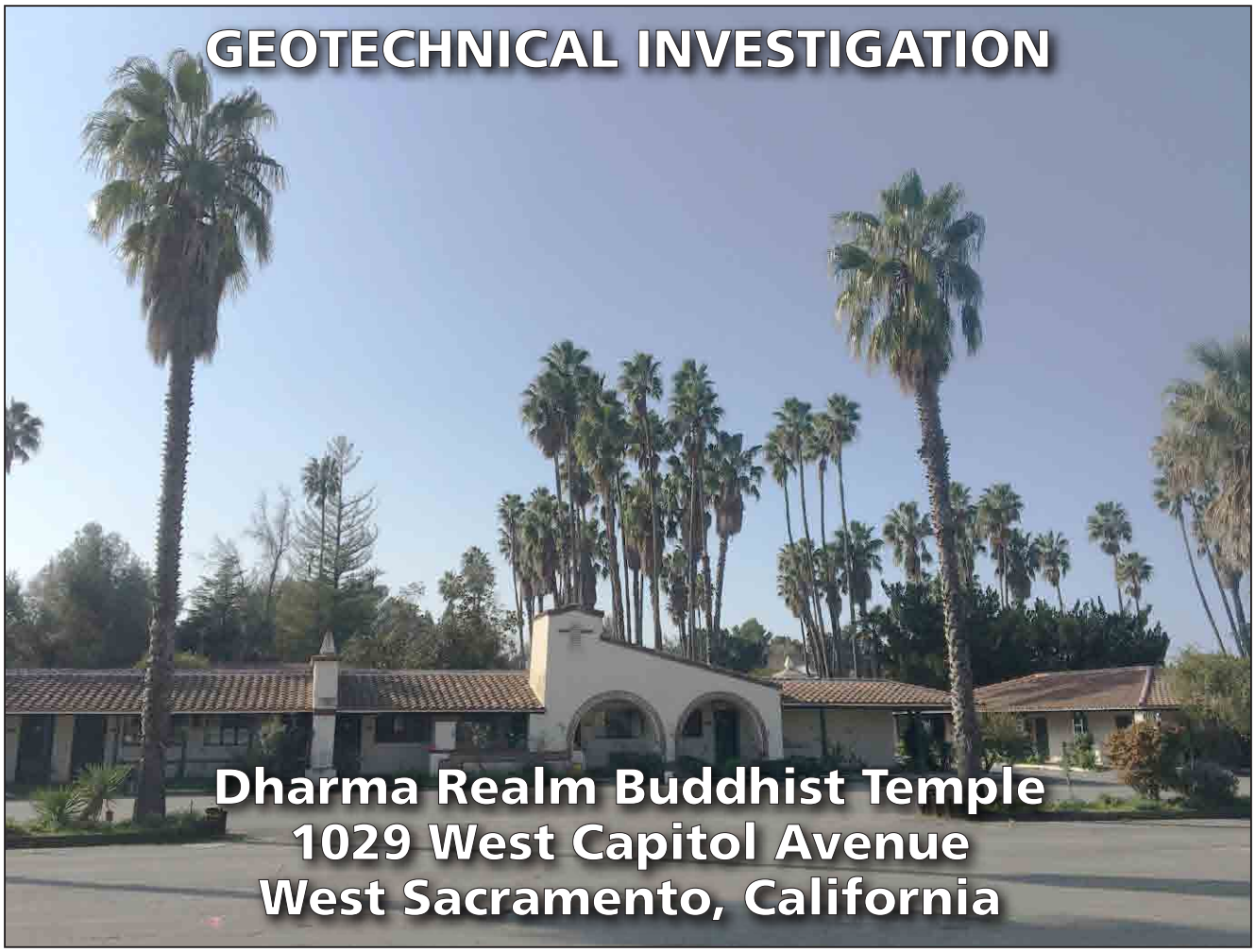
Appendix E
Geotechnical Report

GEOTECHNICAL INVESTIGATION



Dharma Realm Buddhist Temple
1029 West Capitol Avenue
West Sacramento, California

PREPARED FOR:

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GEOCON

GEOCON PROJECT NO. S1483-05-01 **APRIL 2018**

GEOCON
CONSULTANTS, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. S1483-05-01
April 4, 2018

VIA ELECTRONIC MAIL

Ken Paddock
Sr. Project Architect
Pacific Design Group
18071 Irvine Boulevard
Tustin, California 92780

Subject: GEOTECHNICAL INVESTIGATION
DHARMA REALM BUDDHIST TEMPLE
1029 W. CAPITOL AVENUE
WEST SACRAMENTO, CALIFORNIA

Dear Mr. Paddock:

In accordance with your authorization of our proposal (Geocon Proposal No. LS-17-242 dated August 17, 2017), we performed a geotechnical investigation for the proposed Dharma Realm Buddhist Temple located at 1029 West Capitol Avenue in West Sacramento, California.

The accompanying report presents our findings, conclusions, and recommendations regarding geotechnical aspects of redeveloping the site as presently proposed. In our opinion, no adverse geotechnical conditions were encountered that would preclude development at the site provided recommendations of this report are incorporated into the design and construction of the project.

Please contact us if you have any questions regarding this report or if we may be of further service.

Sincerely,

GEOCON CONSULTANTS, INC.

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GEOTECHNICAL INVESTIGATION

1.0 PURPOSE AND SCOPE

This report presents the results of our geotechnical investigation for the proposed Dharma Realm Buddhist Temple located at 1029 West Capitol Avenue in West Sacramento, California. The approximate site location is depicted on the Vicinity Map, Figure 1.

The purpose of our investigation was to evaluate subsurface soil and geologic conditions at the site and provide conclusions and recommendations relative to the geotechnical aspects of designing and constructing the project as presently proposed.

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

- Performed a limited geologic literature review to aid in evaluating the geologic conditions present at the site. A list of referenced material is included 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.
- Notified subscribing utility companies via Underground Service Alert (USA) a minimum of 48 hours (as required by law) prior to performing exploratory excavations at the site.
- Paid required fees and obtained a subsurface exploration permit from the Yolo County Environmental Health Division (YCEHD).
- Performed five exploratory borings (B1 through B5) with a track-mounted CME 75 drill rig using hollow-stem augers to depths ranging from approximately 4 to 41½ feet.
- Performed three in-situ cone penetration test (CPT) soundings (CPT1 through CPT3) using a 20-ton CPT rig to depths ranging from approximately 60 to 89 feet.
- Performed shear wave velocity measurements at approximate 5-foot intervals to a depth of approximately 89 feet in CPT1.
- Obtained representative disturbed and relatively undisturbed soil samples from the exploratory borings.
- Logged the borings in accordance with the Unified Soil Classification System (USCS).
- Upon completion, backfilled the borings and CPT soundings with neat cement grout. Borings in paved areas were capped with cold-patch asphalt concrete.
- Performed laboratory tests to evaluate pertinent geotechnical parameters.
- Prepared this report to summarize our findings, conclusions, and recommendations regarding the geotechnical aspects of developing the site as presently proposed.

Details of our field exploration program including exploratory boring logs are presented in Appendix A. Approximate locations of our borings are shown on the Site Plan, Figure 2. 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 approximately 15-acre site is located on the south side of West Capitol Avenue east of Merkley Avenue. The site currently contains an approximately 188,500-square-foot building and associated paved parking areas. Site-specific topography/survey information was not available for our review as of the date of this report. Based on satellite imagery topographic information (Google Earth Pro, May 19, 2017, WGS84 EGM96 Geoid), the site is relatively flat and level with an approximate elevation of 15 feet. The current site configuration is shown on the Site plan, Figure 2. Photos of the site are presented as Photos 1 and 2.

The project consists of constructing a 94,107-square-foot, three-story temple with a 19,080-square foot pedestrian bridge/stair structure that connects the third floor to a new central plaza below. The temple building will include assembly facilities, a kitchen, and cafeteria. The project configuration is shown on the Proposed Project Plan, Figure 3. Elevation views of the proposed temple and pedestrian bridge/stair structure are shown on the Proposed Project Elevation, Figure 4.

The building will likely be of steel-frame and/or concrete construction. Based on information provided by the project structural engineer, the temple building includes large spans (up to 120 feet). Maximum column loading is on the order of 3,500 kips and maximum wall loading is on the order of 32 kips per foot. Due to the high structural loading and the subsurface conditions at the site, the building will require deep foundations or shallow foundations combined with ground improvement.

Other improvements will likely include underground utility infrastructure, concrete flatwork and driveways, and landscaping. Grading plans are not yet available; however, due to the relatively flat site topography, we anticipate relatively minor grading with cuts and fills on the order of 3 feet or less.

3.0 SOIL AND GEOLOGIC CONDITIONS

We identified geologic and soil conditions by observing and sampling exploratory borings and reviewing the referenced geologic literature (Section 9.0). Soil descriptions below include the USCS symbol where applicable.

3.1 Site and Regional Geology

Based on our review of published geologic maps of the area, the site is located in the west-central portion of the Great Valley geomorphic province of California. The Great Valley (Sacramento and San Joaquin Valleys) of California is a long structural depression or down-warped trough, with the axis of the trough lying close to the eastern front of the Coast Ranges geomorphic province. In general, the southern portion of the Sacramento Valley is underlain by Quaternary continental deposits (alluvial deposits) and late Tertiary age marine and continental sedimentary rocks, which rest on a basement complex consisting of granitic rocks of the Sierra Nevada. Based on the *Preliminary Geologic Map of the*

Sacramento 30' x 60' Quadrangle, California, California Geological Survey (CGS), 2011, the site is underlain by Holocene basin (map symbol Qhb) deposits described as a mixture of sand, silt, and clay.

3.2 Existing Pavement Sections

Table 3.2 summarizes the pavement section material thicknesses encountered in our borings.

TABLE 3.2
SUMMARY OF EXISTING PAVEMENT SECTIONS

Boring ID	HMA (inches)	AB (inches)
B1	1	8
B2	1	8
B3	2	8
B4	1	11
B5	1	11

Notes:
Approximate boring locations shown on the Site Plan, Figure 2
HMA = Hot Mix Asphalt
AB = Aggregate Base

3.3 Alluvium

Below the pavement section, we encountered alluvium in each of our exploratory borings to the maximum depth explored of approximately 41½ feet. The alluvium predominantly consisted of:

- 0 to 15 feet: interbedded, medium-stiff to very stiff lean clay and loose poorly graded sand (SP and silty sand (SM). This material has high in-situ moisture content (approximately 20% to 35%) and is marginally compressible under increased loading. Laboratory Plasticity Index and Expansion Index tests on selected near-surface soil samples indicate relatively low plasticity and corresponding low expansion potential.
- 15 to 45 feet: very soft to medium stiff lean clay (CL). This material has high in-situ moisture content (approximately 30% to 40%) and is highly compressible under increased loading.
- 45 to 90 feet: interbedded layers of medium dense to very dense poorly graded sand (SP) with variable amounts of fine gravel. This layer is considered suitable for deep foundation support.

Soil conditions described in the previous paragraphs are generalized. The exploratory boring logs and CPT 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

We encountered groundwater in our exploratory borings at depths ranging from approximately 8 to 13 feet on February 7, 2018.

The online database operated by the California Department of Water Resources (DWR) includes historical depth to groundwater measurements in four area wells as summarized below:

TABLE 4.0
DWR HISTORIC DEPTH TO GROUNDWATER SUMMARY

DWR Well ID	Distance and Direction from Site	Data Period	Depth to Groundwater Range (Feet)
09N04E34K001M	1½ mile south	1968 – 2004	2 (1986) to 16.5 (1977)
09N04E32R001M	2½ mile southwest	1972 – 1992	10 (1991) to 42 (1977)
09N04E32G001M	2½ mile southwest	1962 – 1980	2 (1969) to 15 (1977)
09N04E27F001M	1½ mile northwest	1953 – 2003	9 (1995) to 50 (1953)

Based on the above information, the average depth to groundwater in the site vicinity ranges from about 5 feet to 15 feet (or deeper). Historical data suggests that groundwater depth could rise as high as 2 feet in the area during sustained high flow stages of the Sacramento River.

It should be noted that fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, and other factors. Depth to groundwater can also vary significantly due to localized pumping, irrigation practices, and seasonal fluctuations. Therefore, it is possible that groundwater may be higher or lower than the level observed during our investigation.

5.0 SEISMICITY AND GEOLOGIC HAZARDS

5.1 Regional Active Faults

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 contained within an Alquist-Priolo Earthquake Fault Zone. Therefore, we consider the potential for ground rupture due to onsite active faulting to be low. In order to determine the distance of known active faults within 50 miles of the site, we used the computer program *EQFAULT*, (Version 3, Blake, 2000). Principal references used within *EQFAULT* are Jennings (1975), Anderson (1984) and Wesnousky (1986). Results are summarized in Table 5.1.

TABLE 5.1
REGIONAL ACTIVE FAULTS

Fault Name	Distance From Site (miles)	Maximum Moment Magnitude (M_W)
Foothills Fault System	23.7	6.5
Great Valley, Segment 4	26.0	6.6
Great Valley, Segment 3	26.7	6.8
Great Valley, Segment 5	29.1	6.5
Hunting Creek - Berryessa	37.7	6.9
Concord – Green Valley	37.7	6.9
Great Valley, Segment 6	40.1	6.7
West Napa	47.3	6.5
Greenville	48.6	6.9

5.2 Ground Shaking

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 6.5 and the estimated Peak Ground Acceleration (PGA) for the Maximum Considered Earthquake (MCE) with a 2,475-year return period is 0.32g.

While listing PGA is useful for comparison of potential effects of fault activity in a region, other considerations are important in seismic design, including frequency and duration of motion and soil conditions underlying the site.

5.3 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 Stockton area. However, soil and groundwater conditions exist at the site that may be susceptible to seismic-induced liquefaction under the design-level seismic event.

The current standard of practice, as outlined in the *Recommended Procedures for Implementation of DMG Special Publication 117A, Guidelines for Analyzing and Mitigating Liquefaction in California"* (Southern California Earthquake Center, 2002) requires liquefaction analysis to a depth of 50 feet below the lowest portion of the proposed structure. 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 assumed a seasonal high groundwater elevation of 5 feet below existing grade, an earthquake moment magnitude of 6.6 (USGS deaggregation), and a MCE (2,475-year return interval event) PGA of 0.321g (PGA_M adjusted for Site Class per American Society of Civil Engineers [ASCE] 7-10, Eq. 11.8-1). We also evaluated liquefaction potential under the design-level ground motion PGA of 0.21 which is 2/3 of the MCE PGA (e.g. $PGA_M/1.5$).

Based on the results of our analyses, there is the potential for liquefaction at the site within sandy soil layers generally present between depths of approximately 10 and 15 feet and 45 to 60 feet.

Consequences of liquefaction may include ground surface settlement, ground loss (sand boils), and lateral slope displacements (lateral spreading). For liquefaction-induced sand boils or fissures to occur, pore water pressure induced within liquefied strata must exert a large enough force to break through overlying, non-liquefiable layers. Based on work by Youd and Garris (1995) which modified and advanced the work of Ishihara (1985), a capping layer of non-liquefiable soil can prevent the occurrence of sand boils or fissures. An apparently continuous, non-liquefiable, silt/clay stratum approximately 10 feet thick is present across the site. In our opinion, this layer is sufficiently thick enough to reduce the potential for ground loss due to sand boils or fissures. Because the site topography is generally flat and level and there is not an adjacent free-face or sloping geometry, lateral spreading is not anticipated.

Therefore, the likely consequence of potential liquefaction at the site is ground settlement. Dynamic settlement of the soils that experience liquefaction may occur after earthquake shaking has ceased. We estimated post-liquefaction settlement under the MCE and design level ground motions using the software CLiq following the methodology of Zhang et al (2002) which is generally based on case histories of liquefaction of hydraulic fills in the San Francisco Bay Area. The results of the analysis are summarized in Table 5.3.

TABLE 5.3
ESTIMATED LIQUEFACTION SETTLEMENT

CPT location	Estimated Total Liquefaction Settlement	
	MCE Ground Motion (PGA_M = 0.32g)[1]	**Design-Level Ground Motion (PGA_M/1.5 = 0.21g)[2]**
CPT1	1.2 inches	0.7 inches
CPT2	0.7 inches	0.2 inches
CPT3	0.8 inches	0.4 inches
Average	0.9 inches	0.4 inches

Notes: 1. MCE liquefaction settlement for structural evaluation of life safety/collapse.
2. Design-Level liquefaction settlement for structural performance evaluation

The distribution of dynamic settlement associated with potential liquefaction is generally less than ¼ inch within each liquefiable layer. This minor amount of potential liquefaction and associated settlement is not significant enough to require site response analysis, substantially reduce pile axial capacity, or induce significant negative skin friction. In our experience, design-level ground motion liquefaction settlements of this magnitude (approximately ½ inch) typically do not require special foundation design or modifications.

5.4 Expansive Soil

Laboratory Plasticity Index and Expansion Index tests on selected near-surface soil samples indicate relatively low plasticity and corresponding low expansion potential. Mitigation and specific design measures with respect to expansive soil is not necessary.

5.5 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

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

6.1.2 Based on our findings, evaluation, and analyses to date, we have identified the following geotechnical constraints for the site:

- Shallow groundwater – Seasonally as shallow as 5 feet (or shallower). This may impact grading and underground utility construction
- Soft, compressible soil between depths of 15 to 45 feet. These soils prohibit the use of conventional shallow foundations due to excessive post-construction settlement.
- Potentially liquefiable sand between 10 and 15 feet and 45 to 60 feet. If liquefaction were to occur, there is a low potential for ground loss (sand boils) and we anticipate only minor post-liquefaction settlements which should not adversely impact foundation performance.

These conditions impact design and construction of the project. Discussion of these geotechnical constraints and specific mitigation, design, and construction recommendations are provided herein.

6.1.3 Given the subsurface conditions and structural loading demands, the building and pedestrian bridge/stair structure may be supported on deep foundations or shallow foundations in conjunction with ground improvement measures, such as drilled displacement columns (DDCs). Specific recommendations are provided in this report.

6.1.4 Conclusions and recommendations provided in this report are based on our review of referenced literature, analysis of data obtained from our exploratory field exploration program, laboratory testing program, and our understanding of the proposed development at this time.

6.1.5 We should review the project plans as they develop further, provide engineering consultation as needed during final design, and perform geotechnical observation and testing services during construction.

6.2 Seismic Site Class / Seismic Design Criteria

6.2.1 We understand that seismic design of the structures will 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 American Society of Civil Engineers (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 5.3 of this report, there is a potential for liquefaction at the site; however, there is a low potential for failure or collapse (sand boils/ground loss) due to liquefaction, 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 measurements performed in conjunction with CPT1. The calculated average shear wave velocity (per ASCE 7-10, Eq. 20.4-1) is approximately 637 feet/sec. Therefore, the site is classified as Site Class "D" per Table 20.3-1 of ASCE 7-10.

6.2.3 We used the USGS web application *US Seismic Design Maps* (http://geohazards.usgs.gov/designmaps/us/ application.php) to evaluate site-specific seismic design parameters in accordance with the 2016 CBC/ASCE 7-10. 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	2013 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	0.699g	Figure 1613.3.1(1) / Figure 22-1
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.299g	Figure 1613.3.1(2) / Figure 22-2
Site Coefficient, F_A	1.241	Table 1613.3.3(1) / Table 11.4-1
Site Coefficient, F_V	1.801	Table 1613.3.3(2) / Table 11.4-2
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	0.867g	Eq. 16-37 / Eq. 11.4-1
Site Class Modified MCE_R Spectral Response Acceleration (1 sec), S_{M1}	0.539g	Eq. 16-38 / Eq. 11.4-2
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.578g	Eq. 16-39 / Eq. 11.4-3
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.359g	Eq. 16-40 / Eq. 11.4-4

6.2.4 Table 6.2.4 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 6.2.4
2016 CBC SITE ACCELERATION DESIGN PARAMETERS

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.239g	Figure 22-7
Site Coefficient, F_{PGA}	1.321	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.316g	Section 11.8.3 (Eq. 11.8-1)

6.2.5 Conformance to the criteria presented in Tables 6.2.3 and 6.2.4 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 Grading and excavations at the site may be accomplished with standard effort using heavy-duty grading/excavation equipment. We do not anticipate grading and excavations to generate cobbles or boulders that would require special handling or placement.

6.3.2 Temporary excavations deeper than 4 feet and entered by workers must meet Cal-OSHA requirements as appropriate. Excavation sloping, benching, the use of trench 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.

6.3.3 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 sloping to prevent damage to any structure or improvements near underground excavations.

6.3.4 Permanent cut and fill slopes 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.

6.3.5 If grading occurs during or after the wet season (typically winter and spring), or in periods of precipitation, in-place and excavated soils will likely be wet. Earthwork contractors should be aware of moisture sensitivity of clayey and fine-grained soils and potential compaction/workability difficulties.

6.3.6 Earthwork and pad preparation operations in these conditions will likely be difficult with low productivity. Often, a period of at least one month of warm and dry weather is necessary to allow the site to dry sufficiently so that heavy grading equipment can operate effectively. Conversely, during dry summer and fall months, dry clay soils may require additional grading effort (discing, mixing, or other means) to attain proper moisture conditioning.

6.3.7 Based on laboratory testing, in-situ moisture content of site soils range from about 20% to 35%, which is significantly higher than optimum moisture content for this type of material. In addition, soil below existing pavements are often wet when exposed. Due to the fine-grained nature of the soils and measured in-situ moisture contents well above optimum, significant soil instability should be expected. Additional drying effort to attain moisture contents suitable for compaction should be anticipated regardless of the time of year. Mitigation alternatives may include aerating/drying the exposed soils (assuming favorable weather conditions), overexcavating 12 to 18 inches and placing geotextile fabric/geogrid covered with aggregate, or chemical treatment (e.g. lime treatment). We can provide specific recommendations during construction based on conditions encountered.

6.4 Materials for Fill

6.4.1 Excavated soils generated from cut operations at the site are suitable for use as fill in structural areas provided they do not contain deleterious matter, organic material, or cementations larger than 6 inches in maximum dimension. Due to high in-situ moisture content, native soils reused as engineered fill will likely require aerating/drying to attain suitable moisture content for compaction, regardless of the time of year.

6.4.2 Import fill material should be primarily granular with a “very low” expansion potential (Expansion Index less than 20), a Plasticity Index less than 15, be free of organic material and construction debris, and not contain rock/cementations larger than 6 inches in greatest dimension. Import soil should also contain a sufficient amount of fines (generally more than 10%) to provide “binder” and reduce potential caving when excavated.

6.4.3 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.

6.5 Grading

6.5.1 All earthwork operations should be observed and all fills tested for recommended compaction and moisture content by a representative of Geocon.

6.5.2 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.

6.5.3 Prior to commencing grading, 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.

6.5.4 Site preparation should begin with removal of existing pavements, surface/subsurface structures, underground utilities, debris, and existing fill. Existing pipelines and overlying trench backfill should be completely removed to expose undisturbed soil. Excavations or depressions resulting from site clearing operations, or other existing excavations or depressions, should be restored with engineered fill in accordance with the recommendations of this report.

6.5.5 Within areas to be developed, any existing trees and associated root systems should be removed. Roots larger than 1 inch in diameter should be completely removed. Smaller roots may be left in-place as conditions warrant and at the discretion of our field representative.

6.5.6 Cut areas, areas left at grade, and areas to receive fill should be scarified at least 12 inches, uniformly moisture-conditioned at or above 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.

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

6.5.8 The top 6 inches of final vehicular pavement (non-pervious) subgrade, whether completed at-grade, by excavation, or by filling, should be uniformly moisture-conditioned at or above 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 aggregate base (AB).

6.5.9 Pipe bedding, shading, and trench backfill should conform to the requirements of the appropriate utility authority. Material excavated from trenches should be adequate for use as general backfill above shading provided it does not contain deleterious matter, vegetation, or cementations larger than 6 inches in maximum dimension. Trench backfill should be placed in loose lifts not exceeding 8 inches, moisture-conditioned at or above optimum and compacted to at least 90% relative compaction. Compaction should be performed by mechanical means only; jetting of trench backfill is not recommended.

6.6 Foundations

Due to the presence of compressible soil within the top 45 feet and the magnitude of anticipated structural loading, the use of shallow foundations for support of the new buildings is not feasible due to excessive post-construction settlement. However, shallow foundations may be used if ground improvement is performed.

We consulted with Farrell Design Build, a geotechnical specialty contractor, to evaluate feasible ground improvement alternatives for the project. Given the significant thickness and fine-grained nature of compressible soil at the site, ground improvement measures such as rammed aggregate piers (RAPs) and vibro-replacement stone columns are not considered to be effective in significantly increasing allowable bearing capacity or reducing post-construction settlements. In addition, technologies such as deep dynamic compaction or chemical grouting are also not considered effective. Based on the subsurface conditions at the site, rigid inclusions, such as drilled displacement columns (DDCs) may provide suitable support for shallow foundations, increase bearing capacity, and reduce post-construction settlements.

If ground improvement is not performed, we recommend using deep foundations consisting of drilled displacement piles for support of the building and pedestrian bridge/stair structure. Recommendations are provided herein.

6.6.1 Ground Improvement – Drilled Displacement Columns

6.6.1.1 Based on our consultation with Farrell Design Build, we anticipate that conventional shallow foundations with spread and continuous footings and floor slab supported on ground improvement can be used. Given the subsurface conditions at the site and anticipated

structural loading, Drill Displacement Column™ (DDC) technology is a suitable form of ground improvement. For DDC, concrete grout is injected under pressure through the displacement drill for discreet ground improvement to install "composite ground" that improves compressible soil, reduces liquefaction potential, and increases the composite allowable bearing pressure for foundation. This method of ground improvement is comparable to large diameter compaction grouting. Typically, the ground improvement is composed of relatively closely-spaced DDC below foundation elements and a wider spacings below the slab section. The drill displacement tool displaces a 16 to 18-inch diameter hole and the finished diameter of the pressure grouted DDC is up to 19 to 21 inches. At this site, we anticipate that the DDC elements would be on the order of 45 to 50 feet long.

6.6.1.2 For preliminary design purposes, DDC elements can be installed in a grid pattern at a spacing of approximately 4 to 6 feet on centers under footings. DDC elements at a spacing of approximately 7 to 12 feet on centers can be used to support the slabs, the spacing will depend on the foundation loading and structural slab design (thickness and reinforcement).

6.6.1.3 Footings supported on DDC-reinforced soils can be designed for an allowable bearing capacity of 5,000 pounds per square foot (psf). The bearing capacity should be verified by a full-scale load test. Footings can be designed to resist lateral loads using an allowable coefficient of friction of 0.40. Lateral sliding resistance is derived at the concrete/aggregate interface below the footing. A 1/3 increase to the allowable bearing capacity and frictional resistance is permitted for short-term seismic and wind loads.

6.6.1.4 Final foundation system and design parameters will be developed in conjunction with the project structural engineer during the ground improvement design-build activities. If the DDC system is selected for structural support, the DDC specialty contractor would provide a complete design-build submittal with design recommendations, plans and specifications stamped and signed by a California registered Professional Engineer. Geocon will need to perform a geotechnical review of the DDC design.

6.6.1.5 In addition to plan review, we recommend geotechnical oversight during DDC construction. Our Quality Assurance (QA) services will supplement the contractor internal Quality Control (QC) program. The contractor QC program should document each DDC element installed. Together the QA/QC program will monitor drill depths, shaft length, grout volume, installation pressures and installation procedures.

6.6.2 Deep Foundations

6.6.2.1 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.

6.6.2.2 Due to the density variations in the soil at the site, the use of fixed-length impact-driven piles can be problematic due to variable embedment depths, 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.

6.6.2.3 Based on the subsurface conditions at the site, we recommend using auger pressure-grout displacement (APGD) piles (also known as drilled displacement piles) or continuous flight auger (CFA) piles for support of the structure. A brief discussion of each pile type follows:

- **Auger Cast Pressure-Grout Displacement (APGD) Piles** are installed using a specialized, plugged auger tool with a displacement body that laterally displaces the soil as it 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. After the auger is removed, the required steel reinforcement is then "wet-set" into the pile to complete the installation. This pile type produces few spoils, approximately 20% or less of the theoretical hole volume.

- **Continuous Flight Auger (CFA) Piles** are similar to APGD piles but are not displacement piles. These 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.

6.6.2.4 From a geotechnical perspective, site soils are well-suited for APGD and CFA piles. CFA and APGD 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. For this site, difficult soil conditions include soft soils on the order of 45 feet thick overlying the medium dense to dense sand bearing layer which will need to be penetrated some distance in order to develop pile capacity. **We strongly recommend performing a comprehensive pile installation**

and load testing program to evaluate constructability as well as capacity. Recommendations are provided in herein.

6.6.2.5 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.

6.6.2.6 Deep foundation systems will generate the majority of their vertical load-carrying capacity from skin friction and some end-bearing in the dense sand and gravel and underlying layers located 45 feet and deeper at the site.

6.6.2.7 Table 6.6.2A presents estimated pile lengths and estimated axial capacities for 18-inch and 24-inch CFA and APGD piles. We note that other pile diameters (such as 16-inch) may also be used, although capacities will be less and longer pile lengths would be needed. **Pile lengths presented in Table 6.6.2 are <u>estimates only</u> based on empirical design procedures. 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 the 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 herein.

TABLE 6.6.2A
AXIAL CAPACITY – ESTIMATED PILE LENGTHS

Pile Type	Estimated Pile Length (feet)	Estimated Allowable Axial Capacity (kips)[1]	Estimated Ultimate Axial Capacity (kips)
18-inch CFA[2] Piles	65	300	600
18-inch APGD[3] Piles	60	300	600
24-inch CFA[2] Piles	55	300	600
24-inch APGD[3] Piles	50	300	600

<u>Notes:</u>
1. *Dead + Live Loading Conditions.*
2. *Continuous Flight Auger Pile (non-displacement)*
3. *Auger Pressure Grout Displacement Pile*

6.6.2.8 Single pile uplift (tension) capacity can be estimated as approximately 50 percent of the allowable downward capacity. Tension piles should be properly reinforced to transfer uplift forces to the pile tip.

6.6.2.9 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.

6.6.2.10 Lateral analysis of the deep foundations may be performed by the pile designer using the computer program LPILE or similar lateral analysis software. Table 6.6.2B summarizes our recommended soil parameters for use in LPILE.

TABLE 6.6.2B
RECOMMENDED SOIL PARAMETERS FOR LPILE ANALYSIS

Soil Layer	Soil Model	Elevation		Effective Unit Weight (pcf)	Undrained Cohesion (psf)	Friction Angle (deg)
		Top	Bottom			
1	Stiff Clay above Groundwater	+15	+10	115	1,000	0
2	Stiff Clay below Groundwater	+10	-25	55	1,000	0
3	Sand	-25	>-25	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

6.6.2.11 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 250 pounds per cubic foot (pcf) equivalent fluid pressure may be used. Frictional resistance along the bottom of the pile caps and grade beams should be neglected.

6.6.2.12 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.

6.6.2.13 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.

6.6.2.14 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.

6.6.2.15 Prior to performing static load testing of the CFA/APGD piles, we recommend performing low-strain impact integrity testing (Pile Integrity Testing or "PIT") in accordance with ASTM D5882. The purpose of the PIT 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 PIT results. As a quality assurance measure during construction, PIT should be performed on 10% of the production piles. The frequency of PIT may be increased by the geotechnical engineer if installation difficulties are encountered. If the results of PIT indicate questionable pile properties, additional high-strain dynamic testing (ASTM D4945) may be required to verify production pile capacity.

6.6.2.16 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, depth into competent material, grout volumes/pressures (for the CFA/APGD piles), and a statement that the foundations can accept the design loading conditions.

6.7 Interior Slabs-on-Grade

6.7.1 Slab thickness and reinforcement should be determined by the structural engineer based on anticipated loading. However, based on our experience, slabs are typically at least 5 inches thick and reinforced with at least No. 4 reinforcing bars placed 18 inches on center, each way. Control joints should be provided at periodic intervals in accordance with American Concrete Institute (ACI) or Portland Cement Association (PCA) recommendations, as appropriate.

6.72 If building pad soils become dry, they should be re-moistened prior to concrete slab-on-grade construction. Building pads should be moistened to at least optimum moisture content at least 48 hours before placing the vapor barrier. Moisture content should be verified by Geocon prior to placing the vapor barrier.

6.8 Concrete Moisture Protection Considerations

6.8.1 Migration of moisture through concrete slabs 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.

6.8.2 In areas where floor coverings are planned, 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.

6.8.3 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.

6.8.4 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.

6.8.5 Proper finishing, curing, and moisture vapor emission testing should be performed in accordance with the latest guidelines provided by the American Concrete Institute, Portland Cement Association, and ASTM.

6.9 Retaining Walls

6.9.1 Design of retaining walls and buried structures may be based on the lateral earth pressures (equivalent fluid pressure) summarized in Table 6.9.

TABLE 6.9
RECOMMENDED LATERAL EARTH PRESSURES

Condition	Equivalent Fluid Density
Active	40 pcf
At-Rest	55 pcf
Passive	250 pcf
Seismic Earth Pressure[1]	10 pcf

1. *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.*

6.9.2 Unrestrained walls should be designed using the active case. Unrestrained walls are those that are allowed to rotate more than 0.001H (where H is the height of the wall). Walls restrained from movement (such as basement walls) should be designed using the at-rest case. 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.

6.9.3 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.

6.10 Concrete Sidewalks and Flatwork

6.10.1 Sidewalk, curb, and gutter within City of West Sacramento right-of-way should be designed and constructed in accordance with the latest City standards and details as applicable.

6.10.2 Onsite exterior concrete flatwork will likely experience seasonal movement. Therefore, some cracking and/or vertical offset should be anticipated. We are providing the following recommendations to reduce distress to concrete flatwork. Recommendations include moisture conditioning subgrade soils, using aggregate underlayment, and providing adequate construction and control joints. It should be noted that even with implementation of these measures, slab movement or cracking could still occur.

- Concrete flatwork and sidewalks should be at least 4 inches thick and underlain by at least 4 inches of Class 2 AB compacted to at least 95% relative compaction. In addition, doweling could be provided at joints to reduce the potential for vertical offset.
- The upper 12 inches of subgrade soil for exterior flatwork and sidewalks should be uniformly moisture-conditioned at or above optimum moisture content and compacted to at least 90% relative compaction prior to placing AB.
- We recommend using construction and control joints in accordance with ACI and/or PCA guidelines. Construction joints that abut building foundations should include a felt strip, or approved equivalent, that extends the full depth of the exterior slab. Exterior slabs should be structurally independent of building foundations except at doorways, where vertical movement could impact doorway operation.

6.11 Hot Mix Asphalt Pavement

6.11.1 We performed Resistance-Value (R-Value) testing on a representative bulk soil sample from proposed at-grade pavement areas. Our testing resulted in an R-Value of 5 (Appendix B). Table 6.11 provides alternative pavement sections based on the design methods of Caltrans' *Highway Design Manual* using a design subgrade R-value of 5.

TABLE 6.11
FLEXIBLE PAVEMENT SECTIONS

	Parking Areas Traffic Index = 5.0	Driveways, Light Truck Traffic, Fire Truck Areas Traffic Index = 6.0
HMA, inches	3.0	3.5
AB, inches	10.0	13.0
Total Section, inches	13.0	16.5

6.11.2 The recommended pavement section is based on the following assumptions:

1. Pavement subgrade soil has an R-Value of at least 5.
2. Class 2 AB has a minimum R-Value of 78 and meets the requirements of Section 26 of Caltrans' *Standard Specifications*.
3. Class 2 AB and the top 6 inches of subgrade are compacted to 95% or higher relative compaction at or near optimum moisture content.

6.11.3 To reduce the potential for water from landscaped areas migrating under pavement into the AB, consideration should be given to using full-depth curbs in areas where pavement abuts irrigated landscaping. The full-depth curbs should extend at least 6 inches or more into the soil subgrade beneath the AB. Alternatively, modified drop-inlets that contain weep-holes may be used to encourage accumulated water to drain from beneath the pavement.

6.11.4 Asphalt pavement section recommendations for driveways and parking areas are based on the design procedures of Caltrans' *Highway Design Manual* (Design Manual), Chapter 600, updated December 20, 2004. It should be noted that most rational pavement design procedures are based

on projected street or highway traffic conditions and, hence, may not be representative of vehicular loading that occurs in parking lots and driveways. Pavement proximity to landscape irrigation, reduced traffic speed and short turning radii increase the potential for pavement distress to occur in parking lots even though the volume of traffic is significantly less than that of an adjacent street. The Design Manual indicates that the resulting pavement sections for parking lots are "minimized to keep initial costs down but are reasonable because additional AC surfacing can be added later, if needed, and generally without incurring traffic hazards or traffic handling problems." It is generally not economically feasible to design and construct the entire parking lot and driveways for the unique loading conditions previously described. Periodic maintenance of the pavement in these areas, therefore, should be anticipated.

6.12 Rigid Concrete Pavement

6.12.1 If rigid PCC pavement is used in automobile and light-truck traffic areas, we recommend that the concrete be at least 6 inches thick. PCC pavement should be underlain by at least 6 inches of Class 2 AB meeting the requirements of Section 26 of Caltrans' *Standard Specifications* and compacted to at least 95% relative compaction. Subgrade soils should be prepared and compacted in accordance with the recommendations of this report.

6.12.2 PCC should have a minimum 28-day compressive strength of 3,500 pounds per square inch (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.

6.12.3 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.

6.13 Site Drainage and Moisture Protection

6.13.1 Adequate 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.

6.13.2 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.

6.13.3 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 use of area drains to collect excess irrigation water and transmit it to drainage structures or impervious above-grade planter boxes. In addition, where landscaping is planned adjacent to the pavement or flatwork, we recommend construction of a cutoff wall (deepened curb) along the edge of the pavement/flatwork that extends at least 4 inches into the soil subgrade below the bottom of the base material.

6.13.4 The soil conditions at the site (low-permeability clays) are not conducive to water infiltration devices such as vegetated swales. However, Low Impact Development (LID) devices can be installed to reduce velocity and the amount of water entering the storm drain system. The LID devices should be properly constructed to prevent water infiltration into the surrounding soil. Water infiltrates the expansive soils, distress may be caused to adjacent pavements, flatwork, or structures. Vegetated swales and basin areas (if used) should be lined with an impermeable liner (e.g. high-density polyethylene, HDPE, with a thickness of about 12 mil or equivalent polyvinyl chloride liner) to reduce infiltration.

6.13.5 We recommend that roof drains be connected to water-tight subdrains that direct the water to the storm drain system. However, we understand that LID and Leadership in Engineering and Environmental Design (LEED) requests disconnecting the roof drains to help obtain certification. 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.

6.13.6 We recommend implementing measures to reduce infiltrating irrigation water near buildings, flatwork, or pavements. Such measures may include:

- Selecting drought-tolerant plants that require little or no irrigation, especially within 3 feet of buildings, slabs-on-grade, or pavements.
- Using drip irrigation or low-output sprinklers.
- Using automatic timers for irrigation systems.
- Using appropriately spaced area drains.

The project landscape architect should consider incorporating these measures into the landscaping plans.

6.13.7 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 Geocon should review the foundation and grading plans 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 (GER) throughout the construction phase and provide 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 or 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.

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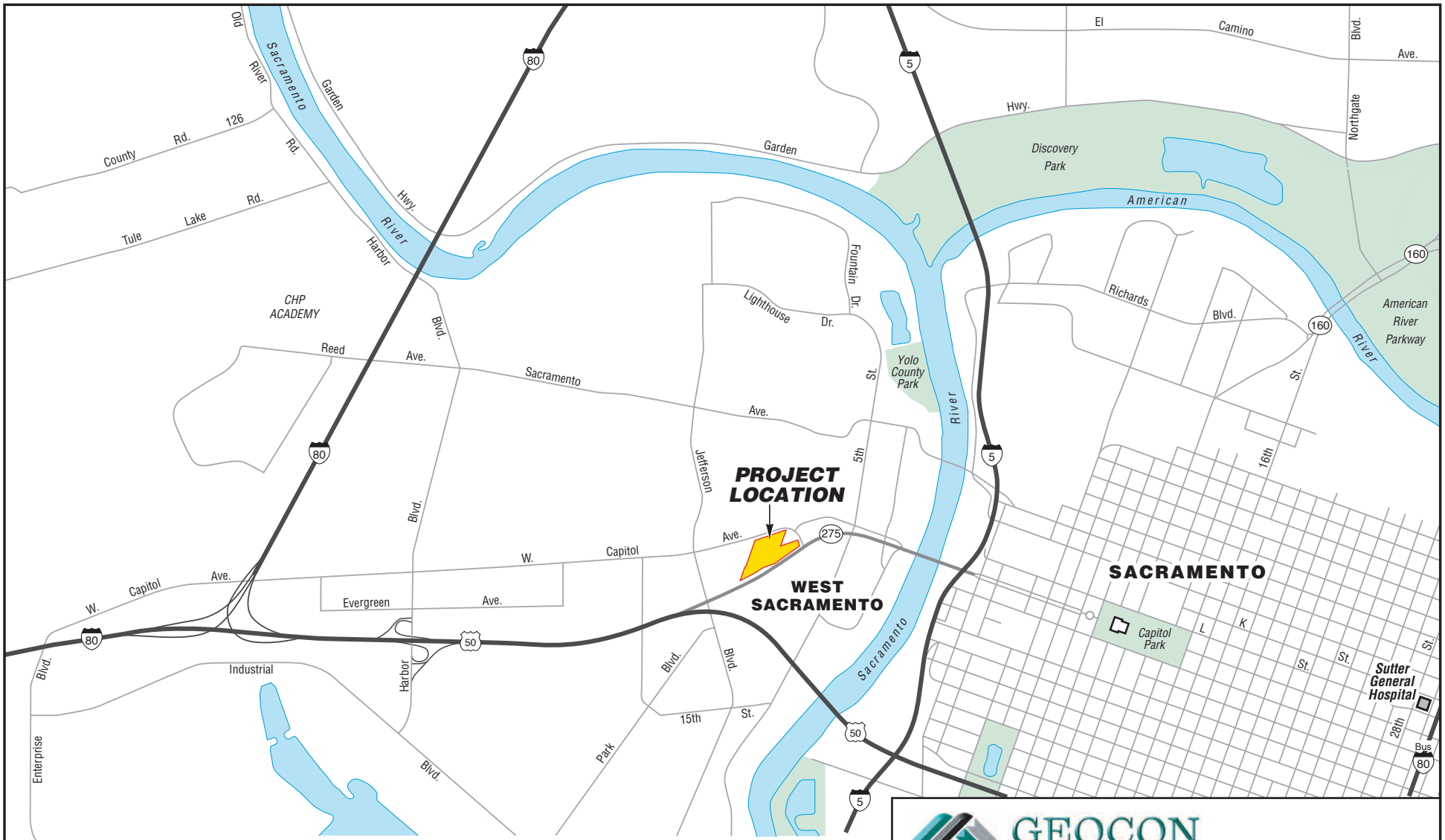

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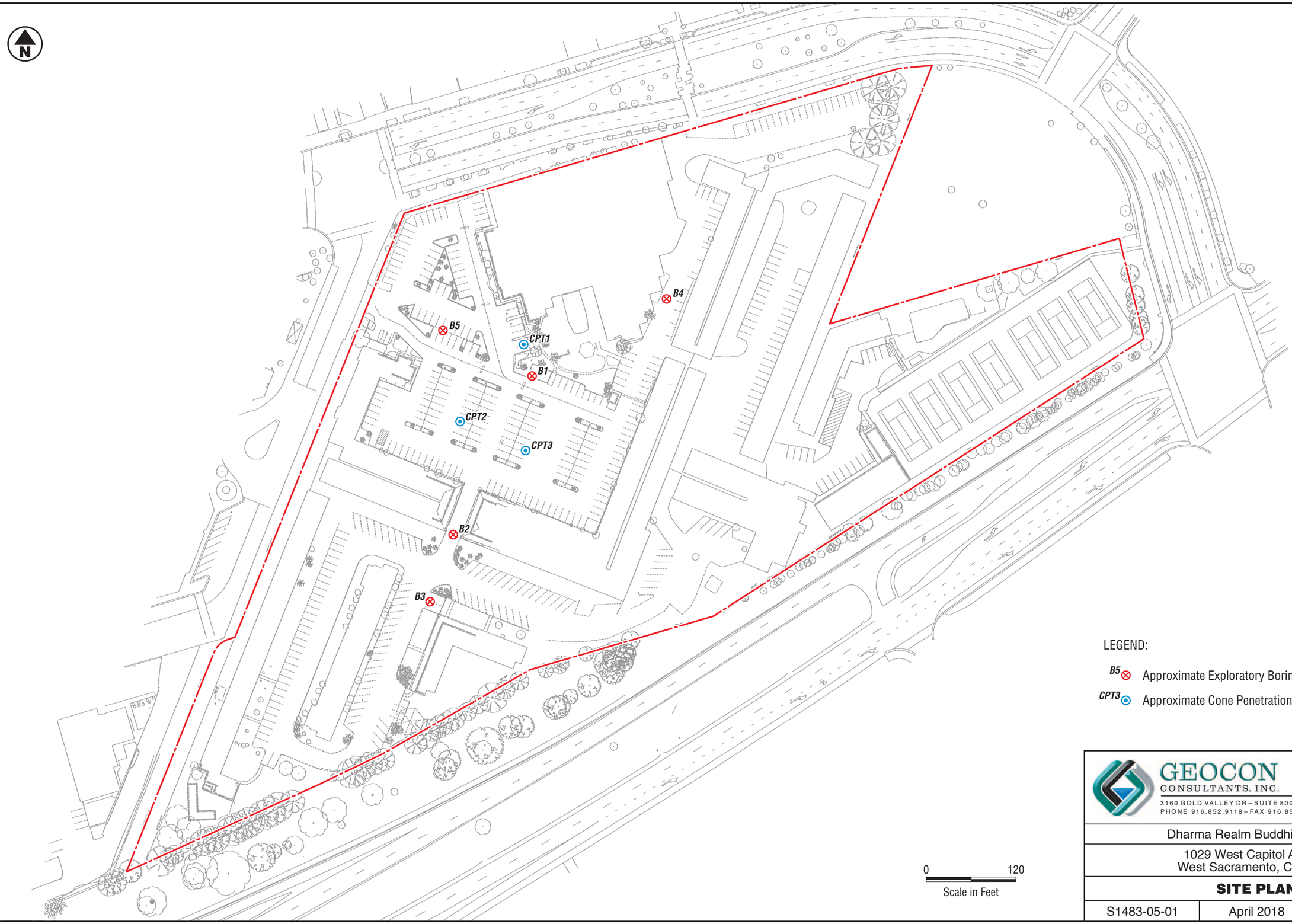
VICINITY MAP

S1483-05-01	April 2018	Figure 1



LEGEND:

B5 ⊗ Approximate Exploratory Boring Location

CPT3 ⊙ Approximate Cone Penetration Test (CPT) Location

0 120

Scale in Feet

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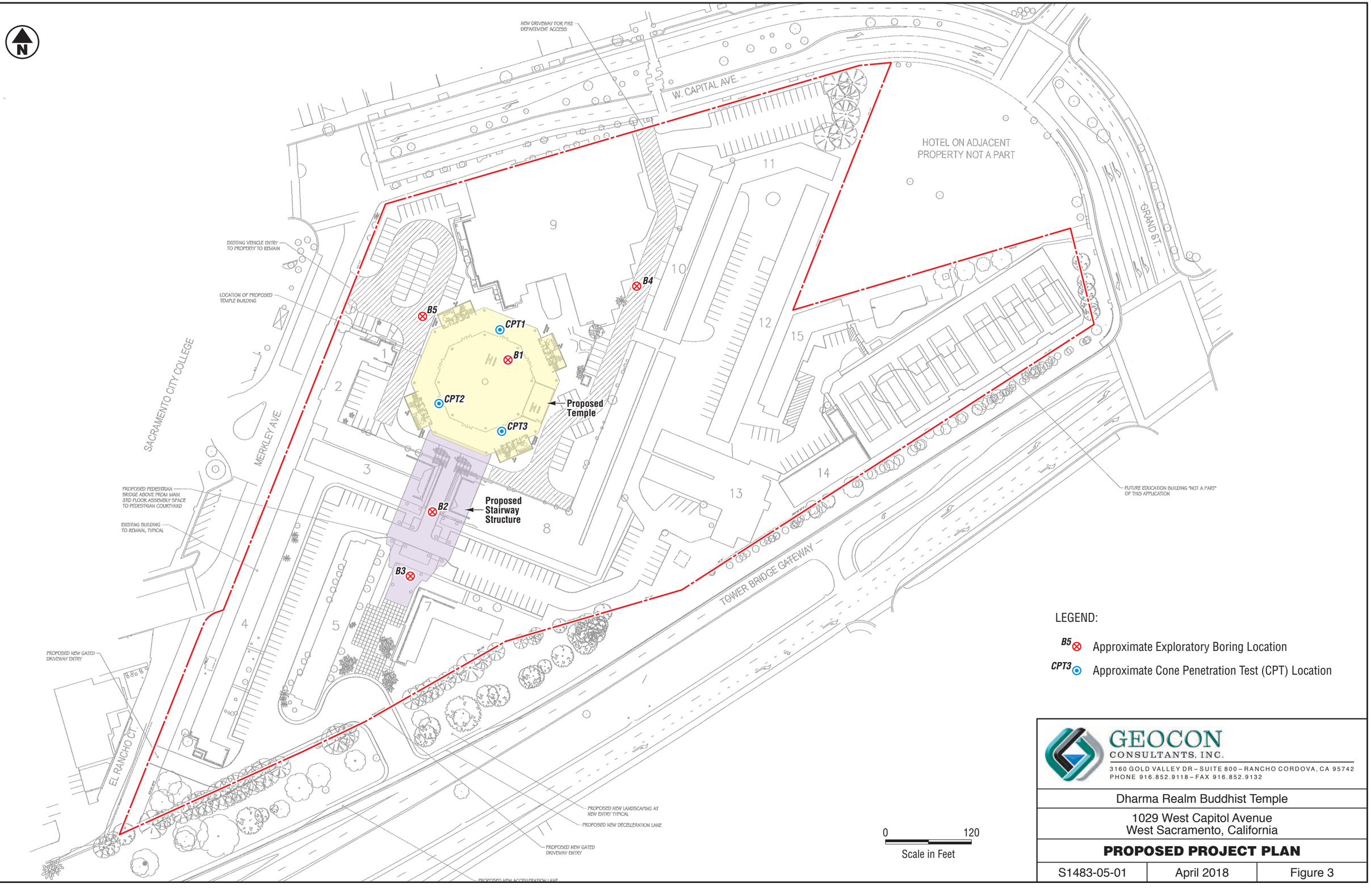
SITE PLAN

S1483-05-01	April 2018	Figure 2



0 120

Scale in Feet

LEGEND:

B5 ⊗ Approximate Exploratory Boring Location

CPT3 ⊙ Approximate Cone Penetration Test (CPT) Location

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PROPOSED PROJECT PLAN

S1483-05-01	April 2018	Figure 3



1 WEST ELEVATION - TEMPLE

NORTH

0 5 10 20 40 FT
SCALE: 1/16" = 1'-0"



2 EAST ELEVATION - TEMPLE

NORTH

0 5 10 20 40 FT.
SCALE: 1/16" = 1'-0"

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PROPOSED PROJECT ELEVATION

S1483-05-01	April 2018	Figure 4



Photo No. 1 Current site conditions



Photo No. 2 Current site conditions

PHOTOS NO. 1 & 2

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Dharma Realm Buddhist Temple

1029 West Capitol Avenue
West Sacramento, California

S1483-05-01	April 2018	

APPENDIX A

APPENDIX A

FIELD EXPLORATION

We performed our geotechnical field exploration on February 1 and 7, 2018. Our field exploration program consisted of drilling five exploratory borings (B1 through B9) and advancing three cone penetration soundings (CPT1 through CPT3) at the approximate locations depicted on the Site Plan, Figure 2 and Proposed Project Plan, Figure 3.

Borings were performed using a track-mounted CME 75 drill rig equipped with 8-inch outside diameter (OD) hollow stem augers. Soil sampling was performed using an automatic 140-pound hammer with a 30-inch drop. We obtained samples using a 3-inch OD split-spoon (California Modified) sampler. We recorded the number of blows required to drive the sampler the last 12 inches (or portion thereof) of the 18-inch sampling interval on the boring logs. Upon completion, the borings were backfilled with either neat cement grout (boring B1) or soil cuttings generated from the borings (borings B2 through B7).

The CPTs were performed using a 20-ton truck-mounted rig. CPT parameters including tip resistance (q_c), sleeve friction (f_s) and dynamic pore pressure (U) were measured at approximately 2-inch intervals as the cone advanced. Soil behavior types were determined using correlations developed by Robertson and Campanella (1988). The cone was equipped with a seismic transducer and shear wave velocity measurements were also collected at approximate 5-foot intervals in CPT1.

We visually examined, classified, and logged the subsurface conditions in the exploratory borings 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 we obtained samples. 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, drill rig penetration rates, excavation characteristics, and other factors. The transition between materials may be abrupt or gradual. Where applicable, we revised the field logs based on subsequent laboratory testing.

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≤S<50	DAMP
INDICATION OF MOISTURE; NO VISIBLE WATER	50≤S<75	MOIST
MINOR VISIBLE FREE WATER	75≤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

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 ⅝-INCH TO 1 FOOT	MODERATELY BEDDED
1 ⅛-INCH TO 3 ⅝-INCH	THINLY BEDDED
⅝-INCH TO 1 ⅛-INCH	VERY THINLY BEDDED
LESS THAN ⅝-INCH	LAMINATED

STRUCTURE DESCRIPTIONS

CRITERIA	DESCRIPTION
ALTERNATING LAYERS OF VARYING MATERIAL OR COLOR WITH LAYERS AT LEAST ⅛-INCH THICK	STRATIFIED
ALTERNATING LAYERS OF VARYING MATERIAL OR COLOR WITH LAYERS LESS THAN ⅛-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
⅛-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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KEY TO LOGS Figure 3

PROJECT NO. **S1483-05-01** PROJECT NAME **Dharma Realm Buddhist Temple**

BORING B1

ELEV. (MSL.) **N/A** DATE COMPLETED **2/7/2018**

ENG./GEO. **Sean Dixon** DRILLER **All Well Abandonment**

EQUIPMENT **CME 75 LAR** 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			**ASPHALT CONCRETE (AC)** 1 inch				
1	B1-Bulk	CL	**AGGREGATE BASE (AB)** 8 inches				PI, CP, CR
2			**ALLUVIUM** Very stiff, moist, gray, Lean CLAY				
3	B1-3.0		- PP = 3.0 tsf				
4	B1-3.5			9	98.9	22.9	
5							
6	B1-5.5 B1-6.0		- light brown - PP = 3.5 tsf	16			CU
7		SP	Loose, wet, gray, Poorly graded SAND, some Silt, fine grained				
8	B1-8.0						
9	B1-8.5			10	95.0	32.2	
10							
11	B1-10.5 B1-10.5			10			
12		CL	Medium stiff, wet, gray, Lean CLAY				
13	B1-13.0				89.6	35.2	
14	B1-13.5			9			
15		SM	Loose, wet, gray, Silty SAND				
16	B1-15.5 B1-16.0		- no recovery	3			
17							
18							
19							
20	B1-20.0			2			
21		CL	Stiff, wet, gray, Lean CLAY				
22							
23							
24							
25	B1-25.0		- PP = 1.75 tsf	4			
26							
27							
28							
29							

Figure A2, Log of Boring, page 1 of 2

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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. **S1483-05-01** PROJECT NAME **Dharma Realm Buddhist Temple**

BORING B1

ELEV. (MSL.) **N/A** DATE COMPLETED **2/7/2018**

ENG./GEO. **Sean Dixon** DRILLER **All Well Abandonment**

EQUIPMENT **CME 75 LAR** 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
30	B1-30.0			CL	- soft to medium stiff, PP = 0.5 tsf	7			PI
31									
32									
33									
34									
35	B1-35.0				- very soft	2			
36									
37									
38									
39									
40	B1-40.0				- medium stiff	6			
41									
					BORING TERMINATED AT 41.5 FEET GROUNDWATER ENCOUNTERED AT 8 FEET BACKFILLED WITH SOIL CUTTINGS				

Figure A3, Log of Boring, page 2 of 2

IN PROGRESS S1483-05-01 DHARMA REALM BUDDHIST TEMPLE.GPJ 03/28/18

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. **S1483-05-01** PROJECT NAME **Dharma Realm Buddhist Temple**

BORING B2

ELEV. (MSL.) **N/A** DATE COMPLETED **2/7/2018**

ENG./GEO. **Sean Dixon** DRILLER **All Well Abandonment**

EQUIPMENT **CME 75 LAR** 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			**ASPHALT CONCRETE (AC)** 1 inch				
1	B2-Bulk	SM	**AGGREGATE BASE (AB)** 8 inches				CP, R
2			**ALLUVIUM**				
3	B2-3.0		Loose, moist, brown, Silty SAND, fine grained				
4	B2-3.5			11	90.9	21.6	
5		CL	Very stiff, moist, light brown, Sandy lean CLAY				
6	B2-5.5 B2-6.0		- PP = 2.5 tsf	13	103.8	22.5	PI
7							
8	B2-8.0						
9	B2-8.5		- stiff, PP = 1.5 tsf	3			CU
10	NR			0			
11							
12		SM	Loose, wet, gray, Silty SAND				
13							
14							
15	B2-15.0			7			GSA
16							
17							
18							
19		SP	Medium dense, wet, gray, Poorly graded SAND, medium grained				
20							
21	B2-20.5 B2-21.0			17	101.4	24.4	
22							
23		CL	Stiff, moist, gray, Lean CLAY				
24							
25							
26	B2-25.5 B2-26.0		- PP = 1.25 tsf	12			
			BORING TERMINATED AT 26.5 FEET GROUNDWATER ENCOUNTERED AT 13 FEET BACKFILLED WITH SOIL CUTTINGS				

Figure A4, Log of Boring, page 1 of 1

IN PROGRESS S1483-05-01 DHARMA REALM BUDDHIST TEMPLE.GPJ 03/28/18

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. **S1483-05-01** PROJECT NAME **Dharma Realm Buddhist Temple**

BORING B3

ELEV. (MSL.) **N/A** DATE COMPLETED **2/7/2018**

ENG./GEO. **Sean Dixon** DRILLER **All Well Abandonment**

EQUIPMENT **CME 75 LAR** 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					**ASPHALT CONCRETE (AC)** 2 inches				
1	B4-Bulk			CL	**AGGREGATE BASE (AB)** 8 inches				CP
2					**ALLUVIUM**				
3	B3-3.0				Stiff, moist, gray, Lean CLAY, fine grained				
4	B3-3.5					15			
5									
6	B3-5.5 B3-6.0				- very stiff, PP = 4.0 tsf	20	94.7	24.9	PI
7									
8	B3-8.0				- stiff, PP = 1.5 tsf				
9	B3-8.5					14	96.8	27.3	
10									
11	B3-10.5 B3-11.0		▼		- very soft, wet, Sandy, PP = 0.0 tsf	7			
12									
13									
14									
15									
16	B3-15.5 B3-16.0				- without Sand - PP = 0.0 tsf	2			
17									
18									
19									
20									
21	B3-20.5 B3-21.0				- 2 feet heave - PP = 0.0 tsf	2			CU
22									
23									
24									
25									
26	B3-25.5 B3-26.0				- stiff to very stiff, PP = 2.0 tsf	6			
					BORING TERMINATED AT 26.5 FEET GROUNDWATER ENCOUNTERED AT 11 FEET BACKFILLED WITH SOIL CUTTINGS				

Figure A5, Log of Boring, page 1 of 1

IN PROGRESS S1483-05-01 DHARMA REALM BUDDHIST TEMPLE.GPJ 03/28/18

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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. **S1483-05-01** PROJECT NAME **Dharma Realm Buddhist Temple**

BORING B4

ELEV. (MSL.) **N/A** DATE COMPLETED **2/7/2018**

ENG./GEO. **Sean Dixon** DRILLER **All Well Abandonment**

EQUIPMENT **CME 75 LAR** 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					**ASPHALT CONCRETE (AC)** 1 inch				
1					**AGGREGATE BASE (AB)** 11 inches				
2	B4-1.5 B4-2.0 B4-2.5			SM	**ALLUVIUM** Medium dense, moist, light brown, Silty SAND, fine grained	17	86.2	31.6	
3				CL		5			
4					Medium stiff, moist, gray, Lean CLAY				
					BORING TERMINATED AT 4.0 FEET GROUNDWATER NOT ENCOUNTERED BACKFILLED WITH SOIL CUTTINGS				

Figure A6, Log of Boring, page 1 of 1

IN PROGRESS S1483-05-01 DHARMA REALM BUDDHIST TEMPLE.GPJ 03/28/18

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. **S1483-05-01** PROJECT NAME **Dharma Realm Buddhist Temple**

BORING B5

ELEV. (MSL.) **N/A** DATE COMPLETED **2/7/2018**

ENG./GEO. **Sean Dixon** DRILLER **All Well Abandonment**

EQUIPMENT **CME 75 LAR** 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					**ASPHALT CONCRETE (AC)** 1 inch				
1					**AGGREGATE BASE (AB)** 11 inches				
2	B5-1.5 B5-2.0 B5-2.5			SM	**ALLUVIUM** Medium dense, moist, light brown, Silty SAND, fine grained - loose	21 4	86.9	18.1	
3									
4					BORING TERMINATED AT 4.0 FEET GROUNDWATER NOT ENCOUNTERED BACKFILLED WITH SOIL CUTTINGS				

Figure A7, Log of Boring, page 1 of 1

IN PROGRESS S1483-05-01 DHARMA REALM BUDDHIST TEMPLE.GPJ 03/28/18

GEOCON

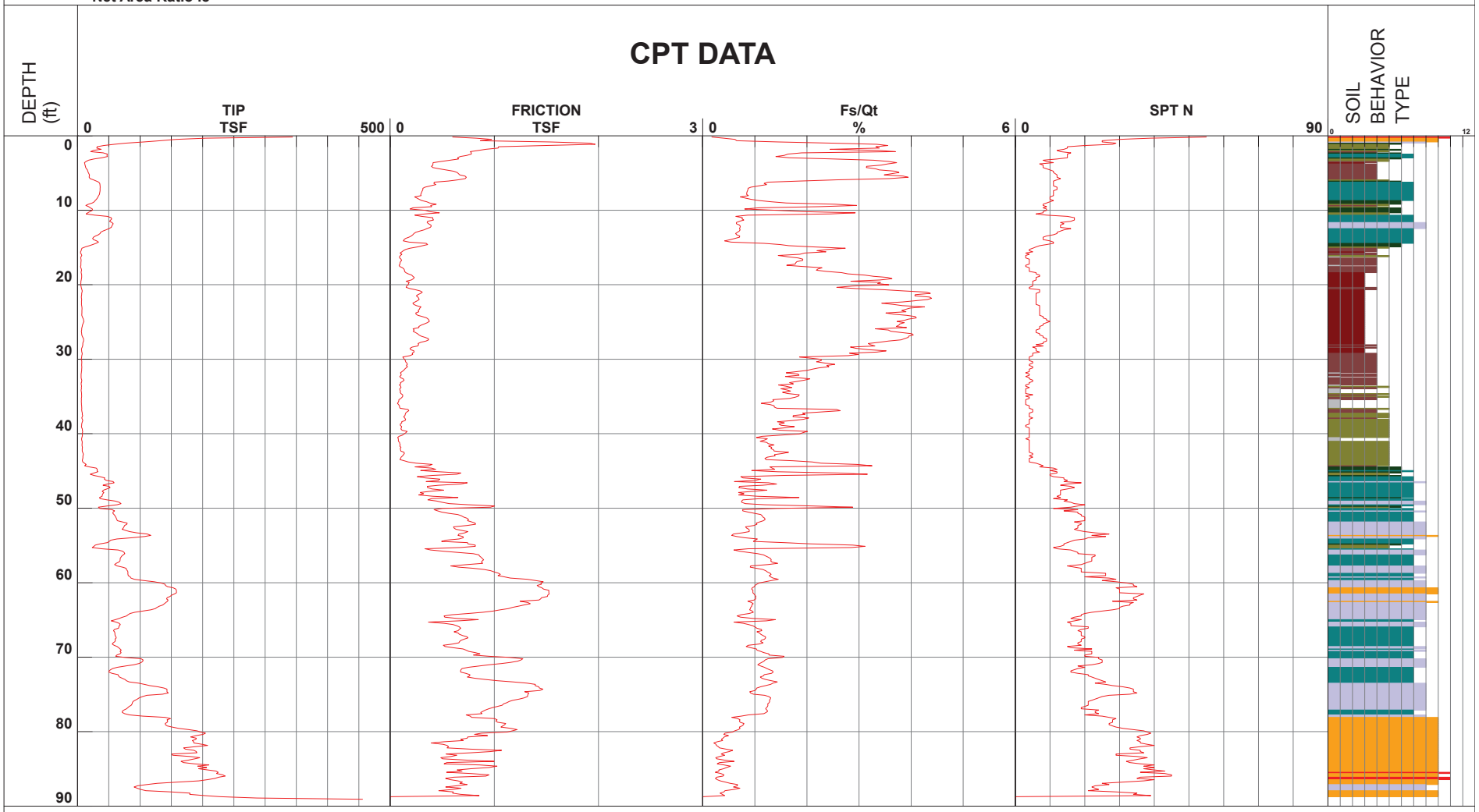
SAMPLE SYMBOLS

- ☐ ... 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

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 10cm squared

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

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CPT1

Dharma Realm Buddhist Temple
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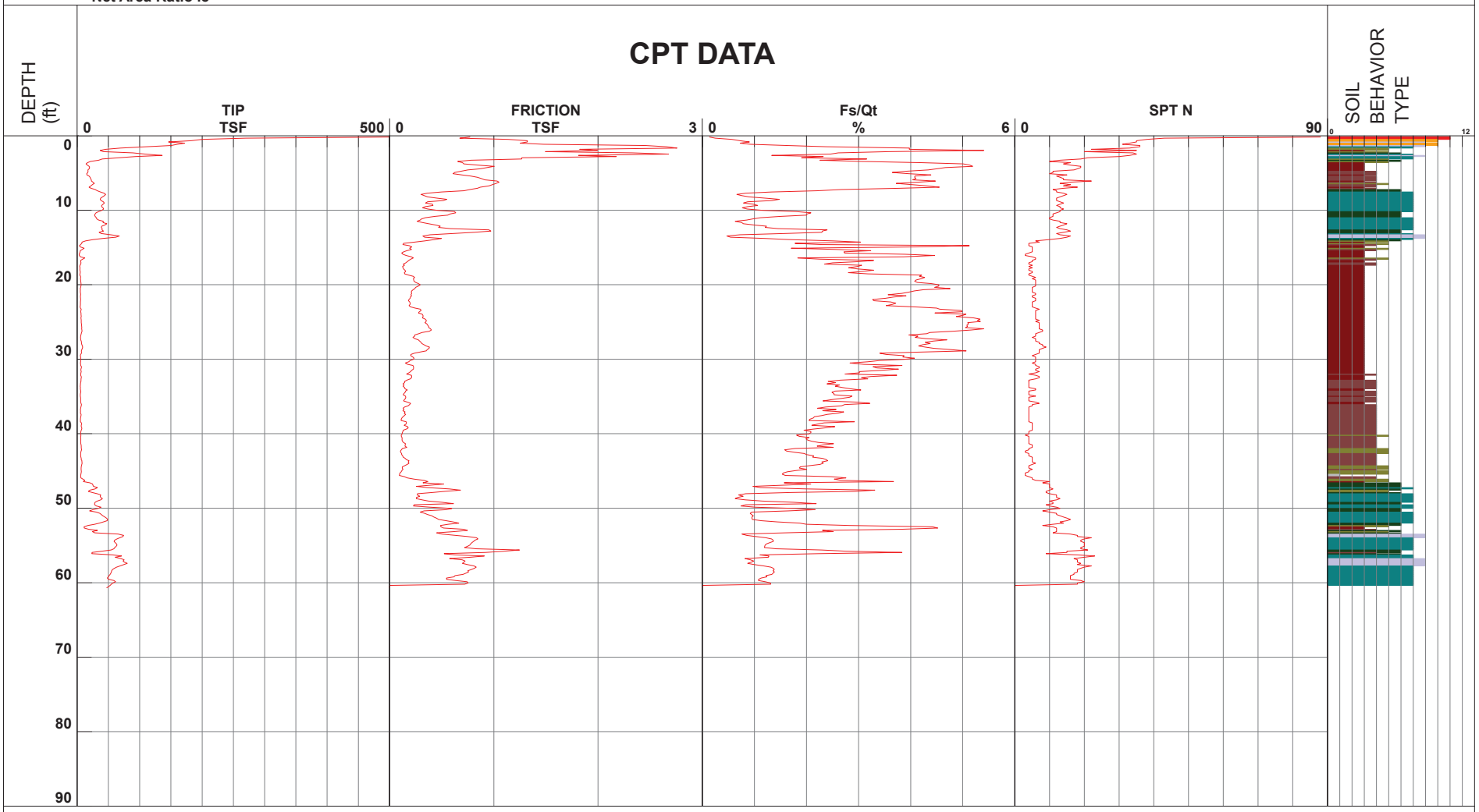
West Sacramento, California

S1483-05-01	April 2018	Figure A8

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 10cm squared

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

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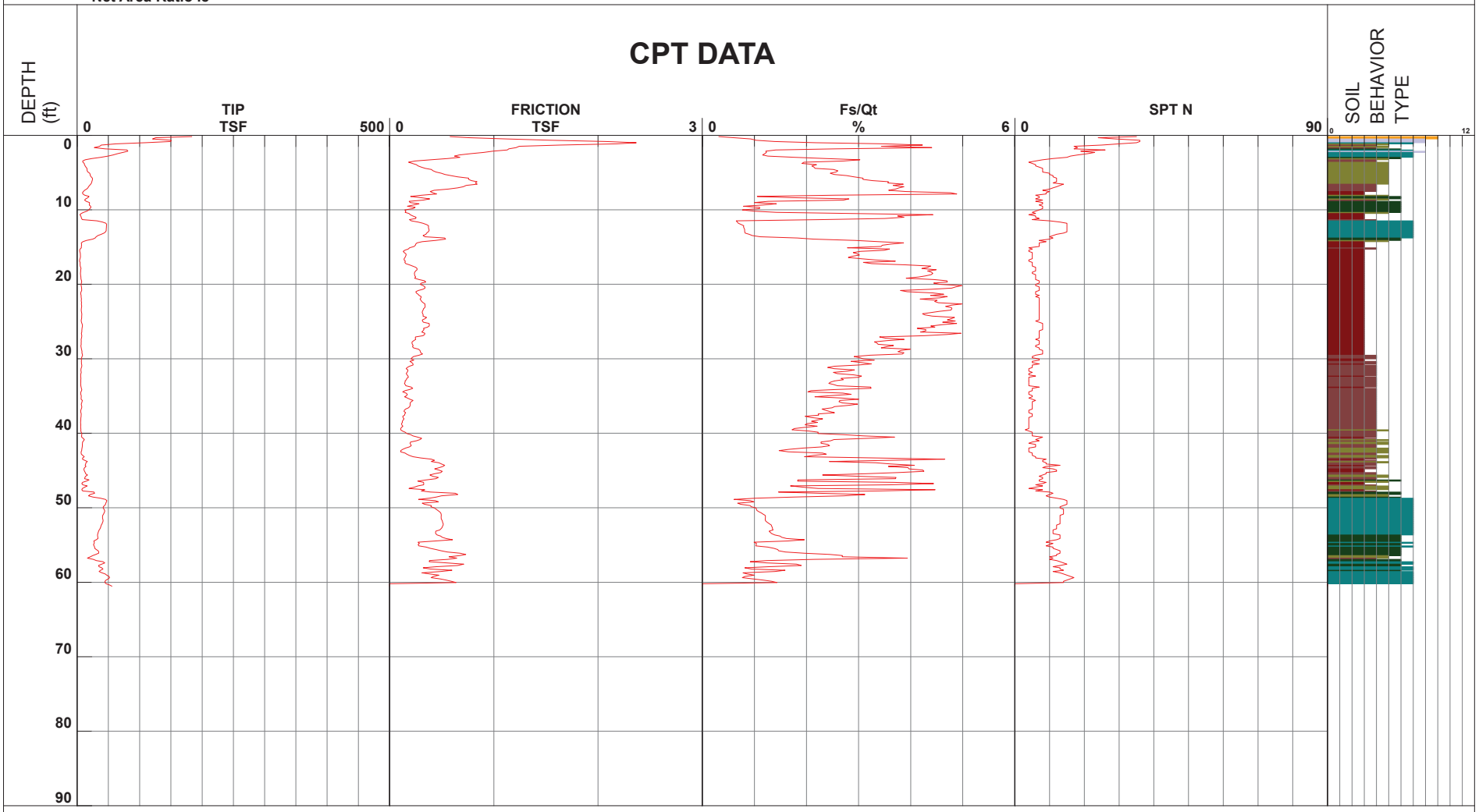
CPT2

Dharma Realm Buddhist Temple
1029 West Capitol Avenue
West Sacramento, California

S1483-05-01	April 2018	Figure A9

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 10cm squared

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

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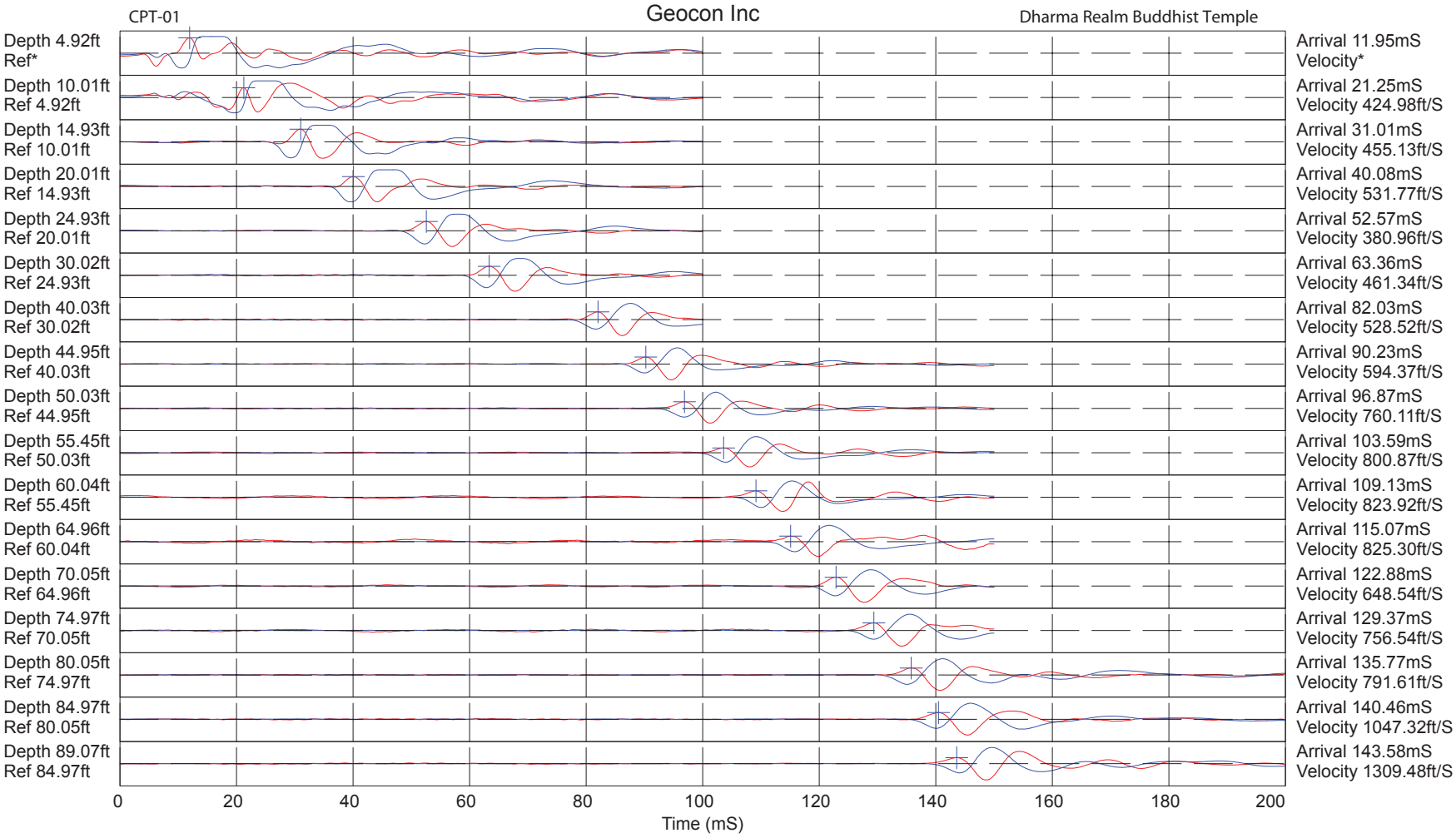
CPT3

Dharma Realm Buddhist Temple
1029 West Capitol Avenue
West Sacramento, California

S1483-05-01	April 2018	Figure A10



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

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Shear Wave Velocity Plots – CPT1

Dharma Realm Buddhist Temple
1029 West Capitol Avenue
West Sacramento, California

S1483-05-01	April 2018	Figure A11

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-place dry density and moisture content, plasticity characteristics, grain size distribution, corrosion potential, and pavement support characteristics. The results of the laboratory tests are presented on the following pages.

TABLE B1
EXPANSION INDEX TEST RESULTS
ASTM D4829

Sample Number	Depth (feet)	Moisture Content (%)		Expansion Index	Classification*
		Before Test	After Test		
B1-Bulk	0 to 5	11.1	24.9	44	Low

**Expansion Potential Classification per ASTM D4829.*

TABLE B2
SOIL 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	0 to 5	7.7	2,140	14.7 / 0.00147%	70.4 / 0.007%

*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.

Sample ID	Depth (feet)	Liquid Limit	Plastic Limit	Plasticity Index	Expansion Index	%<#200 Sieve	Water Content (%)	Dry Density (pcf)
B1-Bulk	1-5	37	21	16	43			
B1-3.5	3.5						22.9	98.9
B1-8.5	8.5						32.2	95.0
B1-13	13					76.3	35.2	89.6
B1-30	30	57	25	32				
B2-3.5	3.5						21.6	90.9
B2-5.5	5.5	32	18	14				
B2-6	6					76.2	22.5	103.8
B2-15	15					25.4		
B2-20.5	20.5					6.4	24.4	101.4
B3-5.5	5.5	45	19	26				
B3-6	6						24.9	94.7
B3-8.5	8.5						27.3	96.8
B4-2	2						31.6	86.2
B5-2	2						18.1	86.9

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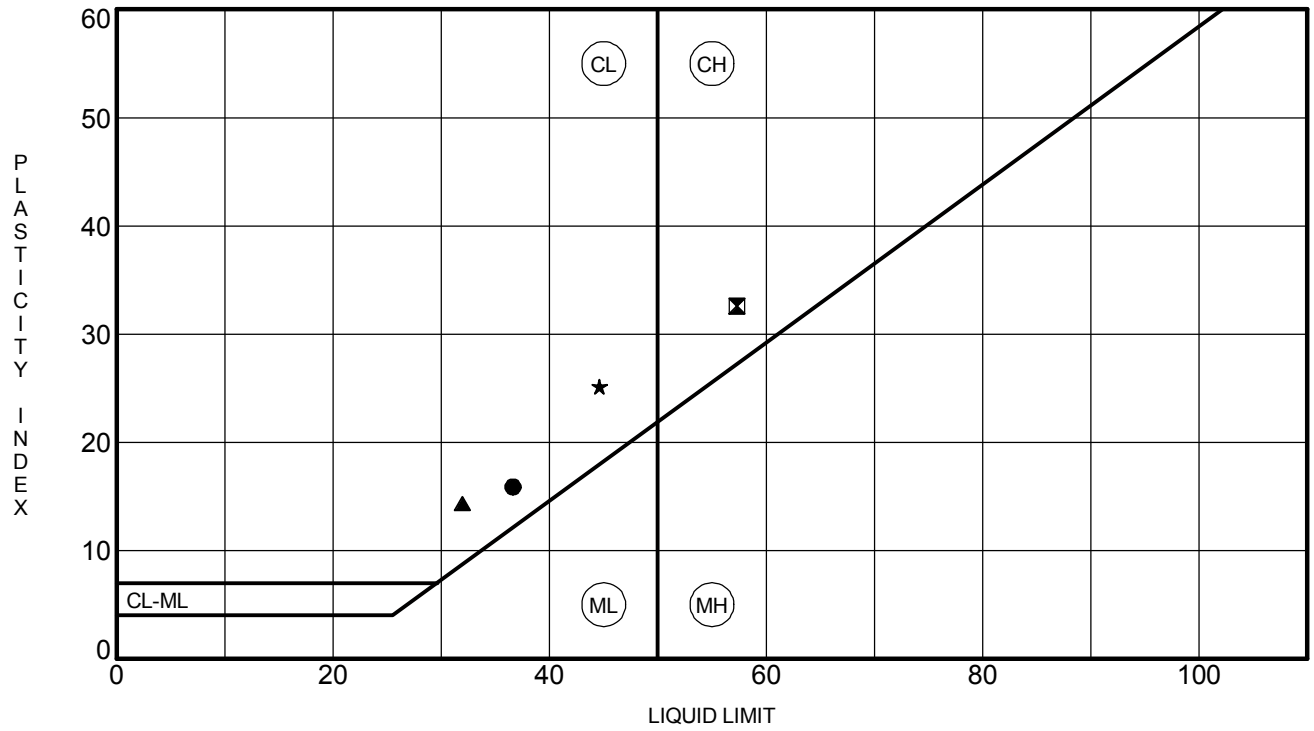
Summary of Laboratory Results

Project: Dharma Realm Buddhist Temple
Location: West Sacramento, CA
Number: S1483-05-01
Figure: B1

US LAB SUMMARY GEOTECH 2 WITH EI COLUMN S1483-05-01 DHARMA REALM BUDDHIST TEMPLE.GPJ US_LAB.GDT 3/21/18



	Sample No.	Liquid Limit	Plastic Limit	Plasticity Index	% Pass #200 Sieve	Unified Soil Classification Description	Preparation Method
●	**B1-Bulk**	**37**	**21**	**16**		**Lean CLAY (CL)**	**dry**
⊠	**B1-30**	**57**	**25**	**32**		**Fat CLAY (CH)**	**dry**
▲	**B2-5.5**	**32**	**18**	**14**		**Lean CLAY (CL)**	**dry**
★	**B3-5.5**	**45**	**19**	**26**		**Lean CLAY (CL)**	**dry**

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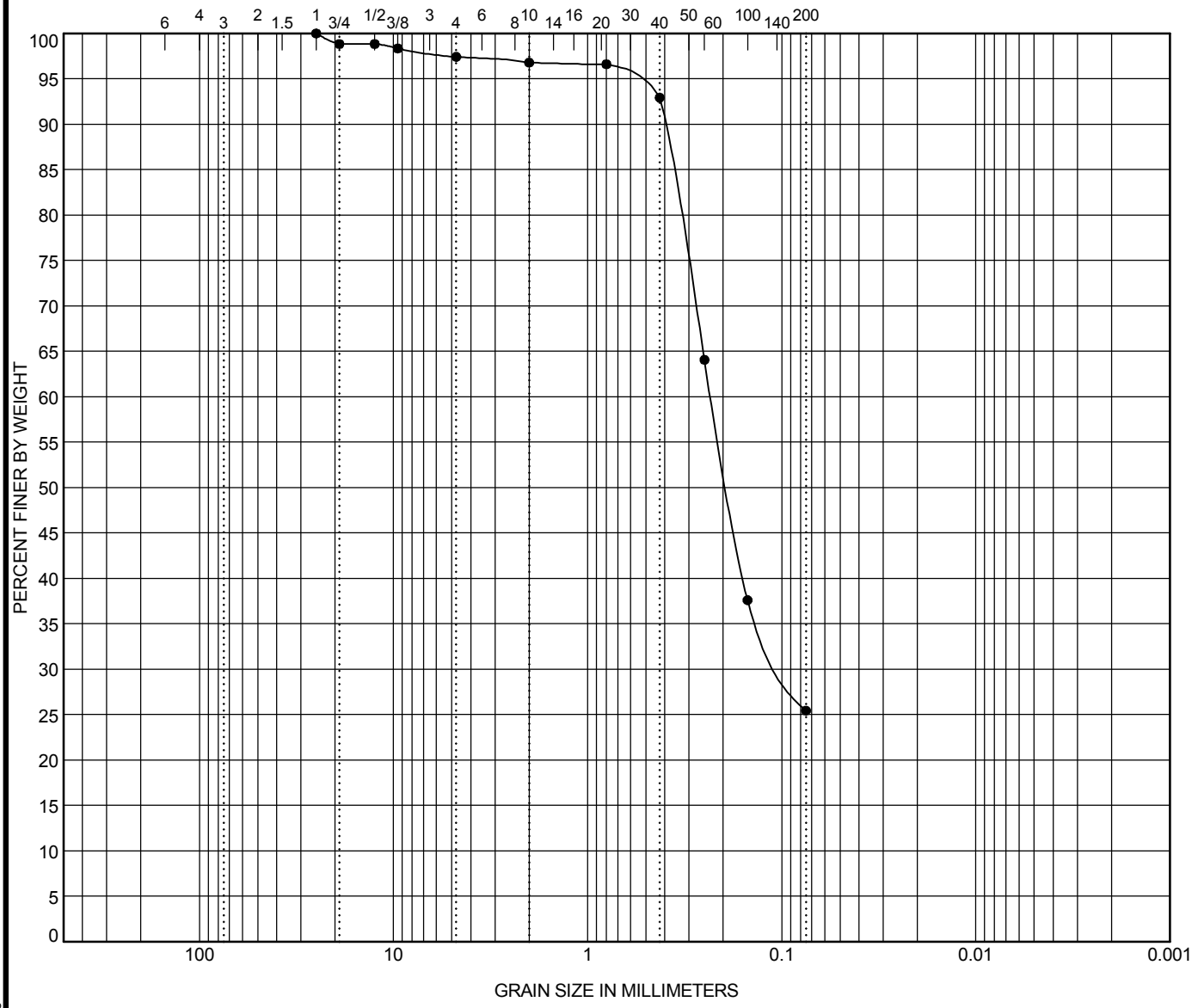
ATTERBERG LIMITS (ASTM D4318)

Project: Dharma Realm Buddhist Temple
Location: West Sacramento, CA
Number: S1483-05-01
Figure: B2

PI COPY 2 S1483-05-01 DHARMA REALM BUDDHIST TEMPLE.GPJ US_LAB.GDT 3/21/18



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample No.	Classification	LL	PL	PI	Cc	Cu
●	**B2-15**	**Silty SAND (SM)**					

	Sample No.	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B2-15**	**25**	**0.231**	**0.097**		**2.6**	**72.0**	**25.4**	

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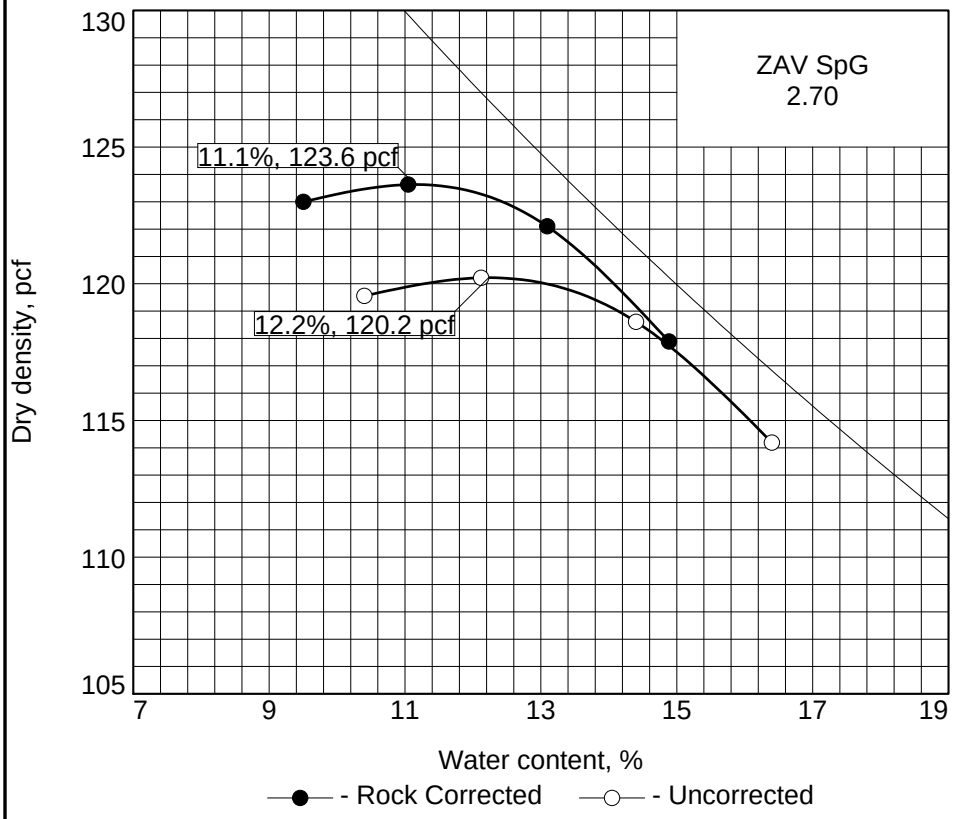
GRAIN SIZE DISTRIBUTION (ASTM D422, D6913)

Project: Dharma Realm Buddhist Temple
Location: West Sacramento, CA
Number: S1483-05-01
Figure: B3

GRAIN SIZE COPY 2 S1483-05-01 DHARMA REALM BUDDHIST TEMPLE.GPJ US_LAB.GDT 3/21/18

COMPACTION TEST REPORT



Curve No. 1

Test Specification:
ASTM 1557 Method A 2017
ASTM D4718-15 Oversize Corr. Applied to Each Test Point

Preparation Method
Hammer Wt. 10
Hammer Drop 18
Number of Layers 5
Blows per Layer 25
Mold Size 0.03327 cu. ft.

Test Performed on Material Passing #4 **Sieve**

NM **LL** **PI**
Sp.G. (ASTM D 854) 2.7
%>#4 10.3 **%<No.200**
USCS **AASHTO**
Date Sampled
Date Tested 2/15/18
Tested By JH

TESTING DATA

	1	2	3	4	5	6
WM + WS	4072.7	4086.3	4044.5	4030.5		
WM	2038.6	2038.6	2038.6	2038.6		
WW + T #1	2263.0	114.4	116.4	110.4		
WD + T #1	2044.5	100.0	100.0	100.0		
TARE #1	241.6	0.0	0.0	0.0		
WW + T #2						
WD + T #2						
TARE #2						
MOISTURE	11.0	13.1	14.9	9.5		
DRY DENSITY	123.6	122.1	117.9	123.0		

ROCK CORRECTED TEST RESULTS	UNCORRECTED
Maximum dry density = 123.6 pcf	120.2 pcf
Optimum moisture = 11.1 %	12.2 %

Material Description
Light Brown Silty Clay w/gravel, Combined Bulks

Remarks:

Project No. S1483-05-01 **Client:**
Project: City of Dharma Realm

○ **Sample Number:** B1,2,3 Bulks

Checked by: MR
Title: Lab Manager

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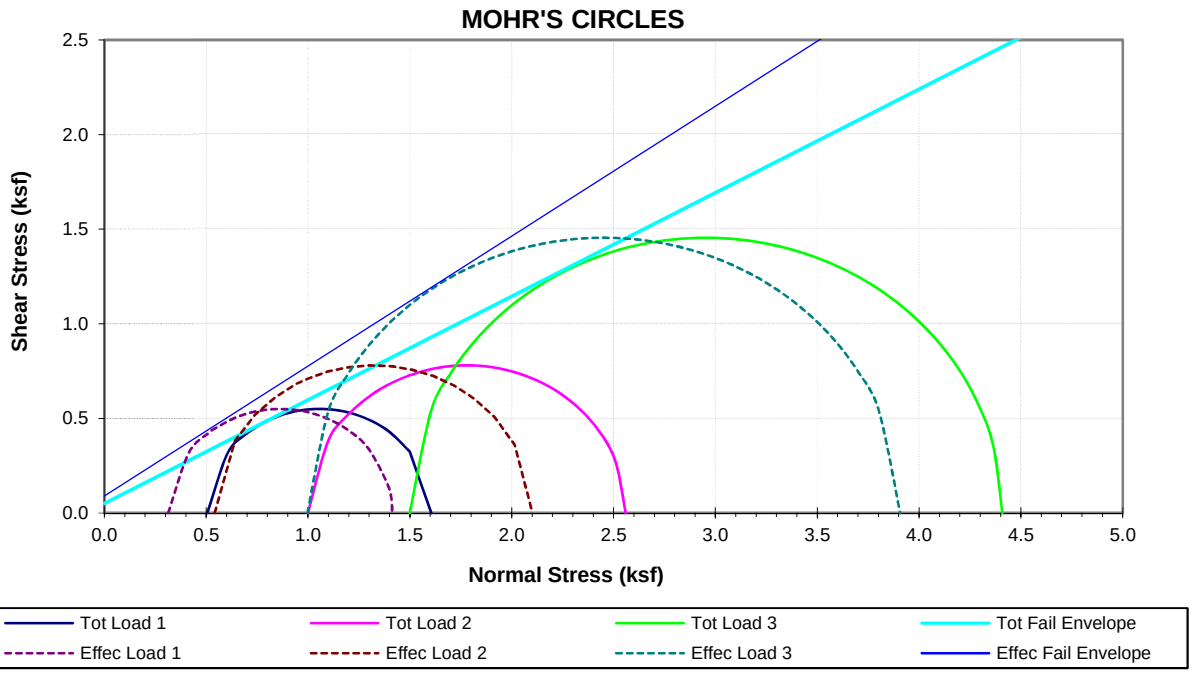
Figure B4

Consolidated Undrained Triaxial Compression - ICU Test ASTM D4767

Boring Number	B1
Sample Number	B1-6
Sample Description	Dark olive gray Sandy lean CLAY



Test Results, At Maximum Principal Stress Ratio		Total		Effective
	Friction Angle ϕ (degrees)	**28.7**		**34.5**
	cohesion (psf)	**50**		**90**
Initial Conditions at Start of Test		stage 1	stage 2	stage 3
Sample ID (psf), Initial Confining Pressure		**500**	**1000**	**1500**
	Height (inch)	4.900	4.840	4.797
	Diameter (inch)	2.405	2.416	2.416
	Moisture Content (%)	19.7	--	--
	Dry Density (pcf)	106.7	--	--
	Saturation (%)	89.1	--	--
After Saturation				
	Dry Density (pcf)	98.9	--	--
After Consolidation				
	Dry Density (pcf)	107.0	--	--
Shear Test Conditions				
	Dry Density (pcf)	107.0	107.9	108.3
	Moisture Content (%)	--	--	20.5
	Saturation (%)	--	--	96.2
	Strain rate (%/hr)	1.91	1.99	2.03
	Cell pressure (psf)	6330	6730	7320
	Initial Back Pressure (psf)	5820	5730	5820
	Initial Effective Confining Pressure (psf)	500	1000	1500
	Total Major Principal Stress At Failure (psf)	1600	2560	4410
	Effective Major Principal Stress At Failure (psf)	1410	2100	3910
	Pore Pressure At Failure (psf)	190	460	500
	Effective Minor Principal Stress At Failure (psf)	310	540	1000

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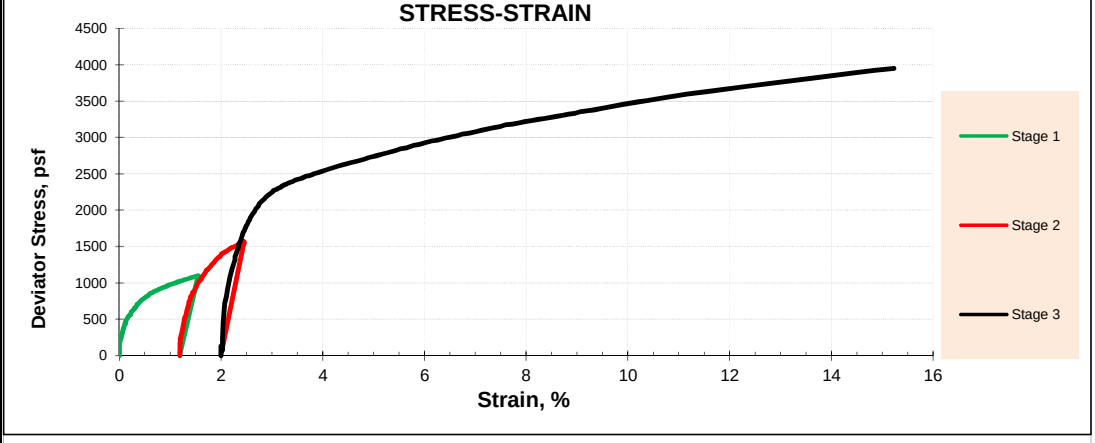
Triaxial Shear Strength - CU Test, ASTM D4767 with Pore Pressure Measurements (staged)

Project: City of Dharma Realm
Location: West Sacramento, CA
Number: S1483-05-01
Figure: B5

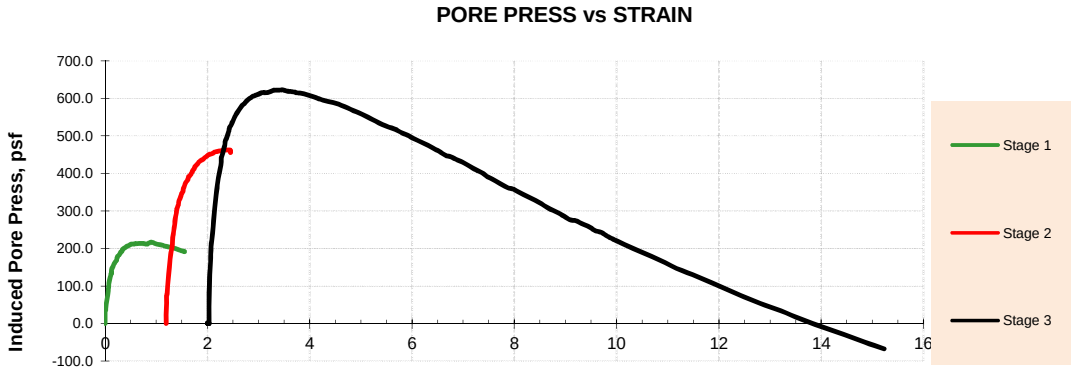
CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION - ICU TEST ASTM D4767

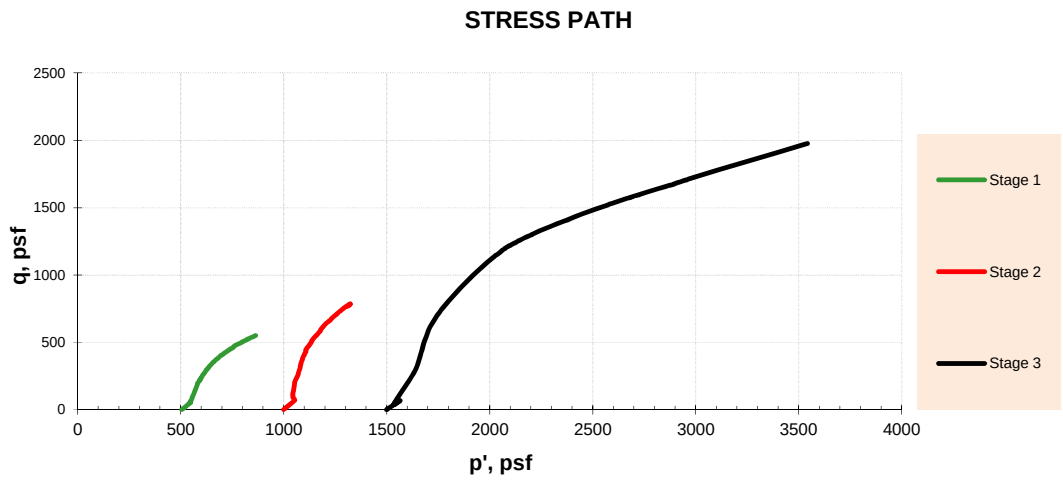
Boring Number	B1
Sample Number	B1-6
Sample Description	Dark olive gray Sandy lean CLAY



After shear photo







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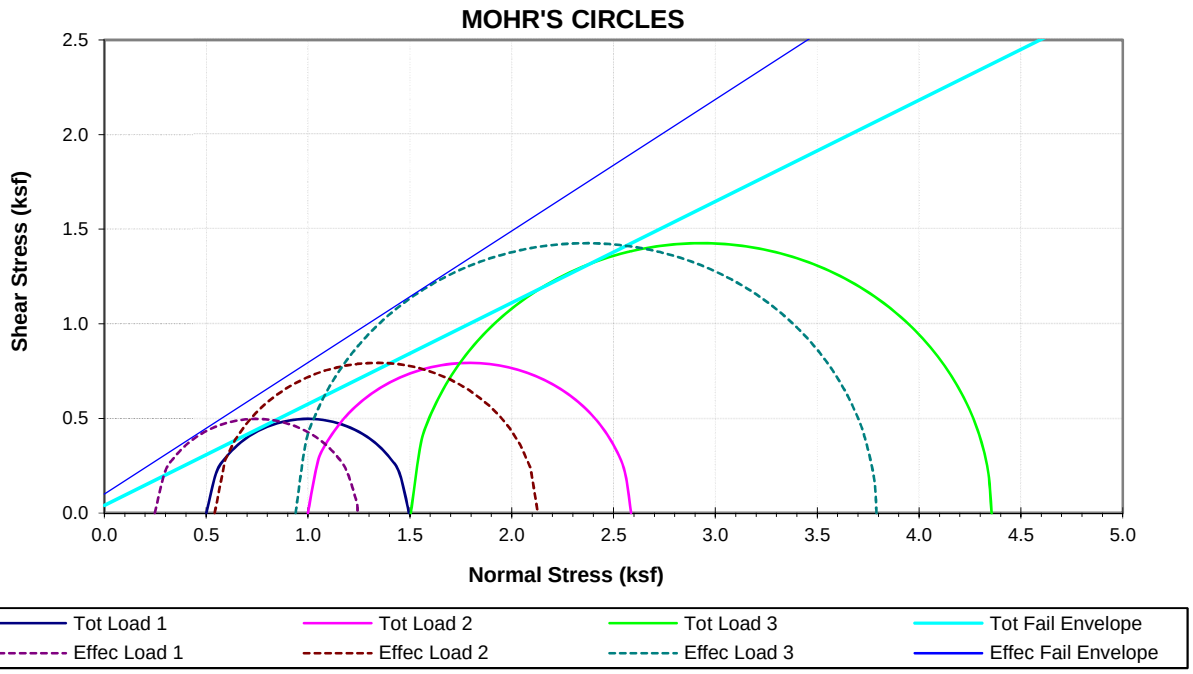
Triaxial Shear Strength - CU Test, ASTM D4767 with pore pressure measurements

Project: City of Dharma Realm
Location: West Sacramento, CA
Number: S1483-05-01
Figure: B5

Consolidated Undrained Triaxial Compression - ICU Test ASTM D4767

Boring Number	B2
Sample Number	B2-8.5
Sample Description	Dark brown Clayey SAND



Test Results, At Maximum Principal Stress Ratio		Total		Effective
	Friction Angle ϕ (degrees)	**28.2**		**34.8**
	cohesion (psf)	**40**		**100**
Initial Conditions at Start of Test		stage 1	stage 2	stage 3
Sample ID (psf), Initial Confining Pressure		**500**	**1000**	**1500**
	Height (inch)	4.820	4.739	4.694
	Diameter (inch)	2.378	2.393	2.396
	Moisture Content (%)	32.4	--	--
	Dry Density (pcf)	93.2	--	--
	Saturation (%)	104.7	--	--
After Saturation				
	Dry Density (pcf)	85.9	--	--
After Consolidation				
	Dry Density (pcf)	93.6	--	--
Shear Test Conditions				
	Dry Density (pcf)	93.6	94.3	95.0
	Moisture Content (%)	--	--	29.7
	Saturation (%)	--	--	100.1
	Strain rate (%/hr)	1.98	2.07	2.00
	Cell pressure (psf)	6250	6730	7240
	Initial Back Pressure (psf)	5750	5730	5740
	Initial Effective Confining Pressure (psf)	500	1000	1500
	Total Major Principal Stress At Failure (psf)	1500	2590	4360
	Effective Major Principal Stress At Failure (psf)	1240	2130	3790
	Pore Pressure At Failure (psf)	250	460	570
	Effective Minor Principal Stress At Failure (psf)	250	540	940

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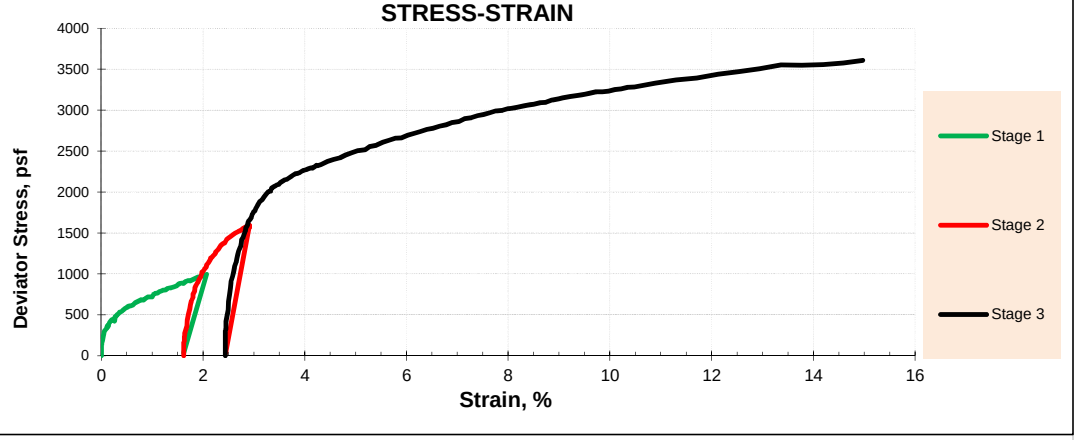
Triaxial Shear Strength - CU Test, ASTM D4767 with Pore Pressure Measurements (staged)

Project: City of Dharma Realm
Location: West Sacramento, CA
Number: S1482-05-01
Figure: B6

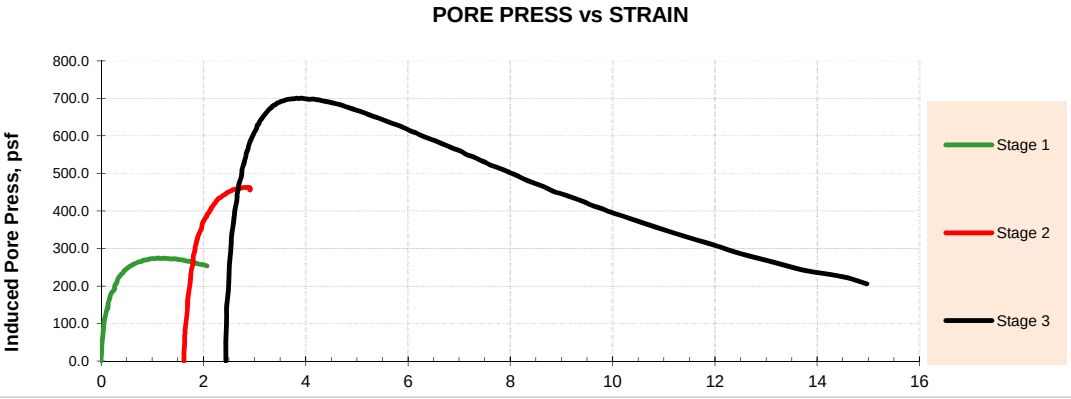
CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION - ICU TEST ASTM D4767

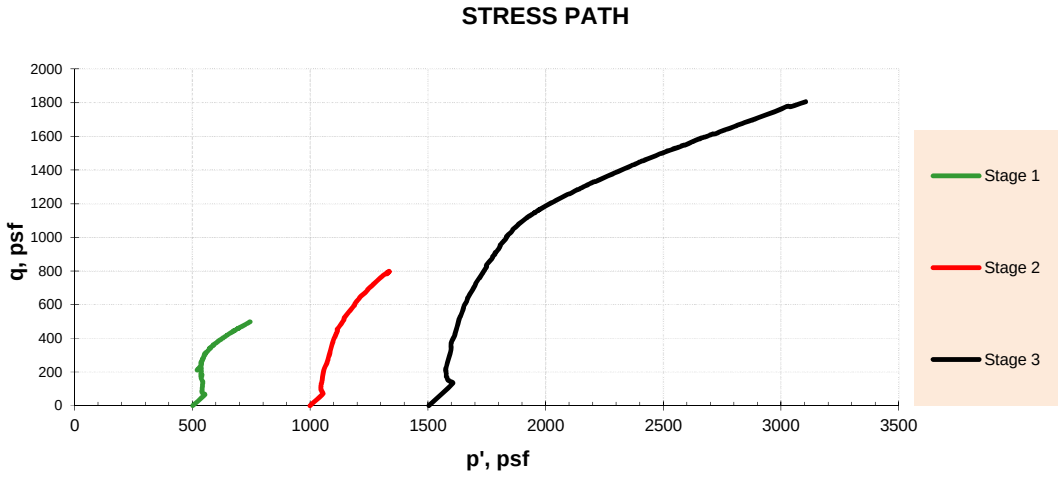
Boring Number	B2
Sample Number	B2-8.5
Sample Description	Dark brown Clayey SAND



After shear photo







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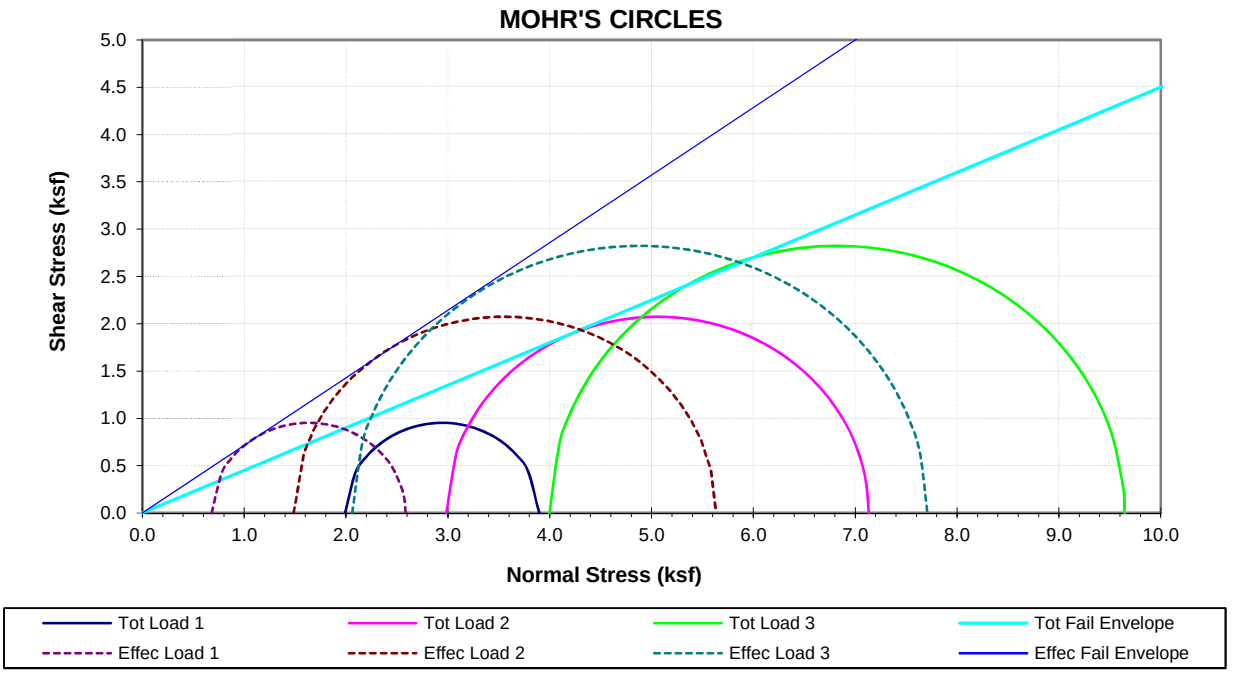
Triaxial Shear Strength - CU Test, ASTM D4767 with pore pressure measurements

Project: City of Dharma Realm
Location: West Sacramento, CA
Number: S1482-05-01
Figure: B6

Consolidated Undrained Triaxial Compression - ICU Test ASTM D4767

Boring Number	B3
Sample Number	B3-21
Sample Description	Dark bluish gray Elastic SILT



Test Results, At Maximum Principal Stress Ratio	Total	Effective
Friction Angle ϕ (degrees)	**24.2**	**35.5**
cohesion (psf)	**0**	**0**

Initial Conditions at Start of Test	stage 1	stage 2	stage 3
Sample ID (psf), Initial Confining Pressure	**2000**	**3000**	**4000**
Height (inch)	4.770	4.582	4.493
Diameter (inch)	2.426	2.414	2.432
Moisture Content (%)	40.6	--	--
Dry Density (pcf)	82.9	--	--
Saturation (%)	110.4	--	--
After Saturation			
Dry Density (pcf)	80.3	--	--
After Consolidation			
Dry Density (pcf)	87.1	--	--
Shear Test Conditions			
Dry Density (pcf)	87.1	87.5	88.4
Moisture Content (%)	--	--	30.6
Saturation (%)	--	--	95.1
Strain rate (%/hr)	0.71	0.75	0.68
Cell pressure (psf)	4900	5910	6900
Initial Back Pressure (psf)	2910	2930	2900
Initial Effective Confining Pressure (psf)	1990	2990	4000
Total Major Principal Stress At Failure (psf)	3900	7130	9640
Effective Major Principal Stress At Failure (psf)	2580	5630	7710
Pore Pressure At Failure (psf)	1310	1500	1940
Effective Minor Principal Stress At Failure (psf)	680	1490	2060

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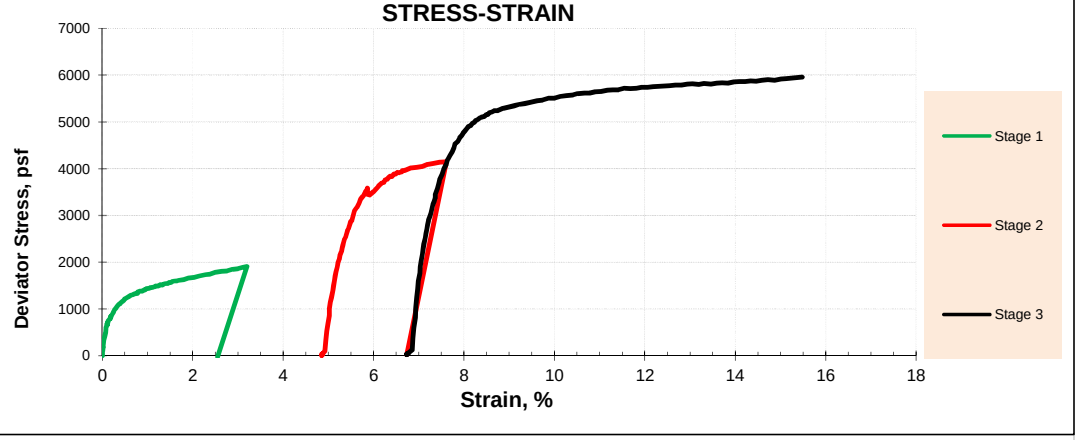
Triaxial Shear Strength - CU Test, ASTM D4767 with Pore Pressure Measurements (staged)

Project: City of Dharma
Location: West Sacramento, CA
Number: S1483-05-01
Figure: B7

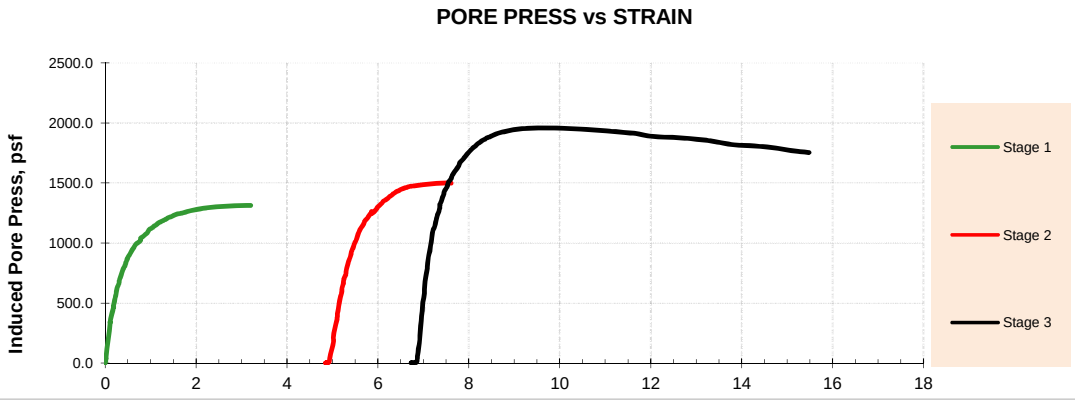
CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION - ICU TEST ASTM D4767

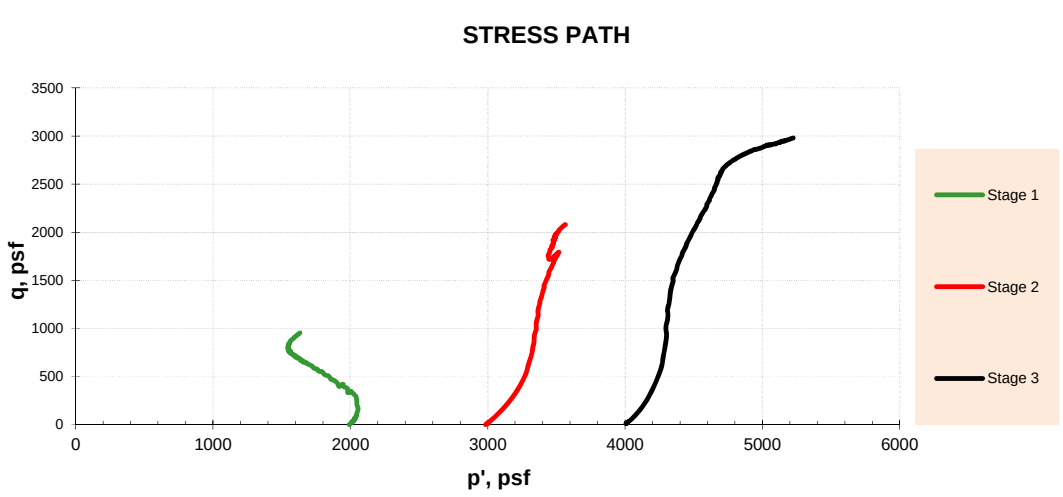
Boring Number	B3
Sample Number	B3-21
Sample Description	Dark bluish gray Elastic SILT



After shear photo







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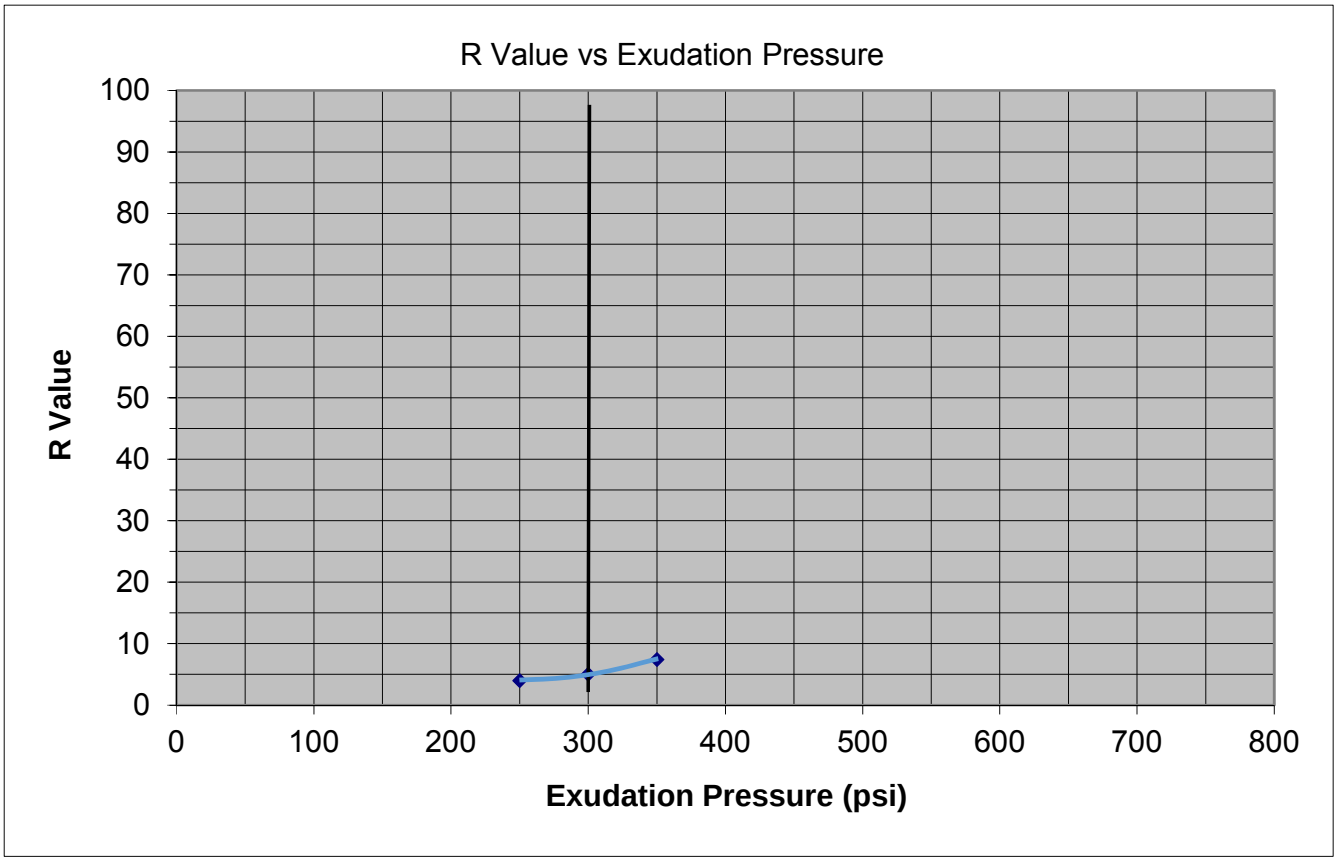
Triaxial Shear Strength - CU Test, ASTM D4767 with pore pressure measurements

Project: City of Dharma
Location: West Sacramento, CA
Number: S1483-05-01
Figure: B7



Sample ID & Description				
Boring Number	B2-Bulk			
Sample Depth (feet)	--			
Material Description	Dark gray CLAY			

Test Data			
Specimen	2044	2054	2111
Exudation Pressure (psi)	350	250	300
Expansion Dial (.0001")	49	6	27
Expansion Pressure (psf)	212.2	26.0	116.9
Resistance 'R' Value	7	4	5
Moisture at test (%)	20.7	22.4	21.5
Dry density at test (pcf)	100.7	100.4	103.5
R Value at 300 psi exudation pressure			**5**
R Value by expansion pressure (TI=5.0)			**6**
R Value by Equillibrium			**5**

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R Value By Exudation

Project:	Dharma Realm
Location:	West Sacramento, CA
Number:	S1483-05-01
Figure:	B8

APPENDIX C

APPENDIX C
LIQUEFACTION ANALYSIS

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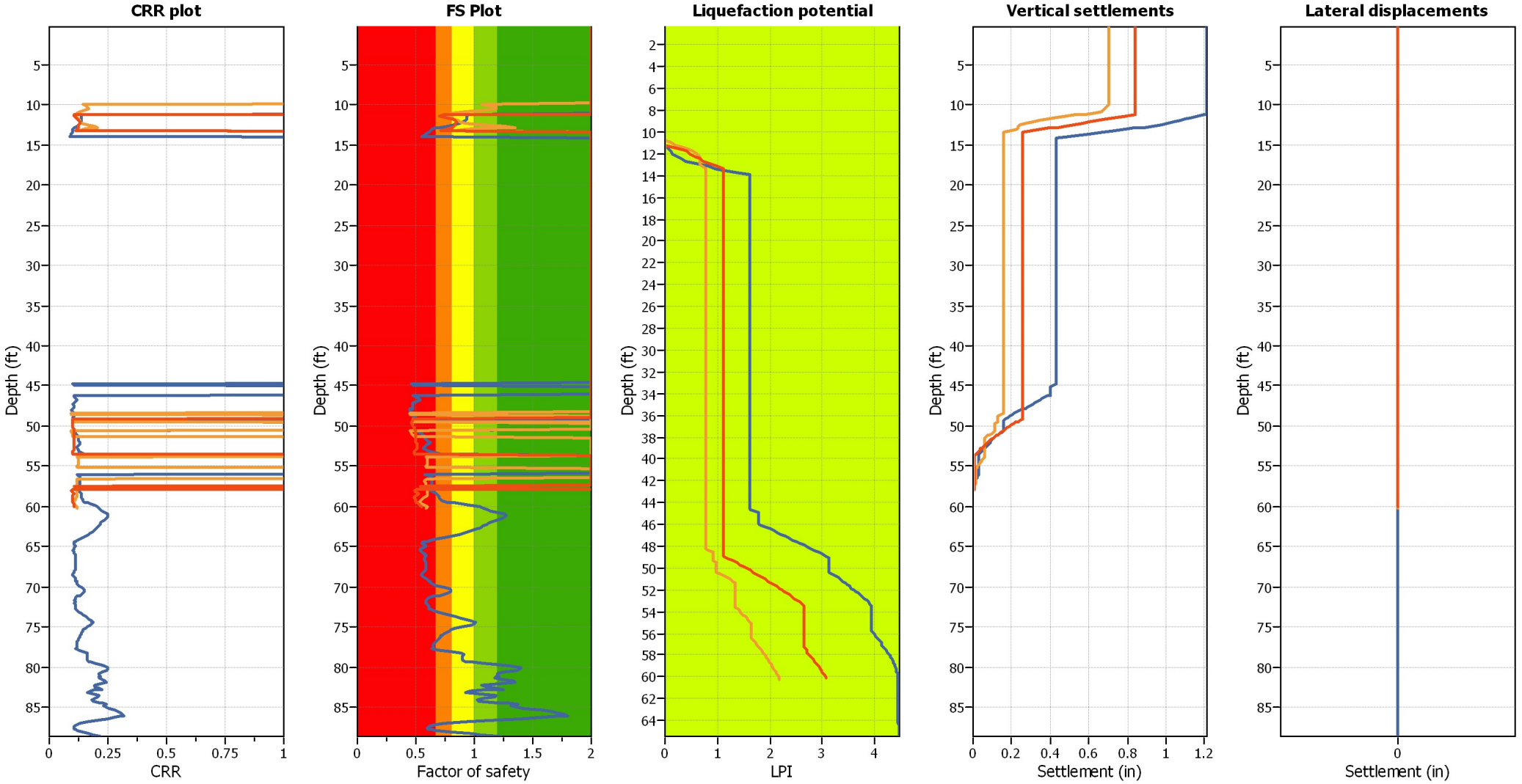
Rancho Cordova, CA 95742

Project: Dharma Realm

Overlay Cyclic Liquefaction Plots



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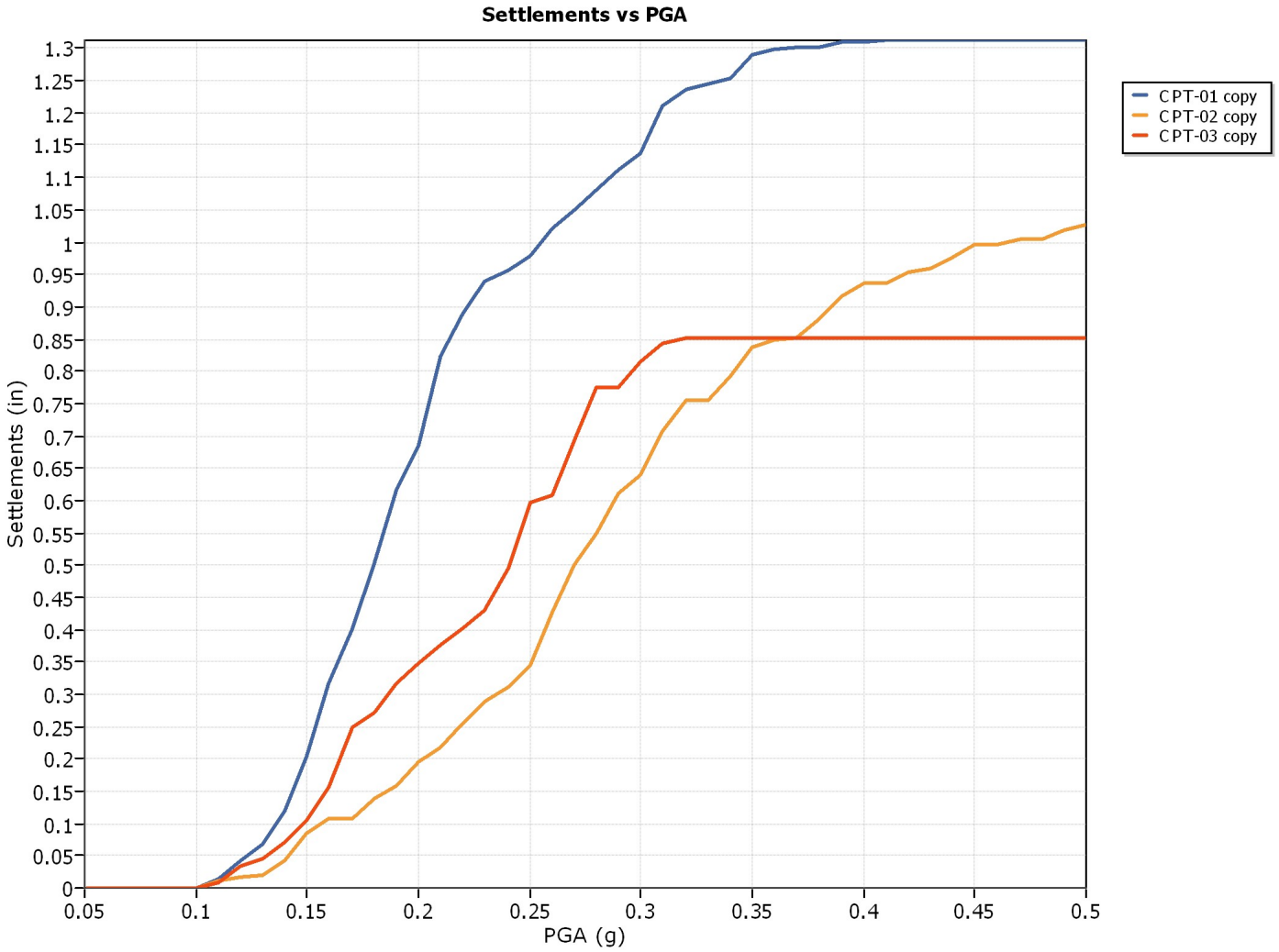
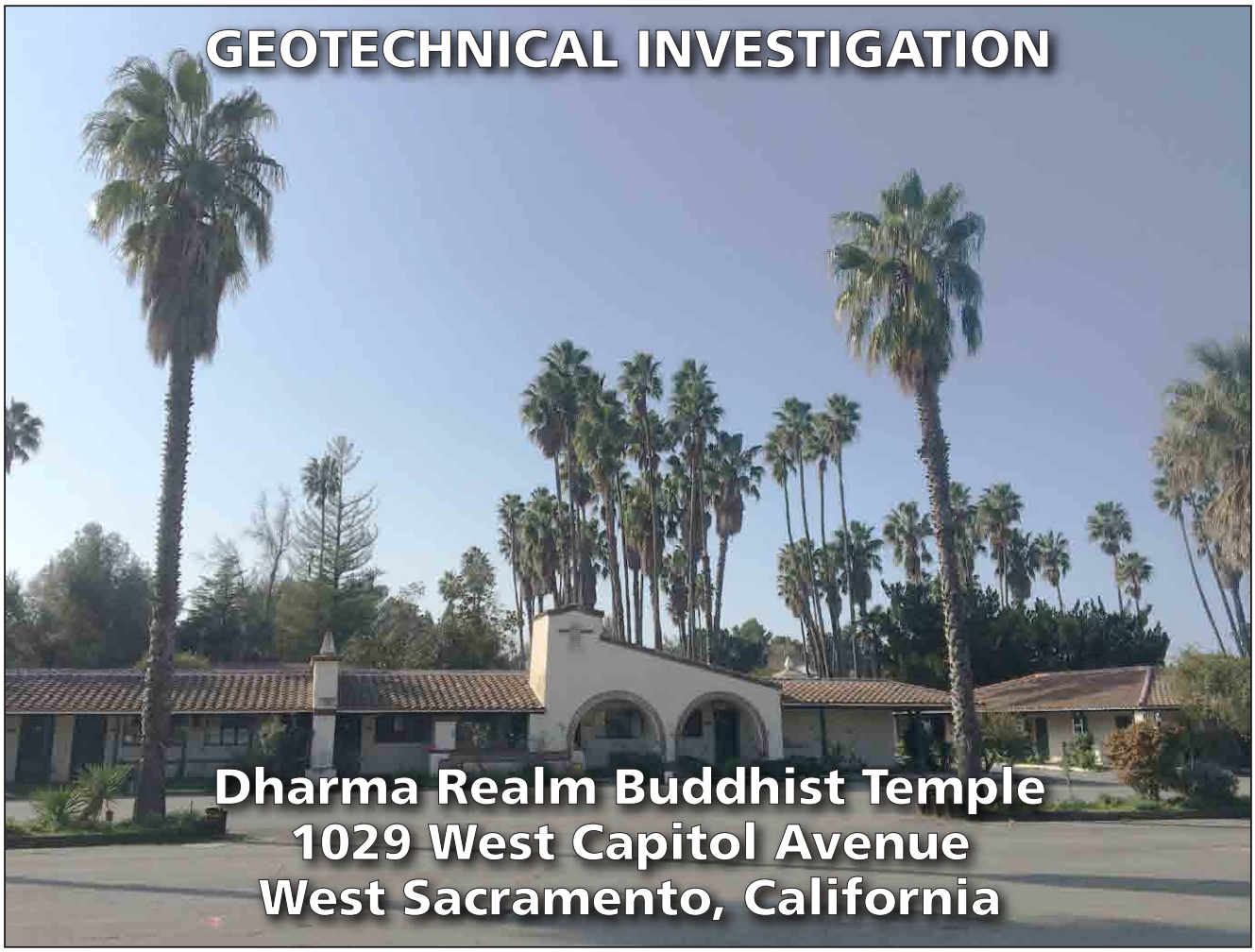
PGA Based Parametric Analysis



:: CPT main liquefaction parameters details ::

CPT Name	Assesment method	Earthquake Mag.	GWT in situ (ft)	GWT earthq. (ft)
CPT-01 copy	NCEER (1998)	6.51	15.00	10.00
CPT-02 copy	NCEER (1998)	6.51	15.00	10.00
CPT-03 copy	NCEER (1998)	6.51	15.00	10.00

GEOTECHNICAL INVESTIGATION



Dharma Realm Buddhist Temple
1029 West Capitol Avenue
West Sacramento, California

PREPARED FOR:

PACIFIC DESIGN GROUP
18071 IRVINE BOULEVARD
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PREPARED BY:

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GEOCON

GEOCON PROJECT NO. S1483-05-01 **APRIL 2018**

GEOCON
CONSULTANTS, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. S1483-05-01
April 4, 2018

VIA ELECTRONIC MAIL

Ken Paddock
Sr. Project Architect
Pacific Design Group
18071 Irvine Boulevard
Tustin, California 92780

Subject: GEOTECHNICAL INVESTIGATION
DHARMA REALM BUDDHIST TEMPLE
1029 W. CAPITOL AVENUE
WEST SACRAMENTO, CALIFORNIA

Dear Mr. Paddock:

In accordance with your authorization of our proposal (Geocon Proposal No. LS-17-242 dated August 17, 2017), we performed a geotechnical investigation for the proposed Dharma Realm Buddhist Temple located at 1029 West Capitol Avenue in West Sacramento, California.

The accompanying report presents our findings, conclusions, and recommendations regarding geotechnical aspects of redeveloping the site as presently proposed. In our opinion, no adverse geotechnical conditions were encountered that would preclude development at the site provided recommendations of this report are incorporated into the design and construction of the project.

Please contact us if you have any questions regarding this report or if we may be of further service.

Sincerely,

GEOCON CONSULTANTS, INC.

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GEOTECHNICAL INVESTIGATION

1.0 PURPOSE AND SCOPE

This report presents the results of our geotechnical investigation for the proposed Dharma Realm Buddhist Temple located at 1029 West Capitol Avenue in West Sacramento, California. The approximate site location is depicted on the Vicinity Map, Figure 1.

The purpose of our investigation was to evaluate subsurface soil and geologic conditions at the site and provide conclusions and recommendations relative to the geotechnical aspects of designing and constructing the project as presently proposed.

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

- Performed a limited geologic literature review to aid in evaluating the geologic conditions present at the site. A list of referenced material is included 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.
- Notified subscribing utility companies via Underground Service Alert (USA) a minimum of 48 hours (as required by law) prior to performing exploratory excavations at the site.
- Paid required fees and obtained a subsurface exploration permit from the Yolo County Environmental Health Division (YCEHD).
- Performed five exploratory borings (B1 through B5) with a track-mounted CME 75 drill rig using hollow-stem augers to depths ranging from approximately 4 to 41½ feet.
- Performed three in-situ cone penetration test (CPT) soundings (CPT1 through CPT3) using a 20-ton CPT rig to depths ranging from approximately 60 to 89 feet.
- Performed shear wave velocity measurements at approximate 5-foot intervals to a depth of approximately 89 feet in CPT1.
- Obtained representative disturbed and relatively undisturbed soil samples from the exploratory borings.
- Logged the borings in accordance with the Unified Soil Classification System (USCS).
- Upon completion, backfilled the borings and CPT soundings with neat cement grout. Borings in paved areas were capped with cold-patch asphalt concrete.
- Performed laboratory tests to evaluate pertinent geotechnical parameters.
- Prepared this report to summarize our findings, conclusions, and recommendations regarding the geotechnical aspects of developing the site as presently proposed.

Details of our field exploration program including exploratory boring logs are presented in Appendix A. Approximate locations of our borings are shown on the Site Plan, Figure 2. 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 approximately 15-acre site is located on the south side of West Capitol Avenue east of Merkley Avenue. The site currently contains an approximately 188,500-square-foot building and associated paved parking areas. Site-specific topography/survey information was not available for our review as of the date of this report. Based on satellite imagery topographic information (Google Earth Pro, May 19, 2017, WGS84 EGM96 Geoid), the site is relatively flat and level with an approximate elevation of 15 feet. The current site configuration is shown on the Site plan, Figure 2. Photos of the site are presented as Photos 1 and 2.

The project consists of constructing a 94,107-square-foot, three-story temple with a 19,080-square foot pedestrian bridge/stair structure that connects the third floor to a new central plaza below. The temple building will include assembly facilities, a kitchen, and cafeteria. The project configuration is shown on the Proposed Project Plan, Figure 3. Elevation views of the proposed temple and pedestrian bridge/stair structure are shown on the Proposed Project Elevation, Figure 4.

The building will likely be of steel-frame and/or concrete construction. Based on information provided by the project structural engineer, the temple building includes large spans (up to 120 feet). Maximum column loading is on the order of 3,500 kips and maximum wall loading is on the order of 32 kips per foot. Due to the high structural loading and the subsurface conditions at the site, the building will require deep foundations or shallow foundations combined with ground improvement.

Other improvements will likely include underground utility infrastructure, concrete flatwork and driveways, and landscaping. Grading plans are not yet available; however, due to the relatively flat site topography, we anticipate relatively minor grading with cuts and fills on the order of 3 feet or less.

3.0 SOIL AND GEOLOGIC CONDITIONS

We identified geologic and soil conditions by observing and sampling exploratory borings and reviewing the referenced geologic literature (Section 9.0). Soil descriptions below include the USCS symbol where applicable.

3.1 Site and Regional Geology

Based on our review of published geologic maps of the area, the site is located in the west-central portion of the Great Valley geomorphic province of California. The Great Valley (Sacramento and San Joaquin Valleys) of California is a long structural depression or down-warped trough, with the axis of the trough lying close to the eastern front of the Coast Ranges geomorphic province. In general, the southern portion of the Sacramento Valley is underlain by Quaternary continental deposits (alluvial deposits) and late Tertiary age marine and continental sedimentary rocks, which rest on a basement complex consisting of granitic rocks of the Sierra Nevada. Based on the *Preliminary Geologic Map of the*

Sacramento 30' x 60' Quadrangle, California, California Geological Survey (CGS), 2011, the site is underlain by Holocene basin (map symbol Qhb) deposits described as a mixture of sand, silt, and clay.

3.2 Existing Pavement Sections

Table 3.2 summarizes the pavement section material thicknesses encountered in our borings.

TABLE 3.2
SUMMARY OF EXISTING PAVEMENT SECTIONS

Boring ID	HMA (inches)	AB (inches)
B1	1	8
B2	1	8
B3	2	8
B4	1	11
B5	1	11

Notes:
Approximate boring locations shown on the Site Plan, Figure 2
HMA = Hot Mix Asphalt
AB = Aggregate Base

3.3 Alluvium

Below the pavement section, we encountered alluvium in each of our exploratory borings to the maximum depth explored of approximately 41½ feet. The alluvium predominantly consisted of:

- 0 to 15 feet: interbedded, medium-stiff to very stiff lean clay and loose poorly graded sand (SP and silty sand (SM). This material has high in-situ moisture content (approximately 20% to 35%) and is marginally compressible under increased loading. Laboratory Plasticity Index and Expansion Index tests on selected near-surface soil samples indicate relatively low plasticity and corresponding low expansion potential.
- 15 to 45 feet: very soft to medium stiff lean clay (CL). This material has high in-situ moisture content (approximately 30% to 40%) and is highly compressible under increased loading.
- 45 to 90 feet: interbedded layers of medium dense to very dense poorly graded sand (SP) with variable amounts of fine gravel. This layer is considered suitable for deep foundation support.

Soil conditions described in the previous paragraphs are generalized. The exploratory boring logs and CPT 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

We encountered groundwater in our exploratory borings at depths ranging from approximately 8 to 13 feet on February 7, 2018.

The online database operated by the California Department of Water Resources (DWR) includes historical depth to groundwater measurements in four area wells as summarized below:

TABLE 4.0
DWR HISTORIC DEPTH TO GROUNDWATER SUMMARY

DWR Well ID	Distance and Direction from Site	Data Period	Depth to Groundwater Range (Feet)
09N04E34K001M	1½ mile south	1968 – 2004	2 (1986) to 16.5 (1977)
09N04E32R001M	2½ mile southwest	1972 – 1992	10 (1991) to 42 (1977)
09N04E32G001M	2½ mile southwest	1962 – 1980	2 (1969) to 15 (1977)
09N04E27F001M	1½ mile northwest	1953 – 2003	9 (1995) to 50 (1953)

Based on the above information, the average depth to groundwater in the site vicinity ranges from about 5 feet to 15 feet (or deeper). Historical data suggests that groundwater depth could rise as high as 2 feet in the area during sustained high flow stages of the Sacramento River.

It should be noted that fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, and other factors. Depth to groundwater can also vary significantly due to localized pumping, irrigation practices, and seasonal fluctuations. Therefore, it is possible that groundwater may be higher or lower than the level observed during our investigation.

5.0 SEISMICITY AND GEOLOGIC HAZARDS

5.1 Regional Active Faults

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 contained within an Alquist-Priolo Earthquake Fault Zone. Therefore, we consider the potential for ground rupture due to onsite active faulting to be low. In order to determine the distance of known active faults within 50 miles of the site, we used the computer program *EQFAULT*, (Version 3, Blake, 2000). Principal references used within *EQFAULT* are Jennings (1975), Anderson (1984) and Wesnousky (1986). Results are summarized in Table 5.1.

TABLE 5.1
REGIONAL ACTIVE FAULTS

Fault Name	Distance From Site (miles)	Maximum Moment Magnitude (M_W)
Foothills Fault System	23.7	6.5
Great Valley, Segment 4	26.0	6.6
Great Valley, Segment 3	26.7	6.8
Great Valley, Segment 5	29.1	6.5
Hunting Creek - Berryessa	37.7	6.9
Concord – Green Valley	37.7	6.9
Great Valley, Segment 6	40.1	6.7
West Napa	47.3	6.5
Greenville	48.6	6.9

5.2 Ground Shaking

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 6.5 and the estimated Peak Ground Acceleration (PGA) for the Maximum Considered Earthquake (MCE) with a 2,475-year return period is 0.32g.

While listing PGA is useful for comparison of potential effects of fault activity in a region, other considerations are important in seismic design, including frequency and duration of motion and soil conditions underlying the site.

5.3 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 Stockton area. However, soil and groundwater conditions exist at the site that may be susceptible to seismic-induced liquefaction under the design-level seismic event.

The current standard of practice, as outlined in the *Recommended Procedures for Implementation of DMG Special Publication 117A, Guidelines for Analyzing and Mitigating Liquefaction in California"* (Southern California Earthquake Center, 2002) requires liquefaction analysis to a depth of 50 feet below the lowest portion of the proposed structure. 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 assumed a seasonal high groundwater elevation of 5 feet below existing grade, an earthquake moment magnitude of 6.6 (USGS deaggregation), and a MCE (2,475-year return interval event) PGA of 0.321g (PGA_M adjusted for Site Class per American Society of Civil Engineers [ASCE] 7-10, Eq. 11.8-1). We also evaluated liquefaction potential under the design-level ground motion PGA of 0.21 which is 2/3 of the MCE PGA (e.g. $PGA_M/1.5$).

Based on the results of our analyses, there is the potential for liquefaction at the site within sandy soil layers generally present between depths of approximately 10 and 15 feet and 45 to 60 feet.

Consequences of liquefaction may include ground surface settlement, ground loss (sand boils), and lateral slope displacements (lateral spreading). For liquefaction-induced sand boils or fissures to occur, pore water pressure induced within liquefied strata must exert a large enough force to break through overlying, non-liquefiable layers. Based on work by Youd and Garris (1995) which modified and advanced the work of Ishihara (1985), a capping layer of non-liquefiable soil can prevent the occurrence of sand boils or fissures. An apparently continuous, non-liquefiable, silt/clay stratum approximately 10 feet thick is present across the site. In our opinion, this layer is sufficiently thick enough to reduce the potential for ground loss due to sand boils or fissures. Because the site topography is generally flat and level and there is not an adjacent free-face or sloping geometry, lateral spreading is not anticipated.

Therefore, the likely consequence of potential liquefaction at the site is ground settlement. Dynamic settlement of the soils that experience liquefaction may occur after earthquake shaking has ceased. We estimated post-liquefaction settlement under the MCE and design level ground motions using the software CLiq following the methodology of Zhang et al (2002) which is generally based on case histories of liquefaction of hydraulic fills in the San Francisco Bay Area. The results of the analysis are summarized in Table 5.3.

TABLE 5.3
ESTIMATED LIQUEFACTION SETTLEMENT

CPT location	Estimated Total Liquefaction Settlement	
	MCE Ground Motion (PGA_M = 0.32g)[1]	**Design-Level Ground Motion (PGA_M/1.5 = 0.21g)[2]**
CPT1	1.2 inches	0.7 inches
CPT2	0.7 inches	0.2 inches
CPT3	0.8 inches	0.4 inches
Average	0.9 inches	0.4 inches

Notes: 1. MCE liquefaction settlement for structural evaluation of life safety/collapse.
2. Design-Level liquefaction settlement for structural performance evaluation

The distribution of dynamic settlement associated with potential liquefaction is generally less than ¼ inch within each liquefiable layer. This minor amount of potential liquefaction and associated settlement is not significant enough to require site response analysis, substantially reduce pile axial capacity, or induce significant negative skin friction. In our experience, design-level ground motion liquefaction settlements of this magnitude (approximately ½ inch) typically do not require special foundation design or modifications.

5.4 Expansive Soil

Laboratory Plasticity Index and Expansion Index tests on selected near-surface soil samples indicate relatively low plasticity and corresponding low expansion potential. Mitigation and specific design measures with respect to expansive soil is not necessary.

5.5 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

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

6.1.2 Based on our findings, evaluation, and analyses to date, we have identified the following geotechnical constraints for the site:

- Shallow groundwater – Seasonally as shallow as 5 feet (or shallower). This may impact grading and underground utility construction
- Soft, compressible soil between depths of 15 to 45 feet. These soils prohibit the use of conventional shallow foundations due to excessive post-construction settlement.
- Potentially liquefiable sand between 10 and 15 feet and 45 to 60 feet. If liquefaction were to occur, there is a low potential for ground loss (sand boils) and we anticipate only minor post-liquefaction settlements which should not adversely impact foundation performance.

These conditions impact design and construction of the project. Discussion of these geotechnical constraints and specific mitigation, design, and construction recommendations are provided herein.

6.1.3 Given the subsurface conditions and structural loading demands, the building and pedestrian bridge/stair structure may be supported on deep foundations or shallow foundations in conjunction with ground improvement measures, such as drilled displacement columns (DDCs). Specific recommendations are provided in this report.

6.1.4 Conclusions and recommendations provided in this report are based on our review of referenced literature, analysis of data obtained from our exploratory field exploration program, laboratory testing program, and our understanding of the proposed development at this time.

6.1.5 We should review the project plans as they develop further, provide engineering consultation as needed during final design, and perform geotechnical observation and testing services during construction.

6.2 Seismic Site Class / Seismic Design Criteria

6.2.1 We understand that seismic design of the structures will 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 American Society of Civil Engineers (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 5.3 of this report, there is a potential for liquefaction at the site; however, there is a low potential for failure or collapse (sand boils/ground loss) due to liquefaction, 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 measurements performed in conjunction with CPT1. The calculated average shear wave velocity (per ASCE 7-10, Eq. 20.4-1) is approximately 637 feet/sec. Therefore, the site is classified as Site Class "D" per Table 20.3-1 of ASCE 7-10.

6.2.3 We used the USGS web application *US Seismic Design Maps* (http://geohazards.usgs.gov/designmaps/us/ application.php) to evaluate site-specific seismic design parameters in accordance with the 2016 CBC/ASCE 7-10. 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	2013 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	0.699g	Figure 1613.3.1(1) / Figure 22-1
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.299g	Figure 1613.3.1(2) / Figure 22-2
Site Coefficient, F_A	1.241	Table 1613.3.3(1) / Table 11.4-1
Site Coefficient, F_V	1.801	Table 1613.3.3(2) / Table 11.4-2
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	0.867g	Eq. 16-37 / Eq. 11.4-1
Site Class Modified MCE_R Spectral Response Acceleration (1 sec), S_{M1}	0.539g	Eq. 16-38 / Eq. 11.4-2
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.578g	Eq. 16-39 / Eq. 11.4-3
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.359g	Eq. 16-40 / Eq. 11.4-4

6.2.4 Table 6.2.4 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 6.2.4
2016 CBC SITE ACCELERATION DESIGN PARAMETERS

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.239g	Figure 22-7
Site Coefficient, F_{PGA}	1.321	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.316g	Section 11.8.3 (Eq. 11.8-1)

6.2.5 Conformance to the criteria presented in Tables 6.2.3 and 6.2.4 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 Grading and excavations at the site may be accomplished with standard effort using heavy-duty grading/excavation equipment. We do not anticipate grading and excavations to generate cobbles or boulders that would require special handling or placement.

6.3.2 Temporary excavations deeper than 4 feet and entered by workers must meet Cal-OSHA requirements as appropriate. Excavation sloping, benching, the use of trench 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.

6.3.3 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 sloping to prevent damage to any structure or improvements near underground excavations.

6.3.4 Permanent cut and fill slopes 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.

6.3.5 If grading occurs during or after the wet season (typically winter and spring), or in periods of precipitation, in-place and excavated soils will likely be wet. Earthwork contractors should be aware of moisture sensitivity of clayey and fine-grained soils and potential compaction/workability difficulties.

6.3.6 Earthwork and pad preparation operations in these conditions will likely be difficult with low productivity. Often, a period of at least one month of warm and dry weather is necessary to allow the site to dry sufficiently so that heavy grading equipment can operate effectively. Conversely, during dry summer and fall months, dry clay soils may require additional grading effort (discing, mixing, or other means) to attain proper moisture conditioning.

6.3.7 Based on laboratory testing, in-situ moisture content of site soils range from about 20% to 35%, which is significantly higher than optimum moisture content for this type of material. In addition, soil below existing pavements are often wet when exposed. Due to the fine-grained nature of the soils and measured in-situ moisture contents well above optimum, significant soil instability should be expected. Additional drying effort to attain moisture contents suitable for compaction should be anticipated regardless of the time of year. Mitigation alternatives may include aerating/drying the exposed soils (assuming favorable weather conditions), overexcavating 12 to 18 inches and placing geotextile fabric/geogrid covered with aggregate, or chemical treatment (e.g. lime treatment). We can provide specific recommendations during construction based on conditions encountered.

6.4 Materials for Fill

6.4.1 Excavated soils generated from cut operations at the site are suitable for use as fill in structural areas provided they do not contain deleterious matter, organic material, or cementations larger than 6 inches in maximum dimension. Due to high in-situ moisture content, native soils reused as engineered fill will likely require aerating/drying to attain suitable moisture content for compaction, regardless of the time of year.

6.4.2 Import fill material should be primarily granular with a “very low” expansion potential (Expansion Index less than 20), a Plasticity Index less than 15, be free of organic material and construction debris, and not contain rock/cementations larger than 6 inches in greatest dimension. Import soil should also contain a sufficient amount of fines (generally more than 10%) to provide “binder” and reduce potential caving when excavated.

6.4.3 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.

6.5 Grading

6.5.1 All earthwork operations should be observed and all fills tested for recommended compaction and moisture content by a representative of Geocon.

6.5.2 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.

6.5.3 Prior to commencing grading, 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.

6.5.4 Site preparation should begin with removal of existing pavements, surface/subsurface structures, underground utilities, debris, and existing fill. Existing pipelines and overlying trench backfill should be completely removed to expose undisturbed soil. Excavations or depressions resulting from site clearing operations, or other existing excavations or depressions, should be restored with engineered fill in accordance with the recommendations of this report.

6.5.5 Within areas to be developed, any existing trees and associated root systems should be removed. Roots larger than 1 inch in diameter should be completely removed. Smaller roots may be left in-place as conditions warrant and at the discretion of our field representative.

6.5.6 Cut areas, areas left at grade, and areas to receive fill should be scarified at least 12 inches, uniformly moisture-conditioned at or above 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.

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

6.5.8 The top 6 inches of final vehicular pavement (non-pervious) subgrade, whether completed at-grade, by excavation, or by filling, should be uniformly moisture-conditioned at or above 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 aggregate base (AB).

6.5.9 Pipe bedding, shading, and trench backfill should conform to the requirements of the appropriate utility authority. Material excavated from trenches should be adequate for use as general backfill above shading provided it does not contain deleterious matter, vegetation, or cementations larger than 6 inches in maximum dimension. Trench backfill should be placed in loose lifts not exceeding 8 inches, moisture-conditioned at or above optimum and compacted to at least 90% relative compaction. Compaction should be performed by mechanical means only; jetting of trench backfill is not recommended.

6.6 Foundations

Due to the presence of compressible soil within the top 45 feet and the magnitude of anticipated structural loading, the use of shallow foundations for support of the new buildings is not feasible due to excessive post-construction settlement. However, shallow foundations may be used if ground improvement is performed.

We consulted with Farrell Design Build, a geotechnical specialty contractor, to evaluate feasible ground improvement alternatives for the project. Given the significant thickness and fine-grained nature of compressible soil at the site, ground improvement measures such as rammed aggregate piers (RAPs) and vibro-replacement stone columns are not considered to be effective in significantly increasing allowable bearing capacity or reducing post-construction settlements. In addition, technologies such as deep dynamic compaction or chemical grouting are also not considered effective. Based on the subsurface conditions at the site, rigid inclusions, such as drilled displacement columns (DDCs) may provide suitable support for shallow foundations, increase bearing capacity, and reduce post-construction settlements.

If ground improvement is not performed, we recommend using deep foundations consisting of drilled displacement piles for support of the building and pedestrian bridge/stair structure. Recommendations are provided herein.

6.6.1 Ground Improvement – Drilled Displacement Columns

6.6.1.1 Based on our consultation with Farrell Design Build, we anticipate that conventional shallow foundations with spread and continuous footings and floor slab supported on ground improvement can be used. Given the subsurface conditions at the site and anticipated

structural loading, Drill Displacement Column™ (DDC) technology is a suitable form of ground improvement. For DDC, concrete grout is injected under pressure through the displacement drill for discreet ground improvement to install "composite ground" that improves compressible soil, reduces liquefaction potential, and increases the composite allowable bearing pressure for foundation. This method of ground improvement is comparable to large diameter compaction grouting. Typically, the ground improvement is composed of relatively closely-spaced DDC below foundation elements and a wider spacings below the slab section. The drill displacement tool displaces a 16 to 18-inch diameter hole and the finished diameter of the pressure grouted DDC is up to 19 to 21 inches. At this site, we anticipate that the DDC elements would be on the order of 45 to 50 feet long.

6.6.1.2 For preliminary design purposes, DDC elements can be installed in a grid pattern at a spacing of approximately 4 to 6 feet on centers under footings. DDC elements at a spacing of approximately 7 to 12 feet on centers can be used to support the slabs, the spacing will depend on the foundation loading and structural slab design (thickness and reinforcement).

6.6.1.3 Footings supported on DDC-reinforced soils can be designed for an allowable bearing capacity of 5,000 pounds per square foot (psf). The bearing capacity should be verified by a full-scale load test. Footings can be designed to resist lateral loads using an allowable coefficient of friction of 0.40. Lateral sliding resistance is derived at the concrete/aggregate interface below the footing. A 1/3 increase to the allowable bearing capacity and frictional resistance is permitted for short-term seismic and wind loads.

6.6.1.4 Final foundation system and design parameters will be developed in conjunction with the project structural engineer during the ground improvement design-build activities. If the DDC system is selected for structural support, the DDC specialty contractor would provide a complete design-build submittal with design recommendations, plans and specifications stamped and signed by a California registered Professional Engineer. Geocon will need to perform a geotechnical review of the DDC design.

6.6.1.5 In addition to plan review, we recommend geotechnical oversight during DDC construction. Our Quality Assurance (QA) services will supplement the contractor internal Quality Control (QC) program. The contractor QC program should document each DDC element installed. Together the QA/QC program will monitor drill depths, shaft length, grout volume, installation pressures and installation procedures.

6.6.2 Deep Foundations

6.6.2.1 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.

6.6.2.2 Due to the density variations in the soil at the site, the use of fixed-length impact-driven piles can be problematic due to variable embedment depths, 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.

6.6.2.3 Based on the subsurface conditions at the site, we recommend using auger pressure-grout displacement (APGD) piles (also known as drilled displacement piles) or continuous flight auger (CFA) piles for support of the structure. A brief discussion of each pile type follows:

- **Auger Cast Pressure-Grout Displacement (APGD) Piles** are installed using a specialized, plugged auger tool with a displacement body that laterally displaces the soil as it 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. After the auger is removed, the required steel reinforcement is then "wet-set" into the pile to complete the installation. This pile type produces few spoils, approximately 20% or less of the theoretical hole volume.

- **Continuous Flight Auger (CFA) Piles** are similar to APGD piles but are not displacement piles. These 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.

6.6.2.4 From a geotechnical perspective, site soils are well-suited for APGD and CFA piles. CFA and APGD 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. For this site, difficult soil conditions include soft soils on the order of 45 feet thick overlying the medium dense to dense sand bearing layer which will need to be penetrated some distance in order to develop pile capacity. **We strongly recommend performing a comprehensive pile installation**

and load testing program to evaluate constructability as well as capacity. Recommendations are provided in herein.

6.6.2.5 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.

6.6.2.6 Deep foundation systems will generate the majority of their vertical load-carrying capacity from skin friction and some end-bearing in the dense sand and gravel and underlying layers located 45 feet and deeper at the site.

6.6.2.7 Table 6.6.2A presents estimated pile lengths and estimated axial capacities for 18-inch and 24-inch CFA and APGD piles. We note that other pile diameters (such as 16-inch) may also be used, although capacities will be less and longer pile lengths would be needed. **Pile lengths presented in Table 6.6.2 are <u>estimates only</u> based on empirical design procedures. 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 the 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 herein.

TABLE 6.6.2A
AXIAL CAPACITY – ESTIMATED PILE LENGTHS

Pile Type	Estimated Pile Length (feet)	Estimated Allowable Axial Capacity (kips)[1]	Estimated Ultimate Axial Capacity (kips)
18-inch CFA[2] Piles	65	300	600
18-inch APGD[3] Piles	60	300	600
24-inch CFA[2] Piles	55	300	600
24-inch APGD[3] Piles	50	300	600

<u>Notes:</u>
1. *Dead + Live Loading Conditions.*
2. *Continuous Flight Auger Pile (non-displacement)*
3. *Auger Pressure Grout Displacement Pile*

6.6.2.8 Single pile uplift (tension) capacity can be estimated as approximately 50 percent of the allowable downward capacity. Tension piles should be properly reinforced to transfer uplift forces to the pile tip.

6.6.2.9 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.

6.6.2.10 Lateral analysis of the deep foundations may be performed by the pile designer using the computer program LPILE or similar lateral analysis software. Table 6.6.2B summarizes our recommended soil parameters for use in LPILE.

TABLE 6.6.2B
RECOMMENDED SOIL PARAMETERS FOR LPILE ANALYSIS

Soil Layer	Soil Model	Elevation		Effective Unit Weight (pcf)	Undrained Cohesion (psf)	Friction Angle (deg)
		Top	Bottom			
1	Stiff Clay above Groundwater	+15	+10	115	1,000	0
2	Stiff Clay below Groundwater	+10	-25	55	1,000	0
3	Sand	-25	>-25	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

6.6.2.11 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 250 pounds per cubic foot (pcf) equivalent fluid pressure may be used. Frictional resistance along the bottom of the pile caps and grade beams should be neglected.

6.6.2.12 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.

6.6.2.13 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.

6.6.2.14 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.

6.6.2.15 Prior to performing static load testing of the CFA/APGD piles, we recommend performing low-strain impact integrity testing (Pile Integrity Testing or "PIT") in accordance with ASTM D5882. The purpose of the PIT 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 PIT results. As a quality assurance measure during construction, PIT should be performed on 10% of the production piles. The frequency of PIT may be increased by the geotechnical engineer if installation difficulties are encountered. If the results of PIT indicate questionable pile properties, additional high-strain dynamic testing (ASTM D4945) may be required to verify production pile capacity.

6.6.2.16 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, depth into competent material, grout volumes/pressures (for the CFA/APGD piles), and a statement that the foundations can accept the design loading conditions.

6.7 Interior Slabs-on-Grade

6.7.1 Slab thickness and reinforcement should be determined by the structural engineer based on anticipated loading. However, based on our experience, slabs are typically at least 5 inches thick and reinforced with at least No. 4 reinforcing bars placed 18 inches on center, each way. Control joints should be provided at periodic intervals in accordance with American Concrete Institute (ACI) or Portland Cement Association (PCA) recommendations, as appropriate.

6.72 If building pad soils become dry, they should be re-moistened prior to concrete slab-on-grade construction. Building pads should be moistened to at least optimum moisture content at least 48 hours before placing the vapor barrier. Moisture content should be verified by Geocon prior to placing the vapor barrier.

6.8 Concrete Moisture Protection Considerations

6.8.1 Migration of moisture through concrete slabs 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.

6.8.2 In areas where floor coverings are planned, 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.

6.8.3 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.

6.8.4 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.

6.8.5 Proper finishing, curing, and moisture vapor emission testing should be performed in accordance with the latest guidelines provided by the American Concrete Institute, Portland Cement Association, and ASTM.

6.9 Retaining Walls

6.9.1 Design of retaining walls and buried structures may be based on the lateral earth pressures (equivalent fluid pressure) summarized in Table 6.9.

TABLE 6.9
RECOMMENDED LATERAL EARTH PRESSURES

Condition	Equivalent Fluid Density
Active	40 pcf
At-Rest	55 pcf
Passive	250 pcf
Seismic Earth Pressure[1]	10 pcf

1. *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.*

6.9.2 Unrestrained walls should be designed using the active case. Unrestrained walls are those that are allowed to rotate more than 0.001H (where H is the height of the wall). Walls restrained from movement (such as basement walls) should be designed using the at-rest case. 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.

6.9.3 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.

6.10 Concrete Sidewalks and Flatwork

6.10.1 Sidewalk, curb, and gutter within City of West Sacramento right-of-way should be designed and constructed in accordance with the latest City standards and details as applicable.

6.10.2 Onsite exterior concrete flatwork will likely experience seasonal movement. Therefore, some cracking and/or vertical offset should be anticipated. We are providing the following recommendations to reduce distress to concrete flatwork. Recommendations include moisture conditioning subgrade soils, using aggregate underlayment, and providing adequate construction and control joints. It should be noted that even with implementation of these measures, slab movement or cracking could still occur.

- Concrete flatwork and sidewalks should be at least 4 inches thick and underlain by at least 4 inches of Class 2 AB compacted to at least 95% relative compaction. In addition, doweling could be provided at joints to reduce the potential for vertical offset.
- The upper 12 inches of subgrade soil for exterior flatwork and sidewalks should be uniformly moisture-conditioned at or above optimum moisture content and compacted to at least 90% relative compaction prior to placing AB.
- We recommend using construction and control joints in accordance with ACI and/or PCA guidelines. Construction joints that abut building foundations should include a felt strip, or approved equivalent, that extends the full depth of the exterior slab. Exterior slabs should be structurally independent of building foundations except at doorways, where vertical movement could impact doorway operation.

6.11 Hot Mix Asphalt Pavement

6.11.1 We performed Resistance-Value (R-Value) testing on a representative bulk soil sample from proposed at-grade pavement areas. Our testing resulted in an R-Value of 5 (Appendix B). Table 6.11 provides alternative pavement sections based on the design methods of Caltrans' *Highway Design Manual* using a design subgrade R-value of 5.

TABLE 6.11
FLEXIBLE PAVEMENT SECTIONS

	Parking Areas Traffic Index = 5.0	Driveways, Light Truck Traffic, Fire Truck Areas Traffic Index = 6.0
HMA, inches	3.0	3.5
AB, inches	10.0	13.0
Total Section, inches	13.0	16.5

6.11.2 The recommended pavement section is based on the following assumptions:

1. Pavement subgrade soil has an R-Value of at least 5.
2. Class 2 AB has a minimum R-Value of 78 and meets the requirements of Section 26 of Caltrans' *Standard Specifications*.
3. Class 2 AB and the top 6 inches of subgrade are compacted to 95% or higher relative compaction at or near optimum moisture content.

6.11.3 To reduce the potential for water from landscaped areas migrating under pavement into the AB, consideration should be given to using full-depth curbs in areas where pavement abuts irrigated landscaping. The full-depth curbs should extend at least 6 inches or more into the soil subgrade beneath the AB. Alternatively, modified drop-inlets that contain weep-holes may be used to encourage accumulated water to drain from beneath the pavement.

6.11.4 Asphalt pavement section recommendations for driveways and parking areas are based on the design procedures of Caltrans' *Highway Design Manual* (Design Manual), Chapter 600, updated December 20, 2004. It should be noted that most rational pavement design procedures are based

on projected street or highway traffic conditions and, hence, may not be representative of vehicular loading that occurs in parking lots and driveways. Pavement proximity to landscape irrigation, reduced traffic speed and short turning radii increase the potential for pavement distress to occur in parking lots even though the volume of traffic is significantly less than that of an adjacent street. The Design Manual indicates that the resulting pavement sections for parking lots are "minimized to keep initial costs down but are reasonable because additional AC surfacing can be added later, if needed, and generally without incurring traffic hazards or traffic handling problems." It is generally not economically feasible to design and construct the entire parking lot and driveways for the unique loading conditions previously described. Periodic maintenance of the pavement in these areas, therefore, should be anticipated.

6.12 Rigid Concrete Pavement

6.12.1 If rigid PCC pavement is used in automobile and light-truck traffic areas, we recommend that the concrete be at least 6 inches thick. PCC pavement should be underlain by at least 6 inches of Class 2 AB meeting the requirements of Section 26 of Caltrans' *Standard Specifications* and compacted to at least 95% relative compaction. Subgrade soils should be prepared and compacted in accordance with the recommendations of this report.

6.12.2 PCC should have a minimum 28-day compressive strength of 3,500 pounds per square inch (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.

6.12.3 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.

6.13 Site Drainage and Moisture Protection

6.13.1 Adequate 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.

6.13.2 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.

6.13.3 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 use of area drains to collect excess irrigation water and transmit it to drainage structures or impervious above-grade planter boxes. In addition, where landscaping is planned adjacent to the pavement or flatwork, we recommend construction of a cutoff wall (deepened curb) along the edge of the pavement/flatwork that extends at least 4 inches into the soil subgrade below the bottom of the base material.

6.13.4 The soil conditions at the site (low-permeability clays) are not conducive to water infiltration devices such as vegetated swales. However, Low Impact Development (LID) devices can be installed to reduce velocity and the amount of water entering the storm drain system. The LID devices should be properly constructed to prevent water infiltration into the surrounding soil. Water infiltrates the expansive soils, distress may be caused to adjacent pavements, flatwork, or structures. Vegetated swales and basin areas (if used) should be lined with an impermeable liner (e.g. high-density polyethylene, HDPE, with a thickness of about 12 mil or equivalent polyvinyl chloride liner) to reduce infiltration.

6.13.5 We recommend that roof drains be connected to water-tight subdrains that direct the water to the storm drain system. However, we understand that LID and Leadership in Engineering and Environmental Design (LEED) requests disconnecting the roof drains to help obtain certification. 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.

6.13.6 We recommend implementing measures to reduce infiltrating irrigation water near buildings, flatwork, or pavements. Such measures may include:

- Selecting drought-tolerant plants that require little or no irrigation, especially within 3 feet of buildings, slabs-on-grade, or pavements.
- Using drip irrigation or low-output sprinklers.
- Using automatic timers for irrigation systems.
- Using appropriately spaced area drains.

The project landscape architect should consider incorporating these measures into the landscaping plans.

6.13.7 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 Geocon should review the foundation and grading plans 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 (GER) throughout the construction phase and provide 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 or 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.

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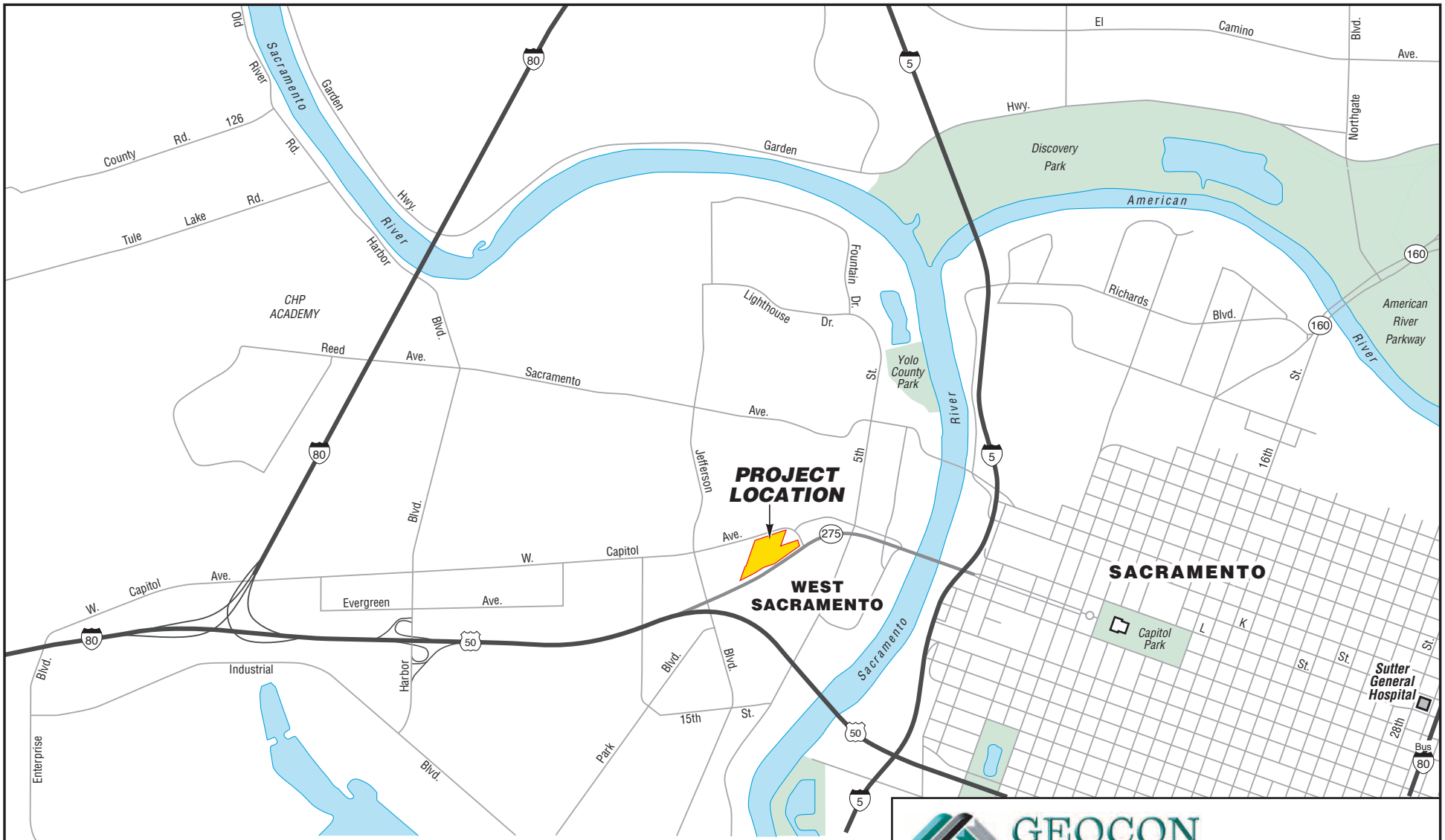

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Dharma Realm Buddhist Temple

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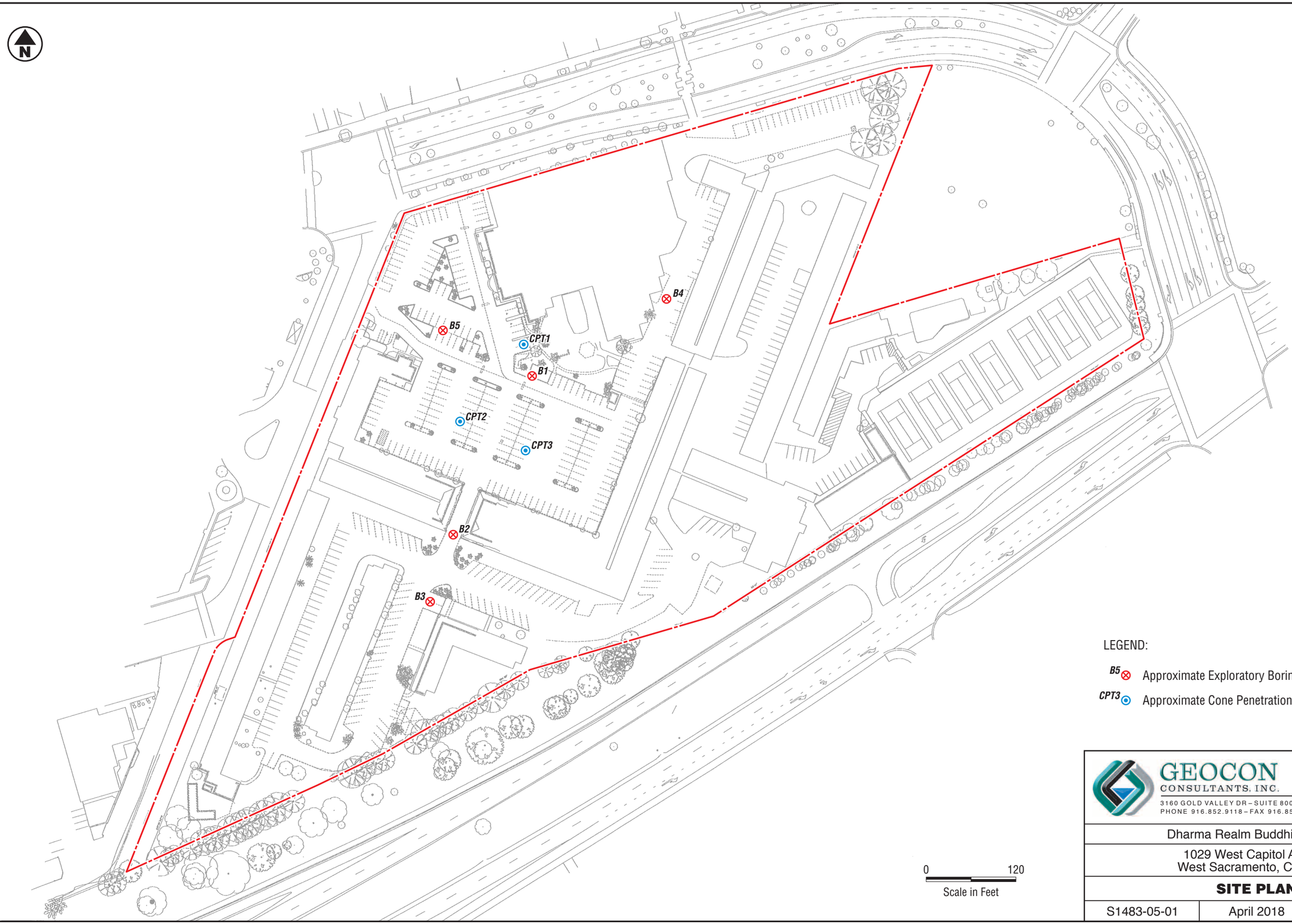
VICINITY MAP

S1483-05-01	April 2018	Figure 1



LEGEND:

B5 ⊗ Approximate Exploratory Boring Location

CPT3 ⊙ Approximate Cone Penetration Test (CPT) Location

0 120

Scale in Feet

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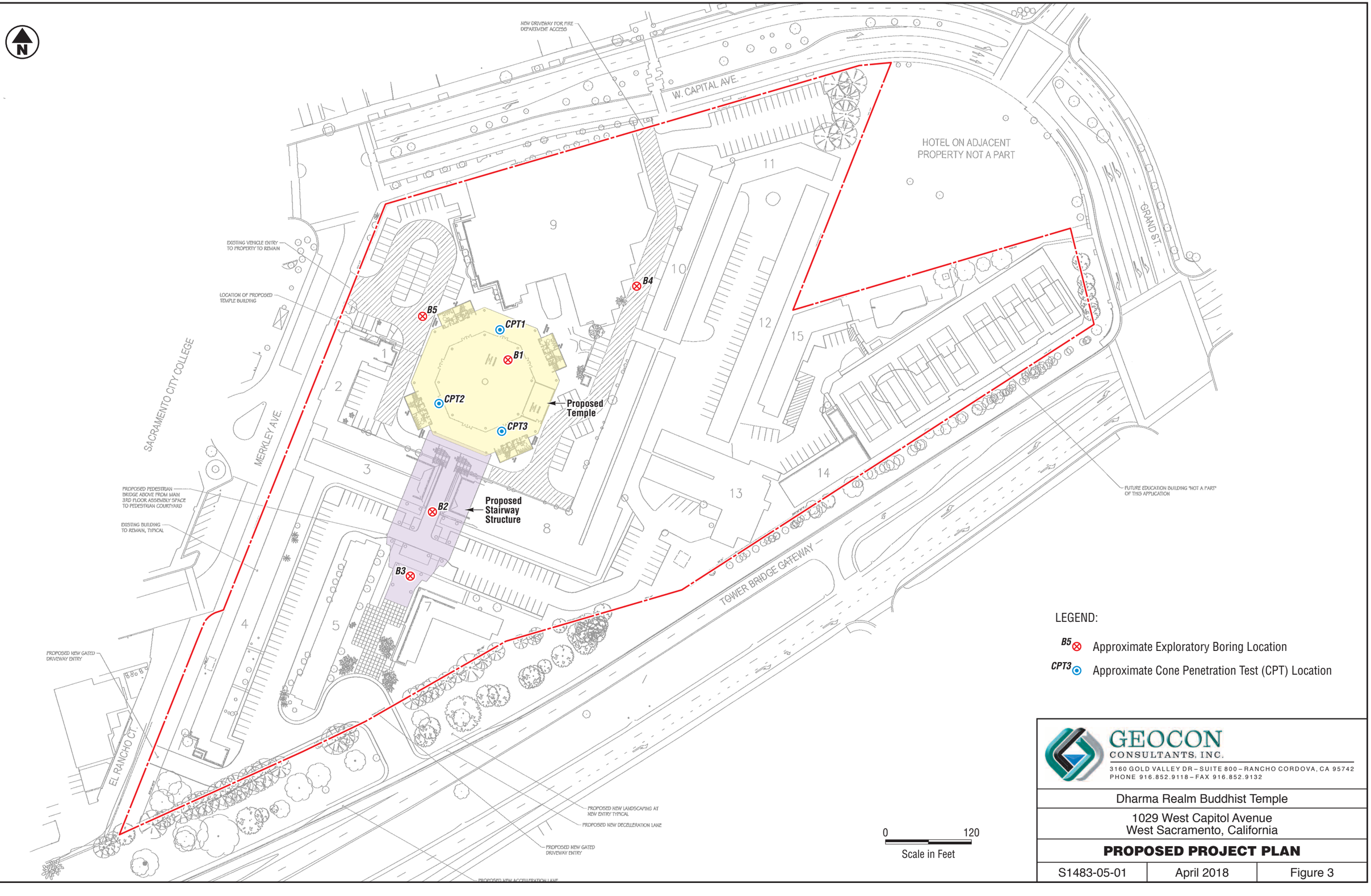
SITE PLAN

S1483-05-01	April 2018	Figure 2



0 120

Scale in Feet

LEGEND:

B5 ⊗ Approximate Exploratory Boring Location

CPT3 ⊙ Approximate Cone Penetration Test (CPT) Location

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West Sacramento, California

PROPOSED PROJECT PLAN

S1483-05-01	April 2018	Figure 3



1 WEST ELEVATION - TEMPLE

NORTH

0 5 10 20 40 FT
SCALE: 1/16" = 1'-0"



2 EAST ELEVATION - TEMPLE

NORTH

0 5 10 20 40 FT.
SCALE: 1/16" = 1'-0"

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PROPOSED PROJECT ELEVATION

S1483-05-01	April 2018	Figure 4



Photo No. 1 Current site conditions



Photo No. 2 Current site conditions

PHOTOS NO. 1 & 2

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S1483-05-01	April 2018	

APPENDIX A

APPENDIX A

FIELD EXPLORATION

We performed our geotechnical field exploration on February 1 and 7, 2018. Our field exploration program consisted of drilling five exploratory borings (B1 through B9) and advancing three cone penetration soundings (CPT1 through CPT3) at the approximate locations depicted on the Site Plan, Figure 2 and Proposed Project Plan, Figure 3.

Borings were performed using a track-mounted CME 75 drill rig equipped with 8-inch outside diameter (OD) hollow stem augers. Soil sampling was performed using an automatic 140-pound hammer with a 30-inch drop. We obtained samples using a 3-inch OD split-spoon (California Modified) sampler. We recorded the number of blows required to drive the sampler the last 12 inches (or portion thereof) of the 18-inch sampling interval on the boring logs. Upon completion, the borings were backfilled with either neat cement grout (boring B1) or soil cuttings generated from the borings (borings B2 through B7).

The CPTs were performed using a 20-ton truck-mounted rig. CPT parameters including tip resistance (q_c), sleeve friction (f_s) and dynamic pore pressure (U) were measured at approximately 2-inch intervals as the cone advanced. Soil behavior types were determined using correlations developed by Robertson and Campanella (1988). The cone was equipped with a seismic transducer and shear wave velocity measurements were also collected at approximate 5-foot intervals in CPT1.

We visually examined, classified, and logged the subsurface conditions in the exploratory borings 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 we obtained samples. 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, drill rig penetration rates, excavation characteristics, and other factors. The transition between materials may be abrupt or gradual. Where applicable, we revised the field logs based on subsequent laboratory testing.

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≤S<50	DAMP
INDICATION OF MOISTURE; NO VISIBLE WATER	50≤S<75	MOIST
MINOR VISIBLE FREE WATER	75≤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

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 ⅝-INCH TO 1 FOOT	MODERATELY BEDDED
1 ⅛-INCH TO 3 ⅝-INCH	THINLY BEDDED
⅝-INCH TO 1 ⅛-INCH	VERY THINLY BEDDED
LESS THAN ⅝-INCH	LAMINATED

STRUCTURE DESCRIPTIONS

CRITERIA	DESCRIPTION
ALTERNATING LAYERS OF VARYING MATERIAL OR COLOR WITH LAYERS AT LEAST ⅛-INCH THICK	STRATIFIED
ALTERNATING LAYERS OF VARYING MATERIAL OR COLOR WITH LAYERS LESS THAN ⅛-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
⅛-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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KEY TO LOGS Figure 3

PROJECT NO. **S1483-05-01** PROJECT NAME **Dharma Realm Buddhist Temple**

BORING B1

ELEV. (MSL.) **N/A** DATE COMPLETED **2/7/2018**

ENG./GEO. **Sean Dixon** DRILLER **All Well Abandonment**

EQUIPMENT **CME 75 LAR** 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			**ASPHALT CONCRETE (AC)** 1 inch				
1	B1-Bulk	CL	**AGGREGATE BASE (AB)** 8 inches				PI, CP, CR
2			**ALLUVIUM** Very stiff, moist, gray, Lean CLAY				
3	B1-3.0		- PP = 3.0 tsf				
4	B1-3.5			9	98.9	22.9	
5							
6	B1-5.5 B1-6.0		- light brown - PP = 3.5 tsf	16			CU
7		SP	Loose, wet, gray, Poorly graded SAND, some Silt, fine grained				
8	B1-8.0						
9	B1-8.5			10	95.0	32.2	
10							
11	B1-10.5 B1-10.5			10			
12		CL	Medium stiff, wet, gray, Lean CLAY				
13	B1-13.0				89.6	35.2	
14	B1-13.5			9			
15		SM	Loose, wet, gray, Silty SAND				
16	B1-15.5 B1-16.0		- no recovery	3			
17							
18							
19							
20	B1-20.0			2			
21		CL	Stiff, wet, gray, Lean CLAY				
22							
23							
24							
25	B1-25.0		- PP = 1.75 tsf	4			
26							
27							
28							
29							

Figure A2, Log of Boring, page 1 of 2

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. **S1483-05-01** PROJECT NAME **Dharma Realm Buddhist Temple**

BORING B1

ELEV. (MSL.) **N/A** DATE COMPLETED **2/7/2018**

ENG./GEO. **Sean Dixon** DRILLER **All Well Abandonment**

EQUIPMENT **CME 75 LAR** 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
30	B1-30.0			CL	- soft to medium stiff, PP = 0.5 tsf	7			PI
31									
32									
33									
34									
35	B1-35.0				- very soft	2			
36									
37									
38									
39									
40	B1-40.0				- medium stiff	6			
41									
					BORING TERMINATED AT 41.5 FEET GROUNDWATER ENCOUNTERED AT 8 FEET BACKFILLED WITH SOIL CUTTINGS				

Figure A3, Log of Boring, page 2 of 2

IN PROGRESS S1483-05-01 DHARMA REALM BUDDHIST TEMPLE.GPJ 03/28/18

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. **S1483-05-01** PROJECT NAME **Dharma Realm Buddhist Temple**

BORING B2

ELEV. (MSL.) **N/A** DATE COMPLETED **2/7/2018**

ENG./GEO. **Sean Dixon** DRILLER **All Well Abandonment**

EQUIPMENT **CME 75 LAR** 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			**ASPHALT CONCRETE (AC)** 1 inch				
1	B2-Bulk	SM	**AGGREGATE BASE (AB)** 8 inches				CP, R
2			**ALLUVIUM**				
3	B2-3.0		Loose, moist, brown, Silty SAND, fine grained				
4	B2-3.5			11	90.9	21.6	
5		CL	Very stiff, moist, light brown, Sandy lean CLAY				
6	B2-5.5 B2-6.0		- PP = 2.5 tsf	13	103.8	22.5	PI
7							
8	B2-8.0						
9	B2-8.5		- stiff, PP = 1.5 tsf	3			CU
10	NR			0			
11							
12		SM	Loose, wet, gray, Silty SAND				
13							
14							
15	B2-15.0			7			GSA
16							
17							
18							
19		SP	Medium dense, wet, gray, Poorly graded SAND, medium grained				
20							
21	B2-20.5 B2-21.0			17	101.4	24.4	
22							
23		CL	Stiff, moist, gray, Lean CLAY				
24							
25							
26	B2-25.5 B2-26.0		- PP = 1.25 tsf	12			
			BORING TERMINATED AT 26.5 FEET GROUNDWATER ENCOUNTERED AT 13 FEET BACKFILLED WITH SOIL CUTTINGS				

Figure A4, Log of Boring, page 1 of 1

IN PROGRESS S1483-05-01 DHARMA REALM BUDDHIST TEMPLE.GPJ 03/28/18

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. **S1483-05-01** PROJECT NAME **Dharma Realm Buddhist Temple**

BORING B3

ELEV. (MSL.) **N/A** DATE COMPLETED **2/7/2018**

ENG./GEO. **Sean Dixon** DRILLER **All Well Abandonment**

EQUIPMENT **CME 75 LAR** 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					**ASPHALT CONCRETE (AC)** 2 inches				
1	B4-Bulk			CL	**AGGREGATE BASE (AB)** 8 inches				CP
2					**ALLUVIUM**				
3	B3-3.0				Stiff, moist, gray, Lean CLAY, fine grained				
4	B3-3.5					15			
5									
6	B3-5.5 B3-6.0				- very stiff, PP = 4.0 tsf	20	94.7	24.9	PI
7									
8	B3-8.0				- stiff, PP = 1.5 tsf				
9	B3-8.5					14	96.8	27.3	
10									
11	B3-10.5 B3-11.0		▼		- very soft, wet, Sandy, PP = 0.0 tsf	7			
12									
13									
14									
15									
16	B3-15.5 B3-16.0				- without Sand - PP = 0.0 tsf	2			
17									
18									
19									
20									
21	B3-20.5 B3-21.0				- 2 feet heave - PP = 0.0 tsf	2			CU
22									
23									
24									
25									
26	B3-25.5 B3-26.0				- stiff to very stiff, PP = 2.0 tsf	6			
					BORING TERMINATED AT 26.5 FEET GROUNDWATER ENCOUNTERED AT 11 FEET BACKFILLED WITH SOIL CUTTINGS				

Figure A5, Log of Boring, page 1 of 1

IN PROGRESS S1483-05-01 DHARMA REALM BUDDHIST TEMPLE.GPJ 03/28/18

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. **S1483-05-01** PROJECT NAME **Dharma Realm Buddhist Temple**

BORING B4

ELEV. (MSL.) **N/A** DATE COMPLETED **2/7/2018**

ENG./GEO. **Sean Dixon** DRILLER **All Well Abandonment**

EQUIPMENT **CME 75 LAR** 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					**ASPHALT CONCRETE (AC)** 1 inch				
1					**AGGREGATE BASE (AB)** 11 inches				
2	B4-1.5 B4-2.0 B4-2.5			SM	**ALLUVIUM** Medium dense, moist, light brown, Silty SAND, fine grained	17	86.2	31.6	
3				CL		5			
4					Medium stiff, moist, gray, Lean CLAY				
					BORING TERMINATED AT 4.0 FEET GROUNDWATER NOT ENCOUNTERED BACKFILLED WITH SOIL CUTTINGS				

Figure A6, Log of Boring, page 1 of 1

IN PROGRESS S1483-05-01 DHARMA REALM BUDDHIST TEMPLE.GPJ 03/28/18

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. **S1483-05-01** PROJECT NAME **Dharma Realm Buddhist Temple**

BORING B5

ELEV. (MSL.) **N/A** DATE COMPLETED **2/7/2018**

ENG./GEO. **Sean Dixon** DRILLER **All Well Abandonment**

EQUIPMENT **CME 75 LAR** 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					**ASPHALT CONCRETE (AC)** 1 inch				
1					**AGGREGATE BASE (AB)** 11 inches				
2	B5-1.5 B5-2.0 B5-2.5			SM	**ALLUVIUM** Medium dense, moist, light brown, Silty SAND, fine grained - loose	21 4	86.9	18.1	
3									
4					BORING TERMINATED AT 4.0 FEET GROUNDWATER NOT ENCOUNTERED BACKFILLED WITH SOIL CUTTINGS				

Figure A7, Log of Boring, page 1 of 1

IN PROGRESS S1483-05-01 DHARMA REALM BUDDHIST TEMPLE.GPJ 03/28/18

GEOCON

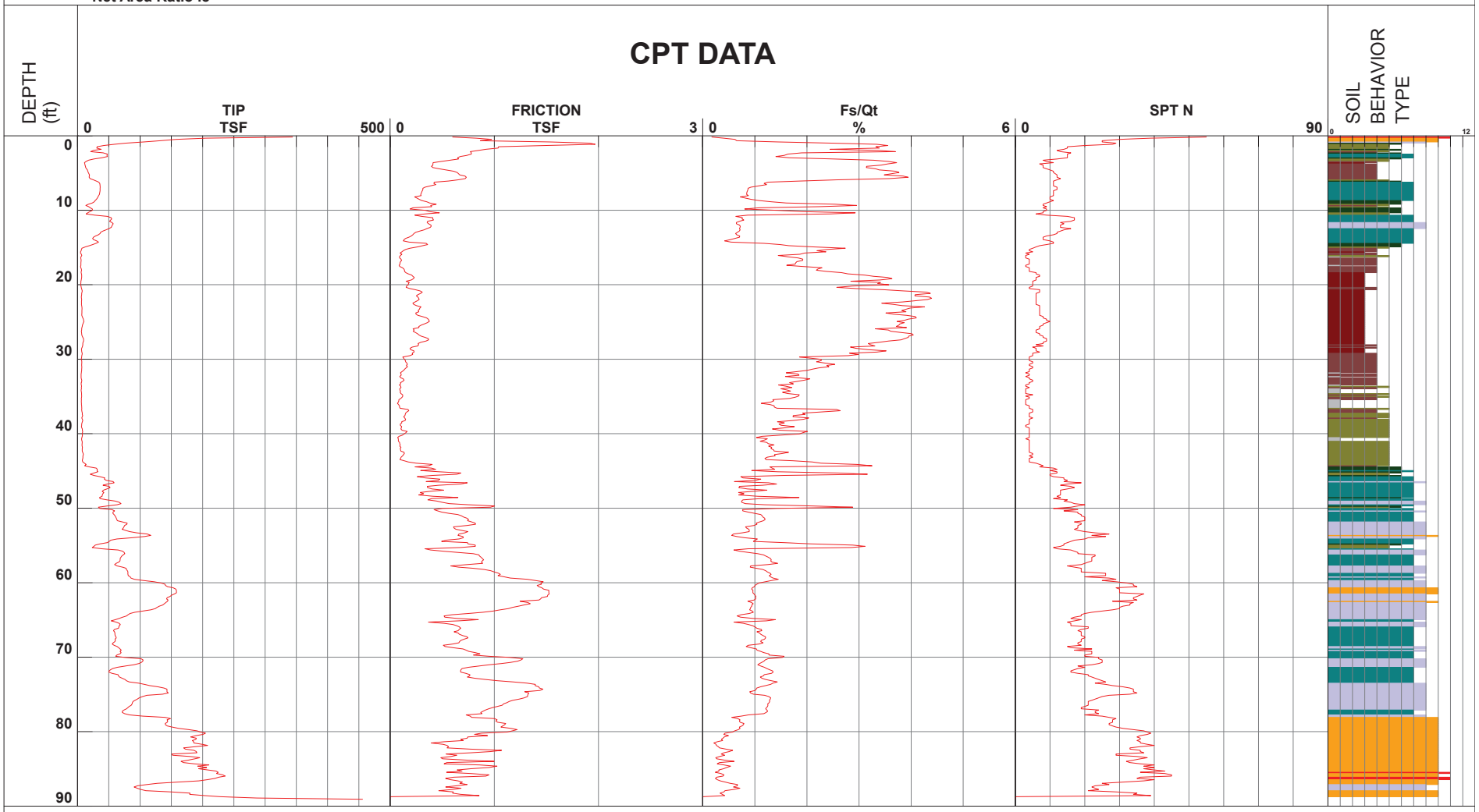
SAMPLE SYMBOLS

- ☐ ... 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

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 10cm squared

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

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CPT1

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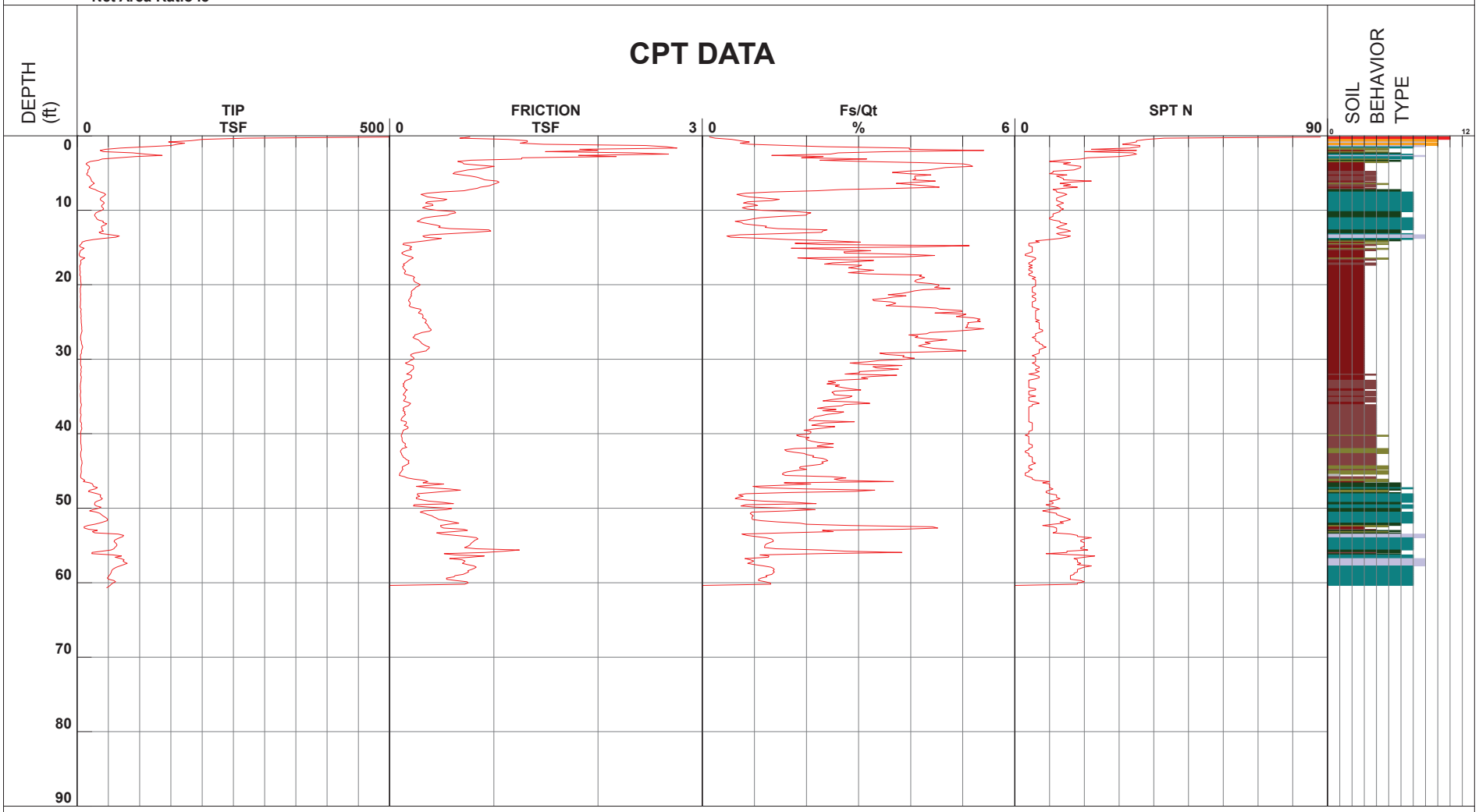
1029 West Capitol Avenue
West Sacramento, California

S1483-05-01	April 2018	Figure A8

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 10cm squared

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

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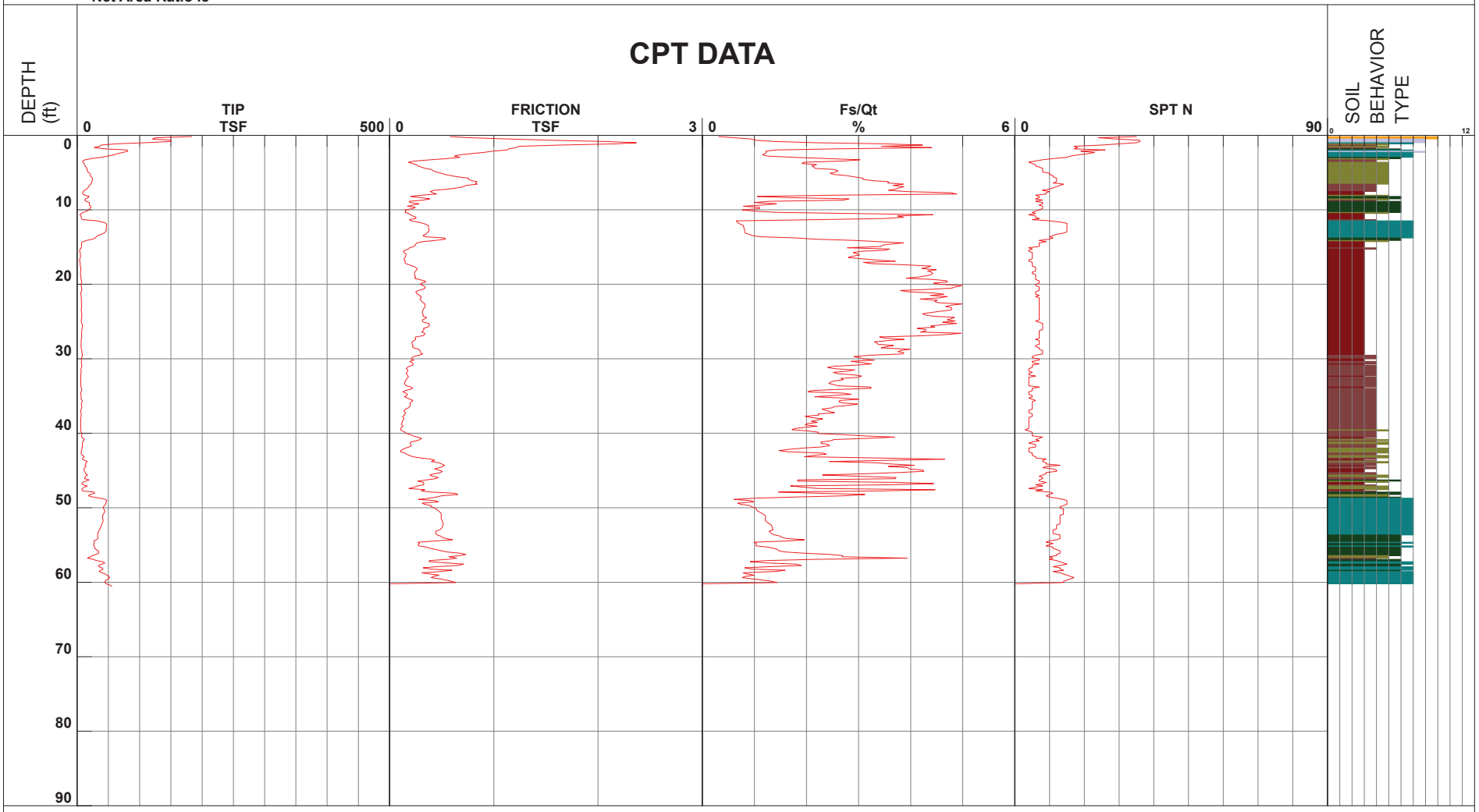
CPT2

Dharma Realm Buddhist Temple
1029 West Capitol Avenue
West Sacramento, California

S1483-05-01	April 2018	Figure A9

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained
2 - organic material
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11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 10cm squared

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

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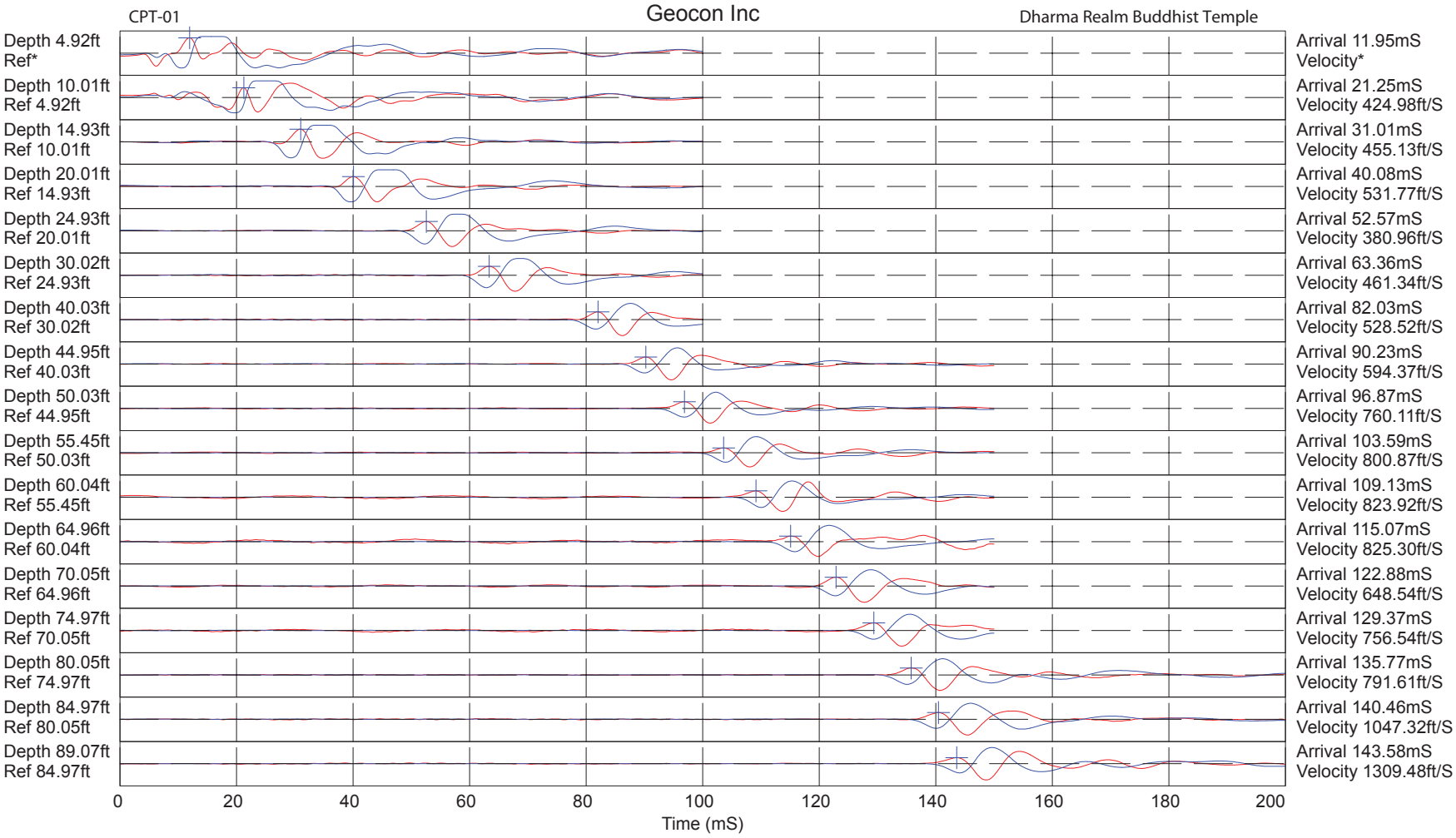
CPT3

Dharma Realm Buddhist Temple
1029 West Capitol Avenue
West Sacramento, California

S1483-05-01	April 2018	Figure A10



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

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Shear Wave Velocity Plots – CPT1

Dharma Realm Buddhist Temple
1029 West Capitol Avenue
West Sacramento, California

S1483-05-01	April 2018	Figure A11

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-place dry density and moisture content, plasticity characteristics, grain size distribution, corrosion potential, and pavement support characteristics. The results of the laboratory tests are presented on the following pages.

TABLE B1
EXPANSION INDEX TEST RESULTS
ASTM D4829

Sample Number	Depth (feet)	Moisture Content (%)		Expansion Index	Classification*
		Before Test	After Test		
B1-Bulk	0 to 5	11.1	24.9	44	Low

**Expansion Potential Classification per ASTM D4829.*

TABLE B2
SOIL 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	0 to 5	7.7	2,140	14.7 / 0.00147%	70.4 / 0.007%

*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.

Sample ID	Depth (feet)	Liquid Limit	Plastic Limit	Plasticity Index	Expansion Index	%<#200 Sieve	Water Content (%)	Dry Density (pcf)
B1-Bulk	1-5	37	21	16	43			
B1-3.5	3.5						22.9	98.9
B1-8.5	8.5						32.2	95.0
B1-13	13					76.3	35.2	89.6
B1-30	30	57	25	32				
B2-3.5	3.5						21.6	90.9
B2-5.5	5.5	32	18	14				
B2-6	6					76.2	22.5	103.8
B2-15	15					25.4		
B2-20.5	20.5					6.4	24.4	101.4
B3-5.5	5.5	45	19	26				
B3-6	6						24.9	94.7
B3-8.5	8.5						27.3	96.8
B4-2	2						31.6	86.2
B5-2	2						18.1	86.9

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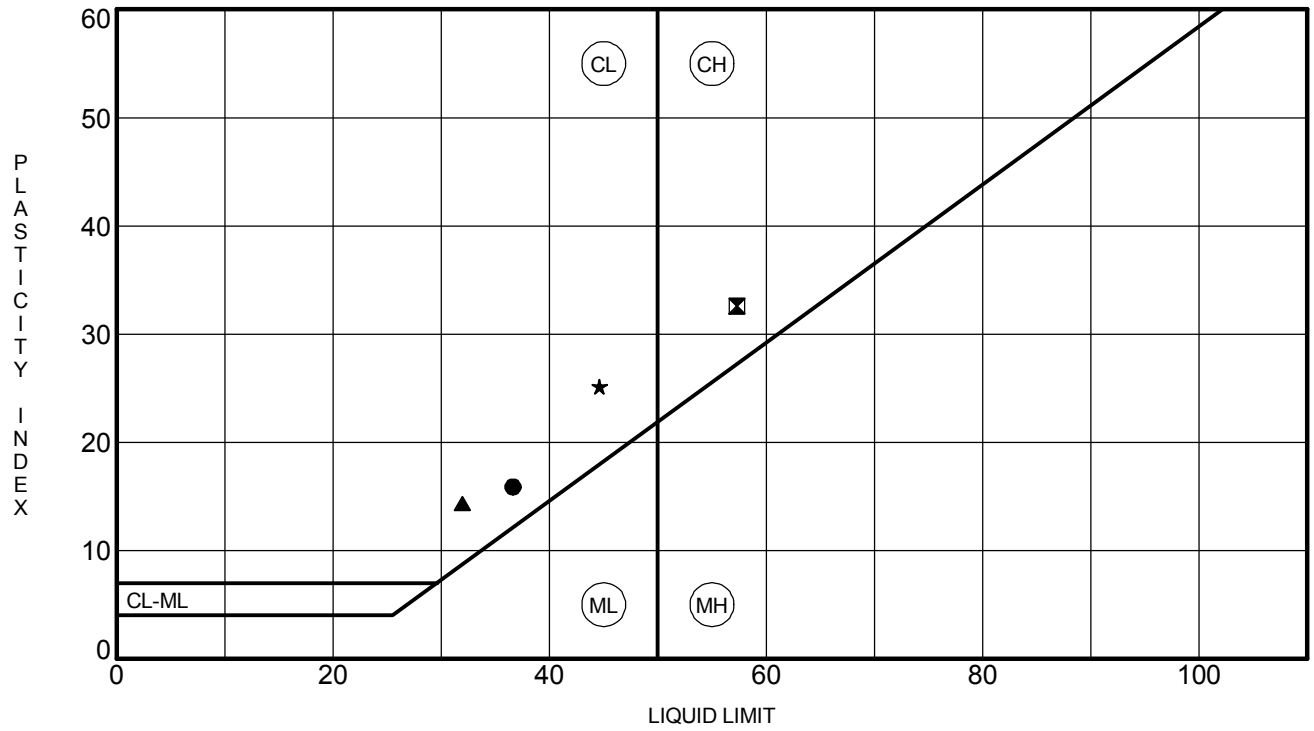
Summary of Laboratory Results

Project: Dharma Realm Buddhist Temple
Location: West Sacramento, CA
Number: S1483-05-01
Figure: B1

US LAB SUMMARY GEOTECH 2 WITH EI COLUMN S1483-05-01 DHARMA REALM BUDDHIST TEMPLE.GPJ US_LAB.GDT 3/21/18



	Sample No.	Liquid Limit	Plastic Limit	Plasticity Index	% Pass #200 Sieve	Unified Soil Classification Description	Preparation Method
●	**B1-Bulk**	**37**	**21**	**16**		**Lean CLAY (CL)**	**dry**
⊠	**B1-30**	**57**	**25**	**32**		**Fat CLAY (CH)**	**dry**
▲	**B2-5.5**	**32**	**18**	**14**		**Lean CLAY (CL)**	**dry**
★	**B3-5.5**	**45**	**19**	**26**		**Lean CLAY (CL)**	**dry**

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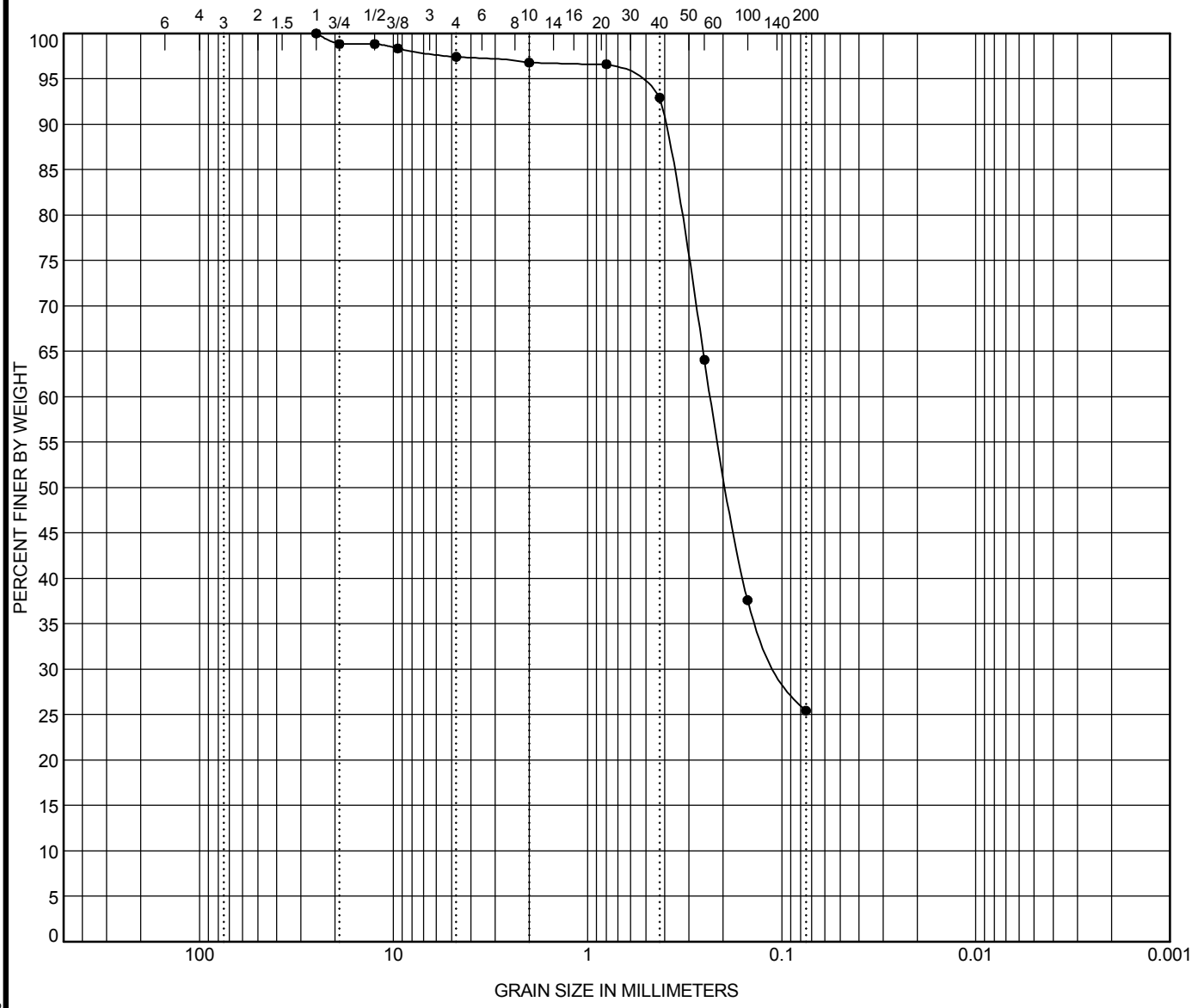
ATTERBERG LIMITS (ASTM D4318)

Project: Dharma Realm Buddhist Temple
Location: West Sacramento, CA
Number: S1483-05-01
Figure: B2

PI COPY 2 S1483-05-01 DHARMA REALM BUDDHIST TEMPLE.GPJ US_LAB.GDT 3/21/18



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample No.	Classification	LL	PL	PI	Cc	Cu
●	**B2-15**	**Silty SAND (SM)**					

	Sample No.	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B2-15**	**25**	**0.231**	**0.097**		**2.6**	**72.0**	**25.4**	

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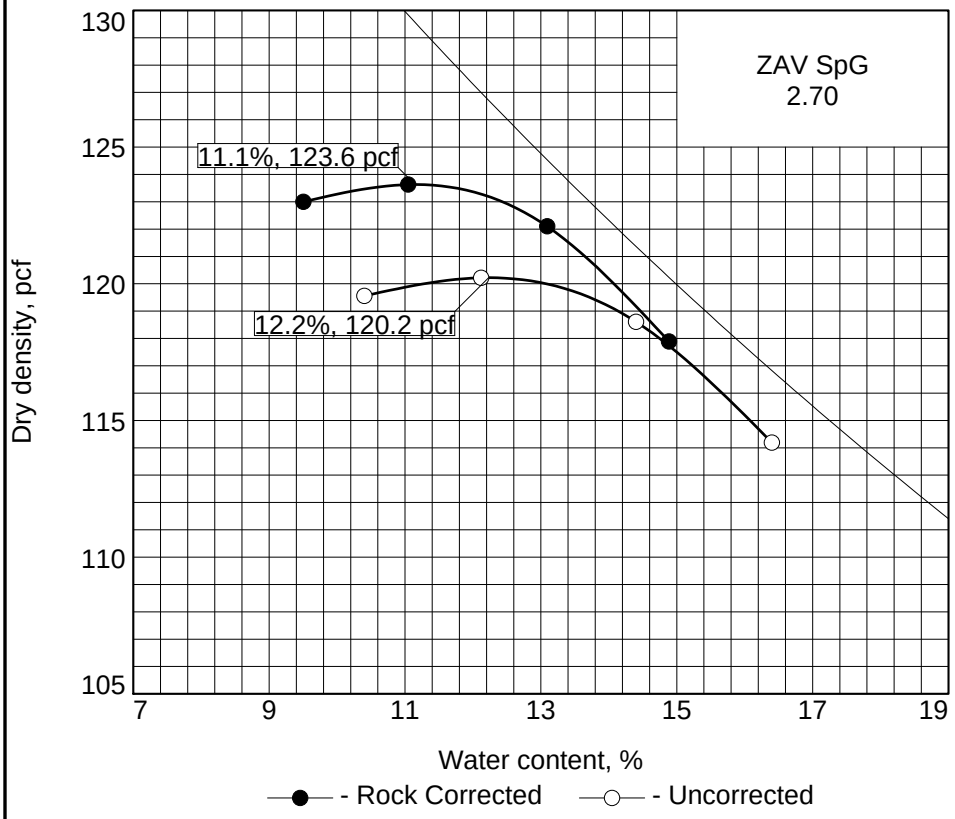
GRAIN SIZE DISTRIBUTION (ASTM D422, D6913)

Project: Dharma Realm Buddhist Temple
Location: West Sacramento, CA
Number: S1483-05-01
Figure: B3

GRAIN SIZE COPY 2 S1483-05-01 DHARMA REALM BUDDHIST TEMPLE.GPJ US_LAB.GDT 3/21/18

COMPACTION TEST REPORT



Curve No. 1

Test Specification:
ASTM 1557 Method A 2017
ASTM D4718-15 Oversize Corr. Applied to Each Test Point

Preparation Method
Hammer Wt. 10
Hammer Drop 18
Number of Layers 5
Blows per Layer 25
Mold Size 0.03327 cu. ft.

Test Performed on Material Passing #4 **Sieve**

NM **LL** **PI**
Sp.G. (ASTM D 854) 2.7
%>#4 10.3 **%<No.200**
USCS **AASHTO**
Date Sampled
Date Tested 2/15/18
Tested By JH

TESTING DATA

	1	2	3	4	5	6
WM + WS	4072.7	4086.3	4044.5	4030.5		
WM	2038.6	2038.6	2038.6	2038.6		
WW + T #1	2263.0	114.4	116.4	110.4		
WD + T #1	2044.5	100.0	100.0	100.0		
TARE #1	241.6	0.0	0.0	0.0		
WW + T #2						
WD + T #2						
TARE #2						
MOISTURE	11.0	13.1	14.9	9.5		
DRY DENSITY	123.6	122.1	117.9	123.0		

ROCK CORRECTED TEST RESULTS	UNCORRECTED
Maximum dry density = 123.6 pcf	120.2 pcf
Optimum moisture = 11.1 %	12.2 %

Material Description
Light Brown Silty Clay w/gravel, Combined Bulks

Remarks:

Project No. S1483-05-01 **Client:**
Project: City of Dharma Realm

○ **Sample Number:** B1,2,3 Bulks

Checked by: MR
Title: Lab Manager

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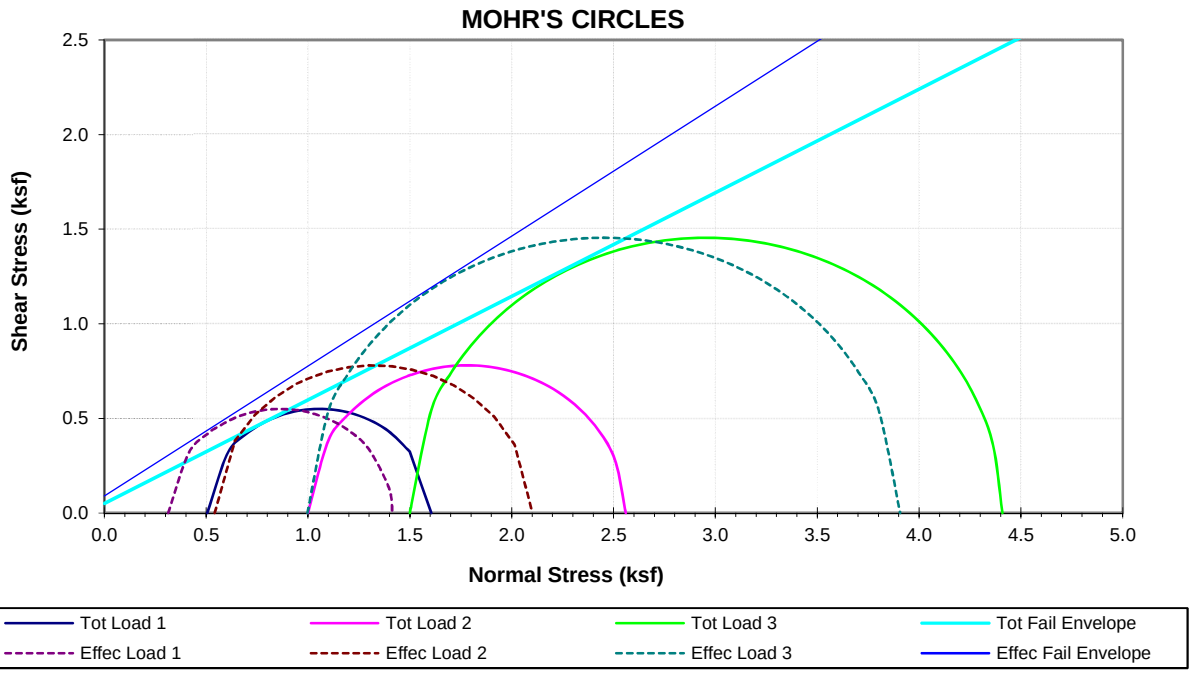
Figure B4

Consolidated Undrained Triaxial Compression - ICU Test ASTM D4767

Boring Number	B1
Sample Number	B1-6
Sample Description	Dark olive gray Sandy lean CLAY



Test Results, At Maximum Principal Stress Ratio		Total		Effective
	Friction Angle ϕ (degrees)	**28.7**		**34.5**
	cohesion (psf)	**50**		**90**
Initial Conditions at Start of Test		stage 1	stage 2	stage 3
Sample ID (psf), Initial Confining Pressure		**500**	**1000**	**1500**
	Height (inch)	4.900	4.840	4.797
	Diameter (inch)	2.405	2.416	2.416
	Moisture Content (%)	19.7	--	--
	Dry Density (pcf)	106.7	--	--
	Saturation (%)	89.1	--	--
After Saturation				
	Dry Density (pcf)	98.9	--	--
After Consolidation				
	Dry Density (pcf)	107.0	--	--
Shear Test Conditions				
	Dry Density (pcf)	107.0	107.9	108.3
	Moisture Content (%)	--	--	20.5
	Saturation (%)	--	--	96.2
	Strain rate (%/hr)	1.91	1.99	2.03
	Cell pressure (psf)	6330	6730	7320
	Initial Back Pressure (psf)	5820	5730	5820
	Initial Effective Confining Pressure (psf)	500	1000	1500
	Total Major Principal Stress At Failure (psf)	1600	2560	4410
	Effective Major Principal Stress At Failure (psf)	1410	2100	3910
	Pore Pressure At Failure (psf)	190	460	500
	Effective Minor Principal Stress At Failure (psf)	310	540	1000

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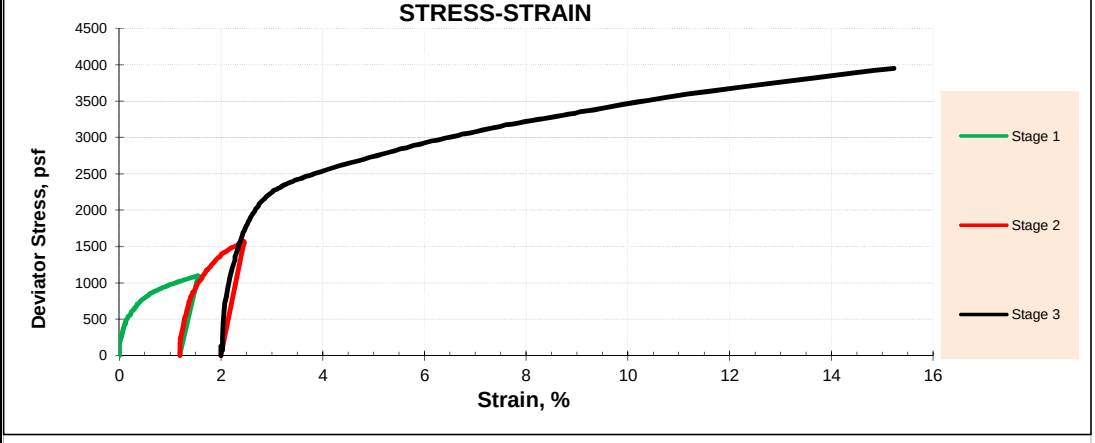
Triaxial Shear Strength - CU Test, ASTM D4767 with Pore Pressure Measurements (staged)

Project: City of Dharma Realm
Location: West Sacramento, CA
Number: S1483-05-01
Figure: B5

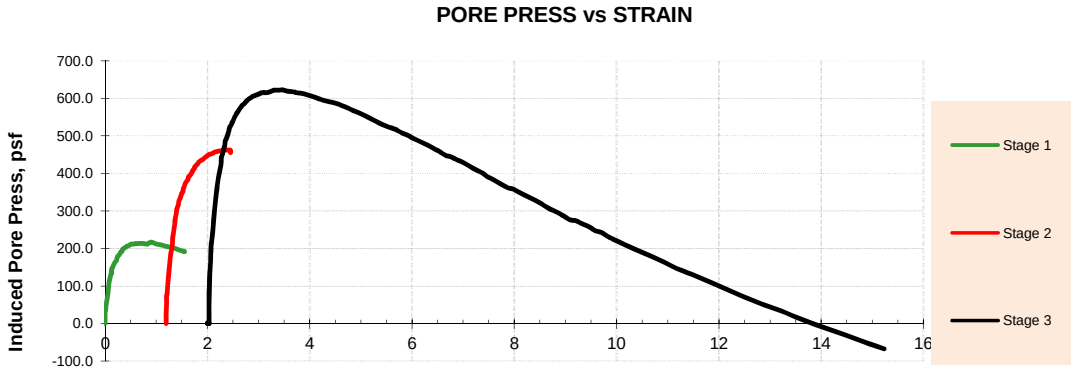
CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION - ICU TEST ASTM D4767

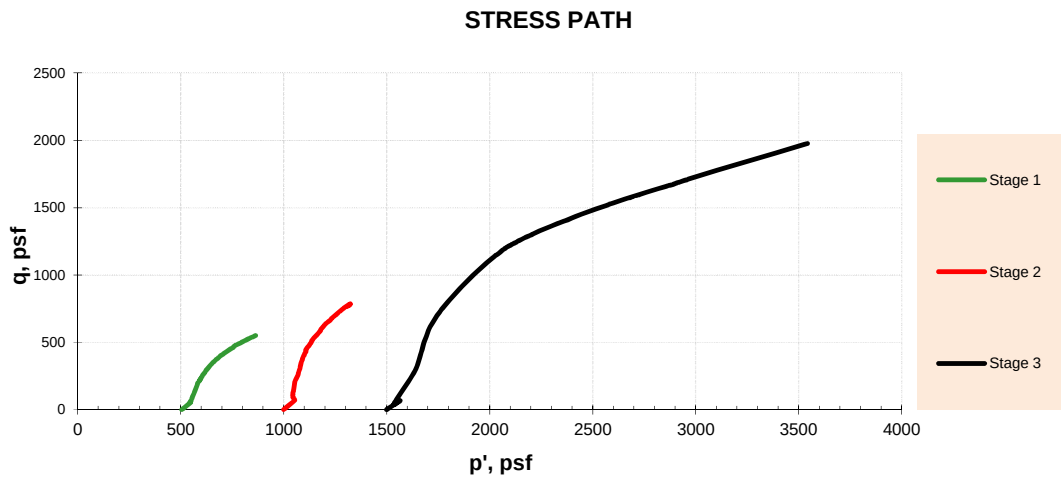
Boring Number	B1
Sample Number	B1-6
Sample Description	Dark olive gray Sandy lean CLAY



After shear photo







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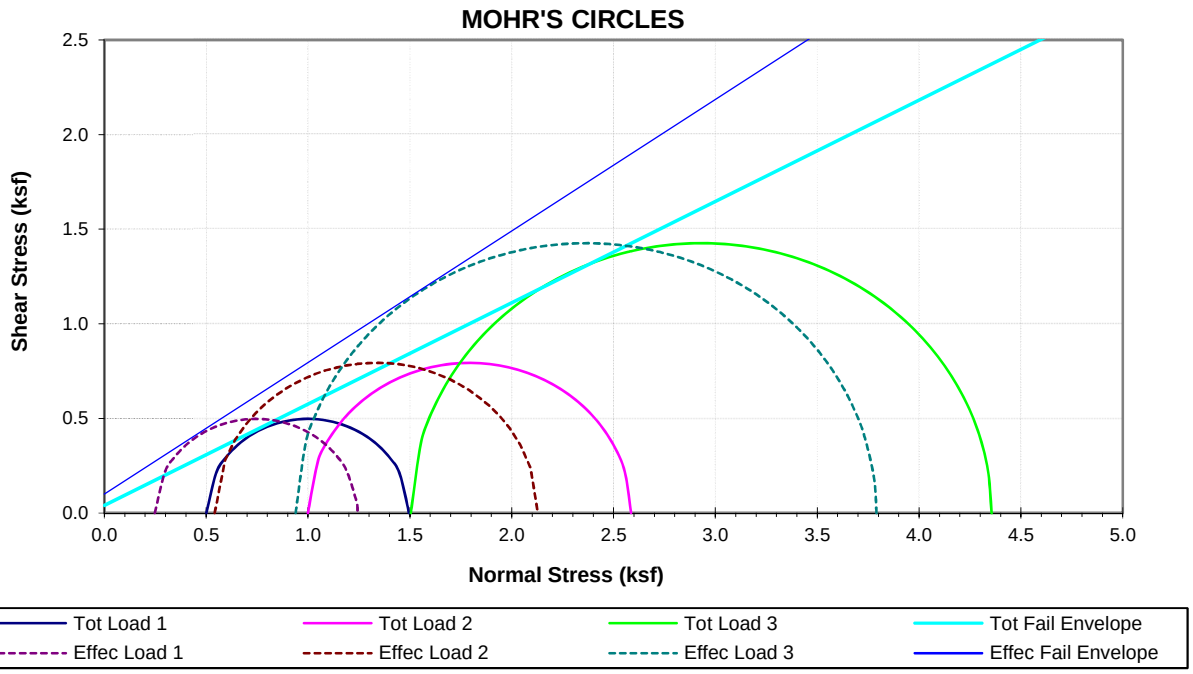
Triaxial Shear Strength - CU Test, ASTM D4767 with pore pressure measurements

Project: City of Dharma Realm
Location: West Sacramento, CA
Number: S1483-05-01
Figure: B5

Consolidated Undrained Triaxial Compression - ICU Test ASTM D4767

Boring Number	B2
Sample Number	B2-8.5
Sample Description	Dark brown Clayey SAND



Test Results, At Maximum Principal Stress Ratio		Total		Effective
	Friction Angle ϕ (degrees)	**28.2**		**34.8**
	cohesion (psf)	**40**		**100**
Initial Conditions at Start of Test		stage 1	stage 2	stage 3
Sample ID (psf), Initial Confining Pressure		**500**	**1000**	**1500**
	Height (inch)	4.820	4.739	4.694
	Diameter (inch)	2.378	2.393	2.396
	Moisture Content (%)	32.4	--	--
	Dry Density (pcf)	93.2	--	--
	Saturation (%)	104.7	--	--
After Saturation				
	Dry Density (pcf)	85.9	--	--
After Consolidation				
	Dry Density (pcf)	93.6	--	--
Shear Test Conditions				
	Dry Density (pcf)	93.6	94.3	95.0
	Moisture Content (%)	--	--	29.7
	Saturation (%)	--	--	100.1
	Strain rate (%/hr)	1.98	2.07	2.00
	Cell pressure (psf)	6250	6730	7240
	Initial Back Pressure (psf)	5750	5730	5740
	Initial Effective Confining Pressure (psf)	500	1000	1500
	Total Major Principal Stress At Failure (psf)	1500	2590	4360
	Effective Major Principal Stress At Failure (psf)	1240	2130	3790
	Pore Pressure At Failure (psf)	250	460	570
	Effective Minor Principal Stress At Failure (psf)	250	540	940

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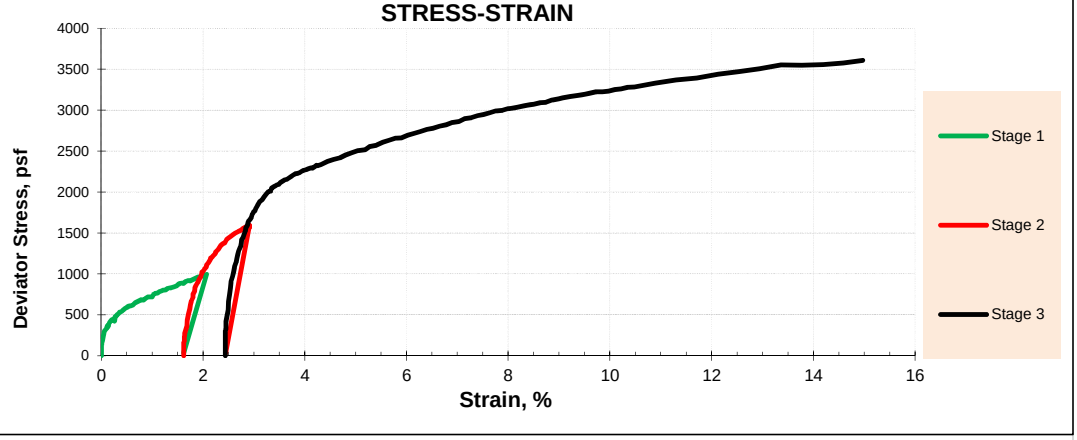
Triaxial Shear Strength - CU Test, ASTM D4767 with Pore Pressure Measurements (staged)

Project: City of Dharma Realm
Location: West Sacramento, CA
Number: S1482-05-01
Figure: B6

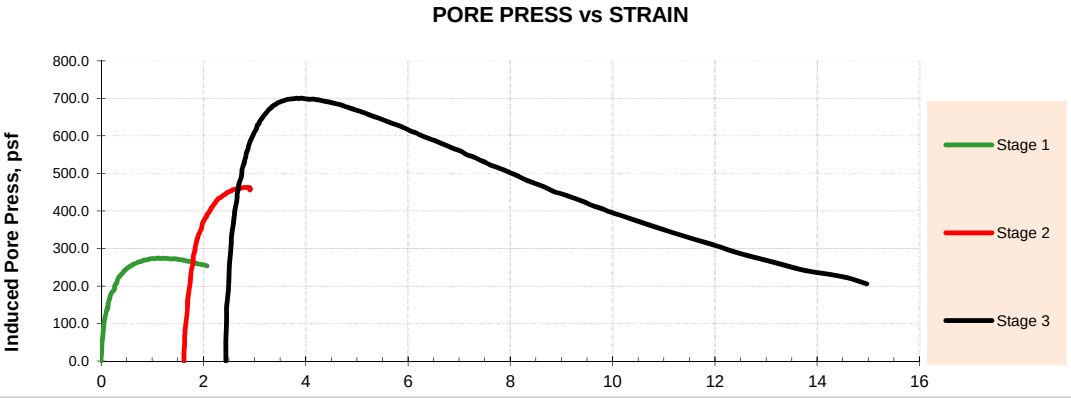
CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION - ICU TEST ASTM D4767

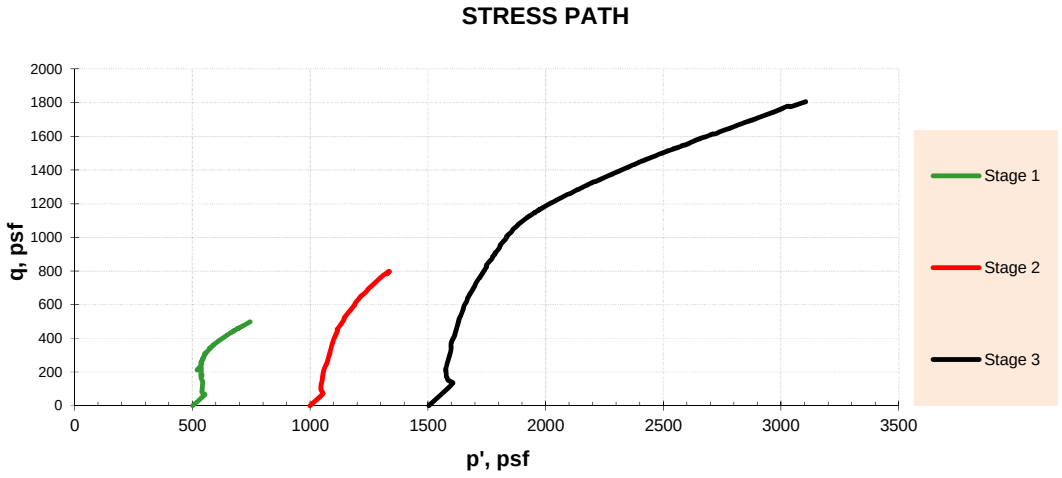
Boring Number	B2
Sample Number	B2-8.5
Sample Description	Dark brown Clayey SAND



After shear photo







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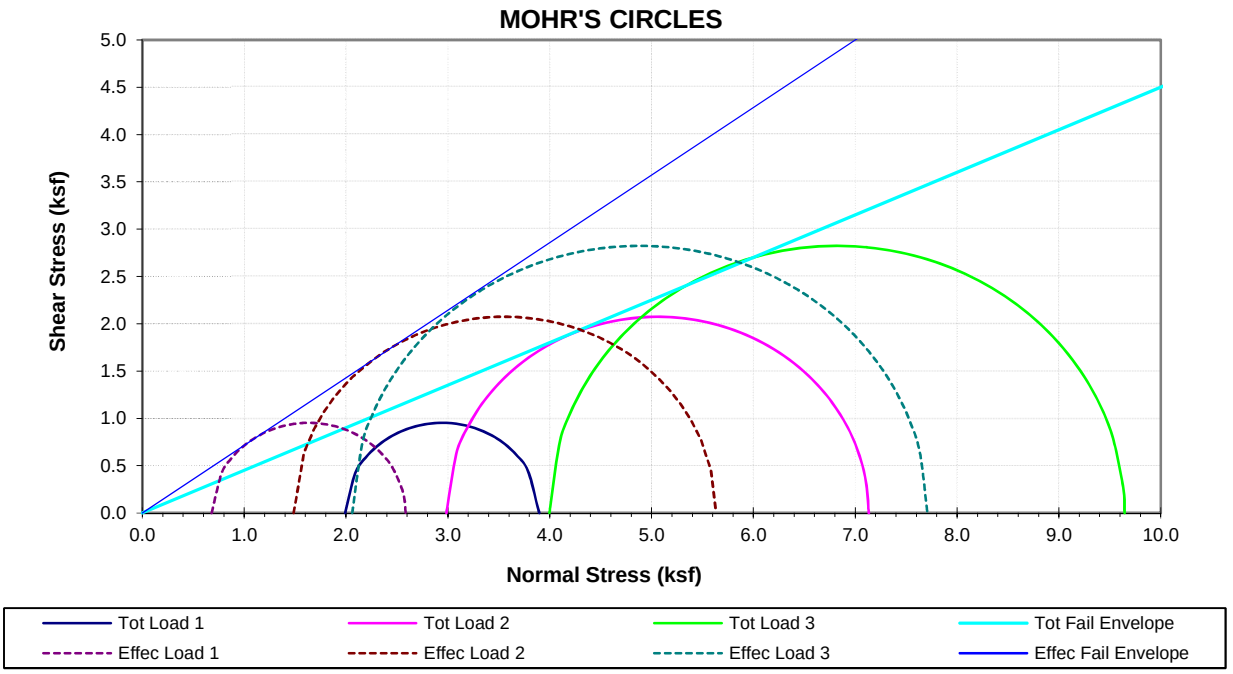
Triaxial Shear Strength - CU Test, ASTM D4767 with pore pressure measurements

Project: City of Dharma Realm
Location: West Sacramento, CA
Number: S1482-05-01
Figure: B6

Consolidated Undrained Triaxial Compression - ICU Test ASTM D4767

Boring Number	B3
Sample Number	B3-21
Sample Description	Dark bluish gray Elastic SILT



Test Results, At Maximum Principal Stress Ratio	Total	Effective
Friction Angle ϕ (degrees)	**24.2**	**35.5**
cohesion (psf)	**0**	**0**

Initial Conditions at Start of Test	stage 1	stage 2	stage 3
Sample ID (psf), Initial Confining Pressure	**2000**	**3000**	**4000**
Height (inch)	4.770	4.582	4.493
Diameter (inch)	2.426	2.414	2.432
Moisture Content (%)	40.6	--	--
Dry Density (pcf)	82.9	--	--
Saturation (%)	110.4	--	--
After Saturation			
Dry Density (pcf)	80.3	--	--
After Consolidation			
Dry Density (pcf)	87.1	--	--
Shear Test Conditions			
Dry Density (pcf)	87.1	87.5	88.4
Moisture Content (%)	--	--	30.6
Saturation (%)	--	--	95.1
Strain rate (%/hr)	0.71	0.75	0.68
Cell pressure (psf)	4900	5910	6900
Initial Back Pressure (psf)	2910	2930	2900
Initial Effective Confining Pressure (psf)	1990	2990	4000
Total Major Principal Stress At Failure (psf)	3900	7130	9640
Effective Major Principal Stress At Failure (psf)	2580	5630	7710
Pore Pressure At Failure (psf)	1310	1500	1940
Effective Minor Principal Stress At Failure (psf)	680	1490	2060

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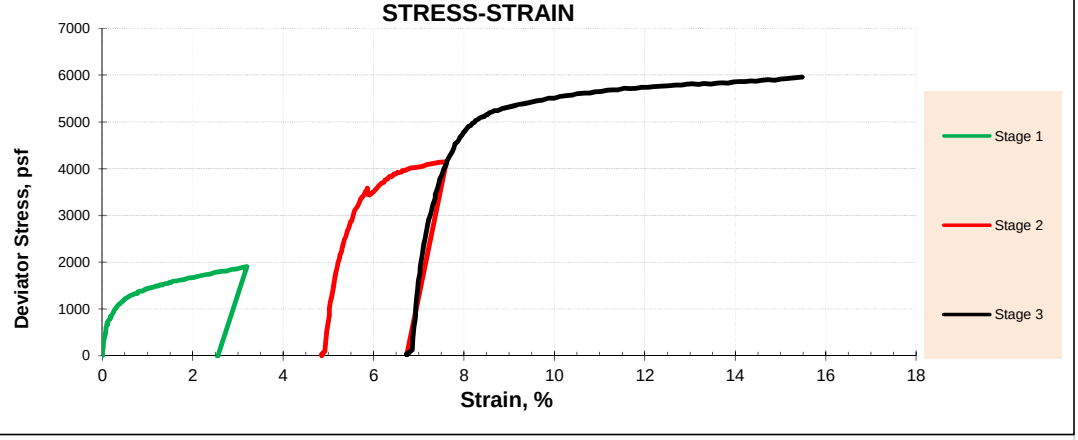
Triaxial Shear Strength - CU Test, ASTM D4767 with Pore Pressure Measurements (staged)

Project: City of Dharma
Location: West Sacramento, CA
Number: S1483-05-01
Figure: B7

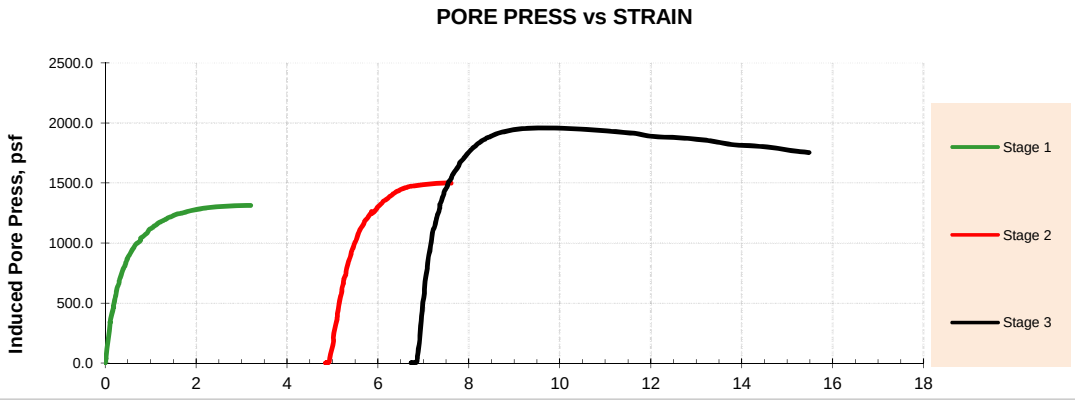
CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION - ICU TEST ASTM D4767

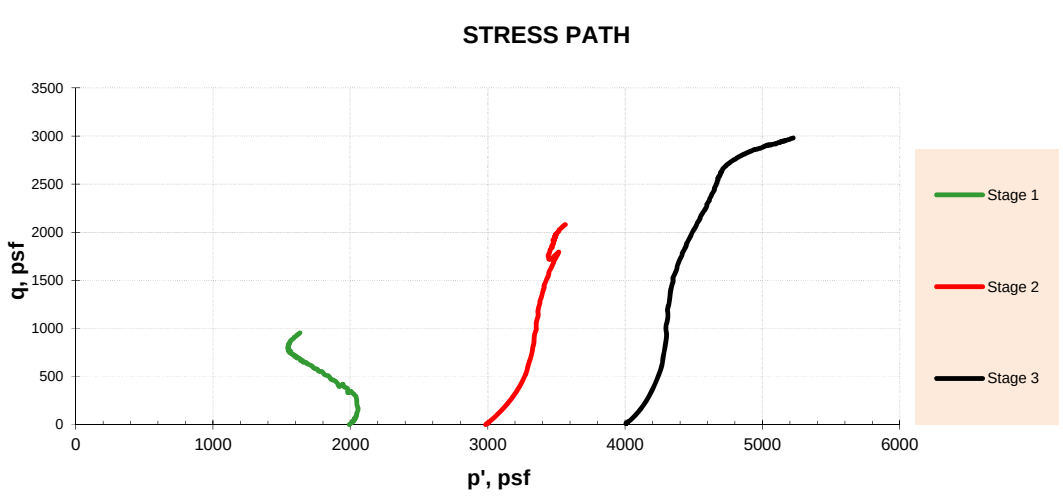
Boring Number	B3
Sample Number	B3-21
Sample Description	Dark bluish gray Elastic SILT



After shear photo







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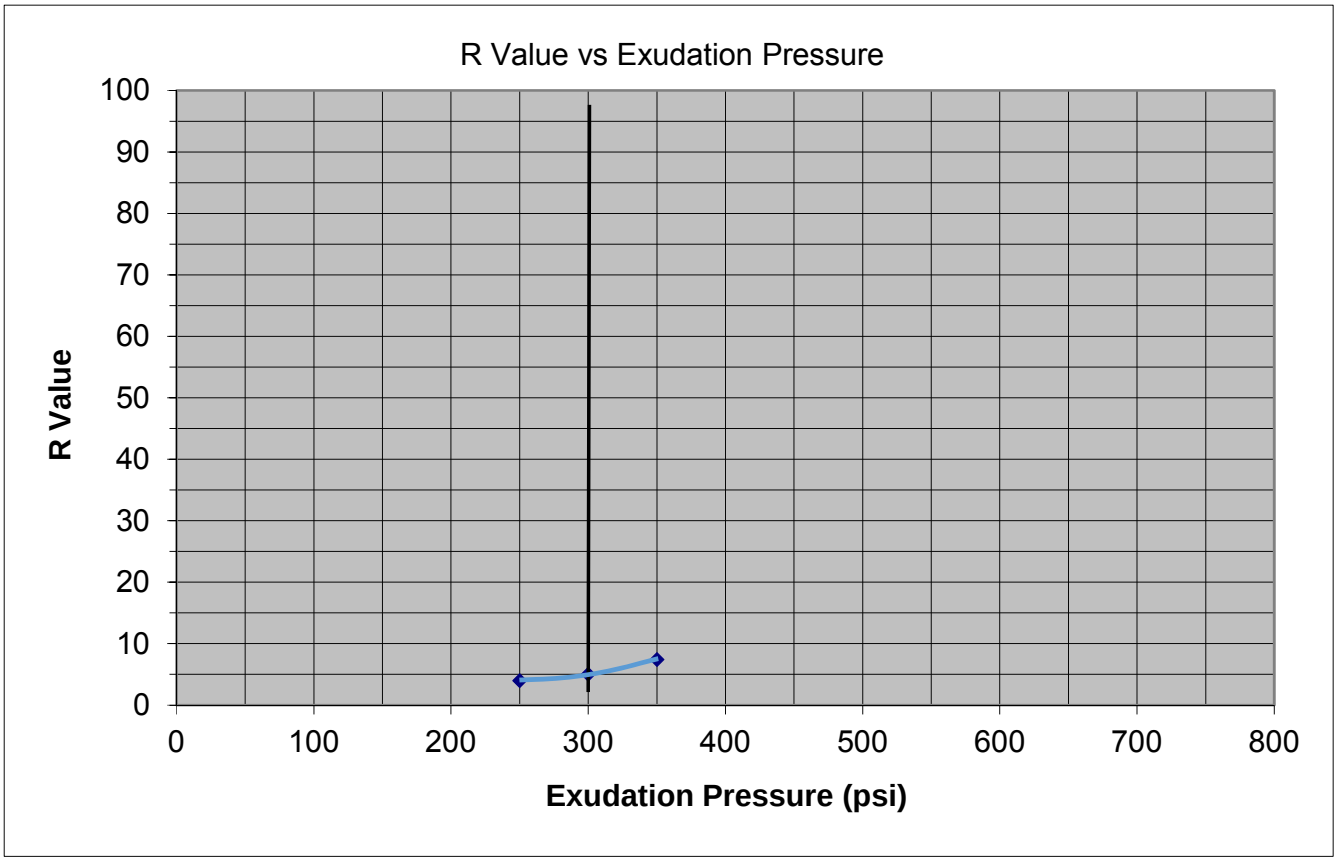
Triaxial Shear Strength - CU Test, ASTM D4767 with pore pressure measurements

Project: City of Dharma
Location: West Sacramento, CA
Number: S1483-05-01
Figure: B7



Sample ID & Description				
Boring Number	B2-Bulk			
Sample Depth (feet)	--			
Material Description	Dark gray CLAY			

Test Data			
Specimen	2044	2054	2111
Exudation Pressure (psi)	350	250	300
Expansion Dial (.0001")	49	6	27
Expansion Pressure (psf)	212.2	26.0	116.9
Resistance 'R' Value	7	4	5
Moisture at test (%)	20.7	22.4	21.5
Dry density at test (pcf)	100.7	100.4	103.5
R Value at 300 psi exudation pressure			**5**
R Value by expansion pressure (TI=5.0)			**6**
R Value by Equillibrium			**5**

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R Value By Exudation

Project:	Dharma Realm
Location:	West Sacramento, CA
Number:	S1483-05-01
Figure:	B8

APPENDIX C

APPENDIX C
LIQUEFACTION ANALYSIS

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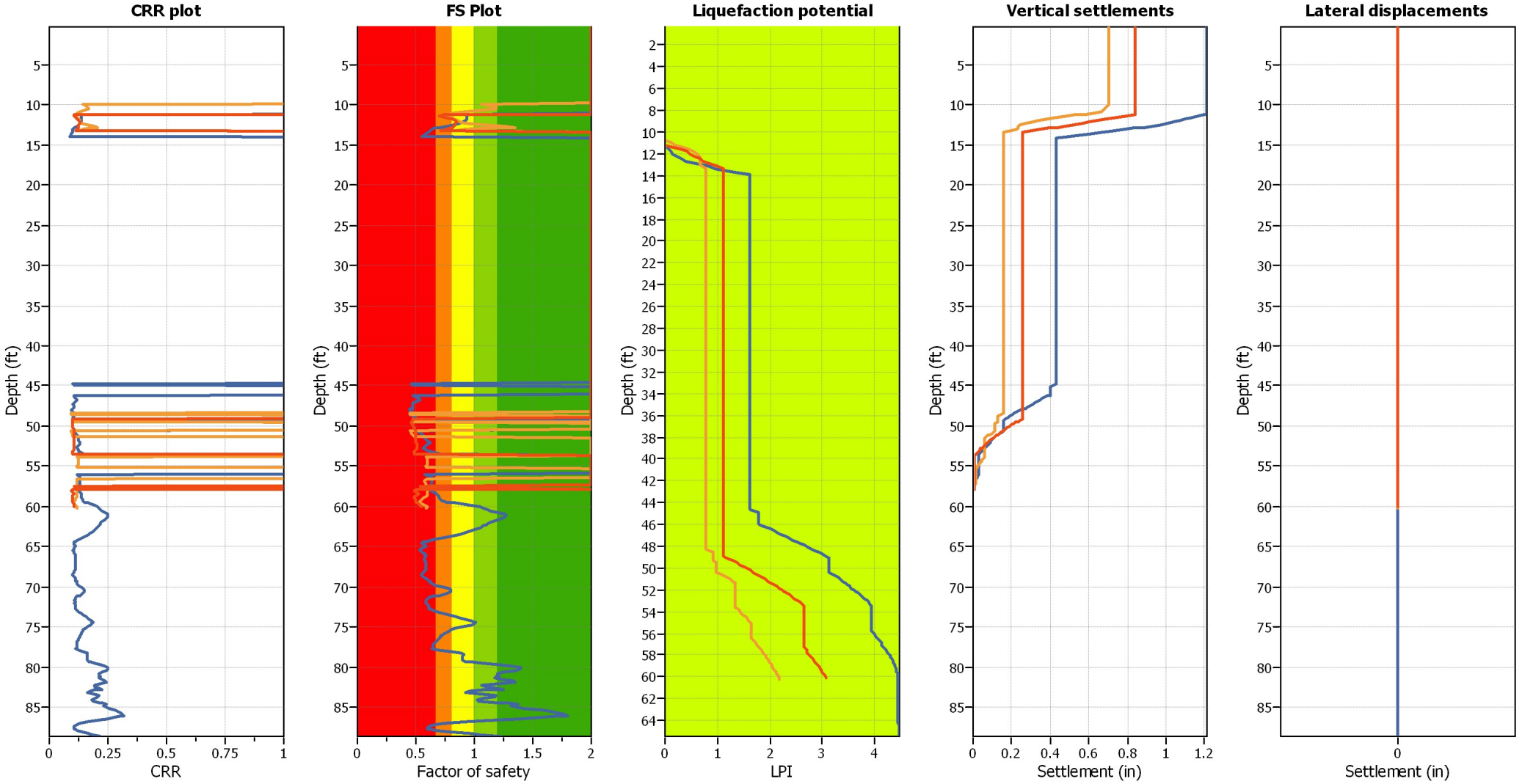
Project: Dharma Realm

Overlay Cyclic Liquefaction Plots



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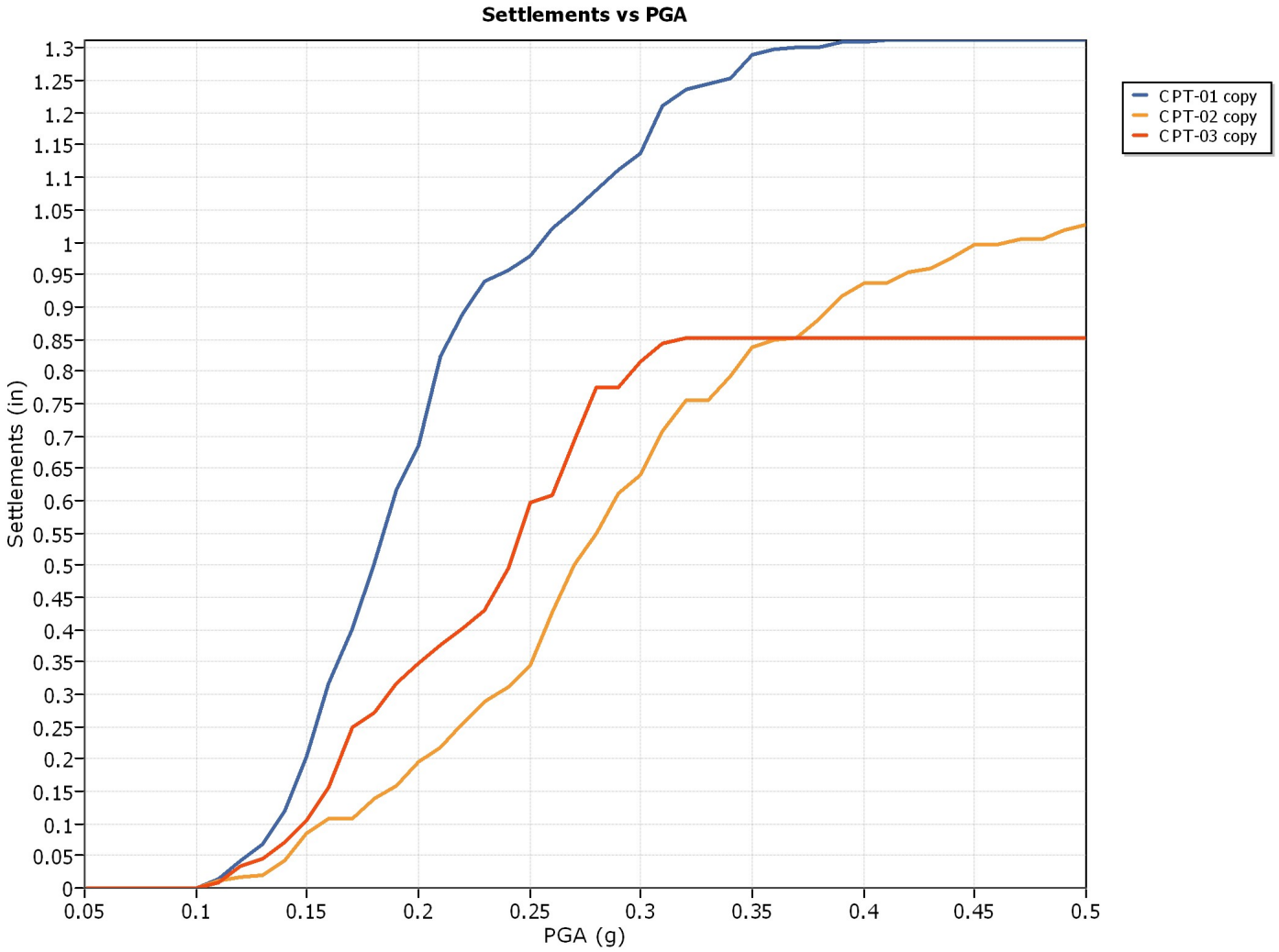
PGA Based Parametric Analysis



:: CPT main liquefaction parameters details ::

CPT Name	Assesment method	Earthquake Mag.	GWT in situ (ft)	GWT earthq. (ft)
CPT-01 copy	NCEER (1998)	6.51	15.00	10.00
CPT-02 copy	NCEER (1998)	6.51	15.00	10.00
CPT-03 copy	NCEER (1998)	6.51	15.00	10.00