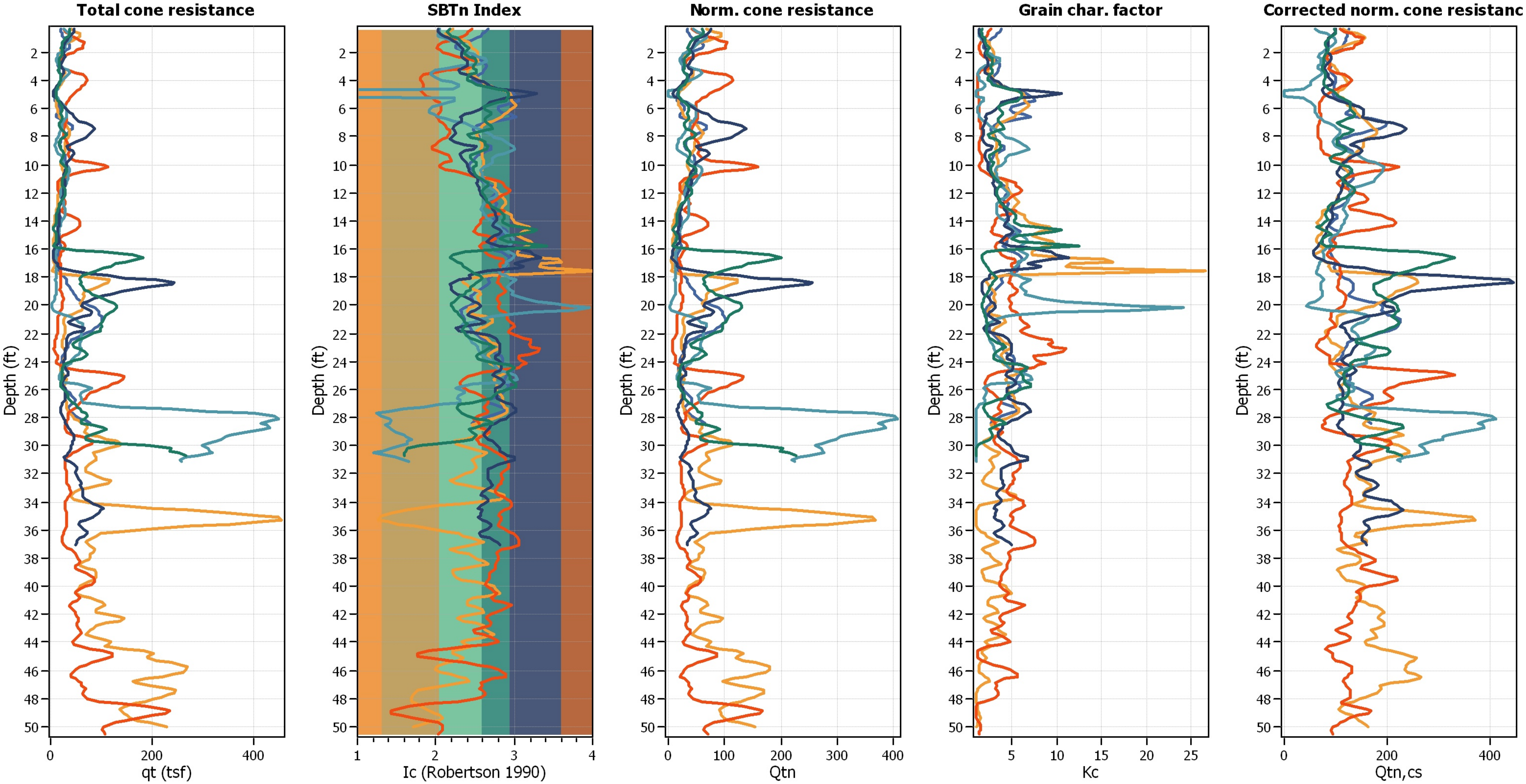
Overlay Intermediate Results



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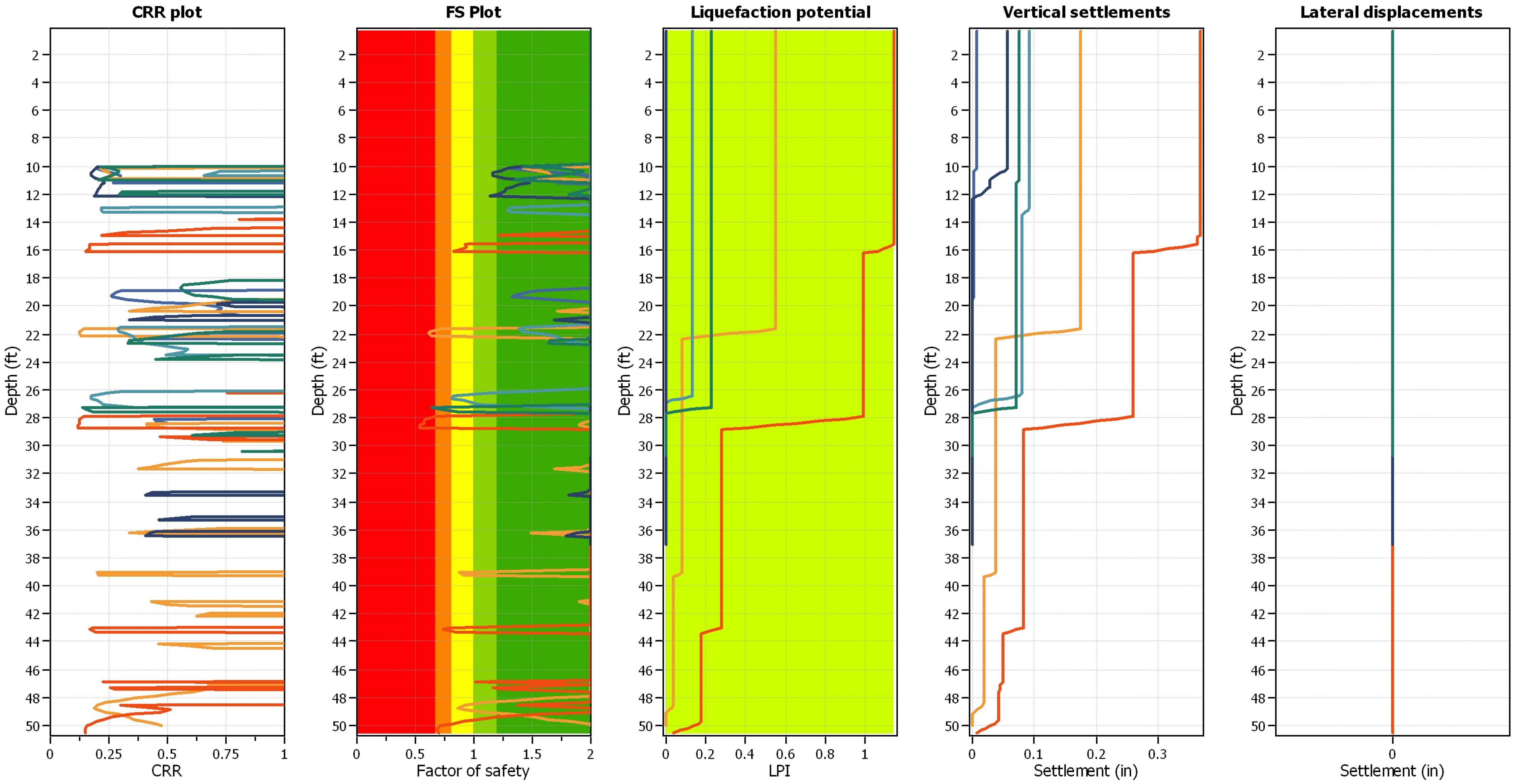
Overlay Cyclic Liquefaction Plots



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Overlay Strength Loss Plots