Appendix D
Geotechnical Report

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GEOTECHNICAL ENGINEERING REPORT
PROPOSED SELF STORAGE FACILITY
21101 VENTURA BOULEVARD
WOODLAND HILLS, LOS ANGELES, CALIFORNIA

Prepared for: **JOHNSON DEVELOPMENT ASSOCIATES, INC.**
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Project Number 13589.002

August 4, 2022

Leighton Consulting, Inc.
A Leighton Group Company

August 4, 2022

Project No. 13589.002

Johnson Development Associates, Inc.
101 North Pacific Coast Highway, Suite 308
El Segundo, California 90245

Attention: Mr. Brian Kearney, West Coast Development Manager

Subject: Geotechnical Engineering Report
Proposed Self Storage Facility
21101 Ventura Boulevard
Woodland Hills (Los Angeles), California

In accordance with our proposal dated May 31, 2022, Leighton Consulting, Inc. (Leighton) is pleased to present this geotechnical engineering report for the subject property located at 21101 Ventura Boulevard, Woodland Hills, Los Angeles, California (the site). The purpose of our study was to evaluate the subsurface soil and groundwater conditions at the site to provide geotechnical recommendations for the design and construction of the proposed concept as currently planned. Several site-specific geotechnical reports provided for two separate design concepts were reviewed in preparation of this report. Applicable data used from those reports is included herein. The results of our literature review, exploration, findings, conclusions and recommendations are presented in this report.

We sincerely appreciate the opportunity to provide our services. If you have any questions or concerns, please contact us at your convenience. The undersigned can be reached at **(866) *LEIGHTON***, specifically at the phone extensions and e-mail addresses listed below.

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Respectfully submitted,

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TABLE OF CONTENTS

Section **Page**

TABLES

ATTACHMENTS

1.0 INTRODUCTION

1.1 Site Description

The subject property is located at 21101 Ventura Boulevard in the Woodland Hills district of the city of Los Angeles, California (site). The project site is a rectangular parcel of land identified by the Los Angeles County Assessor's Office designated the site as a portion of Assessor Parcel Number (APN) 2167-001-010. The overall property is approximately 2.5 acres in size and is currently occupied by 60,000 square foot, 6-story, cast-in-place non-ductile concrete moment frame tower constructed in 1966. Ancillary improvements include asphalt paved parking lot, landscaping, and a pool located north of the tower.

The 0.58 acre project self-storage site is bounded on the north by Clarendon Street, on the east by Alhama Drive, the south by Ventura Boulevard, and the west by a car dealership. U.S. Highway 101 (i.e., 101 Freeway) is located just north and parallel to Clarendon Street. The site location (latitude 34.168144°, longitude -118.59299°) and immediate vicinity are shown on Figure 1, *Site Location Map*. According to the United States Geological Survey (USGS) 7.5-Minute Canoga Park Quadrangle (USGS, 1981), the site surface is relatively flat with an approximate elevation (El.) of ±900 feet mean sea level (msl).

1.1.1 Aerial Imagery and Topography

Review of internet-based aerial imagery (NETR, 2022) including the earliest recorded photographs and topographic maps indicate the site was used for agriculture beginning sometime in the 1940's. Sporadic rows of orchards are visible fronting Ventura Boulevard. The current hotel development circa 1966 was first observed in a photograph dated 1967. Topographic maps produced in 1928 for the Canoga Park Quadrangle indicate a naturally occurring mounded hill in the northwest region of the site at approximate elevation (El.) +900 feet. As discussed in this report, the topographic feature observed in this quadrangle map appears be consistent with the encountered subsurface conditions in which bedrock was encountered at shallow depths (5 to 8 feet bgs) in the northwest descending to approximately 64 feet below ground surface in the northeast along Alhama Drive.

1.1.2 Existing Site Grades

Review of the A.L.T.A. Survey prepared by Surveying and Drafting Services Inc. (April, 2021) for the proposed development indicates existing site grades are relatively flat and level with adjacent roadways. Ground surface elevations within the footprint of the proposed structure descend northerly and range from approximately El. 899-896 feet in the northeastern region to El. 900 to 896 in southwestern region of the subject site.

1.2 Proposed Development

Based on review of conceptual plan (Ware Malcomb, July 3, 2022), we understand the proposed 0.58 development footprint includes the construction of a new 6-story self-storage facility with a footprint of approximately 21,086 square feet (ft^2). The ground floor of the proposed structure will consist of at-grade parking, with 6,774 ft^2 of office with the remainder of the overall footprint consisting of open-air vehicle parking below the upper five levels of the building. A copy of the architectural schematic site plan is included in Appendix A-1 for reference. The overall footprint of the proposed building shown on Plate 1, *Geotechnical Map* included at the end of the report.

Structural load information for the proposed building had not yet been determined at the time of geotechnical analysis. Structural support is understood to consist of columns on the 2nd and 3rd floors located on a 10-foot by 10-foot grid. Structural support of floors 4 to 6 and the roof will consist of light gauge steel framing with walls at 10-foot intervals.

Based upon the proposed configuration and experience with other developments, the five levels of the proposed building above the at-grade parking and limited first story office space is anticipated to be supported by columns at regular intervals along the perimeter of the building footprint at approximate 30- to 40-foot intervals and an additional line of columns oriented along the long (east-west) direction of the building at mid-span. The locations of the columns are anticipated to be coincident with landscape planter regions interspersed between parking stalls.

Structural loads have been assumed to be approximately 625 kips for the perimeter columns and 1250 kips for the interior columns. Actual load information, when developed, should be reviewed by the geotechnical engineer to ensure recommendations remain applicable for the project.

Pavement for the proposed development is expected to be primarily subjected to vehicle loads associated with automobiles, cargo vans and occasional heavy trucks. Pavement recommendations have been presented on the basis of Traffic Index values of 4.0, 5.0 and 6.0 for a 20-year design life.

1.3 Previous Site Investigations

Geotechnologies Inc. (GTI) 2017: In accordance with the 2016 California Building Code (CBC) a geotechnical investigation report was prepared on the subject site to identify the distribution and engineering properties of the earth materials underlying the site, and to provide geotechnical recommendations for the seismic upgrade/retrofit of the existing Courtyard Marriott hotel. This investigation included excavations of two (2) exploratory borings drilled to depths of 50 to 53 feet bgs, collection of representative samples, laboratory testing, engineering analysis and the preparation of their report.

Findings presented in the referenced report indicated approximately 5 feet of fill was reported overlying alluvium ranging in thickness from 10 to 53 feet below ground surface (bgs) with in-situ moisture contents described as moist to wet, stiff to very stiff consistency fine-grained cohesive soils deposited by ancestral stream action. The alluvial sediments cap the claystone and siltstone bedrock formally named as the upper member of Hoots 1931 Modelo Formation, which is equivalent to Dibblee’s (1992 DF-35) Tertiary Unnamed Shale Unit (Tush). Groundwater was encountered in each boring at depths of 21 to 40 feet bgs correlating to elevation EL. 881 and El. 861 feet, respectively.

Liquefaction analyses were performed using a historic high of 10 feet bgs and peak ground acceleration (PGA_M) of 0.68g and moment magnitude (M_W) 6.8 resulting in an opinion that the subsurface soils were not prone to liquefaction. Site soils were characterized as having very low to moderate range of expansion (EI=13 to 86). Micropiles were recommend to provide uplift resistance to the new shear walls proposed as part of the seismic modernization program. Calculated settlement of then-proposed new elements was reported to not exceed an estimated ¼ inch. Boring locations are presented on Plate 1. Boring logs from the prior exploration (Geotechnologies, 2017) are included in Appendix B, *Field Exploration.* Laboratory data reviewed in preparation of this report is included in Appendix C, *Laboratory Test Data.*

Geocon West Inc. (GWI) 2020: In accordance with the 2019 CBC, GWI prepared a geotechnical report that included review of a prior report prepared for the site, field exploration, laboratory testing, engineering analysis, and the preparation of their 2020 report to support demolition of the existing swimming pool and construction of a new four-story hotel structure and new swimming pool within the currently vacant northern region of the property, the region of the property that is the location of the proposed self-storage building that is addressed by this report. Wall and column loads were estimated at 500 kips and 5 kips per lineal foot, respectively.

The site was explored by excavating two 8-inch diameter borings using a truck-mounted hollow-stem auger. The borings were excavated to depths of approximately 50 and 67½ feet beneath the existing ground surface. Supplemental site exploration was performed on December 31, 2019 by advancing four Cone Penetrometer Tests (CPTs) to depths between approximately 5 and 77½ feet below the existing ground surface.

Artificial fill was reported to a maximum depth of 8 feet below existing ground surface characterized as slightly moist to moist and soft to firm dark brown sandy silt. Underlying the fill they encountered Quaternary age alluvial fan deposits characterized primarily dry to wet, and very soft to hard or very loose to dense as grayish brown or brown to dark brown, interbedded clay, silt with clay, silty sand, sand with silt to graded sand. Underlying the fill they encountered Miocene age sedimentary bedrock of the Modelo Formation (Tush equivalent, Dibblee, 1992) in boring B1 and B2 at depths of 64 feet and 9½ feet beneath the existing grade. Bedrock material consisted primarily of fine-grained laminated to interbedded sandstone or silty sandstone with siltstone and claystone that was poorly-bedded to massive and slightly weathered to hard.

1.4 Purpose and Scope

The purpose of our work was to evaluate the subsurface conditions through literature review and exploration at the site relative to the currently proposed development (Ware Malcomb, 2022) and provide geotechnical recommendations to aid in project planning and design. The scope of this evaluation included the following tasks:

- *Background Review* – We reviewed readily available geotechnical reports, literature, aerial photographs, and maps relevant to the site available provided by you or from our in-house library and public domain. We evaluated geological hazards and potential geotechnical issues that may significantly affect the site. The documents reviewed are listed in Appendix A, *References*.

- *Site Reconnaissance* – We performed a visual site reconnaissance to mark our subsurface exploration locations and notify Underground Service Alert (USA) for utility clearance. Site conditions were visually reviewed to evaluate existing topography, development and drainage patterns.

- *Field Exploration* – Our field exploration was performed on May 6^{th} and 9^{th}, 2022 and consisted of advancing five (5) hollow-stem auger borings (designated LB-1 through LB-3) and three (3) Cone Penetrometer Test (CPT) soundings (designated CPT-1 through CPT-3). The hollow-stem auger borings were drilled to approximate depths ranging from 10 to 51½ feet bgs. CPT soundings CPT-1 and CPT-3 were advanced until practical refusal to approximate depths of 46 to 70 feet bgs; CPT-2 was terminated at shallow depth on an obstruction interpreted as a concrete slab (Plate 1). Shear wave velocities were recorded at 5-foot intervals in CPT-1.

  During drilling of the hollow-stem auger borings, both bulk and drive samples were obtained. Drive ring samples were collected from the borings using a Modified California ring-lined sampler with sampling conducted in accordance with ASTM Test Method D 3550. Soil samples were also collected by performing the Standard Penetration Test (SPT) within the borings in accordance with ASTM Test Method D 1586. The ring and SPT samplers were driven for a total penetration of 18 inches using a 140-pound automatic hammer falling freely for 30 inches. The number of blows per 6 inches of penetration was recorded on the boring logs, which are included in Appendix B, *Field Exploration.*

  The borings were logged in the field by an engineer from our technical staff under the supervision of a California Certified Engineering Geologist. Soil samples were reviewed and described in general accordance with the Unified Soil Classification System (USCS). The samples were sealed and packaged for transportation to our laboratory for testing. After completion of drilling the hollow-stem auger borings and CPT explorations were backfilled with soil cuttings and bentonite pellets to existing ground surface. The locations of the borings and CPT soundings are shown on Plate 1, *Geotechnical Map.* Boring and CPT logs are included in Appendix B.

- *Laboratory Testing* – Geotechnical laboratory testing was performed on select soil samples collected during our field exploration to determine the engineering properties of the encountered subsurface soils. The results of laboratory testing are presented in Appendix C, *Laboratory Test Data.*

- *Engineering Analysis* – Geotechnical analysis was performed on the collected data to develop conclusions and recommendations for design and construction of the improvements as currently planned. The results of relevant analyses are presented in Appendix D, *Engineering Analyses*.

- *Report Preparation* – This report presents our findings, conclusions and recommendations for the proposed development.

2.0 GEOTECHNICAL FINDINGS

2.1 Regional Geologic Setting

The project site lies on the northern flanks of the Chalk Hills area of the Santa Monica Mountains on the southern margin of the San Fernando Valley. The Santa Monica Mountains extend westward from the Los Angeles River approximately 45 miles to the Oxnard Plain. The San Fernando Valley is an east-west structural trough bordered on the north and south by mountains that are actively deforming the highlands and anticlinal ridges due to movement along thrust faults. As these ranges have risen and been deformed, the San Fernando Valley has subsided and filled with sediment.

Sedimentary rock in the Chalk Hills is composed of shale and siltstone belonging to the Modelo Formation of Hoots and Kew (1931) and was deposited in a shallow marine environment during the Tertiary period, which is equivalent to Dibblee's (1992) Tertiary Unnamed Shale Unit (Tush). The unnamed shale strikes generally northwest to northeast and dips predominately to the north between 10 to 23 degrees. Shallow mudflows are common in the highlands to the south. Regional geologic units are shown on Figure 2. *Regional Geology Map.*

The site has received sediment from streams emanating in the Santa Monica Mountains to the south. These streams result in deposition of channel deposits, floodplain and overbank alluvium. Composition of these deposits is highly source dependent. Streams eroding the Modelo Formation shale to the south tend to deposit clayey alluvium such as is found to underlie the site (Appendix B).

Erosion over geologic time by streams originating in the highland topography to the south are responsible for incision of the former canyon during low sea level stand approximately 120,000 years ago. Backfill of the canyon occurred during sea level rise beginning approximately 15,000 years ago resulting in the relatively north dipping alluvial fan topography of today and the contrast in subsurface conditions in which bedrock exists at shallow depth adjacent to deeper alluvial deposits, see Plate 2, *Geotechnical Cross Sections AA BB and CC* for subsurface interpretations of alluvial and bedrock geometry.

2.2 Local Geologic Units and Subsurface Conditions

Based on our review of available information (Appendix A, *References)*, and interpreted from our field exploration, the site is underlain by a relatively thin mantle of artificial fill material overlying Quaternary age young alluvial fan deposits in the eastern region and bedrock at shallow depth in the western region. The locations of the subsurface explorations are shown on Plate 1, *Geotechnical Map.*

A general description of the encountered soils and bedrock is presented as follows:

- *Artificial Fill, Undocumented- Map Symbol (Afu):* Artificial fill was encountered in our subsurface explorations to depths of approximately 5 to 8 feet bgs, which is consistent with prior explorations at the site (*References)*. As encountered, the artificial fill soils consisted predominately of mottled greyish brown to dark brown lean clay, silty sand and sand. Localized thicker accumulations of the fill materials may be encountered between explored locations during future earthwork construction.

- *Quaternary Young Alluvial Fan Deposits- Map Symbol (Qyf):* Beneath the mantle of artificial fill, young alluvial fan deposits were encountered to the maximum depth explored of 70 feet bgs. In general, black to light brown to yellowish brown, stiff to medium stiff lean clay, clay, silty sand to medium dense well graded sand was encountered. Detailed descriptions of the geologic units encountered are presented on the boring logs in Appendix B. Geotechnical conditions described on the logs represent the conditions at the actual exploratory excavation locations. Other variations may occur beyond and/or between the excavations. Lines of demarcation between the geologic units and the various earth materials on the logs represent approximated boundaries, and (unless otherwise noted) actual transitions may be gradual.

- *Bedrock: Tertiary Unnamed Shale: (Tush):* Bedrock was encountered beneath the fill and alluvium at depths ranging from 5 to 64 feet below ground surface. As encountered, the bedrock was characterized as thinly- to thickly-bedded to laminated, mottled brown to olive brown, hard moderately to well-cemented silty claystone to siltstone. Bedrock was moderately oxidized and fractured which contained moderately healed to well- healed with iron oxide, gypsum and fine-grained sand. The bedrock exhibited varying degrees of weathering from highly weathered to decomposed, therefore, the upper mantle (approximate 1-3 feet) of bedrock behaves more like soil.

2.3 **Engineering Properties**

Geotechnical engineering properties determined to be relevant for the proposed development were evaluated on the basis of field and laboratory testing; and on review of the interpreted subsurface profiles (Plate 2) and engineering correlations

(Appendix D). The following summarizes the relevant properties evaluated for this project.

2.3.1 Expansive Soil Characteristics

Expansive soils contain significant amounts of clay particles that swell considerably when wetted and which shrink when dried. Foundations constructed on these soils are subject to uplifting forces caused by the swelling. Without proper mitigation measures, heaving and cracking of both building foundations and slabs-on-grade could result.

Laboratory testing of the near surface soils (0-5 ft bgs) included Expansion Index (EI) testing to provide qualitative estimation of expansion potential. The test results on the near surface sample, characterized as clayey sand (SC) from boring LB-1, yielded an EI value of 35 or *Low Expansion* potential.

Prior testing of expansion in boring B-1 (Plate 1, Geocon, 2020) indicated EI=76 or *medium potential* for expansion. Prior testing of expansion in borings B-1 and B-2 (Plate 1, Geotechnologies, 2017) indicate EI=86 and EI=13 or *medium and very low potential* for expansion, respectively.

Based on these results, variance in expansion potential of onsite soil is anticipated; therefore, additional testing is recommended upon completion of site grading and excavation to confirm the expansion potential presented in this report. For purposes of this report, and based upon visual and laboratory characterization of the near surface materials, the expansion potential of these near surface soils is expected to be medium, which should be verified upon completion of earthwork grading.

2.3.2 Soil Plasticity

Select samples of the subsoils were subjected to Atterberg Limits testing to measure plasticity and allow evaluation of the liquefaction susceptibility per the criteria stated in the Bray and Sancio (2006) technical paper.

The data presented in the table below includes the soil descriptions for clarity from current and prior explorations and to demonstrate the soils that correspond to the various classifications of the cohesive soils satisfy the criteria for resistance to liquefaction except where noted. For locations of explorations see Plate 1.

Table 1 – Soil Properties

Boring No.	Soil Description	Sample Data		In-Situ Moisture Content (w_i) (%)	Liquid Limit (LL)	(0.80) x LL	Plasticity Index (PI)
		I.D.	Depth (feet)				
LB-1	Lean Clay (CL)	R-8	25	23.0	49	39.2	26
LB-1	Fat Clay (CH)	ST-4	12.5	23.5	52	41.6	29
LB-1	Lean Clay with Sand (CLs)	ST-10	35	17.2	30	24.0	13
LB-1	Fat Clay (CH)	ST-13	50	23.8	50	40.0	26
B1	Clay (CL)		15	28	45	36	23
B1	Clay (CL)		22.5		47	37.6	23
B1	Clay (CL)		30	31.8	35	28	18
B1	Silt (ML)		40	31.7	43	34.4	17
B1	Clay (CL)		47.5		43	34.4	24
B1	Fat Clay (CH)		50		50	40	31
B1	Clay (CL)		57.5		38	30.4	19
B2	Clay (CL)		15	25.3	48	38.4	28
B2	Clay (CL)		20	31.2	40	32	19
B2	Fat Clay (CH)		25	39.2	53	42.4	30
B2	Clay (CL)		30	31.7	39	31.2	21

Notes: R-8 "Ring" sample collected via ASTM D3550; Modified California Ring Sampler

ST-13 Shelby Tube sample collected via ASTM D1587

Bray and Sancio criteria: w_i < 0.80(LL); or PI > 18

The results in the table above summarized tests conducted on soils classified as clay or silt as encountered at the test boring LB-1 (Plate 1).With the exception of test sample ST-10 in boring LB-1 and Geocon's (2020) boring B-1 at 40 feet in depth, the test results summarized above indicate the tested soils satisfy both criteria established by Bray and Sancio (2006) to allow the materials to be considered not susceptible to liquefaction potential. The samples that exhibited PI < 18 did, however, exhibit in-situ moisture content contents less than 80% of the respective liquid limit (LL), which indicates these soils are also considered to be not susceptible to liquefaction based upon the Bray and Sancio criteria.

2.3.3 Soil Corrosivity

One (1) near-surface bulk soil sample from boring LB-2 obtained during our subsurface exploration was tested to assess corrosion potential to buried concrete and materials that are in contact with the soils. The chemical analysis test results for the onsite soil from our geotechnical exploration are included in Appendix C of this report. The test results from indicate soluble sulfate concentrations of 123 parts per million (ppm), a chloride content of 100 ppm, pH value of 7.80, and minimum resistivity value of 1,000 ohm-cm.

The results of the resistivity testing to date (Geocon, 2020 and Geotechnologies, 2017) indicate the underlying soil is severely corrosive to buried ferrous metals per ASTM STP 1013. Based on the measured water-soluble sulfate content from the soil samples (Geocon, 2020 and Geotechnologies, 2017), concrete in contact with the soil is expected to have negligible exposure to sulfate attack per ACI 318 (ACI, 2014). The sample tested for water-soluble chloride content indicate a low potential for corrosion of steel in concrete due to the chloride content of the soil. Discussion of these results is presented in Sections 5.5 and 5.6 of this report. The results of prior and current testing are presented in Appendix C.

2.3.4 Consolidation

Select samples from test boring LB-1 were subjected to consolidation testing to evaluate the time-dependent compressibility of the clays that underlie the eastern region of the site.

In addition the testing conducted on samples collected from the current exploration, the results of prior testing were reviewed and considered in the evaluation of the site. Accordingly, Leighton accepts responsibility in using test data prepared by other consulting engineers for the subject site.

The results of the consolidation testing are included in Appendix C. The parameters used in the settlement analysis are included in the graphical output of the analysis software.

2.3.5 Soil Collapse

Nine (9) samples of onsite soils were tested to evaluate the response of the soil samples upon wetting at a specific normal pressure. The results of testing indicate a low potential for collapse with collapse strains ranging from 0.24 to 0.86 percent under normal pressures in the range of 2 to 3.2 ksf. The results of prior and current testing are presented in Appendix C.

2.3.6 Shear Strength

Evaluation of the shear strength characteristics of the soils included laboratory direct shear testing. The results of prior and current testing are included in Appendix C and include graphs that provide values of angle of internal friction (ø) and cohesion (c) for use in geotechnical analysis.

2.3.7 Excavation Characteristics

Based on our subsurface exploration, the encountered earth material can be graded with relative ease using conventional equipment in good working order. We anticipate site soils will be considered as Type C soil, which may be prone to raveling and collapse in deeper, unshored excavations. Clay soils may stand near vertical to depths of 4 to 5 feet.

2.3.8 Shear Wave Velocity

Shear wave velocities were profiled at 5-foot intervals to a depth of 70 feet bgs in CPT-1 (Plate 1) to estimate average S-wave velocities of the upper 100 feet (Vs_{100}) and 30 meters (Vs_{30}). The time-weighted average shear wave velocity recorded onsite was approximately 755 feet per second (ft/sec). The shear wave velocity report is included in Appendix B. Based on collected velocities and in accordance with the 2019 California Building Code, the soils at this site classified as Seismic Site Class D.

2.4 Groundwater Conditions

Groundwater was encountered during our current exploration at a depth of 22 feet below grade. Prior excavations encountered groundwater at depths as shallow as 17 feet below grade. The depths to groundwater encountered during the various exploration activity are summarized in Plate 1, *Geotechnical Map.*

Seepage was encountered in the bedrock material in boring LB-2 at 33 feet bgs. Joints and fractures in the bedrock facilitate the hydraulic connectivity allowing infiltration to migrate through the bedrock structure as evidenced by the oxidation features and gypsum precipitation along parting surfaces within bedrock samples.

Based on the current concept groundwater is not expected to pose a constraint to construction. However, due to the relatively low permeability of the alluvial soils encountered on-site, accurate determination of the hydrostatic groundwater table requires monitoring of an observation well. Local fluctuations in groundwater can be expected to occur beneath the site, manifested as zones of perched water possibly on beds of impermeable clay and increased soil moisture in strata of high fines content. Sources of possible groundwater may be due to periods of prolonged and/or locally intense precipitation, excessive landscape irrigation or broken utilities. Due to the clayey nature of subsurface fill and low permeability alluvial soils, infiltration is not considered geotechnically feasible at this site.

3.0 GEOLOGIC AND SEISMIC HAZARDS

Geologic and seismic hazards include surface fault rupture, ground shaking associated with earthquake ground motion, liquefaction, seismically-induced settlement, lateral spreading and seismically-induced landslides. The following sections discuss these hazards and their potential effect on the project site.

The project site is located in a seismically active region of California with a number of mapped and potentially active faults in the vicinity of the site capable of inducing ground motions that may affect the proposed development. Accordingly, the project should be designed in accordance with all applicable current codes and standards utilizing the appropriate seismic design parameters to reduce seismic risk as described in the “Minimum Statewide Safety Standard” in Chapter 2 of the California Geological Survey (CGS) Special Publication 117a (CGS, 2008).

3.1 Surface Fault Rupture

Our review of available in-house literature indicates that no known active faults have been mapped crossing the site, and the site is not located within a designated Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007). Therefore, the potential for surface fault rupture at the site is expected to be low and a surface fault rupture hazard evaluation is not mandated for this site.

The location of the closest active faults to the site was evaluated using the United States Geological Survey (USGS) Earthquake Hazards Program National Seismic Hazard Maps (USGS, 2008c). The closest active faults to the site are the Malibu Coast and Santa Susana, located approximately 9.1 and 9.5 miles from the site, respectively. The San Andreas Fault, which is the largest active tectonic plate boundary in California, is approximately 35 miles northeast of the site. Major regional faults with surface expression in proximity to the site are shown on Figure 3, *Regional Fault and Historical Seismicity Map*.

3.2 Seismicity and Ground Motion

The principal seismic hazard to the site is ground shaking resulting from an earthquake occurring along any of several major active and potentially active faults in southern California. The intensity of ground shaking at a given location depends primarily upon the earthquake magnitude, the distance from the source, and the site response characteristics.

The site should be expected to experience strong ground shaking resulting from an earthquake occurring along one or more of the major regional active faults (Figure 3) and the rupture scenarios currently modeled in the Third Unified California Earthquake Fault Rupture Forecast (UCERF3).

Design of the project should be performed in accordance with all applicable current codes and standards utilizing the appropriate seismic design parameters to reduce seismic risk as defined by California Geological Survey (CGS) Chapter 2 of Special Publication 117a (CGS, 2008). The 2019 edition of the CBC is the current edition of the code. Through compliance with these regulatory requirements and the utilization of appropriate seismic design parameters selected by the design professionals, potential effects relating to seismic shaking can be reduced.

3.2.1 Mapped Seismic Parameters

At a minimum, seismic design of structures should be performed by the project structural engineer in accordance with the 2019 edition of the California Building Code (CBC 2019) with the 2020 City of Los Angeles amendments to accommodate effects of ground shaking produced by regional seismic events.

Based on Table 1613.2.3(2) of the 2019 CBC, the long period site coefficient (F_v) should be determined in accordance with Section 11.4.8 of ASCE 7-16 since the mapped spectral response acceleration at 1 second (i.e., S_1) is greater than 0.2g for Site Class D. Consequently, and in accordance with Section 11.4.8 of ASCE 7-16, a site-specific ground motion hazard assessment is required; however, the values provided in the following table may be utilized if design is performed in accordance with exception (2) in Section 11.4.8 of ASCE 7-16, with special requirements for the seismic response coefficient (C_s) as noted below. The project structural engineer should review the seismic parameters.

The following table lists seismic design parameters based on the 2019 CBC, Section 1613.3 (ASCE 07-16) methodology.

Table 2 – 2019 CBC Seismic Design Parameters

Categorization/Coefficients	Code-Based (1) (2)
Site Longitude (decimal degrees) West	-118.59299°
Site Latitude (decimal degrees) North	33.168144°
Site Class	D
Mapped Spectral Response Acceleration at 0.2s Period, S_s	1.654
Mapped Spectral Response Acceleration at 1s Period, S_1	0.6
Short Period Site Coefficient at 0.2s Period, F_a	1.0
Long Period Site Coefficient at 1s Period, F_v	1.7[3]
Adjusted Spectral Response Acceleration at 0.2s Period, S_{MS}	1.654
Adjusted Spectral Response Acceleration at 1s Period, S_{M1}	1.020[3]
Design Spectral Response Acceleration at 0.2s Period, S_{DS}	1.103
Design Spectral Response Acceleration at 1s Period, S_{D1}	0.680[3]

1. All were derived from the Applied Technology Council web page: https://seismicmaps.org/
2. All coefficients in units of g (spectral acceleration)
3. Per Exception 2 in Section 11.4.8 of ASCE 7-16, seismic response coefficient C_s to be determined by Eq. 12.8-2 for values of $T \leq 1.5T_s$ (see *Note* below)

Note: Long period coefficient (Fv) of 1.7 may be utilized for calculation of Ts, provided that the value of the Seismic Response Coefficient (Cs) is determined by Equation 12.8-2 for values of the fundamental period of the building (T) less than or equal to 1.5Ts, and taken as 1.5 times the value computed in accordance with either equation 12.8-3 for T greater than 1.5 Ts and less than or equal to T_L or equation 12.8-4 for T greater than T_L.

Table 3 – Peak Ground Acceleration

Parameter	Value
MCE_G Peak Ground Acceleration	0.679
F_{PGA} – Site Amplification Factor at PGA	1.1
PGA_M – Site Modified Peak Ground Acceleration	0.747

3.2.2 Site-Specific Response Spectrum

As previously discussed, a site-specific ground motion hazard analysis need not be performed for structures that satisfy certain requirements depending upon the Site Class determined for the subsurface profile and the manner in which the coefficient C_S is calculated for use in structural analysis in accordance with Section 11.4.8 of ASCE 7-16. If desired a site-specific analysis can be performed upon request.

As an alternate to the use of mapped parameters and the underlying requirement for determination of the coefficient C_s based on structural period, a site-specific seismic analysis has been conducted in accordance with section 11.4.8 of ASCE 7-16. The site-specific seismic design parameters are summarized below. Details of the analysis and the site-specific response spectra are presented in Appendix D, *Engineering Analyses*.

Table 4 – Site-Specific 2019 CBC Seismic Design Parameters

Categorization/Coefficients	Design Value[1]
Adjusted Spectral Response Acceleration at 0.2s Period, S_{MS}	2.141
Adjusted Spectral Response Acceleration at 1s Period, S_{M1}	1.298
Design Spectral Response Acceleration at 0.2s Period, S_{DS}	1.427
Design Spectral Response Acceleration at 1s Period, S_{D1}	0.865
Site-Specific Peak Ground Acceleration, PGA_M	0.776

Notes: 1 Values determined in accordance with Section 21.4 of ASCE 7-16 and may be used according to this section for Equivalent Lateral Force procedure.

2 In accordance with Section 21.5 of ASCE 7-16

3.2.3 Hazard Disaggregation

Disaggregation of the seismic hazard for the subject site was conducted using the *Unified Hazard Tool* developed by the USGS. Disaggregation was conducted for the ground motion that corresponds to average return period (ARP) of 2,475 years and 475 years. The specified ARP values correspond to a probability of exceedance of 2 percent and 10 percent, respectively, for a 50-year exposure period.

Table 5 – Hazard Disaggregation

Average Return Period (years):	2,475	475
Probability of Exceedance for 50-year Exposure Period	2%	10%
Mean Magnitude (M)	6.63	6.57
Distance (km)	12.69	15.62
Peak Ground Acceleration (g)	0.815	0.503

3.3 Liquefaction Potential

The State of California Seismic Hazard Zone Map for the Canoga Park Quadrangle (CGS, 1998), indicates the project site is located within an area susceptible to liquefaction, and is designated as an area of required investigation for liquefaction (Figure 4, Seismic Hazard Map).

Phenomenon and Consequences – Liquefaction is the phenomenon of the reduction in strength and stiffness of certain soil types that exist below the groundwater table when subjected to strong ground shaking such as typically associated with seismic activity. Soils most susceptible to liquefaction are saturated granular deposits that exhibit loose to medium dense relative density.

Liquefaction occurs due to the generation of excess pore pressures in the soils resulting from the tendency for loose granular deposits to densify during strong ground shaking. The pressure of the water contained in the soil void space (pores) increases as the densification or consolidation process is impeded due to the presence of the pore water and the temporary inability of the water to drain. Consequently, the pore pressures within the saturated soils increase above hydrostatic pressure. Liquefaction occurs when this excess pore pressure (i.e. magnitude above hydrostatic) approaches the magnitude of the in-situ lithostatic stress and, therefore, the difference between the in-situ stress and excess pore pressure approaches zero (i.e., zero effective stress).

In general, liquefaction hazards are the most severe in the upper 50 feet below ground surface for structures supported at-grade on shallow foundations. Depending upon the depth below ground surface and the extent of the susceptible deposits, the effects of liquefaction at sites with level grade may consist of the following:

- Settlement of the ground surface as pore pressure equilibrium is reestablished in the soils after ground shaking;
- Ground failure in which large, intact blocks of the non-liquefied soils above the water table (the non-liquefied crust) develop fissures. These blocks tend to oscillate during ground shaking, thereby resulting in the potential for differential horizontal displacements (i.e. ground oscillation); and
- Loss of bearing strength and failure of foundations.

Triggering Analysis – The potential for liquefaction to occur was assessed at CPT soundings CPT-1 and CPT-3 using the software package, *CLiq* (Geologismiki, 2006). The potential for liquefaction in Boring LB-1, extended to a depth 50 feet, was assessed using the *LiqSVs* software (Geologismiki, 2007). Liquefaction triggering analysis was performed using the simplified procedure (Youd, 2001). The depth to groundwater used in analysis was 10 feet. The depth to groundwater was considered to be conservative considering the data presented in Plate 1.2 contained in SHZR 07 (CDMG, 1997) which indicates little data with respect to historic high groundwater exists for the subject site.

Evaluation of the potential for liquefaction to be triggered was performed for two scenarios:

- Geomean Maximum Considered Event (MCE_G); and
- Design Basis Earthquake (DBE).

The MCE_G is based upon a 2,475 year average return period (ARP) and ground motion corresponding to the PGA_M value; the DBE is based upon a 475-year ARP with the ground motion used in this analysis being two-thirds of the PGA_M value. In regard to the subject site, these design scenarios correspond to the following:

- PGA_M value of 0.747g identified in conjunction with determination of the seismic design parameters, and earthquake magnitude (M_W) of 6.63; and
- DBE peak ground acceleration value of 0.503g (i.e., two-thirds of the PGA_M) and earthquake magnitude (M_W) of 6.57.

Liquefaction triggering analysis conducted on the CPT soundings provides greater specificity in evaluating the effect of liquefaction and potential consequences due to the refinement in subsurface stratigraphy provided by the continuous record of lithology recorded during advancement of the CPT probe. When compared to test borings in which samples of finite length/dimension are typically collected at regular intervals on the order of several feet, some conservatism is applied as these represent a discretized depiction of the subsurface profile.

Review of the stratigraphy depicted in the graphical “stick” logs of the tests borings and CPT soundings was found to show significant correlation between the interpreted soil conditions of the CPT as provided by the CPT Soil Behavior Type and the soil classifications determined by visual/manual review of the collected samples (ASTM D2488) supplemented by laboratory testing (ASTM D2487). As a result, the use of the stratigraphy provided by the CPT soundings was considered to be valid in adjusting the thicknesses of layers potentially susceptible to liquefaction for use in the liquefaction triggering analysis of the test borings.

However, only a single strum of sand was identified at boring LB-1; no adjustment to the thickness of this layer was made in the analysis.

Liquefaction analysis of the test borings is presented in Appendix D, *Engineering Analysis*. In those analyses, the materials identified as clay in the test borings were deemed not susceptible to liquefaction and were defined as such in the analyses. The potential susceptibility to liquefaction was defined in the "Can Liquefy" column of the *Field Input Data* section of the LiqSVs software output file.

The justification for omitting specific strata from liquefaction triggering analysis is demonstrated by the data summarized in Table 1 (Section 2.3.2 Soil Properties) using the criteria stated in SP117 and LADBS IB P/BC 2020-151.

Summary of Results – The results of the analysis are presented in Appendix D and indicate a low potential for liquefaction to occur in the alluvial soils with the potential for liquefaction to occur within the relatively thin layers of granular soils within the alluvial deposits in the eastern region of the site.

3.4 Seismically-Induced Ground Deformation

The response to seismically-induced ground motion may include settlement of the ground surface and structures supported on-grade due to densification of soils as well as construction of soils susceptible to liquefaction; lateral displacement of sites with level grade where liquefaction is triggered in one or more strata at depth; and instability of slopes. Evaluation of these modes of seismically-induced deformation is discussed below.

3.4.1 Settlement

Seismically-induced settlement consists of dynamic settlement of unsaturated soil (above groundwater) and liquefaction-induced settlement (below groundwater). These settlements occur primarily within low density sandy soil due to reduction in volume during and shortly after an earthquake event.

Evaluation of the potential for such seismically-induced settlements to occur was performed for two scenarios: Geomean Maximum Considered Event (MCE_G) and the Design Basis Earthquake (DBE) as discussed above. The MCE_G is based upon a 2,475 year average return period (ARP) while the DBE is based upon a 475-year ARP with the ground motion used in this analysis being two-thirds of the PGA_M value.

Analysis of the potential magnitude of seismically-induced settlement was performed in conjunction with the liquefaction triggering analysis.

Table 6 – Seismically-Induced Settlement

MCE_G Seismic Event (2,475 ARP)

Design Scenario		Exploration I.D.	Settlement (inches)		
PGA	M_W		Liquefaction(1)	Non-Liquefaction(2)	Total
0.75	6.63	CPT-1	2.15	0.58	2.73
		CPT-3	1.30	< 0.1	1.30
		LB-1	1.60	< 0.1	1.60

Notes: (1) Liquefaction-induced settlement; Design Groundwater at depth of 10 feet
(2) Non-liquefaction induced ("dry sand") settlement

Design-Basis Seismic Event (475 ARP)

Design Scenario		Exploration I.D.	Settlement (inches)		
PGA	M_W		Liquefaction(1)	Non-Liquefaction(2)	Total
0.50	6.57	CPT-1	1.60	0.15	1.75
		CPT-3	0.71	< 0.1	0.71
		LB-1	1.60	< 0.1	1.60

Notes: (1) Liquefaction-induced settlement; Design Groundwater at depth of 10 feet
(2) Non-liquefaction induced ("dry sand") settlement

Analysis of the potential seismically-induced settlements was performed under two design scenarios to allow evaluation building performance for different criteria:

- Settlements associated with the MCE_G event (2,475-year ARP) are recommended to be considered as an extreme event with building performance criteria being structural integrity and prevention of collapse to protect occupants and maintain safety in accordance with the intent of the building code (Section 12.13.9 of ASCE 7-16).
- Settlements associated with the Design Basis Event (475-year ARP) are recommended to be considered under service load conditions with

building performance criteria being tolerable settlements and distortions that are not expected to adversely affect the building and normal operations but may require cosmetic and minor structural repairs.

The results of the analysis of seismically-induced settlement indicate total settlement in the range of 2½ to 2¾ under the ground motion corresponding to the 2,475 ARP. The differential settlement is estimated to be approximately 1½ inches over a minimum span of 50 feet.

Settlement associated with the ground motion corresponding to the 475 ARP was in the range of 1½ to 2 inches with a differential settlement of approximately 1 inch over a minimum span of 50 feet.

3.4.2 Lateral Displacements

Lateral displacements of the ground surface on a site resulting from liquefaction consist of lateral spreading and ground oscillation. Lateral spreading is a phenomenon in which the upper crust of non-liquefied soils progressively translates down-gradient travelling over the liquefied soils.

The occurrence of lateral spreading typically requires the liquefiable zone to be continuous, unconstrained laterally, and free to move along gently dipping strata of low friction toward an unconfined area such as a shoreline or riverbank, although lateral spreading may occur in areas of low topographic relief and gently sloping terrain. Although the low potential for liquefaction to occur has been identified, the potential for lateral spreading is anticipated to also be low based on the regional topography, and the discontinuous nature of the strata identified as being potentially susceptible to liquefaction under the design groundwater conditions (i.e., groundwater at depth of 17 feet).

3.4.3 Landsliding

The potential for seismically-induced landsliding to occur at the site is not a consideration due to the relatively flat topography and absence of significant slopes on or adjacent to the site. In addition, the State of California Seismic Hazard Zones Map for the Canoga Park Quadrangle (CGS, 1998), indicates the site is **not** located within an area of potential landslide susceptibility (Figure 4, *Seismic Hazard Map*). Slopes planned as part of the development, although not anticipated, should be engineered and constructed at a gradient of 2:1 (horizontal:vertical) or flatter.

3.5 Flooding

Review of Figure 7-2 Mapped Flood Areas in South valley APC presented in the City of Los Angeles Floodplain Management Plan (City of LA, October 2020) indicate the site is not within mapped flood area.

3.6 Methane

Our review of State of California Geologic Energy Management Division (CalGEM) records (https://maps.conservation.ca.gov/doggr/wellfinder) indicate the project site is not within a 500 foot radius of any active, inactive or abandoned oil or natural gas well. In addition, the site is not located within an active of former wellfield. Therefore, the potential for methane intrusion and mitigation requirements is considered low.

3.7 Static Settlement

Analysis of settlement for the proposed development at the subject site is considered to be represented by two distinct profiles:

- Eastern region the building where underlain by laterally continuous, well-correlated deposits of fine-grained alluvial soils; and
- Central to western region of the building supported by material associated with thin layer of undocumented fill overlying bedrock that does not include potentially compressible clays.

The structural support of the proposed building is anticipated to consist of isolated columns positioned at regularly spaced intervals around the perimeter of the building with one additional row of columns within the interior. The columns will be supported by thickened sections of the mat foundation of specific length and width dimensions. The thickened regions below isolated perimeter columns are anticipated to be square in plan.

As discussed in conjunction with the description of the building, structural loads have been estimated to be approximately 625 kips per perimeter columns and 1250 kips for interior columns. Based upon the maximum allowable soil bearing pressure, the typical column support region of the mat foundation will be approximately 13.5 x 13.5 feet in plan dimension. The interior columns are anticipated to be supported by combined foundations consisting of long rectangular regions in which the actual contact pressure is anticipated to be less than the pressure below the columns.

Based upon the assumptions described above regarding structural support, the static settlement in the eastern region of the building where underlain by alluvial deposits has been estimated to be approximately 1¾ inches for the perimeter column supports; the settlement of the interior regions of the foundation will be dependent upon the contact pressure distribution determined from structural analysis, which should be reviewed by Leighton and additional settlement analysis performed as considered to be necessary. Settlement within the western region of the building area where bedrock exists at shallow depth (i.e., at the contact with structural compacted fill placed as part of the building pad preparation), is estimated to be less than 1 inch.

4.0 SUMMARY OF FINDINGS AND CONCLUSIONS

Based on this study, construction of the proposed self-storage development is considered feasible from a geotechnical viewpoint provided the recommendations presented in this report are implemented during future design and confirmed prior to construction. No severe geologic or soils related issues were identified that would preclude development of the site for the proposed improvements. The most significant geotechnical issues at the site are those related to the potential for strong seismic shaking, saturated alluvial soils and compressible clay. Presented below is a summary of findings based upon the results of our geotechnical exploration of the site. Geotechnical recommendations for design and construction are presented in Section 5.0 of this report.

- The site is not located in a designated Alquist-Priolo Earthquake Fault Zone. The closest active faults to the site are the Malibu Coast and Santa Susana, located approximately 9.1 and 9.5 miles from the site, respectively. An earthquake along these fault zones, or others in the nearby southern California area, will generate moderate to strong ground shaking at the site.
- The site **is** located within an area shown as susceptible to liquefaction according to the State of California Seismic Hazard Zones Map for the Canoga Park Quadrangle.
- Seismically-induced settlements of the soils are anticipated to be within tolerable levels for structural performance based upon evaluation criteria previously described and the remedial grading recommended to be performed to prepare the pad. The seismically-induced settlement and estimated distortions should be reviewed by the project structural engineer.
- The site is underlain by undocumented artificial fill to depths of approximately 5 to 8 feet below ground surface (bgs) overlying young alluvial fan deposits of Holocene age and bedrock. The alluvial unit consists predominantly of compressible clay.
- Groundwater was encountered during exploration conducted for the current geotechnical evaluation of the site. Groundwater is not anticipated to be a constraint to site grading or construction. Design groundwater elevation is considered to be 17 feet below current grade.
- Development of the site associated with a mat foundation system should include overexcavation and recompaction of the undocumented fill soils.
- Excavation along the northern, eastern and western property lines to prepare the building pad as described in this report may require either slot-cut techniques or

installation of temporary shoring to allow continuous excavation and protect the improvements on the adjacent property. Recommendations for slot-cutting are presented in Section 5.1.6.

- The proposed structure may be supported by a mat foundation system supported on structural compacted fill replacing existing fill soils in which the foundation system is designed to reduce the effects of static and seismically-induced settlement.

5.0 RECOMMENDATIONS

Geotechnical recommendations for the proposed development are presented in the following sections and are intended to provide sufficient geotechnical information to develop the project plan in accordance with 2019 CBC requirements. The following recommendations may be superseded by more restrictive requirements of the architect, structural engineer, and/or local building official.

The recommendations below are based upon the exhibited geotechnical engineering properties of the soils and their anticipated response both during and after construction. The recommendations are also predicated upon proper field observation and testing during construction. The project geotechnical engineer should be notified of suspected variances in field conditions to determine the effect upon the recommendations subsequently presented.

Leighton should review the grading plan, foundation plan and specifications as they become available to verify that the recommendations presented in this report have been properly interpreted and incorporated into the plans prepared for the project.

5.1 Earthwork

Based upon the anticipated conceptual plan, site grading is not expected to require significant cut or fill, exclusive of removal quantities of undocumented fill (i.e., to depths of 5 to 8 feet below grade) required to properly prepare the site. All site grading should be performed in accordance with the project specifications that are prepared by the appropriate design professional. The general earthwork and grading recommendations presented in Appendix E may be used as a guideline in developing the project specifications.

We recommend that earthwork and grading operations be observed and appropriate testing be performed by representatives of our firm to verify that the site is properly prepared, the selected fill materials are satisfactory, and that placement and compaction of fills has been performed in accordance with our recommendations and the project specifications.

5.1.1 Site Preparation

Initial preparation of the site should include removal of any remnants of any previously existing improvements including all foundations and underground utilities that will not be salvaged with the proposed development or would otherwise interfere with construction. Utilities that will remain active should be properly rerouted to preserve their function. Several explorations encountered shallow refusal in the mid-central region of the site. Buried concrete slabs or rubble should be expected during grading.

The proposed building area extends into the area currently occupied by a swimming pool associated with the existing hotel. Preparation of the building area for the proposed facility will require demolition and removal of the pool shell along with associated facilities and utilities that service the pool. The pool bottom may remain in-place provided the slab is perforated at regular intervals of 2 to 3 feet on-center and the initial lift of backfill consists of coarse granular, free-draining material. A non-woven geosynthetic filtration fabric is recommended to be placed over the granular fill prior to placing fill soils; or complete removal of the shell should be performed.

5.1.2 <u>Overexcavation and Recompaction</u>

Existing fill was encountered to depths ranging from approximately 5 to 8 feet below existing grade. The fill is considered to be “uncertified” in that documentation addressing the placement and compaction of the fill under engineering controlled conditions was not included in the documentation provided by the client for the subject site. If such documentation is provided for review, the recommendations subsequently presented for preparation of the building area may be revised. In the absence of documentation, the City of Los Angeles requires complete removal and reconditioning of undocumented fill and placement under engineering controlled condition to satisfy City requirements of Primary Structural Fill.

Based upon the conditions encountered at the test borings performed for the current exploration, preparation of the building area should include overexcavation and recompaction of existing fill soil to expose suitable bearing native soils or bedrock prior to placement of fill to establish building pad grade. Due to the variance in the depth of existing fill encountered at the test boring locations, some variation in the actual depth of excavation may be required during site grading. As indicated, the depth to which existing, uncertified fill was encountered ranged from approximately 5 to 8 feet below existing grade. In addition to removal of existing fill, the minimum depth of overexcavation is recommended to be 6 feet below existing grade or planned pad grade, whichever is lower in elevation.

Care must be used and precautions implemented in performing earthwork and grading operations along the property lines. **It is essential that excavation not undermine existing adjacent improvements.** Overexcavation performed along property lines that may extend to depths greater than 5 feet below grade and/or remove support of adjacent foundations is recommended to be performed using slot-cutting techniques to reduce the potential for adversely affecting the adjacent improvements. Recommendations for slot-cutting are presented in Section 5.1.6 of this report.

5.1.3 Subgrade Preparation

Exposed subgrades, including all excavation or removal bottoms, should be observed by a representative of the geotechnical engineer prior to placement of fill or other improvements to verify that suitable bearing soil is exposed. The structural fill subgrade in the building area may also require observation by a Deputy Grading Inspector for the city of Los Angeles prior to fill placement. Subgrade surfaces determined to be suitable for fill placement or other improvements should be scarified to a depth of at least 8 inches, moisture-conditioned or dried back to 2 to 3 percent above the optimum moisture content and then compacted to a minimum of 90 percent of the ASTM Test Method D1557 (Modified Proctor) laboratory maximum density.

5.1.4 General Fill Placement and Compaction

The onsite soil, free of organic material, cobbles, boulders, rubble, and rock less than 8 inches in largest dimension, is suitable to be used as structural fill. Material excavated form the building pad are generally considered to be suitable for use as structural fill but should be reviewed by the geotechnical engineer prior to reuse.

All fill soil should be placed in loose lifts no greater than 8 inches in thickness or suitable thickness comparable to the equipment being used, moisture-conditioned as necessary to a moisture content 2 to 3 percentage points above optimum moisture content or dried back and compacted using proper equipment to the minimum standard as noted below, unless stated otherwise in the specific sections.

- Fill soil should be moisture-conditioned and compacted to a minimum of 90 percent relative compaction as determined by the Modified Proctor compaction test (ASTM Test Method D1557).
- Fill soils that exhibit less than 15 percent of particles finer than 0.005 mm should be compacted to at least 95 percent of the Modified Proctor density.
- Base course material should be compacted to a minimum of 95 percent relative compaction.
- Utility trench backfill is discussed in Section 5.4.

Material imported to the site if any for use as fill should be reviewed and approved by the geotechnical engineer prior to import to the site and placement as fill. Imported soils should be low in expansion potential (EI less than 30); non-corrosive to metals and concrete; and be free of hazardous substances.

Moisture Sensitive Soils: Near surface soils at the site and soils that expected to be exposed during grading and excavating activities are sensitive to water. Soils of this nature will become unstable if exposed to increases in moisture content as may occur due to seasonal precipitation or water influx related to construction activities.

In areas where subgrade instability occurs, subgrade stabilization by mechanical or chemical modification may be necessary if scarification, aeration and recompaction is not feasible. Mechanical methods consist of placement of coarse crushed aggregate, perhaps in conjunction with a geogrid or geotextile fabric. Chemical modification includes mixing the unstable soils with specific minimum quantities of hydrated lime or Portland cement followed by proper compaction to develop a stable subgrade. Site clays that are already exist at in-place moisture contests over optimum moisture content or become wet due to other factors, will be more difficult to compact compared to sands.

Rock Stabilization: Subgrade stabilization in this method consists of placing 2- to 3-inch (nominal diameter) crushed rock in lifts and compacting each lift of the aggregate into the subgrade to improve stability. Rock should be mechanically compacted under the weight of the equipment to push the rock into the underlying clay soils. Vibratory equipment **should**

not be used to work in the rock blanket as the vibrations may aggravate locally soft saturated clays causing pumping conditions to expand laterally and destabilize the subgrade further.

***Chemical Modification**:* Disking, blending, cement and/or lime treatment may also be considered by the earthwork contractor to facilitate compaction. However, additional sulfate testing will be required prior to treating/mixing soils with lime, to avoid an adverse sulfate heave reaction. Lime and/or cement treatment also require specialized equipment to blend plastic clay thoroughly with cement or lime, to be effective.

Choice of means and methods to mitigate wet clay compaction difficulty will be at the discretion of the contractor based on weather at the time of earthwork, available materials and equipment, among other considerations specific to the contractor. However, any proposed cement and/or lime treatment must be reviewed and approved by Leighton prior to implementation.

5.1.5 Shrinkage

The change in volume of excavated and recompacted soil varies according to soil type and location. This volume change is represented as a percentage increase (bulking) or decrease (shrinkage) in volume of fill after removal and recompaction. Field and laboratory data used in our calculations included laboratory-measured maximum dry density for the general soil type encountered at the subject site, the measured in-place densities of near surface soils encountered and our experience.

Based upon the results of the in-place density and the moisture-density relationship exhibited by representative bulk samples of the near surface soils, recompaction of the soils is anticipated to result in volume shrinkage in the range of 5 to 10 percent. Due to the lack of in-situ density data for the upper 5 feet, shrinkage values are intended solely as an estimate based on visual classification and judgement. The estimated shrinkage does not include material losses due to removal of organic material or other unsuitable bearing materials (debris, rubble) and the actual shrinkage that occurs during grading may vary throughout the site. Some adjustments to earthwork volume should be anticipated during grading of the site.

5.1.6 Slot-Cut Technique

Based on our understanding of the currently proposed development (Appendix A-1, *Development Concept*), the footprint of the proposed storage building is planned be located adjacent to the northern, eastern and western property lines with ample clearance along all sides of the building to allow building pad preparation to be performed by open-cut techniques with the excavation sidewalls sloped for stability and safety.

Overexcavation required along the property lines to remove existing soils and prepare the building area as described in Section 5.1 may require the use of special excavation techniques to avoid undermining or otherwise adversely affecting the adjacent properties. As typically required, excavations that encroach an imaginary plane projected down at 1H:1V from the adjacent property line must either be shored or performed in a slot-cut manner.

Slot-cut excavation consists of designating a repeating series of three slots (e.g., “A”, “B” and “C”) and performing the excavation in which every third section (e.g., each “A” slot) is excavated and backfilled before the next slot (e.g., “B”) is excavated and backfilled, followed by a similar procedure for the remaining sections (“C” slots).

The results of the analysis indicated the maximum safe span (FS ≥ 1.25) for excavation to depth of 8 feet is 6 feet.

The span stated above may require review and revision after specific site plans have been developed that accurately depict the location of the building (and associated extent of excavation) relative to the property lines, and may also require field-revision depending upon the actual stability exhibited by the excavations.

5.2 Foundation Design – Structural Mat

Preparation of the building area should be performed as described in Section 5.1 of this report. Upon completion of the building pad preparation, the soils that will provide support of the foundation are expected to consist of properly moisture-conditioned structural compacted fill layer that exhibits medium expansion potential underlain by suitable bearing native soils in the eastern region of the site and bedrock in the central to western regions. If the soils exposed at foundation grades are disturbed during the foundation excavation process, they should be scarified, reconditioned and recompacted as engineered fill as described in Section 5.1.

A structural mat foundation is recommended for use at the proposed site due to the estimated total combined settlement (i.e., static and seismically-induced) and the requirements of the city of Los Angeles. In consideration of the anticipated structural support system in which isolated columns will be used to support the upper four stories of the building above the at-grade parking lot, the mat foundation is anticipated to include locally thickened sections of a specific length and width to provide column support interconnected by a reinforced structural slab supported by the prepared building pad. Once foundation plans are prepared and building loads provided they should be reviewed by the geotechnical engineer.

5.2.1 Vertical Load Capacity

Design of a structural mat foundation to support building loads is recommended to be based upon a maximum allowable soil bearing pressure of 3,500 psf supported by a subgrade consisting of structural compacted fill. The recommended allowable bearing capacity may be increased by one-third when considering short-term, transient loads such as wind or seismic forces.

Mat foundations typically experience some deflection due to loads placed on the mat and the reaction of the soils underlying the slab. A design coefficient of subgrade reaction (k_{v1}) of 100 pounds per cubic inch (pci) may be used for evaluating such deflections at the site. This value is based on the soil conditions encountered and is considered as applied to a unit square foot area. The value should be adjusted for the size of the mat and the effective reaction area at point and line loads. The coefficient of subgrade reaction K_b for a mat of a specific width may be evaluated using the following equation.

$K_b = k_{v1} [(B+1) / 2B]^2$ B is the effective diameter of slab reaction area

5.2.2 Lateral Load Resistance

Lateral loads may be resisted by friction between the footings and the supporting subgrade and passive resistance of the soil adjacent to the vertical side of the foundation provided the foundations are poured neat against properly compacted fill or undisturbed existing soils. The coefficient of sliding friction is recommended to be 0.30.

The passive resistance against the sides of the mat foundation may be modeled as an equivalent fluid with density of 230 pounds per cubic foot (pcf) to a maximum of 2,300 psf where the sides of the mat are supported within structural compacted fill placed as part of the recommended preparation of the building area. No reduction will be needed to any of the above two components for computing the total resistance to lateral loads.

The use of passive resistance in lateral load analysis requires the foundations to be cast neat against vertical, undisturbed trench walls or any voids along the footing and be backfilled with structural compacted fill to maintain continuous contact between the foundation concrete and adjacent soils.

5.2.3 Settlement Estimates

On a preliminary basis, the total static settlement of a structural mat foundation designed in accordance with the assumptions and parameters stated in this report is estimated to be 1¾ inches. The estimated differential settlement is estimated to be approximately 1¼ inch across a minimum span of 30 feet, but will be dependent upon the distribution of load demand applied to the mat slab.

The settlement estimates discussed above are based upon the response of the building to the structural loads imposed upon the bearing soils. Settlement of the foundation includes a component associated with seismically-induced settlement. The magnitude of this settlement was discussed in Section 3.4.1 of this report considering two levels of design ground motion. The values stated in Section 3.4.1 should be combined with the static settlement stated above for structural analysis. As discussed in Section 3.4.1, the criteria for evaluation of the magnitude of settlement can be differentiated based upon the level of ground motion associated with the seismic events (i.e., 2,475-year vs. 475-year average return period).

Detailed analysis to evaluate total and differential settlement should be conducted once the actual contact pressure distribution has been determined by the project structural engineer and reviewed by the geotechnical engineer.

5.3 Temporary Excavations

All temporary excavations, including utility trenches, retaining wall excavations, and other excavations should be performed in accordance with project plans, specifications and all Occupational Safety and Health Administration (OSHA) requirements. The sides of excavations should be shored or sloped in accordance with OSHA regulations. OSHA allows the sides of unbraced excavations, up to a maximum height of 20 feet, to be cut to a ¾H:1V (horizontal:vertical) slope for Type A soils, 1H:1V for Type B soils, and 1½H:1V for Type C soils. The soil type should be verified or revised based on geotechnical observation and testing during construction, as soil classifications may vary over short horizontal distances.

Excavations to a depth of 5 feet or less may be excavated near vertical provided the soil conditions are evaluated and found to be suitable by the "competent person" present in site during trenching and excavating activities as defined by OSHA regulations. However, due to the presence of fill that underlie the site and the potential for variances in soil composition, unsupported excavation sidewalls may cave, thereby requiring proper back-sloping for excavation stability and safety.

OSHA regulations are applicable in areas with no restriction of surrounding ground deformations. Shoring should be designed for areas with deformation restrictions and in situations where the excavations may be subjected to surcharge loads such as material stockpiles and heavy machinery if the surcharge loads cannot be maintained a minimum distance equivalent to the excavation height or 5 feet, whichever is greater.

Excavations that extend below an imaginary plane inclined at 45 degrees below the edge of any adjacent existing site foundation should be properly shored to maintain support of the adjacent structures or be performed in a slot-cut manner as discussed in Section 5.1.6 of this report. Earth pressure recommendations for design of shoring are presented in Section 5.9 of this report. Design of shoring should include the effect surcharges where appropriate.

5.4 Trench Backfill

Utility trenches should be backfilled with compacted fill in accordance with the *Standard Specifications for Public Works Construction,* (SSPWC, "Greenbook"), current edition. Utility trenches can be backfilled with onsite material free of rubble, debris, organic and oversized material up to 3 inches in largest dimension.

Trench backfill may be subdivided in to zones relative to the pipe (i.e., “pipe zone”) and the remaining material required to complete backfill (i.e., “over pipe zone”). Materials used as backfill in each zone includes specific requirements.

5.4.1 Pipe Zone

Prior to backfilling trenches, pipes should be bedded in and covered with either:

(1) **Granular Bedding:** a) ½-inch open grade aggregate; or b) a uniform sand material with a Sand Equivalent (SE) greater-than-or-equal-to (≥) 30, passing the No. 4 U.S. Standard Sieve (or as specified by the pipe manufacturer).

(2) **CLSM:** Controlled Low Strength Material (CLSM) conforming to Section 201-6 of the SPWC. CLSM bedding should be placed to 1-foot (0.3 m) over the top of the conduit, and vibrated. CLSM should **not** be jetted.

Pipe bedding should extend at least 4-inches below the pipeline invert and at least 12 inches over the top of the pipeline. The bedding and shading sand is recommended to be densified in place by vibratory, lightweight compaction equipment. Jetting of the pipe bedding is not recommended to achieve densification; mechanical compaction should be used.

Utility lines below the mat slab can provide a pathway for migration of water from irrigation or other exterior sources of water from entering the building pad and potentially activating expansive characteristics of the soils. Utility trenches that cross the perimeter of the foundation are recommended to include an imperious plug below perimeter and/or where trenches transition from planter areas as bedding sand in trenches can collect water from exterior sources such as landscape irrigation and migrate under the slab.

In regions where the trenches may provide an avenue for water infiltration below the foundation, material used in the pipe zone may consist of a cement-bentonite mixture to fill the trench that is a minimum of 2 feet in length or Controlled Low Strength Material (CLSM) consisting of at least one sack of Portland cement per cubic yard of sand, conforming to Section 201-6 of the 2021 Edition of the *Standard Specifications for Public Works Construction* (Greenbook). The impervious pipe zone backfill is recommended to extend at least 5 feet beyond the transition from landscape to foundation.

5.4.2 <u>Above Pipe Zone</u>

Above the pipe zone, trenches can be backfilled with excavated on-site soils free of debris, organic and oversized material greater than (>) 3 inches in largest dimension. As an option, the entire trench can be backfilled with one-sack CLSM same as presented above for the pipe bedding zone. Oversized rock (>3 inches) should either be removed from any backfill, or pulverized for use in backfill only above the pipe zone.

Native soil backfill over the pipe-bedding zone should be placed in thin lifts, moisture conditioned or dried back, as necessary, and mechanically compacted using a minimum standard of 90% relative compaction (relative to the laboratory modified Proctor maximum **<u>dry</u>** density), relative to the ASTM D1557 laboratory maximum dry density within the building footprint and hardscape areas, or 85% under landscape areas. Backfill above the pipe zone should **<u>not</u>** be jetted. In any case, backfill above the pipe zone (bedding) should be observed and tested by Leighton.

5.5 <u>Cement Type</u>

In general, soil environments that are detrimental to concrete have high concentrations of soluble sulfates and/or pH values of less than 5.5. Section 4.3 of ACI 318 (ACI, 2014) provides specific guidelines for the concrete mix-design when the soluble sulfate content of the soil exceeds 0.1 percent by weight or 1,000 parts per million (ppm).

The results of laboratory testing indicated concentrations of soluble sulfate less than 0.1 percent. The test results indicate a sulfate Exposure Class designation of “S0” appears to be appropriate for the project site based upon criteria presented in ACI 318. However, if the concrete is expected to be in contact with reclaimed water, Type V cement and a water/cement ratio of 0.45 should be used. Samples should be collected from the compacted fill subgrade upon completion of grading to confirm the initial findings of this exploration.

5.6 <u>Corrosion Protection Measures</u>

In general, soil resistivity, which is a measure of how easily electrical current flows through soils, is the most influential factor for corrosion to ferrous metals. The following table presents an approximate relationship between soil resistivity and soil corrosiveness.

Table 7 – Soil Corrosivity as a Function of Resistivity

Soil Resistivity (ohm-cm)	Classification of Soil Corrosiveness
0 to 900	Very severe corrosion
900 to 2,300	Severely corrosive
2,300 to 5,000	Moderately corrosive
5,000 to 10,000	Mildly corrosive
10,000 to >100,000	Very mildly corrosive
Note: [1] ASTM STP 1013 titled *Effects of Soil Characteristics on Corrosion* (February, 1989).	

In addition to resistivity, the concentration of chloride ions can also be used to evaluate corrosive potential to steel, either in the form of reinforcement protected by concrete cover or plain steel substructures, such as steel pipes. As a general guideline, the chloride threshold adopted by Caltrans is a concentration of 500 ppm or greater as determined by California Test 532. Concentrations of chloride ions above the stated concentration or other characteristics such as soil resistivity or redox potential may warrant special corrosion protection measures. The results of corrosivity testing are summarized in the following table.

Table 8 – Summary of Corrosivity Testing

Test Parameter	Test Results LB-2	General Classification of Hazard
Water-Soluble Sulfate in Soil (ppm)	123	Negligible sulfate exposure to buried concrete
Water-Soluble Chloride in Soil (ppm)	100	Non-corrosive to buried reinforced concrete
pH	7.80	Mildly alkaline
Minimum Resistivity (saturated, ohm-cm)	1,000	Severely corrosive to buried ferrous pipes

The results of the resistivity tests indicate that the near surface soil exhibits a severely corrosive potential to buried ferrous metals per ASTM STP 1013. The samples tested for water-soluble chloride content indicate a low potential for corrosion of steel in concrete due to the chloride content of the soil.

Based on the test results, ferrous pipes buried in moist to wet site earth materials should be avoided by using high-density polyethylene (HDPE), polyvinyl chloride (PVC) and/or other non-ferrous pipe when possible provided the pipe walls possess sufficient strength for the embedment and external loading to which the pipe will be subjected. Ferrous pipe can also be protected by polyethylene bags, tape or coatings, di-electric fittings or other means to separate pipe from on-site soils. Samples should be collected from the compacted fill subgrade upon completion of grading to confirm the initial findings of this exploration.

5.7 Surface Drainage

Positive drainage of surface water away from structures is very important. Water should not be allowed to pond adjacent to buildings or new site improvements. Water should not be allowed to flow directly over the slope faces. Positive drainage may be accomplished by providing drainage away at a minimum of 2 percent for earthen surfaces for a lateral distance of at least five feet and further maintained by a swale or drainage path at a gradient of at least 1 percent. Where necessary, drainage paths may be shortened by the use of area drains and collector pipes. Eave gutters are recommended and should reduce water infiltration into the subgrade materials. Downspouts should be connected to appropriate outlet devices.

Irrigation of landscaping should be controlled to maintain, as much as possible, consistent moisture content sufficient to provide healthy plant growth without over watering.

5.8 Pavements

5.8.1 Pavement Subgrades

The pavement subgrades should be prepared as described in Section 5.1 of this report. Adequate drainage (both surface and subsurface) should be provided such that the subgrade soils and aggregate base materials are not allowed to become wet.

Landscape areas must be separated from pavements with concrete curbs and/or edge drains. Excessive over-irrigation will have an adverse impact on adjacent pavements. Irrigation adjacent to pavements, without a deep curb or other cutoff to separate landscaping from paving, will result in premature pavement failure.

5.8.2 Hot Mix Asphalt (HMA) Pavement Sections

Asphalt pavements are anticipated to be feasible for the proposed development. Based on the anticipated subgrade conditions and the results of laboratory testing, the following recommendations for pavement design have been based on assumed design R-value of 5 for the anticipated subgrade. R-value tests should be performed upon completion of grading to verify the design value is appropriate for the actual subgrade conditions. Based on the design procedures outlined in the current Caltrans Highway Design Manual and using an R-value of 78 for the pavement base course, the following flexible pavement sections may be used for Traffic Index values that correspond to various levels of vehicle traffic.

Table 9 – Asphalt Pavement Sections

Traffic Index	Asphalt Concrete (inches)	Base Course (inches)	
		CAB	CMB
4 or less	4	6	7
5.0	4	8	9
6.0	4	12	13
Notes: CAB – Crushed Aggregate Base Course; Caltrans Class 2, Section 26 or SSPWC Section 200-2.2			
CMB – Crushed Miscellaneous Base Course; SSPWC Section 200-2.4			

The asphalt concrete should conform to the specifications outlined in Section 203-6 of the Standard Specifications for Public Works Construction (SSPWC, a.k.a. “Green Book”), and asphalt concrete construction methods should meet the requirements of Section 302-5 of the Green Book.

The base course should conform to requirements of Section 26 of State of California Department of Transportation Standard Specifications (Caltrans), latest edition, or meet the specifications for untreated base as defined in Section 200-2.2 of the SSPWC (Green Book). As an alternate, the base course may comply with the specifications for Crushed Miscellaneous Base per SSPWC Section 200-2.4 with an appropriate increase in thickness.

Prior to placement of the base course, the subgrade soil should be processed to a minimum depth of 8 inches, moisture-conditioned to 2 to 3 percent above optimum moisture content, and recompacted to a minimum of 90 percent relative compaction. Base course should be placed in thin lifts, moisture conditioned, as necessary, and compacted to a minimum of 95 percent relative compaction (ASTM D1557).

5.8.3 Portland Cement Concrete (PCC) Pavements

Portland cement concrete is anticipated for areas of the parking lot subjected to large loads and truck loading/unloading areas. A minimum 6-inch thick Portland Concrete Cement (PCC) pavement section reinforced with No. 3 rebar at a maximum on-center spaced at 18 inches in each direction is recommended. The recommended PCC pavement section was based on the procedures described in the *Guide for Design and Construction of Concrete Parking Lots by the American Concrete Inst*itute (ACI 330R-08). All PCC pavements should have a minimum 28-day concrete compressive strength of 3,000 pounds-per-square-inch (psi). The reinforcement should be evaluated by the civil/structural engineer to accommodate any additional structural loading requirements. Control joints are recommended to be installed at a maximum on-center spacing of 12 feet and should form square panels.

Concrete pavement may be underlain by a minimum 4-inch thick layer of compacted granular base to serve as a leveling mat for construction and to assist in load transfer at construction joints. Due to the planned construction, we anticipate the subgrade will consist of a minimum 2 feet of properly compacted fill compacted to at least 90 percent relative compaction in accordance to ASTM Test Method D1557.

Integral curbs should be used at the perimeter of PCC pavement. Longitudinal joints should be avoided near curbs and gutters. Use of concrete cutoff or edge barriers should be considered at the perimeter of common parking or driveway areas when abutting either open (unfinished) or landscaped areas.

5.8.4 Construction and Performance

All pavement construction should be performed in accordance with the *Standard Specifications for Public Works Construction* (SSPWC, “Greenbook”) or the project specifications as prepared by others. Field observation and periodic testing, as needed during placement of the base course materials, should be undertaken to ensure that the requirements of the project specifications are fulfilled.

The recommended pavement sections are based upon the assumed traffic intensity and the subgrade conditions that are expected to result upon completion of the recommended site preparation. Care should be used in the selection of the pavement section to ensure the traffic intensity is consistent with the design assumption. Premature pavement wear and possible distress should be expected where the pavement section is not appropriate for the actual traffic loading conditions.

The recommended asphalt pavement sections have been based upon a 20-year design life. Operation budgets for the facility are recommended to include allowances for regular pavement maintenance (i.e., seal cracks, slurry seal coat reconditioning, etc.) and the need for periodic repairs to minimize pavement damage. Pavement rehabilitation at 8- to 10-year intervals should also be anticipated to be necessary to achieve the design life.

5.9 Lateral Earth Pressures

Based upon our understanding of the proposed development, permanent earth retaining structures are not planned. However, temporary excavation support may be necessary in conjunction with utility construction and subterranean elevator pits are expected to be required for the facility. Design of these structures are recommended to be based upon either an equivalent active fluid pressure of 45 psf per foot of retained height (pcf) or 65 pcf under at-rest conditions. If temporary excavations are braced at the top of the excavation and at specific design intervals, design may be based upon a uniform horizontal pressure of 29H where H is the height of retained material.

The earth pressure condition, i.e., At-Rest vs. Active, is dependent upon the allowable rotation or deflection of the wall. Free-standing walls separate from the proposed buildings may be designed on the basis of the Active earth pressure condition. However, the walls that are not free to rotate are recommended to be designed under the At-Rest earth pressure condition, such as the walls of underground vault structures.

5.10 Exterior Concrete Flatwork

Exterior concrete slabs-on-grade should have a minimum thickness of 4 inches. Common Type II cement should be adequate for concrete flatwork not exposed to recycled water. Type V cement and a water:cement ratio of 0.45 should be used for concrete exposed to recycled water.

Concrete flatwork should be placed on properly compacted fill. If this material has been disturbed or become dry, the subgrade soil to a depth of 12 inches should be moisture conditioned 2 to 3 percent above the optimum moisture content and recompacted to minimum 90 percent relative compaction.

Minor cracking of concrete after curing due to expansion, drying and shrinkage is normal and should be expected. However, cracking is often aggravated by a high water-to-cement ratio, high concrete temperature at the time of placement, small nominal aggregate size, and rapid moisture loss due to hot, dry, and/or windy weather conditions during placement and curing. Cracking due to temperature and moisture fluctuations can also be expected. Inclusion of joints at frequent intervals and reinforcement will help control the locations of the cracks, and thus improve aesthetics. In addition, the use of low-slump concrete or low water/cement ratios can reduce the potential for shrinkage cracking. Construction or weakened plane joints should be spaced at intervals of 6 feet or less for sidewalks, curbs and gutters. The spacing may be increased to 10 feet for ramps and driveways. If cracking occurs, repairs may be needed to mitigate the trip hazard and/or improve the appearance.

Slab cracking and/or uneven slab surfaces may result due to the presence of uncontrolled fill if the fill is left in place below the slabs. Slab distortion may result in tripping hazards. Standard methods to mitigate trip hazards include the following:

- Saw Cutting
- Grinding
- Patching and ramping
- Removing and replacement

A regular maintenance program should be implemented to mitigate trip hazards and comply with safety requirements.

5.11 Additional Geotechnical Services

The geotechnical recommendations presented in this report are based on subsurface conditions as interpreted from limited subsurface explorations, limited laboratory testing and information available at the time the report is prepared. Leighton Consulting, Inc. should review the site and grading plans when available and comment further on the geotechnical aspects of the project. Geotechnical observation and testing should be conducted during excavation and all phases of grading operations. Our conclusions and recommendations should be reviewed and verified by Leighton Consulting Inc. during earthwork construction and revised accordingly if geotechnical conditions encountered vary from our preliminary findings and interpretations.

Geotechnical observation and testing should be provided during the following activities:

- Grading and excavation of the site;
- During overexcavation and removal/reconditioning of unsuitable soil;
- Subgrade preparation;
- Compaction of all fill materials;
- Utility trench backfilling and compaction;
- Footing excavation and slab-on-grade preparation;
- Pavement subgrade and base preparation;
- Placement of asphalt concrete and/or concrete; and
- When conditions are encountered on site that are not consistent with the conditions described in this report.

6.0 LIMITATIONS

Leighton’s work was performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. No other warranty, express or implied, is made as to the conclusions and professional opinions included in this report.

As in many projects, conditions revealed in excavations may be at variance with preliminary findings. If this occurs, the changed conditions must be evaluated by the geotechnical consultant and additional recommendations be developed as needed.

The identification and testing of hazardous, toxic or contaminated materials was outside the scope of Leighton's work. Should such materials be encountered at any time, or their existence be suspected, all measures stipulated in local, County, State and Federal regulations, as applicable, should be implemented.

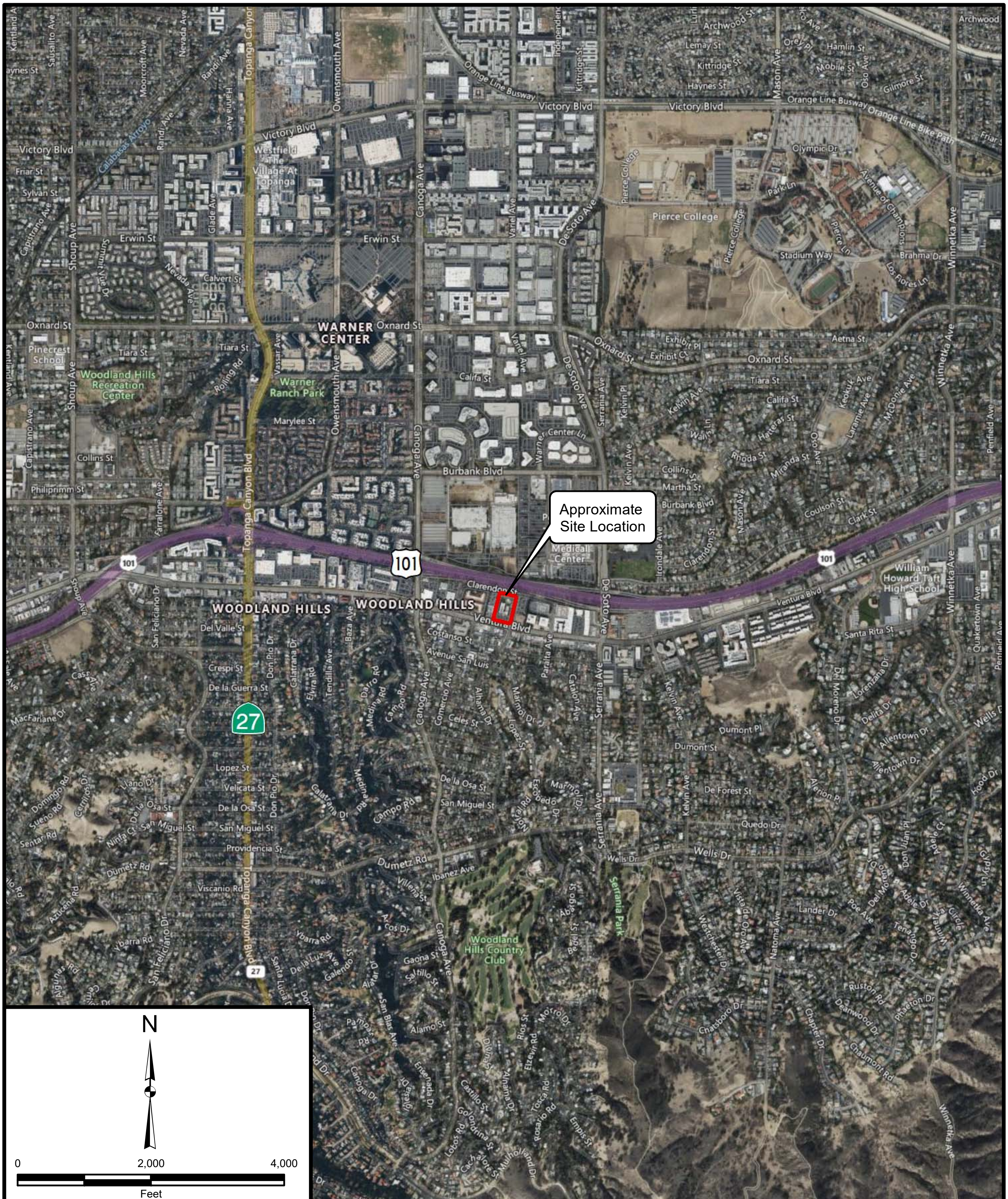
This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the necessary design consultants for the project and incorporated into the plans; and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

The conclusions and recommendations in this report are based in part upon data that were obtained from a necessarily limited number of observations, site visits, excavations, samples and tests. Such information can be obtained only with respect to the specific locations explored, and therefore may not completely define all subsurface conditions throughout the site. The nature of many sites is that differing geotechnical or geological conditions can occur within small distances and under varying climatic conditions. Furthermore, changes in subsurface conditions can and do occur over time. Therefore, the findings, conclusions and recommendations presented in this report can be relied upon only if Leighton has the opportunity to perform additional investigation of the site, finalize and complete the conclusions and recommendations presented in this report, and finally to observe the subsurface conditions during grading and construction of the project, in order to verify that our preliminary findings are representative of the site.

This report is intended only for the use of Johnson Development Associates, Inc. and its design consultants, and only as related expressly to evaluation of the feasibility of developing the subject site with the proposed development and for preliminary planning and design.

If parties other than Leighton are engaged to provide construction geotechnical services, they must be notified that they will be required to assume complete responsibility for the geotechnical phase of the project by concurring with the findings and recommendations in this report or by providing alternative recommendations.

Any persons using this report for bidding or construction purposes should perform such independent explorations as they deem necessary to satisfy themselves as to the surface and subsurface conditions to be encountered and the procedures to be used in the performance of work on the subject site.



Project: 13589.002	Eng/Geol: JEH/JAR
Scale:1 " = 2,000 '	Date: July 2022

Reference: © 2022 Microsoft Corporation © 2022 Maxar ©CNES (2022) Distribution Airbus

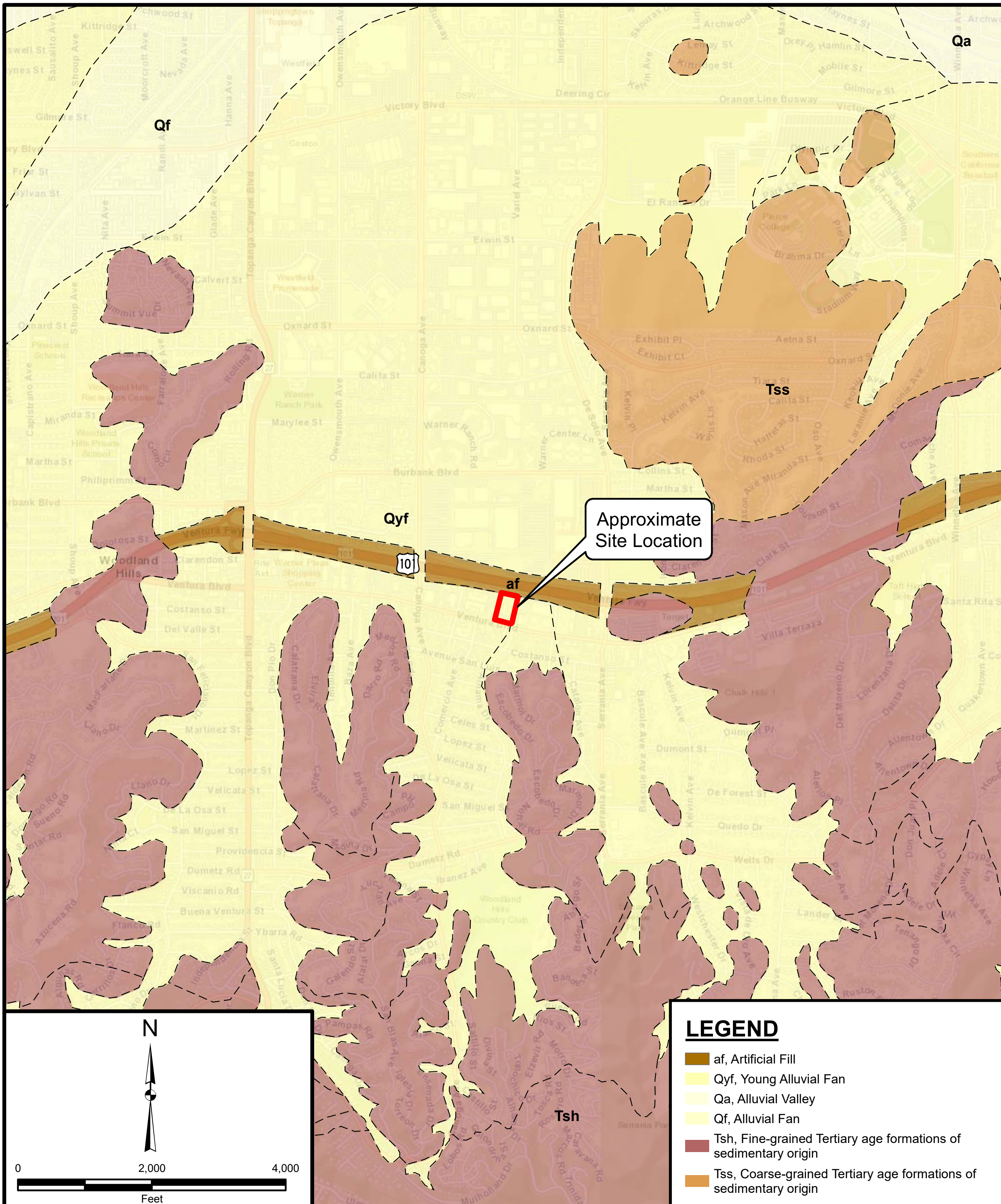
SITE LOCATION MAP

21101 Ventura Boulevard
Woodland Hills, California

FIGURE 1

Leighton

Map Saved as J:\Drafting\13589\002\Maps\13589-002_F01_SLM_2022-07-13.mxd on 7/12/2022 5:28:24 PM Author: KVM (btran)



LEGEND

- af, Artificial Fill
- Qyf, Young Alluvial Fan
- Qa, Alluvial Valley
- Qf, Alluvial Fan
- Tsh, Fine-grained Tertiary age formations of sedimentary origin
- Tss, Coarse-grained Tertiary age formations of sedimentary origin

Project: 13589.002	Eng/Geol: JEH/JAR
Scale:1 " = 2,000 '	Date: August 2022

Reference:
State of California, Department of Conservation, California Geological Survey, 2012.

REGIONAL GEOLOGY MAP

21101 Ventura Boulevard
Woodland Hills, California

FIGURE 2

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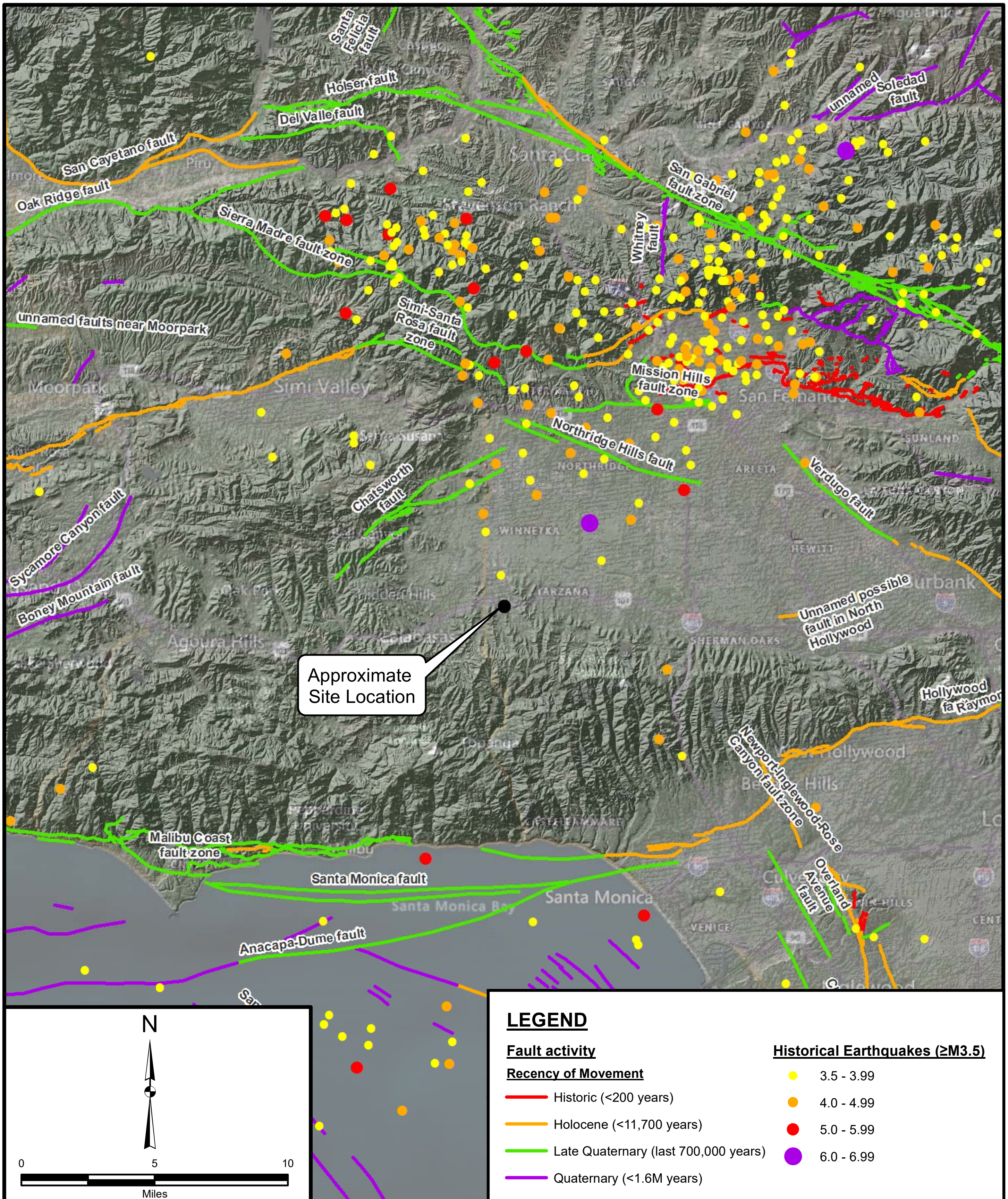
Map Saved as J:\Drafting\13589\002\Maps\13589-002_F03_RGM_2022-07-13.mxd on 7/13/2022 10:37:55 AM Author: KVM (btran)



Project: 13589.002	Eng/Geol: JEH/JAR
Scale: 1 " = 5 miles	Date: August 2022

Base Map: ESRI ArcGIS Online 2022
Reference: maps.conservation.ca.gov

REGIONAL FAULT AND HISTORIC SEISMICITY MAP

21101 Ventura Boulevard
Woodland Hills, California

FIGURE 3

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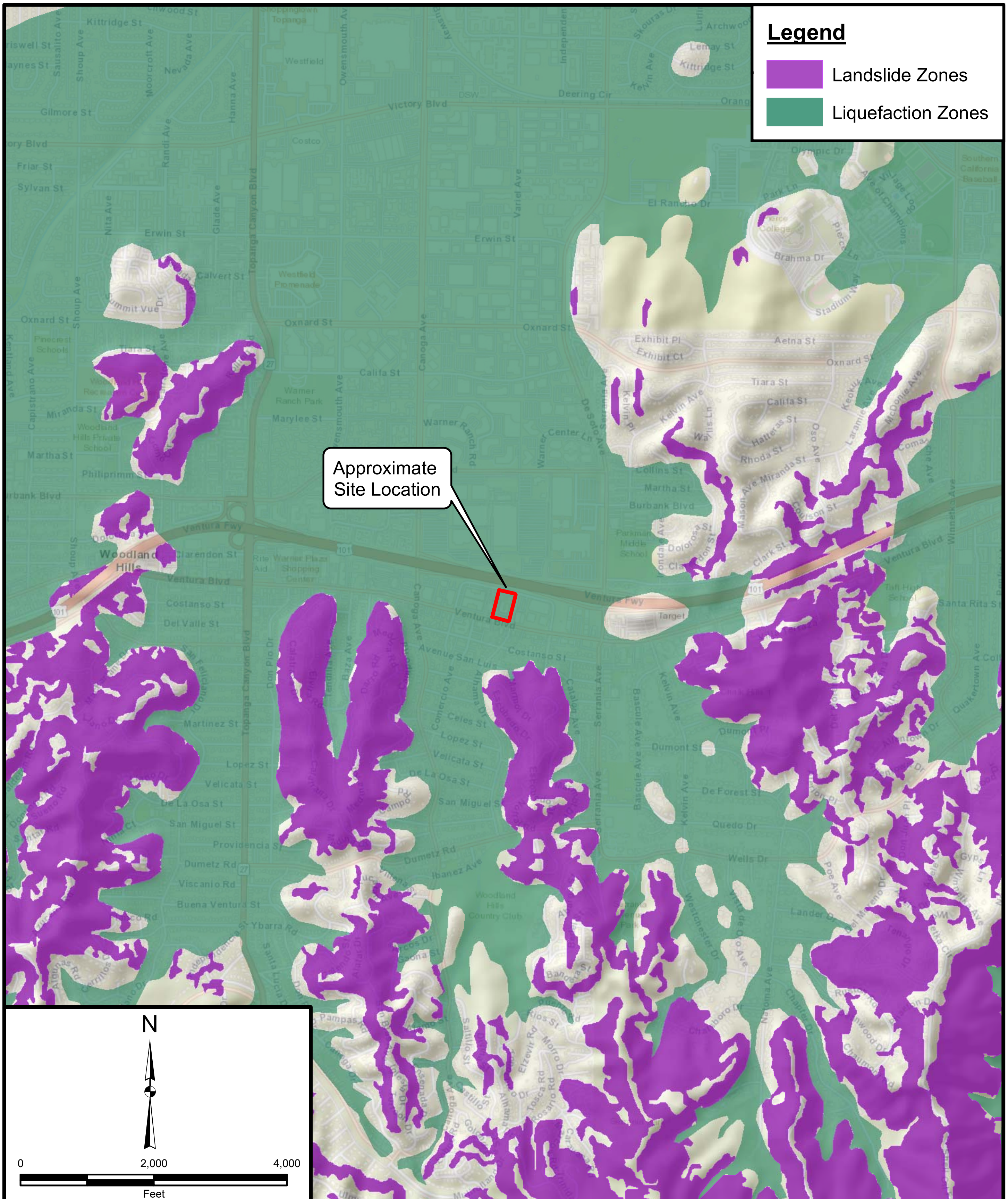
Map Saved as J:\Drafting\13589\002\Maps\13589-002_F04_RF&HSM_2022-07-13.mxd on 7/13/2022 10:46:00 AM Author: KVM (btran)



Project: 13589.002	Eng/Geol: JEH/JAR
Scale:1 " = 2,000 '	Date: August 2022

Base Map: ESRI ArcGIS Online 2022
Reference: maps.conservation.ca.gov

SEISMIC HAZARD MAP

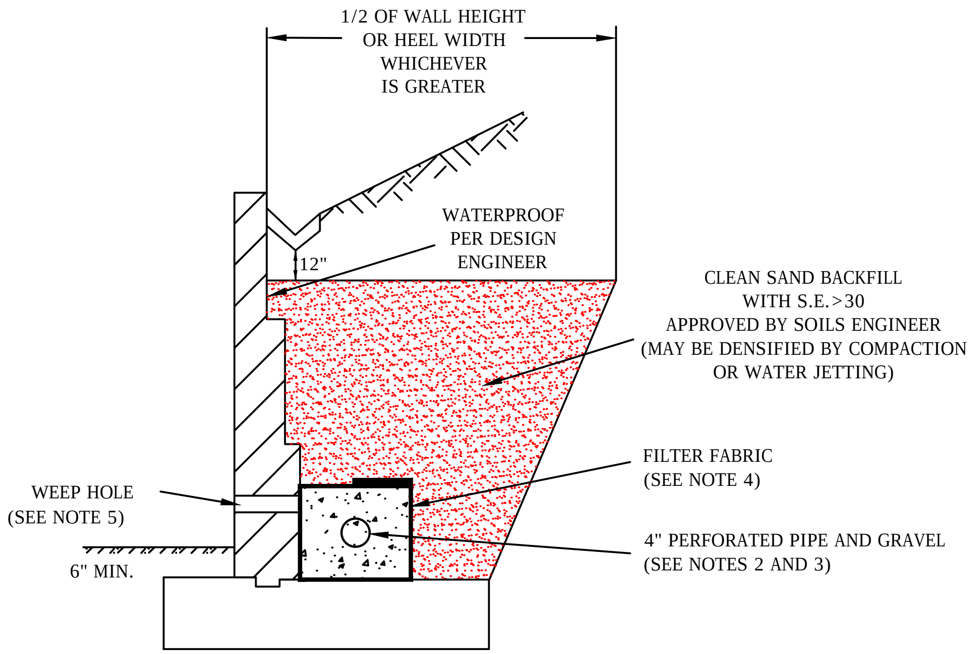
21101 Ventura Boulevard
Woodland Hills, California

FIGURE 4

Leighton

Map Saved as J:\Drafting\13589\002\Maps\13589-002_F05_SHM_2022-07-13.mxd on 7/13/2022 10:43:59 AM Author: KVM (btran)

SUBDRAIN OPTIONS AND BACKFILL WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF >50



NOTE: AS AN ALTERNATE TO CLEAN SAND BACKFILL, CLEAN GRAVEL MAY BE UTILIZED WITH APPROVED FILTER FABRIC. A SECOND ALTERNATE IS TO UTILIZE AN AGGREGATE BASE MATERIAL COMPACTED TO 90% RELATIVE COMPACTION. A SAMPLE OF THE PROPOSED BASE MUST BE APPROVED BY THE GEOTECHNICAL CONSULTANT PRIOR TO BACKFILL FOR SUITABILITY. COMPACTION SHOULD BE ACHIEVED WITHOUT DAMAGING THE WALL.

GENERAL NOTES:

* Waterproofing should be provided where moisture nuisance problem through the wall is undesirable.
* Water proofing of the walls is not under purview of the geotechnical engineer
* All drains should have a gradient of 1 percent minimum
*Outlet portion of the subdrain should have a 4-inch diameter solid pipe discharged into a suitable disposal area designed by the project engineer. The subdrain pipe should be accessible for maintenance (rodding)
*Other subdrain backfill options are subject to the review by the geotechnical engineer and modification of design parameters.

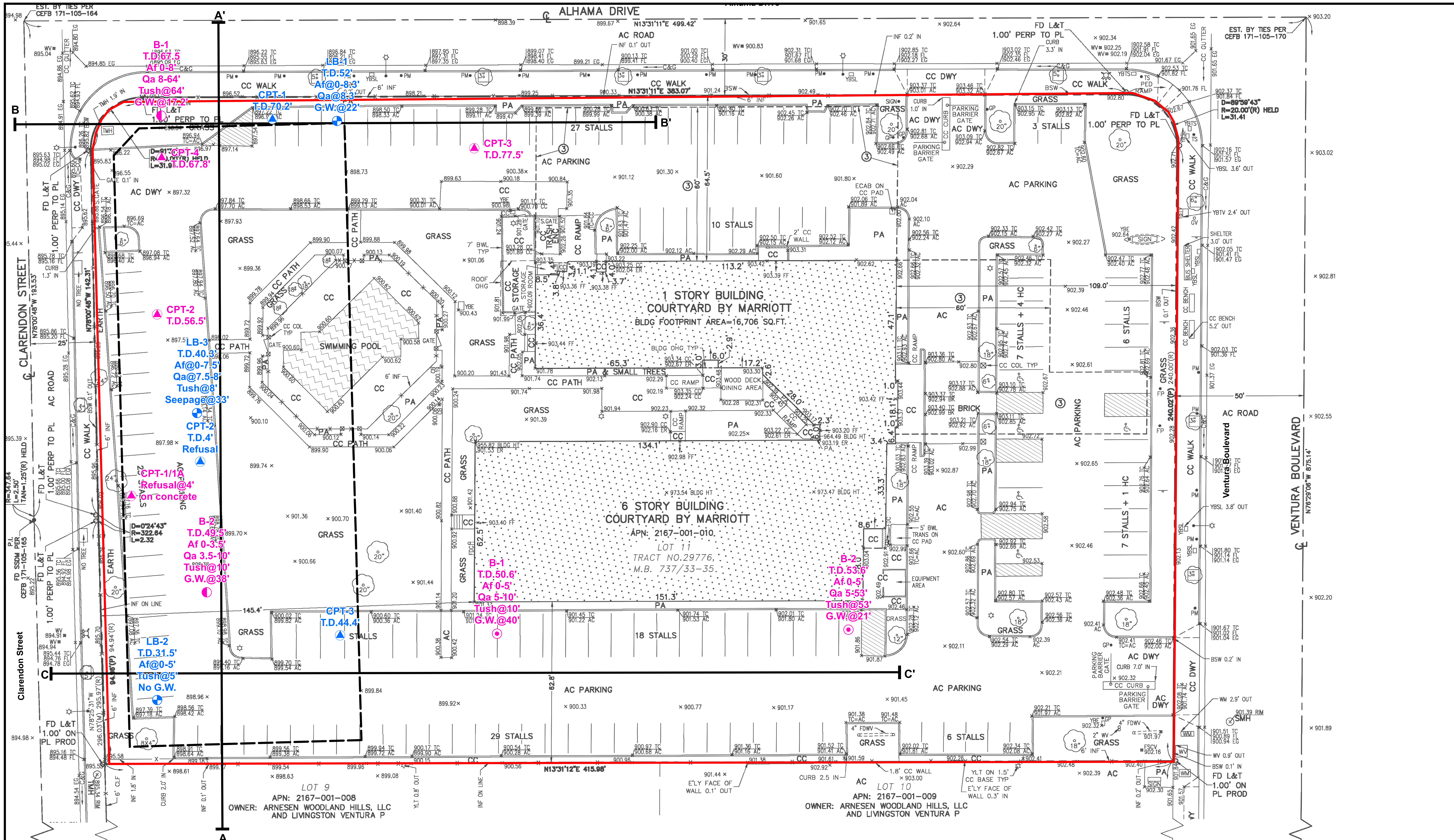
Notes:
1) Sand should have a sand equivalent of 30 or greater and may be densified by water jetting.
2) 1 Cu. ft. per ft. of 1/4- to 1 1/2-inch size gravel wrapped in filter fabric
3) Pipe type should be ASTM D1527 Acrylonitrile Butadiene Styrene (ABS) SDR35 or ASTM D1785 Polyvinyl Chloride plastic (PVC), Schedule 40, Armco A2000 PVC, or approved equivalent. Pipe should be installed with perforations down. Perforations should be 3/8 inch in diameter placed at the ends of a 120-degree arc in two rows at 3-inch on center (staggered)
4) Filter fabric should be Mirafi 140NC or approved equivalent.
5) Weephole should be 3-inch minimum diameter and provided at 10-foot maximum intervals. If exposure is permitted, weepholes should be located 12 inches above finished grade. If exposure is not permitted such as for a wall adjacent to a sidewalk/curb, a pipe under the sidewalk to be discharged through the curb face or equivalent should be provided. For a basement-type wall, a proper subdrain outlet system should be provided.
6) Retaining wall plans should be reviewed and approved by the geotechnical engineer.
7) Walls over six feet in height are subject to a special review by the geotechnical engineer and modifications to the above requirements.

RETAINING WALL BACKFILL AND SUBDRAIN DETAIL

WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF >50

Leighton

FIGURE 5



NOTES

1. REFERENCE DOCUMENT FOR LEGAL DESCRIPTION AND EASEMENTS: FIRST AMERICAN TITLE COMPANY PRELIMINARY REPORT ORDER NUMBER NCS-734482-LA2, DATED JUNE 1, 2015.
2. SITE ADDRESS: 21101 VENTURA BLVD, WOODLAND HILLS, CA 91364.
3. ASSESSOR'S PARCEL NUMBER: 2167-001-010.
4. LAND AREA: 109,035 SQ. FT. 2.503 ACRES
5. BUILDING AREAS ARE BASED ON OUTSIDE FOOTPRINT DIMENSIONS.
6. CURRENT ZONING:
   C4-1LD (COMMERCIAL) AND P-1LD (AUTOMOBILE PARKING-SURFACE AND UNDERGROUND)
7. BUILDING SETBACKS PER CITY OF LOS ANGELES ZONING:

   ZONE C4:
   FRONT: NONE
   SIDE AND REAR : NONE FOR COMMERCIAL USES;
   SAME AS R4 ZONE FOR RESIDENTIAL USES AT LOWEST RESIDENTIAL STORY.

   ZONE P:
   FRONT: 10FT IN COMBINATION WITH AN A OR R ZONE; OTHERWISE NONE
   SIDE AND REAR: AREA REGULATIONS OF A AND R ZONE APPLY FOR RESIDENTIAL USE.
8. HEIGHT RESTRICTION FOR ZONE C4-1LD AND P-1LD:
   NOT TO EXCEED 6 STORIES OR 75 FT PER CITY OF LOS ANGELES ZONING.
9. FLOOR SPACE AREA RESTRICTION: 3:1 PER CITY OF LOS ANGELES ZONING.
10. EXISTING PARKING STALLS
    REGULAR: 143
    HANDICAPPED: 5
11. FLOOD ZONE NOTE:

THE PROPERTY SHOWN ON THIS SURVEY DOES NOT LIE WITHIN A SPECIAL FLOOD HAZARD AREA AS DEFINED BY THE FEDERAL EMERGENCY MANAGEMENT AGENCY (FEMA). THE PROPERTY LIES WITHIN ZONE X (OUTSIDE FLOOD AREA) PER FLOOD INSURANCE RATE MAP OF THE CITY OF LOS ANGELES AS COMMUNITY PANEL NUMBER 06037C-1290F DATED SEPTEMBER 26, 2008.

12. NO OBSERVED EVIDENCE OF CURRENT EARTH MOVING WORK, BUILDING CONSTRUCTION OR BUILDING ADDITIONS.
13. NO PROPOSED CHANGES IN STREET RIGHT OF WAY LINES, IF INFORMATION IS AVAILABLE FROM THE CONTROLLING JURISDICTION. NO OBSERVED EVIDENCE OF RECENT STREET OR SIDEWALK CONSTRUCTION OR REPAIRS.
14. NO OBSERVED EVIDENCE OF SITE USE AS A SOLID WASTE DUMP, SUMP OR SANITARY LANDFILL.
15. NO LOCATION OF WETLAND AREAS AS DELINEATED BY APPROPRIATE AUTHORITIES.

LEGAL DESCRIPTION

LOT 11 OF TRACT NO. 29776, IN THE CITY OF LOS ANGELES, COUNTY OF LOS ANGELES, STATE OF CALIFORNIA, AS PER MAP RECORDED IN BOOK 737 PAGES 33 TO 35 INCLUSIVE OF MAPS, IN THE OFFICE OF THE COUNTY RECORDER OF SAID COUNTY.

EASEMENT NOTE

3 TO MAINTAIN A 60 FEET YARD UNOBSTRUCTED FROM GROUND TO CEILING AS STATED IN THE TERMS AND PROVISIONS CONTAINED IN THE DOCUMENT ENTITLED "COVENANT AND AGREEMENT" RECORDED APRIL 1, 1965 AS INSTRUMENT NO. 3886 IN BOOK M-1817 PAGE 832 OF OFFICIAL RECORDS. PLOTTED.

BASIS OF BEARINGS

THE CENTERLINE OF ALHAMA DRIVE, BEING N 13°31'11" E PER TRACT NO. 29776, M.B. 737/33-35.

BENCH MARK

CITY OF LOS ANGELES BENCH MARK NO. 06-05942

ELEV: 902.624'
DATUM: NAVD 1988
YEAR OF ADJUSTMENT: 2000

SURVEY CERTIFICATION

TO: BANK OF AMERICA, N.A., AS LENDER, WH HOTEL, LLC, A DELAWARE LIMITED LIABILITY COMPANY, AS BORROWER, AND FIRST AMERICAN TITLE COMPANY, ITS SUCCESSORS AND ASSIGNS, AS TITLE COMPANY:

THIS IS TO CERTIFY THAT THIS MAP OR PLAT AND THE SURVEY ON WHICH IT IS BASED WERE MADE IN ACCORDANCE WITH THE 2011 MINIMUM STANDARD DETAIL REQUIREMENTS FOR ALTA/ACSM LAND TITLE SURVEYS, JOINTLY ESTABLISHED AND ADOPTED BY ALTA, ACSM AND NSPS, AND INCLUDES ITEMS 1, 2, 3, 4, 5, 6(a), 6(b), 7(a), 7(b)(1), 7(c), 8, 9, 11(a), 13, 14, 16, 17, 18, 19, 20(a), 20(b) AND 21 OF TABLE A THEREOF. PURSUANT TO THE ACCURACY qTZ1hkhxs8PnxiOPWÝ?Ū?1ŏ????.?Ðœc?STANDARDS AS ADOPTED BY ALTA, NSPS AND ACSM AND IN EFFECT ON THE DATE OF THIS CERTIFICATION, UNDERSIGNED FURTHER CERTIFIES THAT THE SURVEY MEASUREMENTS WERE MADE IN ACCORDANCE WITH THE "MINIMUM ANGLE, DISTANCE, AND CLOSURE REQUIREMENTS FOR SURVEY MEASUREMENTS WHICH CONTROL LAND BOUNDARIES FOR ALTA/ACSM LAND TITLE SURVEYS." THE FIELD WORK WAS COMPLETED ON MAY 29, 2015.

Ofer Shapira
OFER SHAPIRA
LICENSE NO.: L.S. 7123
EXPIRES: 12/31/2015

6-3-15
DATE

LEGEND

LB-3	Approximate location of hollow stem auger boring showing total depth (T.D.), depth to earth units and groundwater (G.W.) in feet below ground surface (Leighton, 2022)
CPT-3	Approximate location of cone penetrometer test (CPT) in feet below ground surface, (Leighton, 2022)
B-2	Approximate location of hollow stem auger boring showing total depth (T.D.), depth to earth units and groundwater (G.W.) in feet below ground surface (Geocon, 2017)
B-2	Approximate location of hollow stem auger boring showing total depth (T.D.), depth to earth units and groundwater (G.W.) in feet below ground surface (Geocon, 2020)
CPT-9	Approximate location of cone penetrometer test (CPT) sounding showing total depth (T.D.), depth to earth units and groundwater (G.W.) in feet below ground surface (Geocon, 2020)
Af	Artificial Fill
Qa	Quaternary alluvium sandy clay to clayey silt
Tush	Tertiary unnamed shale. Bedrock, siltstone and claystone
	Limit of Proposed Building
C C'	Geotechnical Cross Section, See Plate 2
	Approximate Site Boundary

YARD LIGHT
FIRE HYDRANT
SURFACE DRAIN
SIGN
10" TREE WITH TRUNK DIAMETER
(M) MEASURED
(P) PRORATED
(R) TRACT NO. 29776, M.B. 737/33-35
1 PRELIMINARY TITLE REPORT SCHEDULE B, ITEM NUMBER SEE EASEMENT NOTE

ABBREVIATIONS

AC	ASPHALT CONCRETE	L&T	LEAD AND TACK
BK	BRICK	OHG	OVERHANG
BLDG	BUILDING	PA	PLANTING AREA
BSW	BACK OF SIDEWALK	PED	PEDESTRIAN
BWL	BLOCK WALL	PERP	PERPENDICULAR
C&G	CURB & GUTTER	PM	PARKING METER
CC	CONCRETE	S. GATE	SLIDE GATE
CLF	CHAIN LINK FENCE	SMH	SEWER MANHOLE
COL	COLUMN	SQ. FT.	SQUARE FEET
DWY	DRIVEWAY	TC	TOP OF CURB
ECAB	ELECTRICAL CABINET	TMH	TELEPHONE MANHOLE
EG	EDGE OF GUTTER	TRANS	TRANSFORMER
ENC	ENCLOSURE	TS	TRAFFIC SIGNAL
ER	EARTH	TW	TOP OF WALL
FDC	FIRE DEPARTMENT CONNECTION	TYP	TYPICAL
FDWV	FIRE DEPARTMENT WATER VALVE	WM	WATER METER
FF	FINISH FLOOR	WV	WATER VALVE
FL	FLOW LINE	YBE	YARD BOX ELECTRICAL
FSCV	FIRE SPRINKLER CONTROL VALVE	YBSL	YARD BOX STREET LIGHT
GP	GUARD POST	YBTS	YARD BOX TRAFFIC SIGNAL
GV	GAS VALVE	YBTV	YARD BOX TELEVISION
INF	IRON FENCE	YLT	YARD LIGHT

PLATE 1	GEOTECHNICAL MAP Proposed Self Storage Facility 21101 Ventura Boulevard Woodland Hills (Los Angeles), California
Scale: 1 " = 20 feet	
Date: August 2022	
Proj: 13589.002	Leighton
Eng/Geol: JEH/JAR	

Base Map: ALTA/ACSM LAND TITLE SURVEY, Sheet 1 of 1, Dated: 6/3/2015

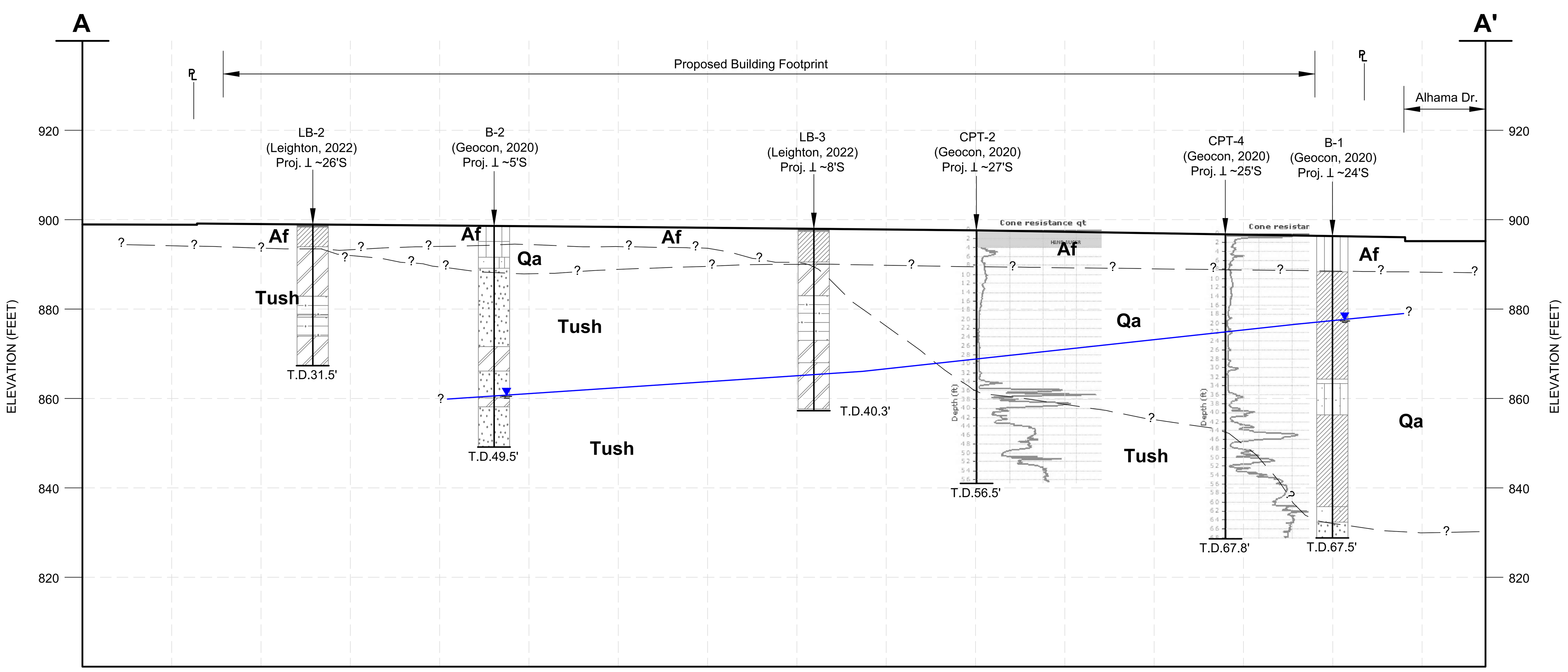
0 20 40
Feet

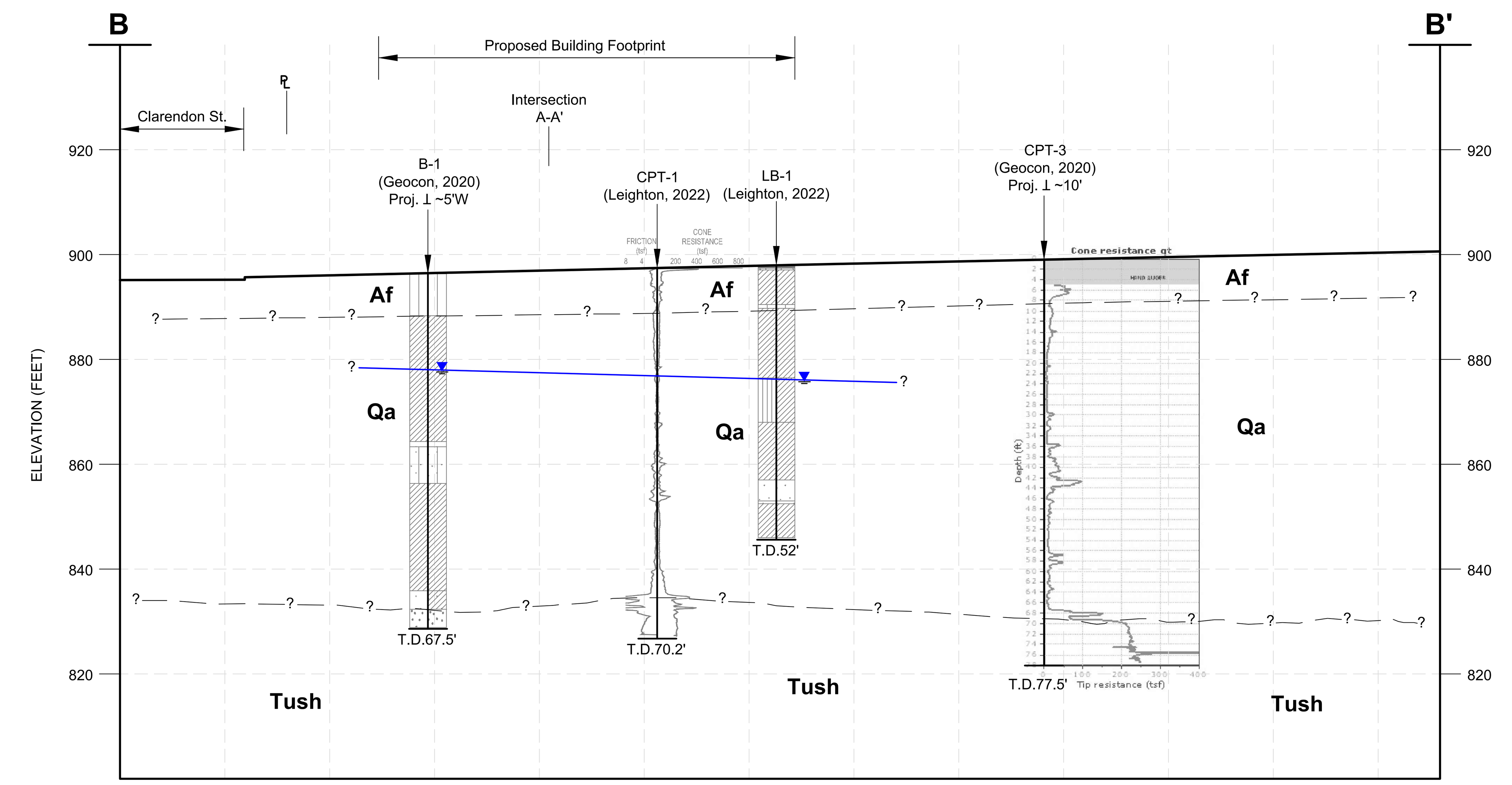
Surveying & Drafting Services, Inc.
901 Seward Street, Los Angeles, CA., 90038
Tel: (323) 366-2882 Fax: (323) 857-1079
Email: mailbox@surveyinganddrafting.com www.surveyinganddrafting.com

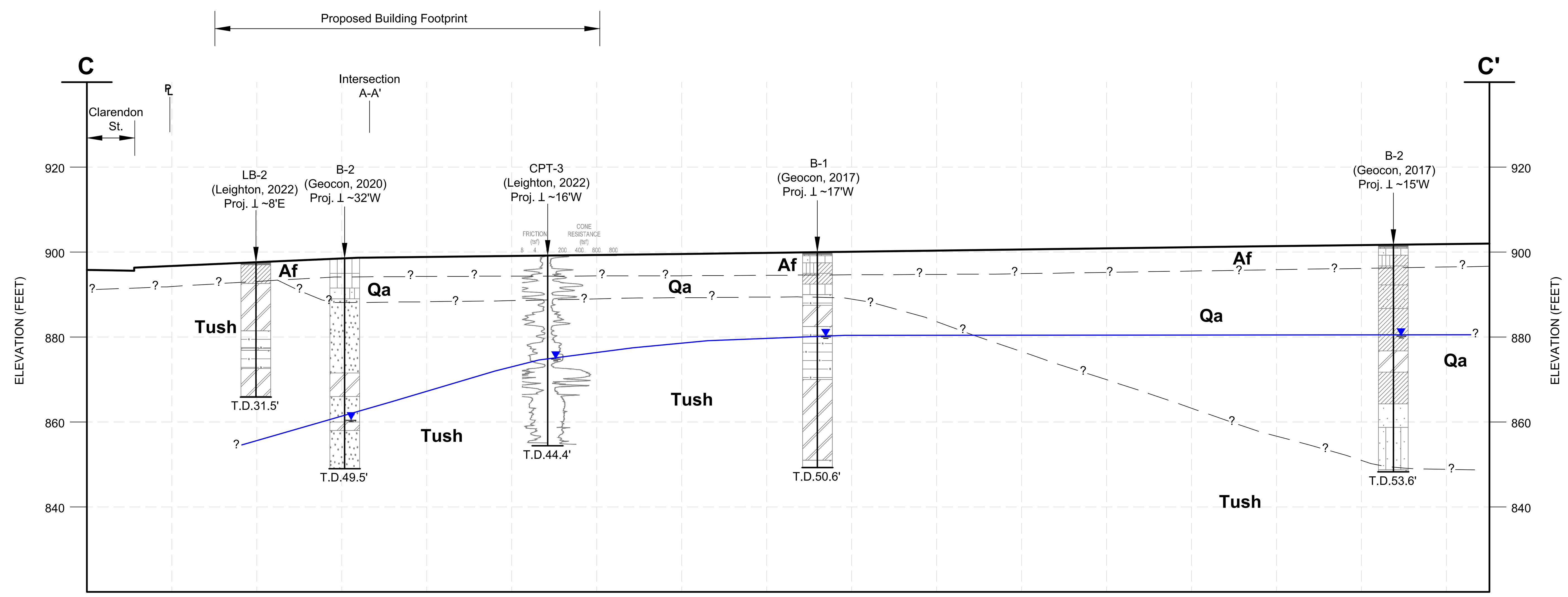
LICENSED LAND SURVEYOR OFER SHAPIRA NO. 7123 Exp. 12-31-15 STATE OF CALIFORNIA

ALTA/ACSM LAND TITLE SURVEY
THE COURTYARD BY MARRIOTT HOTEL
21101 VENTURA BLVD. WOODLAND HILLS, CALIFORNIA 91364

DATE: 6-3-15	DRAWN BY: A.S. , V.L.
DATE OF SURVEYS: 10-25-11, 5-23-13, 5-29-15	CHECKED BY: O.S.
JOB NAME: WOLFF/COURTYARD BY MARRIOTT	SHEET: 1 OF 1







Note:

See Plate 1 for explanation of units

Approximate Groundwater Depth

GEOTECHNICAL CROSS SECTIONS A-A' THROUGH C-C'	PLATE 2
21101 Ventura Boulevard Woodland Hills, California	Scale: 1"=20'
	Date: August 2022
Leighton	Proj: 13589.002
	Eng/Geol: JEH/JAR

J:\DRAFTING\13589\002\CAD\2022-07-13\13589-002_P02-CS-ABC_2022-08-03.DWG (08-03-22 1:34:05PM) Plotted by: btran

APPENDIX A

REFERENCES

Leighton

APPENDIX A

REFERENCES

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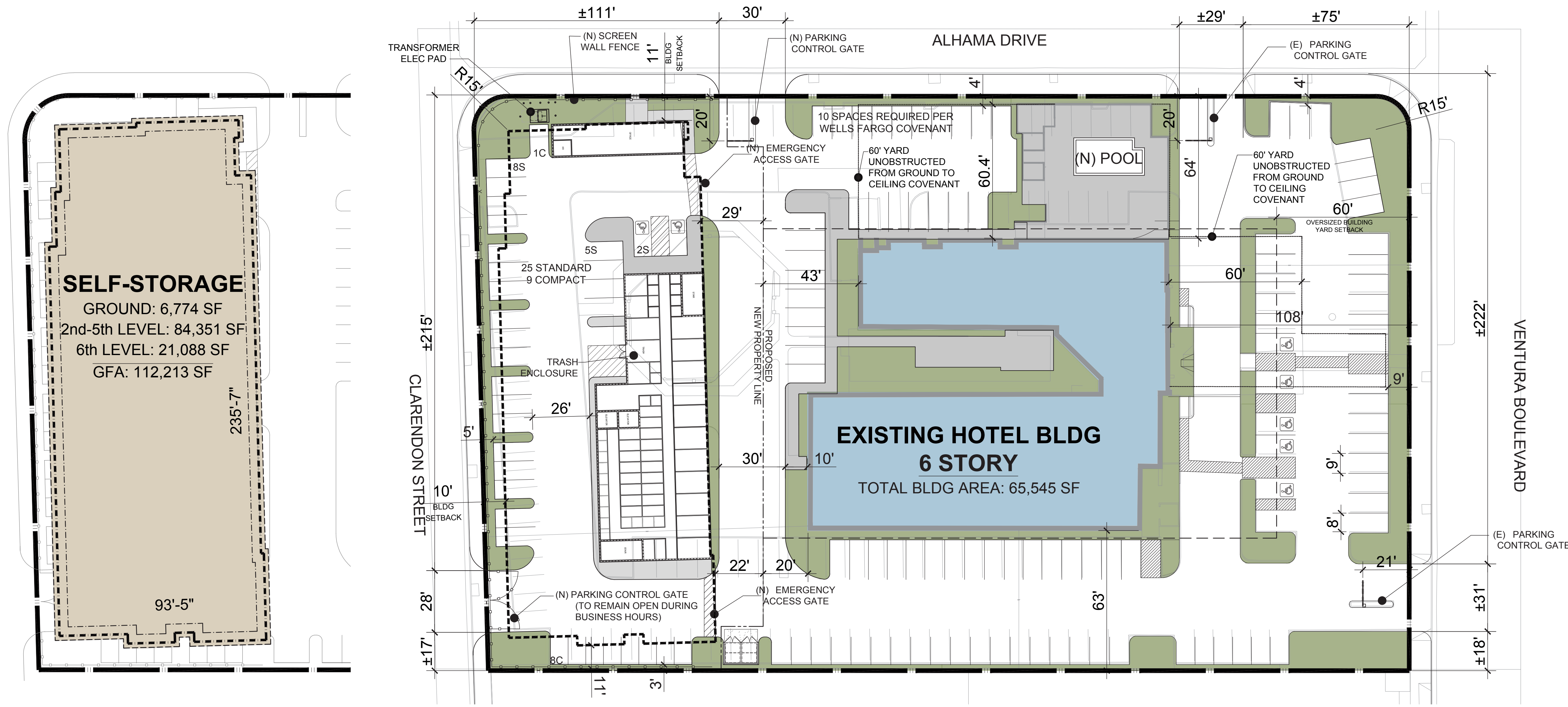
APPENDIX A1

DEVELOPMENT CONCEPT

Leighton



DEVELOPMENT STANDARDS:

ZONING:	SP (VCBC)
MAX. F.A.R.:	1.25:1
MAX. COVERAGE:	75%
MAX. HEIGHT:	45 FT
BUILDING SETBACKS:	
FRONT:	10 FT
SIDE:	0 FT [2]
REAR:	15 FT [3]
LANDSCAPE SETBACKS:	
FRONT:	10 FT
SIDE:	10 FT
REAR:	10 FT
LANDSCAPE REQ.:	15%
OFF-STREET PARKING:	
STANDARD:	8.5X18
COMPACT:	8X15
COMPACT %:	40%
DRIVE AISLE:	28 FT
REQ. PARKING RATIO BY USE:	
STORAGE:	1/500 SF [1]

NOTES:

1 Plus 1 per 5000 beyond the first 10,000sf.

2 No side yards except that an accessway which may include a max of 20' wide driveway, a max of 4 fot walk way and landscape buffers of 18" to 5 feeton either side may be provided.

3 if rear lot line is adjacent to street, then there should be a min of 15' rear yard. 20' If adjacent to residential

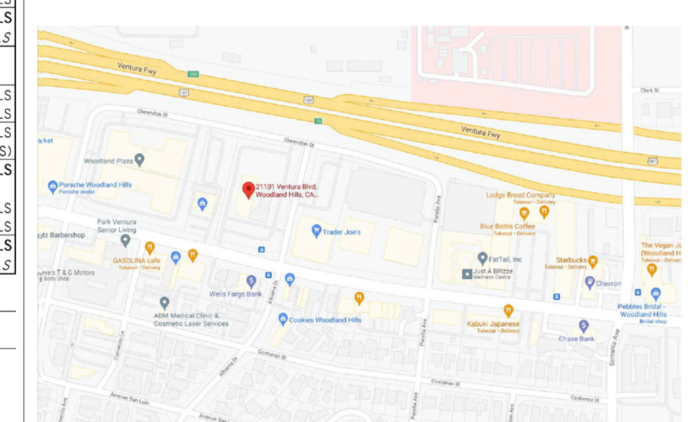
PROJECT DATA:		
SITE AREA		
EXISTING HOTEL PARCEL		
PROPOSED SELF- STORAGE		
GROSS:		2.50 AC
		109,065 SF
BUILDING AREA:		
EXISTING HOTEL		
GROUND FLOOR		18,080 SF
SECOND FLOOR		9,493 SF
THIRD FLOOR		9,493 SF
FOURTH FLOOR		9,493 SF
FIFTH FLOOR		9,493 SF
SIXTH		9,493 SF
TOTAL HOTEL BLDG AREA		**65,545 SF**
PROPOSED SELF-STORAGE		
GROUND FLOOR & OFFICE/RETAIL		6,060 SF
SECOND FLOOR		21,130 SF
THIRD FLOOR		21,130 SF
FOURTH FLOOR		21,130 SF
FIFTH FLOOR		21,130 SF
SIXTH FLOOR		19,890 SF
TOTAL SELF-STORAGE AREA:		**110,470 SF**
GRAND TOTAL BUILDING AREA:		**176,015 SF**
FAR:		
EXISTING HOTEL		
PROVIDED:		#DIV/0!
MAXIMUM ALLOWED		1.25
MAXIMUM ALLOWED (IN SF)		136,331 SF
PROPOSED LANDSCAPE	#DIV/0!	**23,156 SF**
PROPOSED SELF-STORAGE		
PROVIDED:		**1.61**
MAXIMUM ALLOWED		1.25
MAXIMUM ALLOWED (IN SF)		136,331 SF
PROPOSED LANDSCAPE	21%	**23,156 SF**

PARKING REQUIRED:		
HOTEL		
TOTAL ROOMS		122 UNITS
1st 30 GUESTROOMS	1 SPACE/UNIT	30 STALLS
2nd 30 GUESTROOMS	0.5 SPACE/UNIT	15 STALLS
REMAINING GUESTROOMS	0.3 SPACE/UNIT	23 STALLS
SUB-TOTAL		**68 STALLS**
PARKING PROVIDED:		
STANDARD:		61 STALLS
COMPACT:	38%	37 STALLS
TOTAL:		**98 STALLS**
REQ. ACCESSIBLE		*4 STALLS*
PARKING REQUIRED:		
SELF STORAGE		
1ST 10K	1/500 SF	20 STALLS
10K over	1/5000 SF	20 STALLS
SUB-TOTAL		40 STALLS
20% BICYCLES SWAP REDUCTION		(-8 STALLS)
TOTAL PARKING REQUIRED:		**32 STALLS**
PARKING PROVIDED:		
STANDARD:		25 STALLS
COMPACT:	26%	9 STALLS
TOTAL:		**34 STALLS**
REQ. ACCESSIBLE		*2 STALLS*

BUILDING HEIGHT (MAX HEIGHT ALLOWED IS 45'-0")	
EXISTING HOTEL	
EXISTING BLDG HEIGHT:	± 71'-6"
PROPOSED SELF-STORAGE	
GROUND FLOOR	16'-0"
SECOND FLOOR	10'-8"
THIRD FLOOR	10'-8"
FOURTH FLOOR	10'-8"
FIFTH FLOOR	10'-8"
SIXTH FLOOR	10'-8"
TOTAL	69'-4"
TOTAL (AT HIGHEST POINT OF PARAPET)	80'-4"

This conceptual design is based upon a preliminary review of entitlement requirements and on unverified and possibly incomplete site and/or building information, and is intended merely to assist in exploring how the project might be developed.

Boundary Source:
PDF ALTA SURVEY



1" =20'

0 10 20 40 100

NORTH

JOHNSON DEVELOPMENT ASSOCIATES, INC.
Part of TheJohnsonGroup

This conceptual design is based upon a preliminary review of entitlement requirements and on unverified and possibly incomplete site and/or building information, and is intended merely to assist in exploring how the project might be developed. Signage shown is for illustrative purposes only and does not necessarily reflect municipal code compliance. All colors shown are for representative purposes only. Refer to material samples for actual color verification.

SITE PLAN

JDA WOODLAND HILLS

WOODLAND HILLS, CA LAX21-0081-00

WARE MALCOMB 11.21.2022

APPENDIX B

FIELD EXPLORATION

BORING LOG NUMBER 1

Wolff Urban Management | **Date: 05/15/17** | **Elevation: 900.8'***

File No. 21430 | **Method: 8-inch diameter Hollow Stem Auger**

km | *Reference: Survey Map by Surveying & Drafting Services, Inc., dated 3/26/12

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0		7-inch Thick Asphalt over 3-inch Thick Base
				1		FILL: Silty Sand to Sandy Silt, dark brown, moist, fine grained
				2		
2.5	40	19.0	104.1			
				3		Sandy Silt to Silty Clay, dark brown and gray, moist
				4		
5	21	17.9	SPT	5		
					CL	ALLUVIUM: Sandy Lean Clay, dark gray, moist, very stiff, fine Sand
				6		
				7		
7.5	15	16.3	102.2			
				8		Sandy to Clayey Silt, dark brown and gray mottling, moist, stiff
				9		
10	13	27.0	SPT	10		
						BEDROCK (MODELO FORMATION): Siltstone, light olive gray, moist, medium hard
				11		
				12		
12.5	41	41.8	77.0			
				13		Claystone, brown to olive gray
				14		
15	16	49.1	SPT	15		
						yellowish brown
				16		
				17		
17.5	63	27.0	97.9			
				18		Clayey Siltstone, olive gray and orange mottling
				19		
20	76	24.1	SPT	20		
						Siltstone, olive gray and orange mottling, fine grained
				21		
				22		
				23		
				24		
				25		

BORING LOG NUMBER 1

Wolff Urban Management

File No. 21430

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26 --		
				27 --		
				28 --		
				29 --		
30	62	47.6	72.7	30 --		Claystone, dark brown, moist, cemented
				31 --		
				32 --		
				33 --		
				34 --		
				35 --		
				36 --		
				37 --		
				38 --		
				39 --		
40	100/8"	38.9	77.8	40 --		
				41 --		
				42 --		
				43 --		
				44 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				45 --		Used 8-inch diameter Hollow-Stem Auger
				46 --		140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				47 --		SPT=Standard Penetration Test
				48 --		
				49 --		
50	100/7"	24.6	97.9	50 --		Siltstone, black, moist, some Clay
				51 --		Total Depth 50.6 feet; Water at 40 feet; Fill to 5 feet

BORING LOG NUMBER 2

Wolff Urban Management — **Date: 05/15/17** — **Elevation: 901.9'***

File No. 21430 — **Method: 8-inch diameter Hollow Stem Auger**

km — *Reference: Survey Map by Surveying & Drafting Services, Inc., dated 3/26/12

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		7-inch Thick Asphalt over 4-inch Thick Base
				-		
				1 --		FILL: Silty Clay, dark and medium brown, moist, stiff
				-		
				2 --		
2.5	22	19.6	105.4	-		
				3 --		Silty Clay, dark brown and gray, moist, stiff
				-		
				4 --		
				-		
5	39	18.2	107.4	5 --		
				-	CL	ALLUVIUM: Sandy Lean Clay, dark brown, moist, very stiff,
				6 --		some silt
				-		
				7 --		
7.5	28	19.1	93.8	-		
				8 --		dark brown, moist, some silt
				-		
				9 --		
				-		
10	15	18.7	SPT	10 --	SC	Clayey Sand, dark brown and yellowish brown mottling, moist,
				-		medium dense, fine grained, trace fine gravel, some cemented
				11 --		thin layers
				-		
				12 --		
12.5	30	16.2	100.4	-		
				13 --		medium brown, moist, medium dense, fine grained, minor
				-		caliche, some carbonate stringers
				14 --		
				-		
15	14	25.3	SPT	15 --		
				-	CL	Sandy Lean Clay, olive brown, very moist, stiff, minor caliche
				16 --		
				-		
				17 --		
17.5	11	35.9	83.2	-		
				18 --		
				-		
				19 --		
				-		
20	6	31.2	SPT	20 --		brown, very moist to wet, medium stiff, fine sand, some silt
				-		
				21 --		
				-		
				22 --		
22.5	10	40.4	81.7	-		
				23 --		
				-		
				24 --		
				-		
25	6	39.2	SPT	25 --		
				-	CH	Fat Clay, brown, very moist to wet, medium stiff

BORING LOG NUMBER 2

Wolff Urban Management

File No. 21430

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
27.5	8	32.7	90.7	-		
				28 --		
				-		
				29 --		
				-		
30	6	31.7	SPT	30 --		
				-	CL	Sandy Lean Clay, brown, wet, medium stiff, fine sand, some caliche
				31 --		
				-		
				32 --		
32.5	18	No Recovery		-		
				33 --		
				-		
				34 --		
				-		
35	9	25.0	SPT	35 --		becomes stiff
				-		
				36 --		
				-		
				37 --		
37.5	20	No Recovery		-	SP	Poorly Graded Sand
				38 --		
				-		
				39 --		
				-		
40	18	23.6	SPT	40 --		
				-		yellowish brown, very moist, medium dense, fine grained, some Silt
				41 --		
				-		
				42 --		
42.5	21	22.3	99.4	-		
				43 --		
				-	SM	Silty Sand, yellowish brown, wet, medium dense, fine grained, trace fine gravel
				44 --		
				-		
45	19	23.8	SPT	45 --		
				-		
				46 --		
				-		
				47 --		
47.5	24	20.9	102.2	-		
				48 --		
				-		
				49 --		
				-		
50	20	22.7	SPT	50 --		yellowish brown, wet, medium dense, fine grained
				-		

BORING LOG NUMBER 2

Wolff Urban Management

File No. 21430

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				- 51 -- - 52 --		
52.5	20	No Recovery	SPT	- 53 --		BEDROCK (MODELO FORMATION): Siltstone, olive gray, moist, medium hard
				- 54 -- - 55 -- - 56 -- - 57 -- - 58 -- - 59 -- - 60 -- - 61 -- - 62 -- - 63 -- - 64 -- - 65 -- - 66 -- - 67 -- - 68 -- - 69 -- - 70 -- - 71 -- - 72 -- - 73 -- - 74 -- - 75 -- -		Total Depth 53.5 feet Water at 21 feet Fill to 5 feet NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual. Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted SPT=Standard Penetration Test

BORING 1

ELEV. (MSL.) -- DATE COMPLETED 11/5/19

EQUIPMENT HOLLOW STEM AUGER BY: CB

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**ARTIFICIAL FILL** Sandy Silt, soft, moist, dark brown, fine-grained, some organic content.			
2								
4					- brown mottles			
	B1@5'				- trace medium-grained sand	11	104.4	16.2
6								
	B1@7.5'					2		
8					**ALLUVIUM** Sandy Clay, firm, moist, brown, fine-grained, trace medium-grained, trace fine gravel.			
10	B1@10'					18	99.0	22.0
					- trace calcium carbonate stringers			
12	B1@12.5'				- soft	3		
14					- orange brown mottles			
	B1@15'					18	90.6	28.1
16								
	B1@17.5'		▼			4		
18					- soft, wet			
				CL	- fine-grained			
20	B1@20'					4	88.6	33.8
22	B1@22.5'				- very soft, saturated	PUSH		
24								
	B1@25'				- soft	4	85.0	38.3
26								
	B1@27.5'					2		
28					- very soft, some silty sand lenses, fine- to medium-grained, orange brown			

Figure A1,
Log of Boring 1, Page 1 of 3

W1089-06-01 BORING LOGS.GPJ

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.

GEOCON

BORING 1

ELEV. (MSL.) -- DATE COMPLETED 11/5/19

EQUIPMENT HOLLOW STEM AUGER BY: CB

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B1@30'			CL	- soft	6	90.7	31.8
32	B1@32.5'			SP	Sand, poorly graded, very loose, wet, brown, fine-grained.	2		
					Silty Sand, very loose, wet, brown, fine-grained.			
34								
	B1@35'					4	83.9	35.0
36				SM	- trace gravel (to 3")			
	B1@37.5'					2		
38								
40	B1@40'				Sandy Clay, soft, saturated, brown, fine-grained, some medium-grained.	9	90.5	31.7
42	B1@42.5'					4		
44								
	B1@45'					7	104.3	22.8
46								
	B1@47.5'					7		
48					- moist, brown with dark brown mottles, fine-grained			
50	B1@50'			CL		10		
52	B1@52.5'					7		
54								
	B1@55'					11	95.0	30.3
56								
	B1@57.5'					16		
58					- stiff, brown			

W1089-06-01 BORING LOGS.GPJ

Figure A1,
Log of Boring 1, Page 2 of 3

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.

GEOCON

PROJECT NO. W1089-06-01

BORING 1

ELEV. (MSL.) -- DATE COMPLETED 11/5/19

EQUIPMENT HOLLOW STEM AUGER BY: CB

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B1@60'			CL		33	99.5	27.1
					Sand with Clay, poorly graded, dense, slightly moist, dark brown with black mottles, fine-grained.			
62	B1@62.5'			SP-SC		48		
64					**MODELO FORMATION**			
	B1@64.5'				Sandstone, fine-grained, moderately hard, slightly moist, gray, fine-grained, poorly bedded.	50 (4")	89.6	35.9
66								
	B1@67'					50 (4")		

Total depth of boring: 67.5 feet
Fill to 8 feet.
Groundwater encountered at 17.2 feet.
Backfilled with grout.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

W1089-06-01 BORING LOGS.GPJ

Figure A1,
Log of Boring 1, Page 3 of 3

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.

GEOCON

PROJECT NO. W1089-06-01

BORING 2

ELEV. (MSL.) -- DATE COMPLETED 11/5/19

EQUIPMENT HOLLOW STEM AUGER BY: CB

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Sandy Silt with Clay, firm, slightly moist, dark brown with brown mottles, some organic material, trace fine gravel, fine- to medium-grained.			
4					**ALLUVIUM** Silt with Clay, some sand, hard, slightly moist, grayish brown with brown mottles, fine-grained with some medium- to coarse-grained sand, trace fine gravel.			
	B2@5'			ML		46	83.3	15.4
	B2@7.5'			SP-SM	Sand with Silt, medium dense, dry, grayish brown, fine-grained, trace fine cobbles.	16		
10	B2@9.5'				**MODELO FORMATION** Sandstone, grayish brown with reddish brown mottles, fine- to medium-grained, hard, completely weathered.	50 (6")	105.6	15.6
	B2@12.5'					52		
	B2@14.5'				- Siltstone bed (up to 1" thick), moderately weathered	50 (4")	100.3	25.1
	B2@17.5'				- soft	12		
	B2@19.5'				- olive gray with reddish brown mottles, medium-grained, medium hard, moderately weathered	50 (3")	92.4	33.3
	B2@22.5'					24		
	B2@25'				- highly weathered	61	84.7	35.2
	B2@27.5'				Claystone, black, poorly bedded to massive, medium hard, slightly weathered.	50 (5")		
	B2@29'					50 (5")	83.9	32.7

Depth scale: 0, 2, 4, 6, 8, 10, 12, 14, 16, 18, 20, 22, 24, 26, 28

W1089-06-01 BORING LOGS.GPJ

Figure A2,
Log of Boring 2, Page 1 of 2

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.

GEOCON

BORING 2

ELEV. (MSL.) -- DATE COMPLETED 11/5/19

EQUIPMENT HOLLOW STEM AUGER BY: CB

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30								
32	B2@32.5'				Sandstone, gray with reddish brown mottles, fine-grained, poorly bedded, medium hard to hard, moderately weathered, moist.	40		
34	B2@34'					50 (6")	96.0	29.0
36								
	B2@37.5'		▼			70		
38					- medium hard			
					Claystone, dark brown to black with reddish streaking, massive, hard, moderately weathered, dry.			
40	B2@39.5'					50 (4")	91.9	28.8
					Silty Sandstone, dark brown, poorly bedded, fine-grained, medium hard to hard, dry, moderately weathered.			
42	B2@42'					50 (5")		
44	B2@44'					50 (5")	85.5	33.2
46								
	B2@47'					50 (6")		
48								
	B2@49'					50 (5")	83.7	30.7

Total depth of boring: 49.5 feet
Fill to 3.5 feet.
Perched groundwater encountered at 38 feet.
Backfilled with grout.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

W1089-06-01 BORING LOGS.GPJ

Figure A2,
Log of Boring 2, Page 2 of 2

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	⊠ ... DISTURBED OR BAG SAMPLE	◩ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

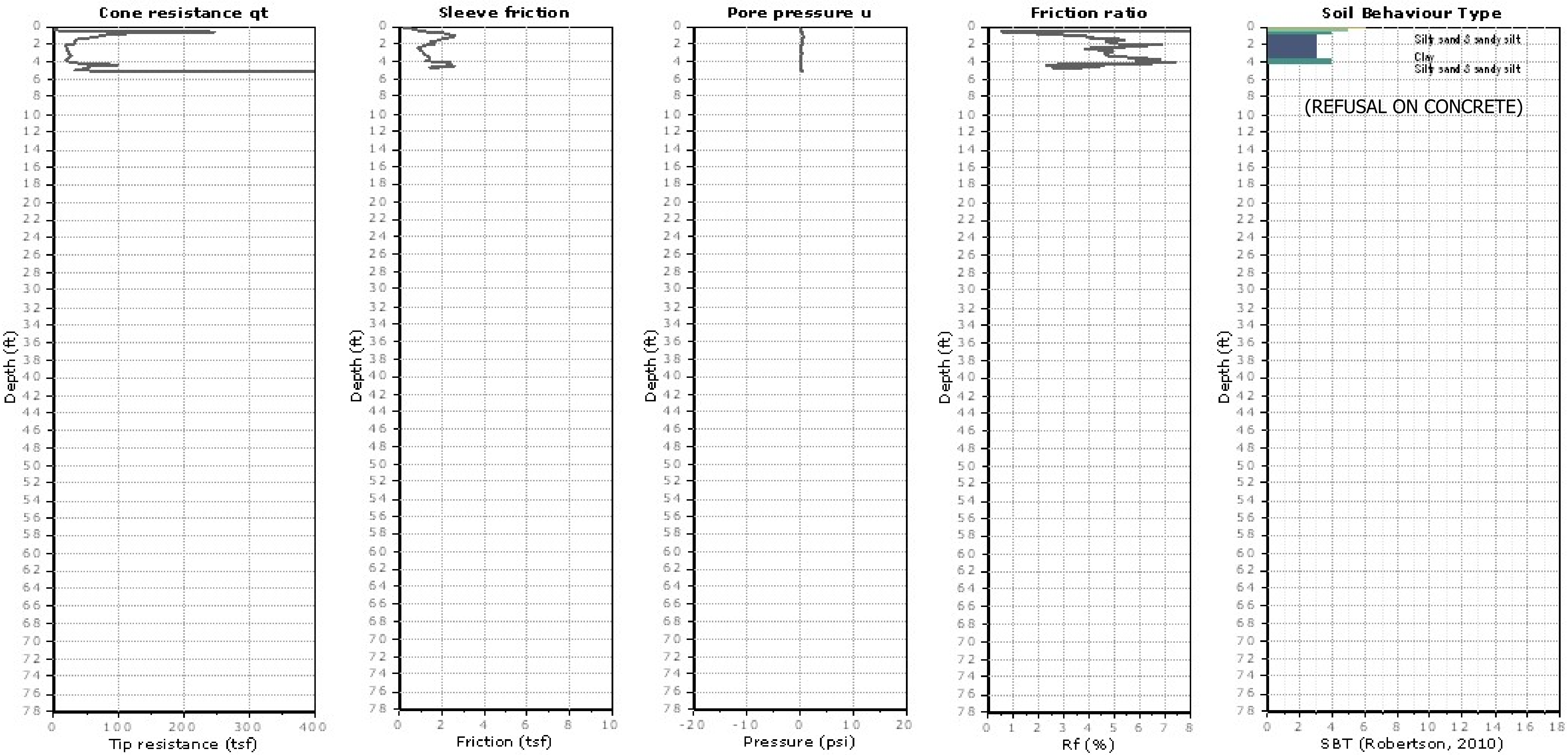
NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

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

Project: **Geocon West**
Location: **2101 Ventura Blvd, Woodland Hills, CA**

CPT-1A
Total depth: 5.12 ft, Date: 12/31/2019



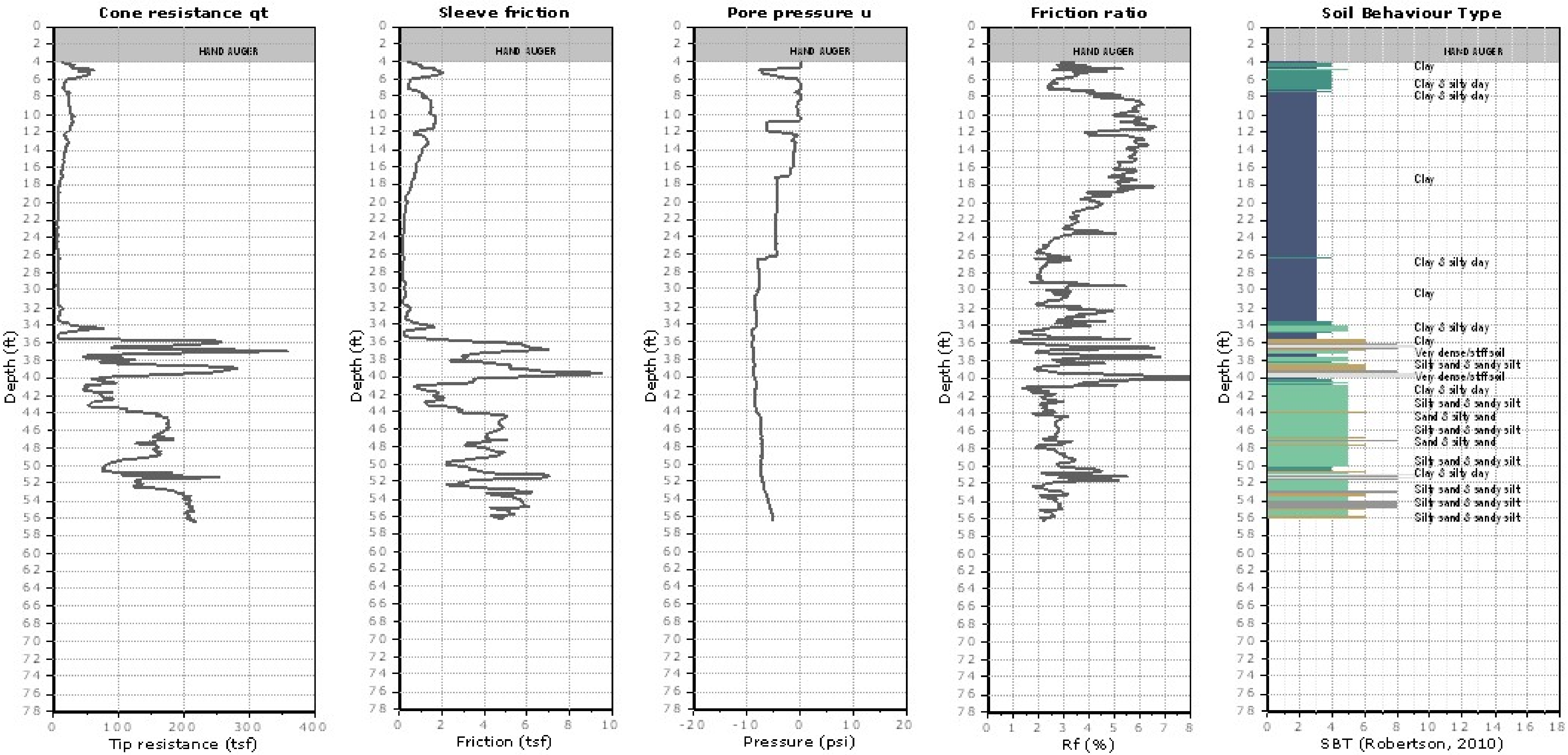
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: Geocon West

Location: 2101 Ventura Blvd, Woodland Hills, CA

CPT-2

Total depth: 56.52 ft, Date: 12/31/2019



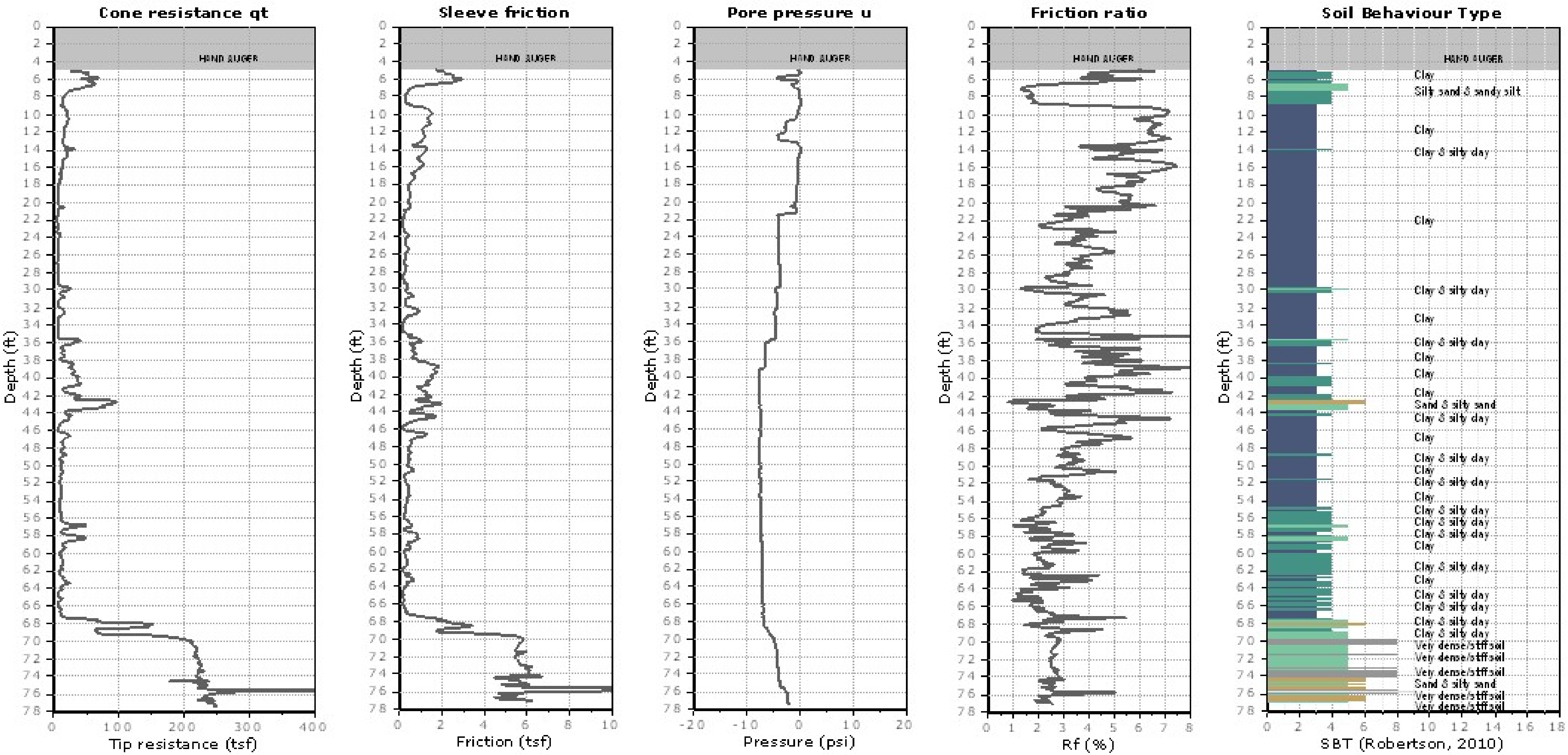
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: Geocon West

Location: 2101 Ventura Blvd, Woodland Hills, CA

CPT-3

Total depth: 77.45 ft, Date: 12/31/2019



Kehoe Testing and Engineering
714-901-7270
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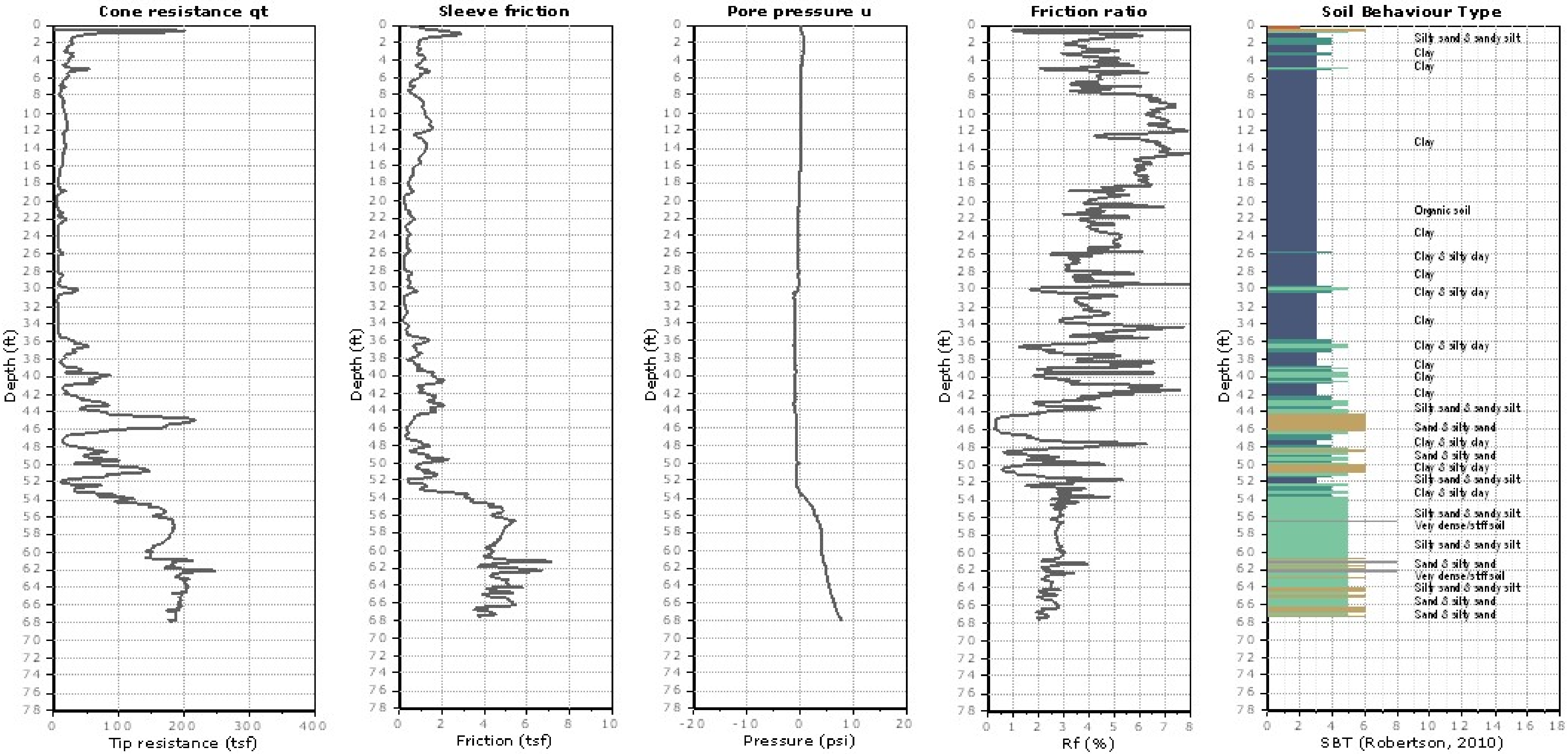
Project: Geocon West **CPT-4**

Location: 2101 Ventura Blvd, Woodland Hills, CA Total depth: 67.92 ft, Date: 12/31/2019



GEOTECHNICAL BORING LOG LB-1

Project No.	13589.002	**Date Drilled**	6-30-22
Project	Proposed Storage	**Logged By**	EDB
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	898'
Location	See Plate 1 - *Geotechnical Map*	**Sampled By**	EDB

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0								**Artificial fill, undocumented (Afu):** @0': 7-inches Asphalt Concrete over 4-inches Aggregate Base, fabric noted @1-inch	
				B-1				SC	@11-inches: Clayey SAND (SC), mottled brown and dark gray to black, moist, low plasticity, trace fine sand	EI
895										
	5			R-1	8 16 19			CL	@5': Lean CLAY (CL), black, moist, very stiff to hard, low plasticity, trace fine silt, PP >4.50 @6': Few fine gravel, black streaks	
890				R-2	12 23 28			SM CL	@7.5': Silty SAND (SM), yellowish brown, moist, dense, fine sand, trace fine gravel, trace construction debris (metal) **Quaternary Alluvium (Qa):** @8.25': Lean CLAY (CL), black, moist, low to medium plasticity, carbonate, trace fine gravel	
	10			R-3	8 13 21				@10': Very stiff, PP > 4.50	
885				ST-4				CH	@12.5': 24-inch ST Sample, Fat CLAY (CH), dark brown, moist, high plasticity, trace silt noted at bottom of sample	CN, AL
	15			R-5	5 9 11			CL	@15': Lean CLAY (CL), light brown to brown, moist, low to medium plasticity, carbonate stringers, 1/16-inch lamination of CL/CH, black, moist	
880				R-6	4 6 8				@17.5': Trace sand, PP = 2.50	
	20			ST-7				CL-ML	@20': 24-inch ST Sample, @Bottom of sample is CLAY (CL), light brown, very moist, medium plasticity, PP = 1.50, some silt	
									@22': Static depth to groundwater, measured after 15 minutes.	
875										
	25			R-8	2 3 6			CL	@25': Lean CLAY (CL), brown, very moist, medium stiff, medium plasticity, trace fine subrounded gravel and silt	CN, AL
870										
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

Leighton

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG LB-1

Project No.	13589.002	**Date Drilled**	6-30-22
Project	Proposed Storage	**Logged By**	EDB
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	898'
Location	See Plate 1 - *Geotechnical Map*	**Sampled By**	EDB

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
	30			R-9	2 2 3			CL	**Quaternary Alluvium (Qa), continued:** @30': Lean CLAY (CL), light brown, very moist to wet, soft to medium stiff, medium plasticity, fine sand, trace fine gravel, black streaks, carbonate staining, PP = 0.50	
865										
	35			ST-10					@35': 24-inch ST Sample, Silty CLAY with sand (CL), light brown, wet, low to medium plasticity, fine sand, slight mica	CN, AL
860										
	40			R-11	4 8 17			SP	@40': Lean CLAY (CL), brown, wet, stiff, low plasticity, fine to medium sand, trace fine gravel, PP = 1.00 @41': Poorly-graded SAND (SP), yellowish brown, medium dense, fine to medium sand	
855									@43': Groundwater encountered, waited 15 minutes for static reading.	
	45			R-12	8 6 5			SW CL	@45': Well-graded SAND (SW), dark yellowish brown, wet, medium dense, fine to coarse sand, trace coarse gravel @45.5': Lean CLAY (CL), dark brown, very moist, stiff, low to medium plasticity, fine sand, slight oxidation	
850										
	50			ST-13				CH	@50': 24-inch ST Sample, Fat CLAY, olive brown, very moist, high plasticity, fine sand, slight oxidation	CN, AL
845									Total Depth of Boring: 52 Feet Groundwater encountered initially @43.2'; rose to 22' after 15 minutes Boring backfilled with soil cuttings, tamped, and capped with approximately 6-inches of Asphalt Cold Patch Mix upon completion of drilling on 6-30-2022.	
	55									
840										
	60									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

Leighton

* * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * *

GEOTECHNICAL BORING LOG LB-2

Project No.	13589.002	**Date Drilled**	6-30-22
Project	Proposed Storage	**Logged By**	EDB
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	898'
Location	See Plate 1 - *Geotechnical Map*	**Sampled By**	EDB

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			B-1				CL	**Artificial fill, undocumented (Afu):** @0': 7-inches Asphalt Concrete over Subgrade, fabric noted @1-inch; @7-inches: Lean CLAY (CL), mottled brownish gray and dark brown, moist, fine sand, trace fine gravel	CR
895										
	5			R-1	4 10 17			Tush	**Tertiary Age Unnamed Shale (Tush):** @5': CLAYSTONE, light brownish gray, moist, low plasticity, weak induration, carbonte spots	
890				R-2	50/4"				@7.5': Partial Recovery, gravel stuck in shoe; @8': Hard drilling, broken gravel bits coming out	
	10			R-3	10 19 27					
885										
	15			R-4	50/5"				@15': Mottled olive, brownish gray, moist, oxidized, FeO staining, well indurated, hard, trace sand; @16.1': SILTSTONE, gray, moist, non-plastic, weak cementation, oxidized, FeO staining	
880										
	20			R-5	18 31 50/5"				@20.25': 6-inch interbed of Silty CLAYSTONE, hard, brownish gray, moist, low plasticity, oxidized, FeO stains	
875										
	25			S-6	6 31 50/4"				@25': Silty CLAYSTONE, hard, mottled olive brown and tannish brown, moist, low plasticity, oxidized, FeO stains	
870										
	30									

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

Leighton

*** This log is a part of a report by Leighton and should not be used as a stand-alone document. ***

GEOTECHNICAL BORING LOG LB-2

Project No.	13589.002	**Date Drilled**	6-30-22
Project	Proposed Storage	**Logged By**	EDB
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	898'
Location	See Plate 1 - *Geotechnical Map*	**Sampled By**	EDB

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	30			R-7	14 27 48			Tush	**Tertiary Age Unnamed Shale (Tush), continued:** @30': Silty CLAYSTONE, hard, mottled olive brown and dark brown, moist, low plasticity, heavily oxidized, FeO stains, mica	
865									Total Depth of Boring: 31.5 Feet No Groundwater Boring backfilled with soil cuttings, tamped, and capped with approximately 6-inches of concrete with black dye upon completion of drilling on 6-30-2022.	
	35									
860										
	40									
855										
	45									
850										
	50									
845										
	55									
840										
	60									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

Leighton

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG LB-3

Project No.	13589.002	**Date Drilled**	6-30-22
Project	Proposed Storage	**Logged By**	EDB
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	897'
Location	See Plate 1 - *Geotechnical Map*	**Sampled By**	EDB

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
	0			B-1				CL	**Artificial Fill, undocumented (Afu):** @0': 6-inches Asphalt Concrete over 2-inches Aggregate Base, fabric noted @1-inch @8-inches: Lean CLAY (CL), mottled olive gray and black, moist, low plasticity, fine sand, trace fine gravel	DS, RV, MD
895										
	5			R-1	4 6 9				@5': Trace silt and asphalt debris, little coarse subangular gravel, PP = 4.50	
890										
				R-2	10 28 36			CH Tush	@7.5': High plasticity "Fat" CLAY (CH), black, moist, base of fill (scarified), some wood debris **Tertiary Age Unnamed Shale (Tush):** @8': CLAYSTONE, light brownish gray, dry, moderate induration, low plasticity, carbonate spots, few coarse gravel	
	10			R-3	8 22 46				@10': Hard, Mottled olive gray and brownish gray, slightly moist, weak to moderate induration, carbonate in matrix as pockets and laminations, FeO stains	DS
885										
				R-4	8 23 50/5"				@12.5': Silty CLAYSTONE, hard, olive brown and grayish brown, slightly moist, low plasticity, slightly oxidized, PP > 4.50	
	15			R-5	26 50/5"				@15': Sandy SILTSTONE, hard, well cemented, mottled light brown and gray, moist, non-plastic, heavily oxidized, FeO stains, carbonate lamination, PP > 4.50	
880										
	20			R-6	6 25 50/4"					DS
875										
	25			S-7	17 31 36				@25': CLAYSTONE with Silt, hard, olive brown to brown, moist, medium plasticity, oxidized, FeO stains	
870										
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

Leighton

*** This log is a part of a report by Leighton and should not be used as a stand-alone document. ***

GEOTECHNICAL BORING LOG LB-3

Project No.	13589.002	**Date Drilled**	6-30-22
Project	Proposed Storage	**Logged By**	EDB
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	897'
Location	See Plate 1 - *Geotechnical Map*	**Sampled By**	EDB

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	**SOIL DESCRIPTION** *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	30			R-8	40 50/4"			Tush	**Tertiary Age Unnamed Shale (Tush), continued:** @30.25': CLAYSTONE, hard, very dark brown, moist, low plasticity, oxidized, FeO stains, thin laminations of rock/sandstone, black MnO on parting surfaces, slightly moist, strong cementation, micaceous	DS
865										
	35			S-9	30 50/5"				@35': Interbedded CLAYSTONE and SILTSTONE, CLAYSTONE is very dark brown, moist, hard, low plasticity, oxidized, SILTSTONE is mottled light gray and brownish gray, moist, non-plastic, moderate carbonate cementation, oxidized @36': CLAYSTONE, hard, very dark brown, moist, low to medium plasticity, oxidized, carbonate inmatrix in pockets and laminations	
860										
	40			R-10	50/4"				@40': Hard, laminated, interbeds of calystone and sandstone, black MnO on parting surfaces, slightly moist, strong cementation, micaceous, PP > 4.50	
855									Total Depth of Boring: 40.3 Feet No Groundwater Boring backfilled with soil cuttings, tamped, and capped with approximately 6-inches of concrete with black dye upon completion of drilling on 6-30-2022.	
	45									
850										
	50									
845										
	55									
840										
	60									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

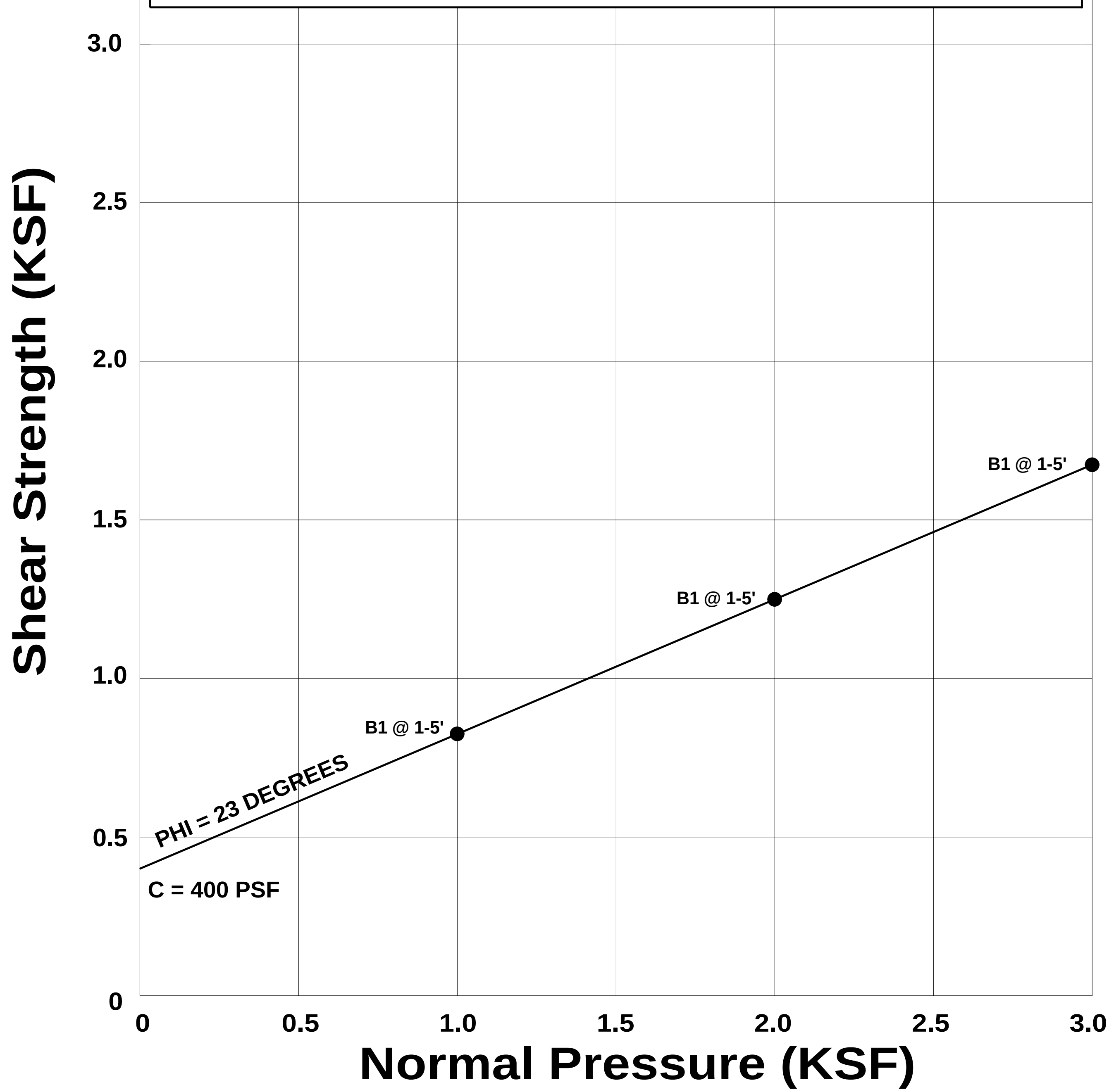
Leighton

APPENDIX C

LABORATORY TEST DATA

BULK SAMPLE REMOLDED TO 90 PERCENT OF THE MAXIMUM LABORATORY DENSITY

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B1 @ 1-5'	SM/ML/CL	107.6	11.9	24.6



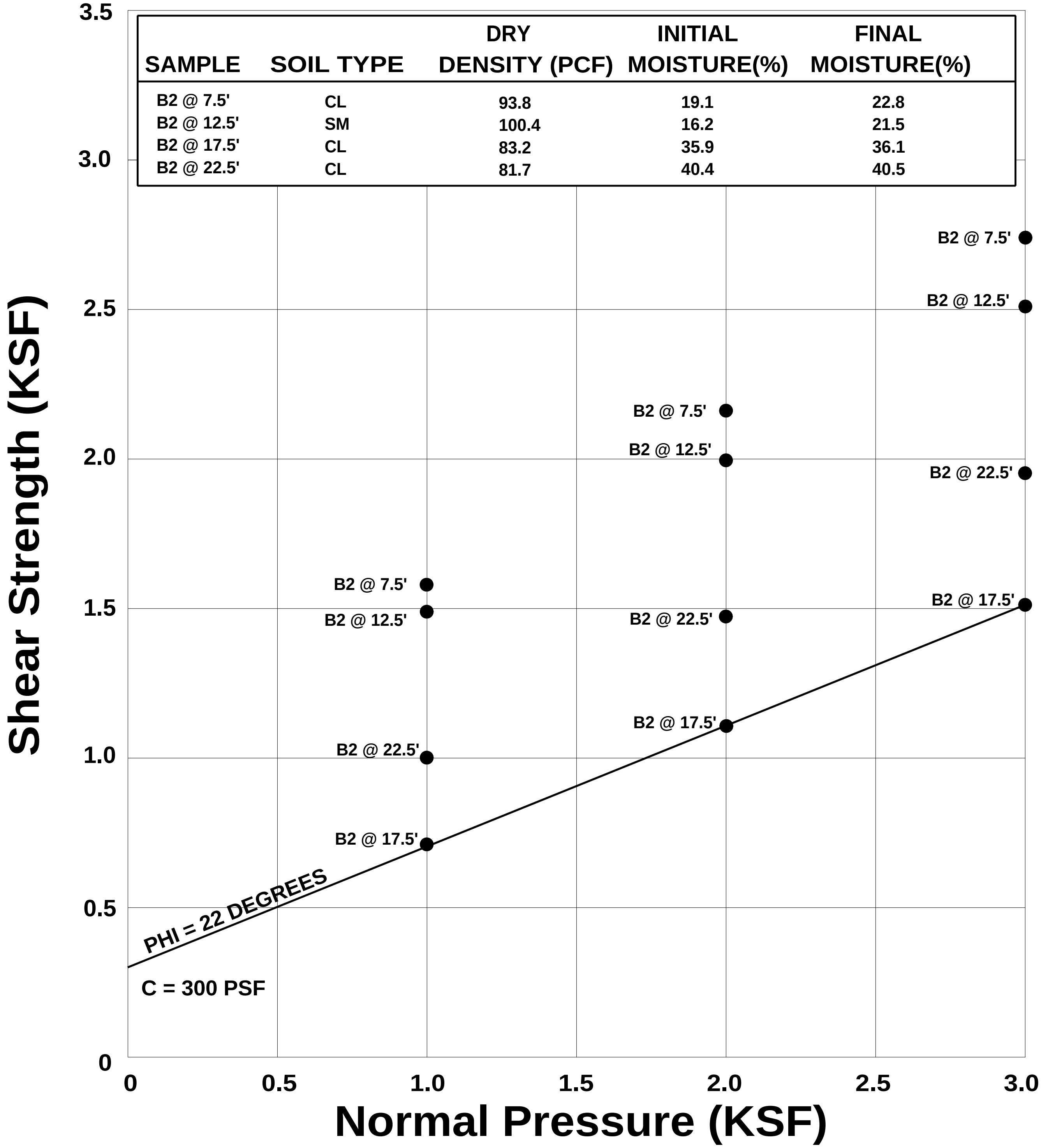
● Direct Shear, Saturated

SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

WOLFF URBAN MANAGEMENT	
FILE NO. 21430	PLATE: B-1

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B2 @ 7.5'	CL	93.8	19.1	22.8
B2 @ 12.5'	SM	100.4	16.2	21.5
B2 @ 17.5'	CL	83.2	35.9	36.1
B2 @ 22.5'	CL	81.7	40.4	40.5



● Direct Shear, Saturated (ALLUVIUM)

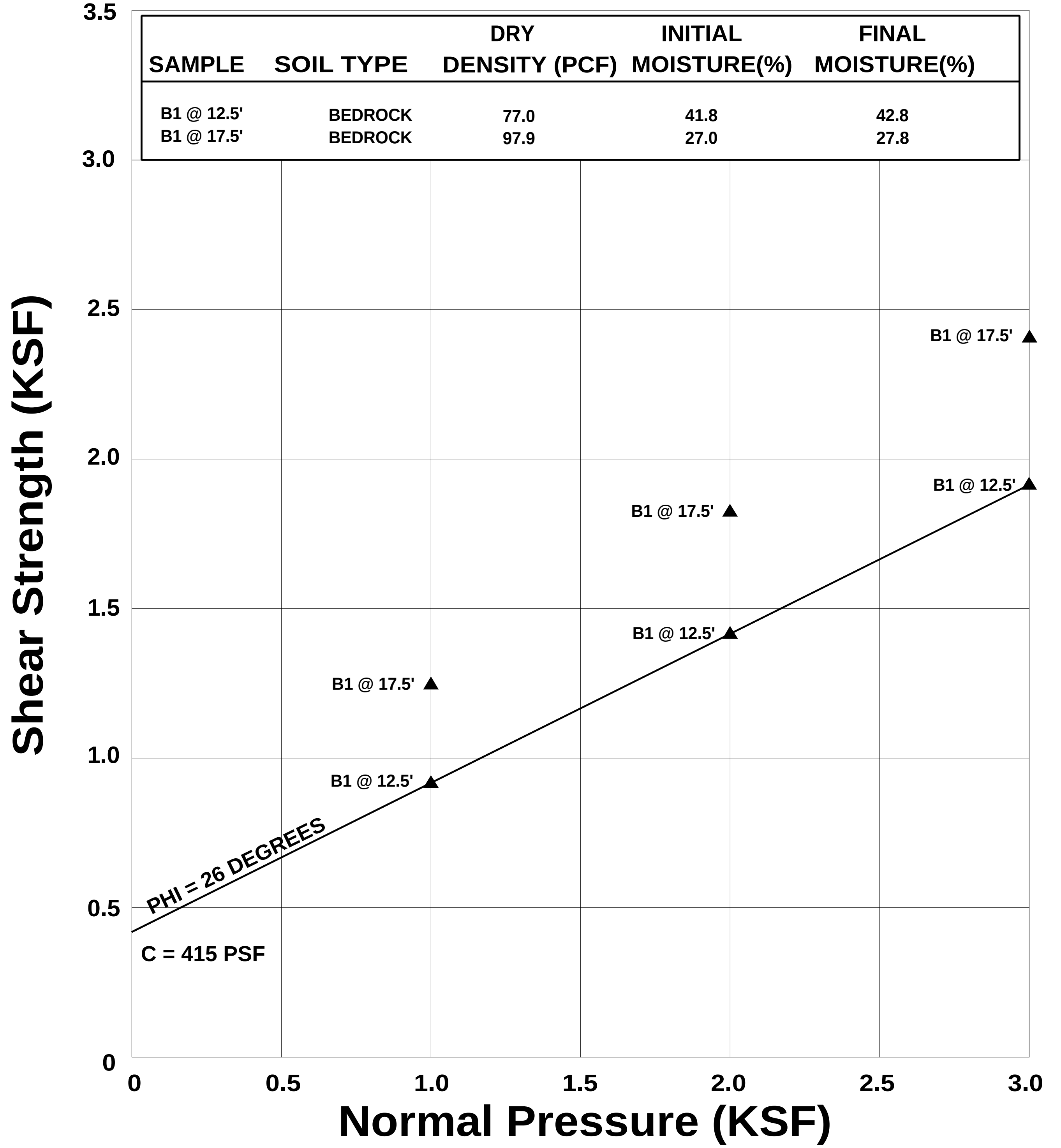
SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

WOLFF URBAN MANAGEMENT

FILE NO. 21430 | **PLATE: B-2**

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B1 @ 12.5'	BEDROCK	77.0	41.8	42.8
B1 @ 17.5'	BEDROCK	97.9	27.0	27.8



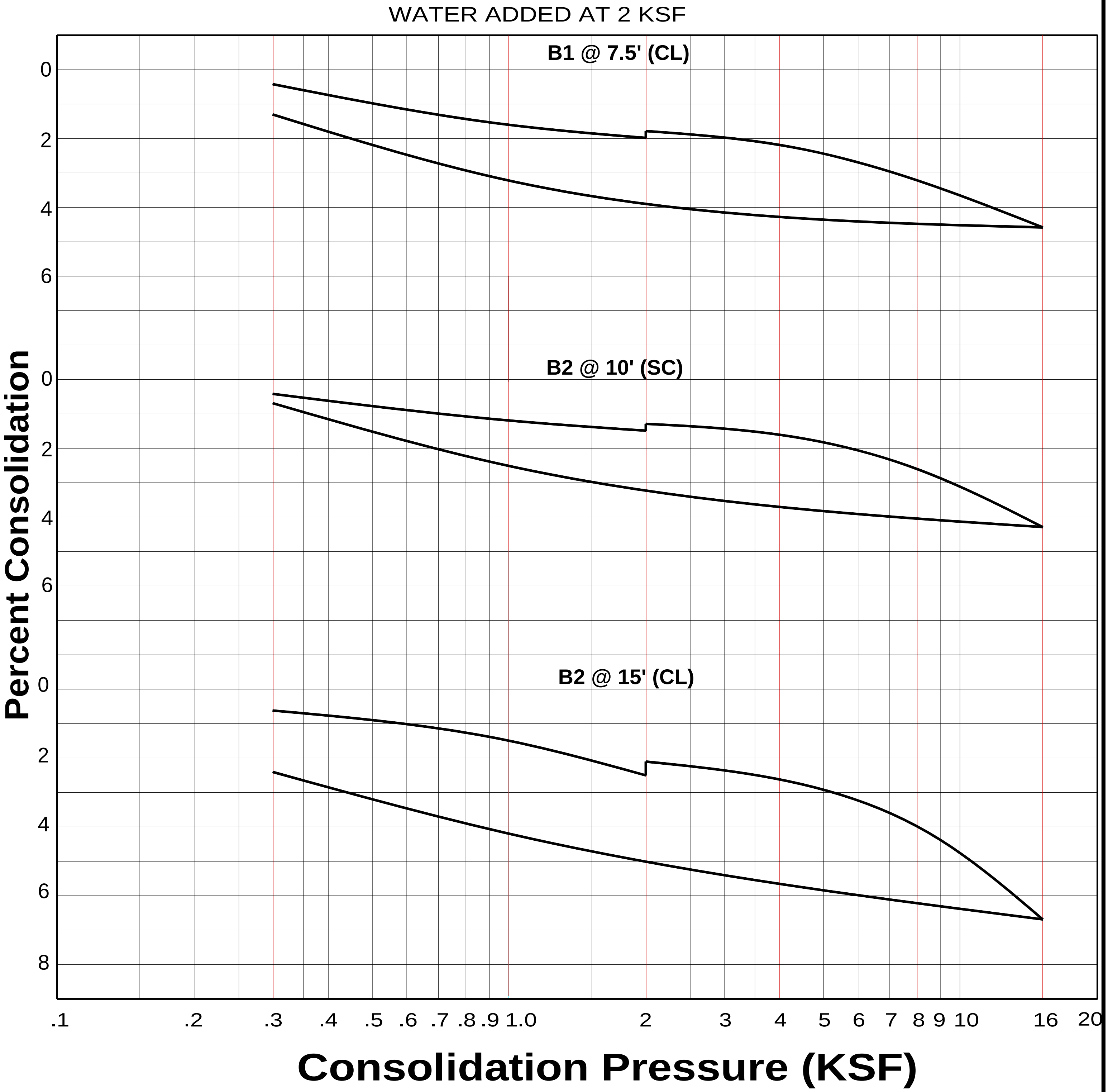
▲ Direct Shear, Saturated (BEDROCK)

SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

WOLFF URBAN MANAGEMENT

FILE NO. 21430 | **PLATE: B-3**



CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

WOLFF URBAN MANAGEMENT

FILE NO. 21430

PLATE: C

ASTM D-1557

SAMPLE	B1 @ 1-5'
SOIL TYPE:	SM/ML/CL
MAXIMUM DENSITY pcf.	119.5
OPTIMUM MOISTURE %	11.9

ASTM D 4829-03

SAMPLE	B1 @ 1- 5'	B2 @ 5'
SOIL TYPE:	SM/ML/CL	CL
EXPANSION INDEX UBC STANDARD 18-2	86	13
EXPANSION CHARACTER	MODERATE	VERY LOW

SULFATE CONTENT

SAMPLE	B1 @ 1-5'	B1 @ 7.5'	B2 @ 1-5'	B2 @ 5'	B2 @ 12.5'
SULFATE CONTENT: (percentage by weight)	< 0.2 %	< 0.1 %	< 0.2 %	< 0.1 %	< 0.1 %

COMPACTION/EXPANSION/SULFATE DATA SHEET

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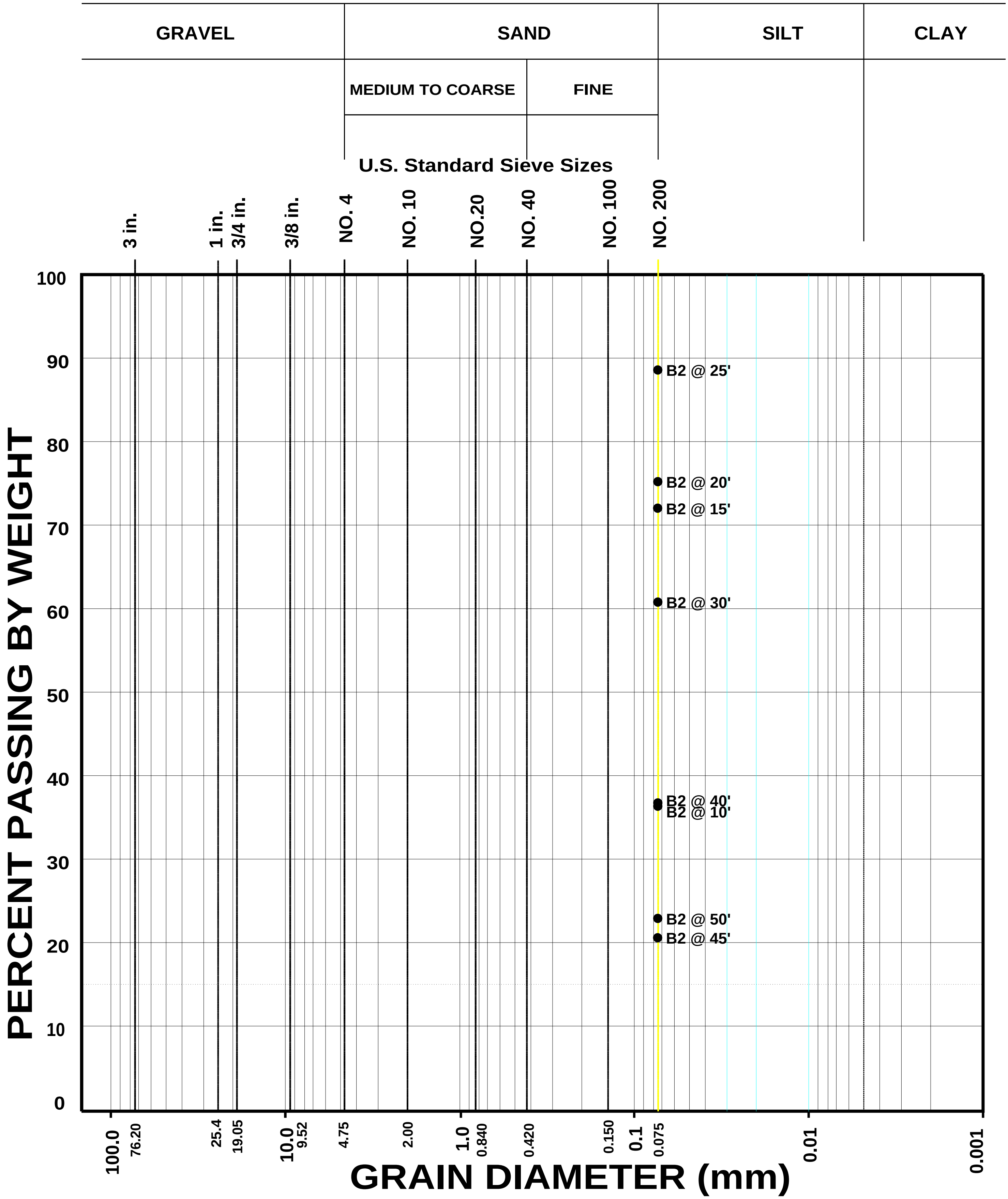
Consulting Geotechnical Engineers

WOLFF URBAN MANAGEMENT

FILE NO. 21430 | PLATE: D



GRAIN SIZE DISTRIBUTION

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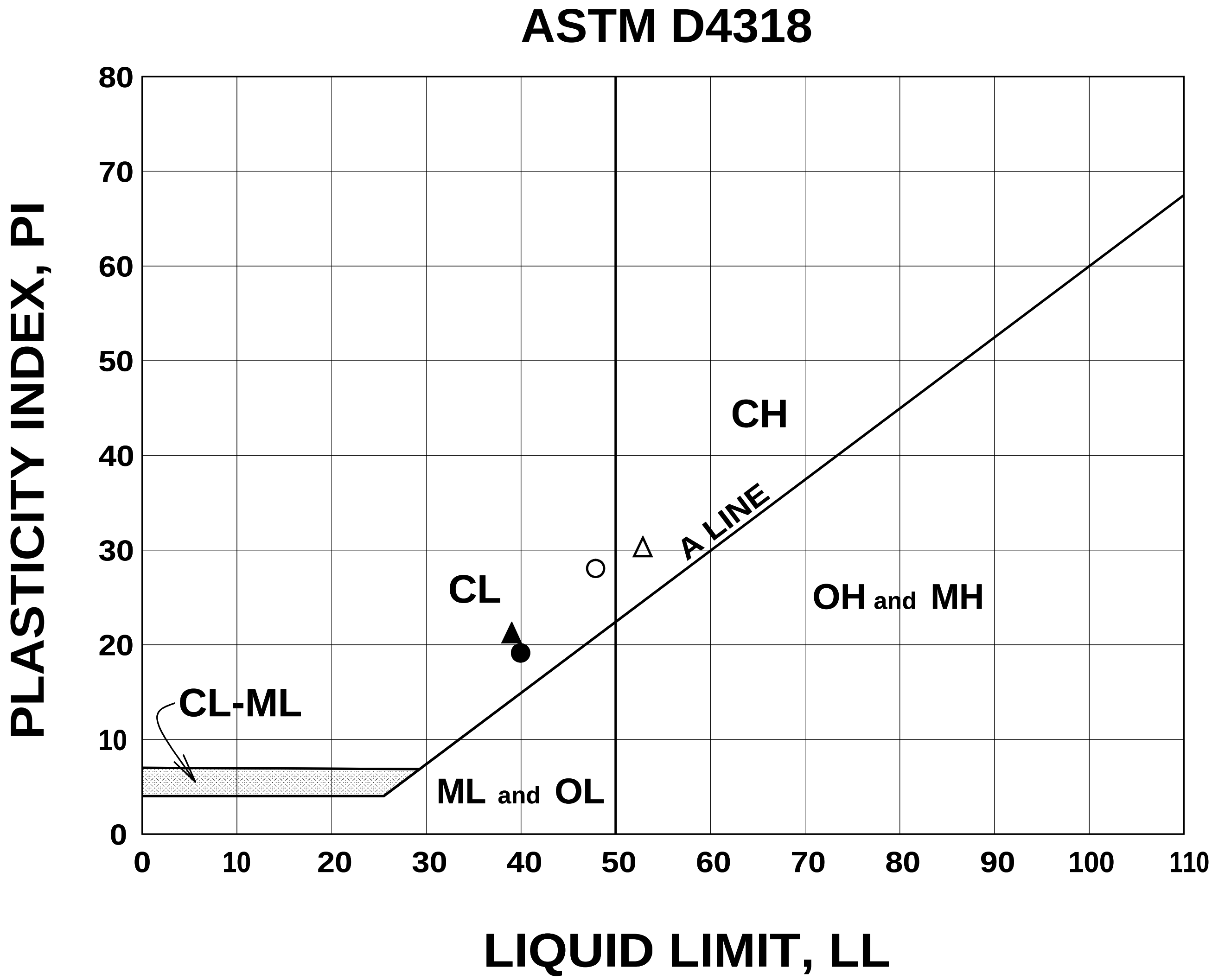
Consulting Geotechnical Engineers

WOLFF URBAN MANAGEMENT

FILE NO. 21430

PLATE: E



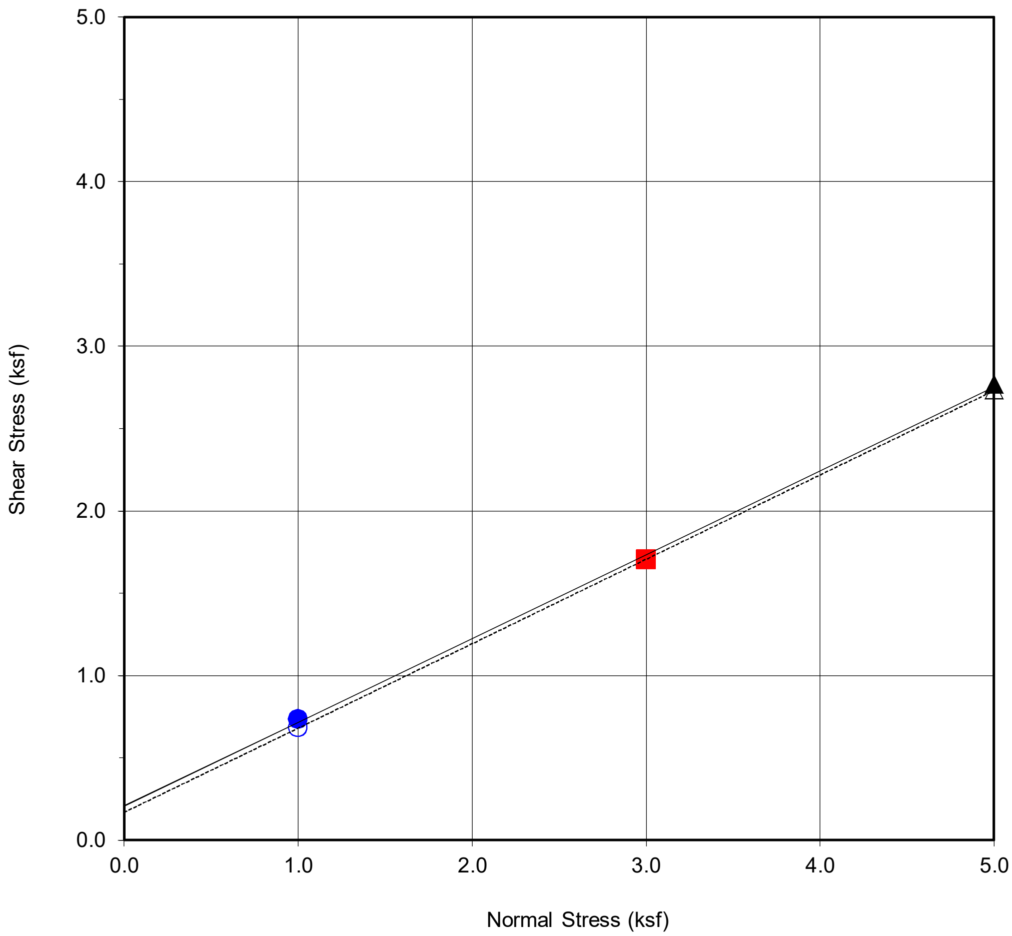
BORING NUMBER	DEPTH (FEET)	TEST SYMBOL	LL	PL	PI	DESCRIPTION
B2	15	○	48	20	28	CL
B2	20	●	40	21	19	CL
B2	25	△	53	23	30	CH
B2	30	▲	39	18	21	CL

ATTERBERG LIMITS DETERMINATION

Geotechnologies, Inc.
Consulting Geotechnical Engineers

WOLFF URBAN MANAGEMENT

FILE NO. 21430 | PLATE: F



Boring No.	B1
Sample No.	B1@0-5'
Depth (ft)	0-5'
Sample Type:	Ring

Soil Identification:		
Dark Brown Sandy Silt (ML)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	207	27.0
Ultimate	169	27.1

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.73	■ 1.70	▲ 2.77
Shear Stress @ End of Test (ksf)	○ 0.68	□ 1.70	△ 2.73
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	13.3	13.3	13.3
Initial Dry Density (pcf)	104.0	104.0	104.0
Initial Degree of Saturation (%)	57.9	57.9	57.9
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	23.6	21.4	20.3

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

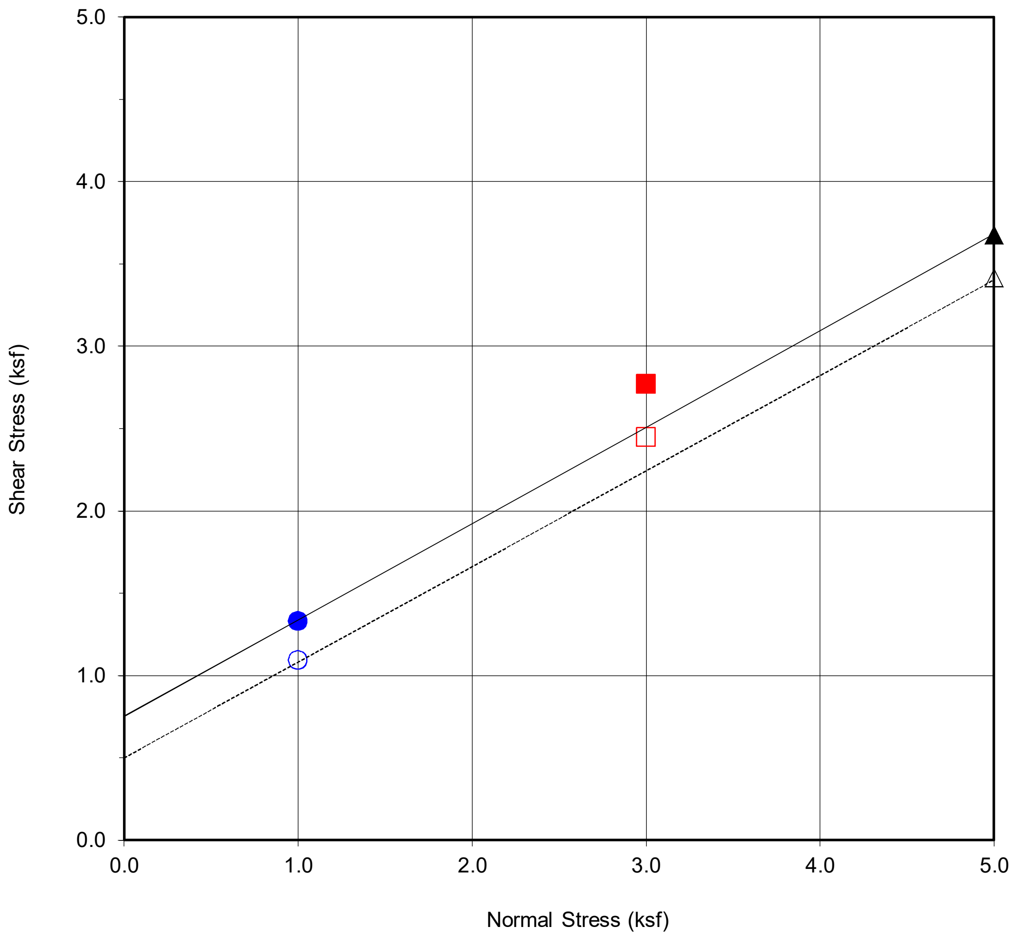
Checked by: PZ

Project No.: W1089-06-01

21101 West Ventura Boulevard
Los Angeles, California

Jan. 2020

Figure B1



Boring No.	B1
Sample No.	B1@5
Depth (ft)	5
Sample Type:	Ring

Soil Identification:		
Dark Brown Sandy Silt (ML)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	750	30.4
Ultimate	500	30.1

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.33	■ 2.77	▲ 3.68
Shear Stress @ End of Test (ksf)	○ 1.09	□ 2.45	△ 3.42
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	32.7	28.0	26.0
Initial Dry Density (pcf)	85.5	92.8	97.1
Initial Degree of Saturation (%)	90.8	92.7	95.6
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	34.8	28.5	25.1

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: PZ

Project No.: W1089-06-01

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Los Angeles, California

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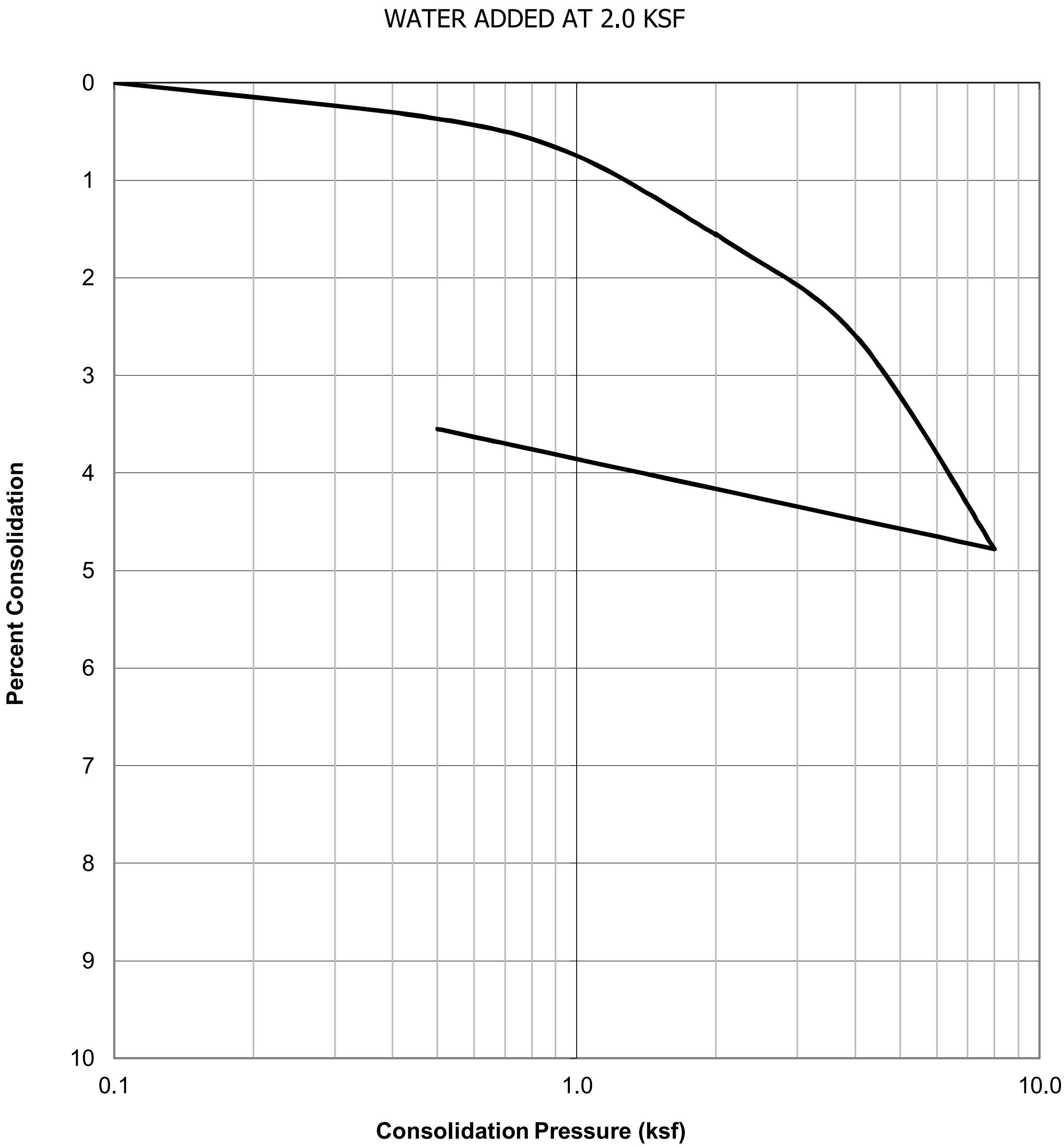
Figure B2



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@5	Dark Brown Sandy Silt (ML)	97.7	16.2	15.2

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: PZ

Project No.: W1089-06-01

21101 West Ventura Boulevard
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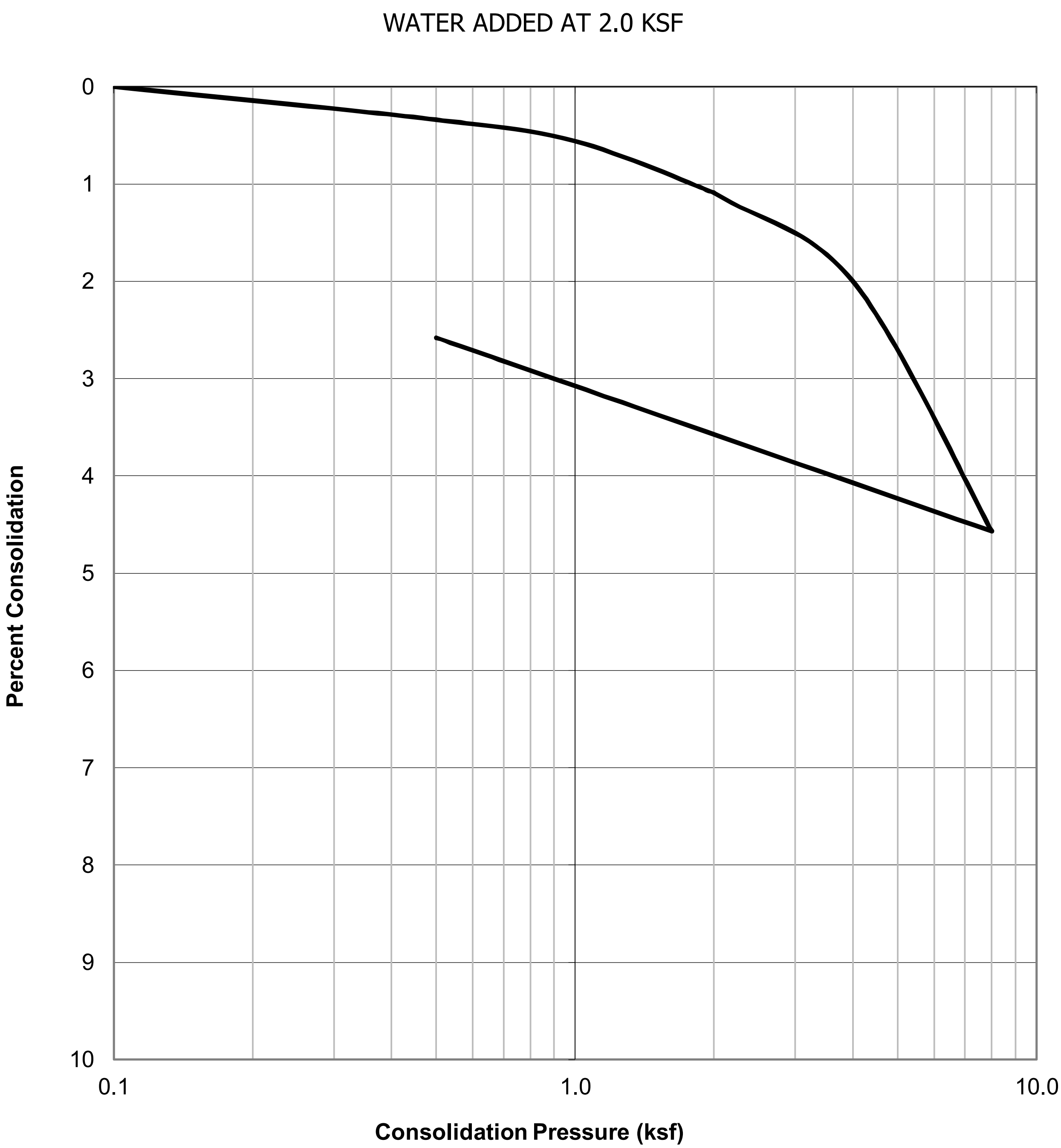
Figure B3



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@5	Grayish Brown Sandy Silt (ML)	56.9	68.8	68.7

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: PZ

Project No.: W1089-06-01

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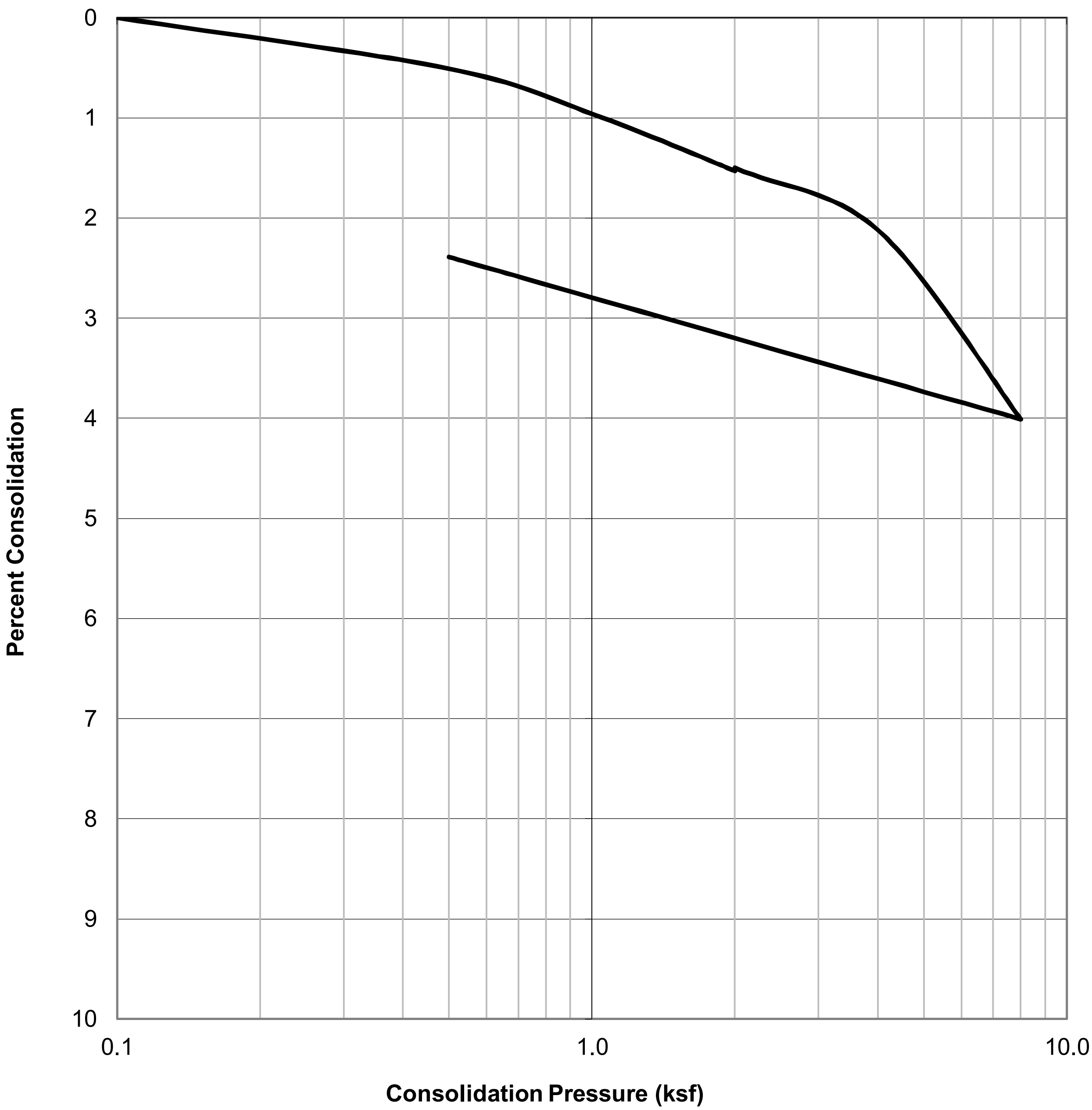
Figure B4

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@10	Dark Brown Sandy Silt (ML)	99.9	22.0	23.1

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: PZ

Project No.: W1089-06-01

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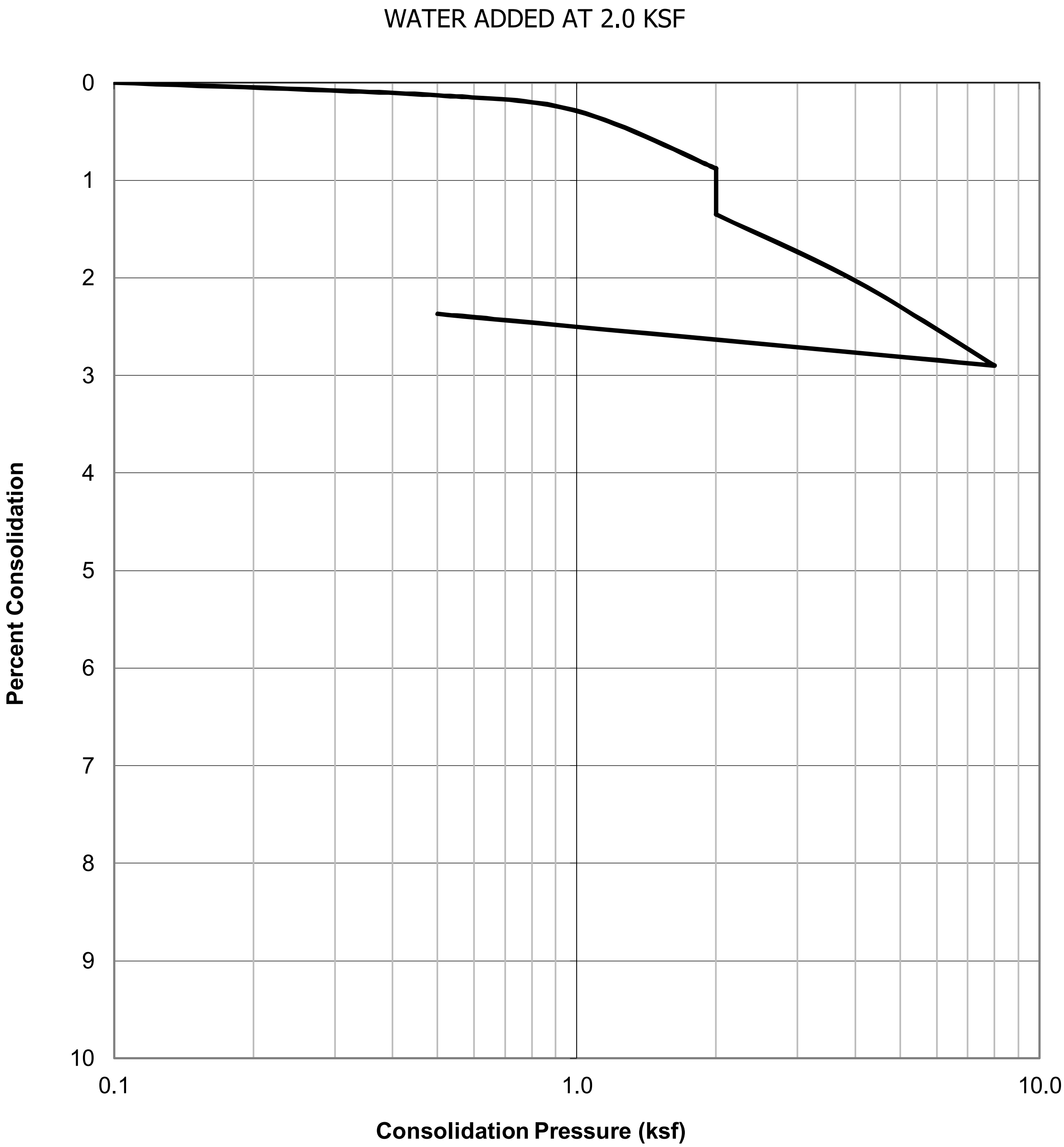
Figure B5



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@9.5	Grayish Brown Silty Sand (SM)	95.8	15.6	23.2

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: PZ

Project No.: W1089-06-01

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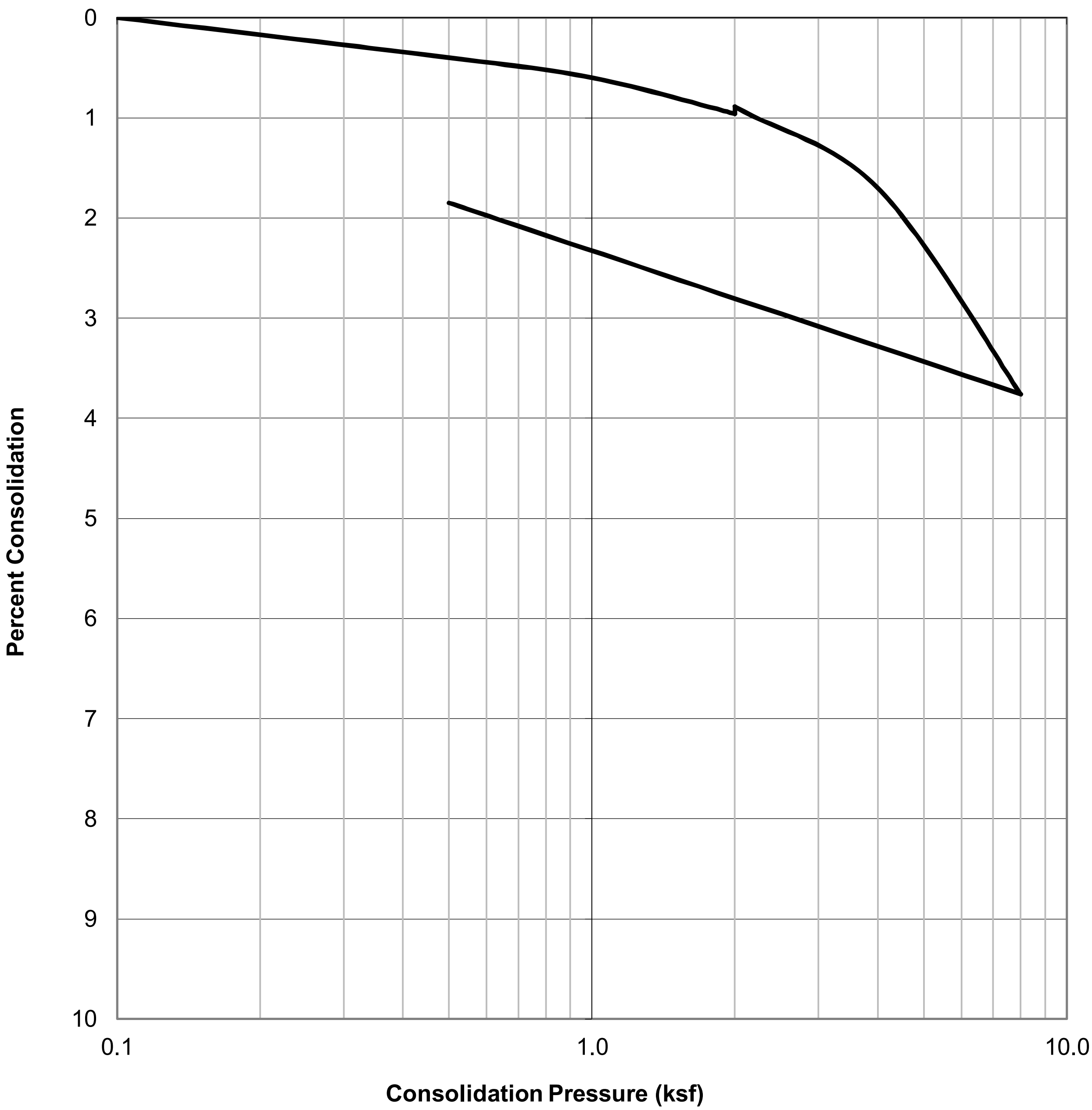
Figure B6

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@15	Brown Sandy Silt (ML)	91.8	28.1	29.2

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: PZ

Project No.: W1089-06-01

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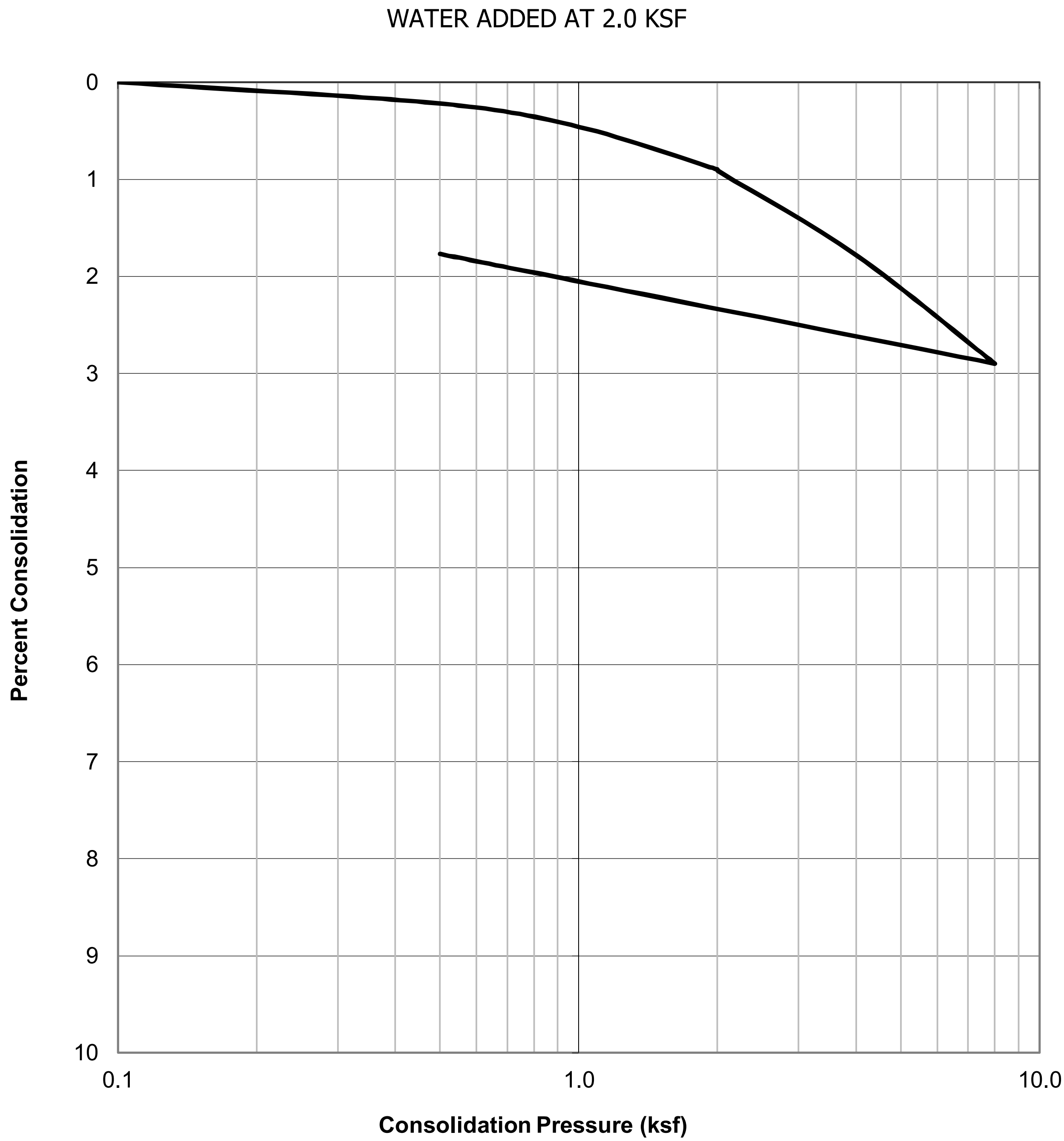
Figure B7



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@14.5	Grayish Brown Silty Sand (SM)	94.7	25.1	26.2

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: PZ

Project No.: W1089-06-01

21101 West Ventura Boulevard
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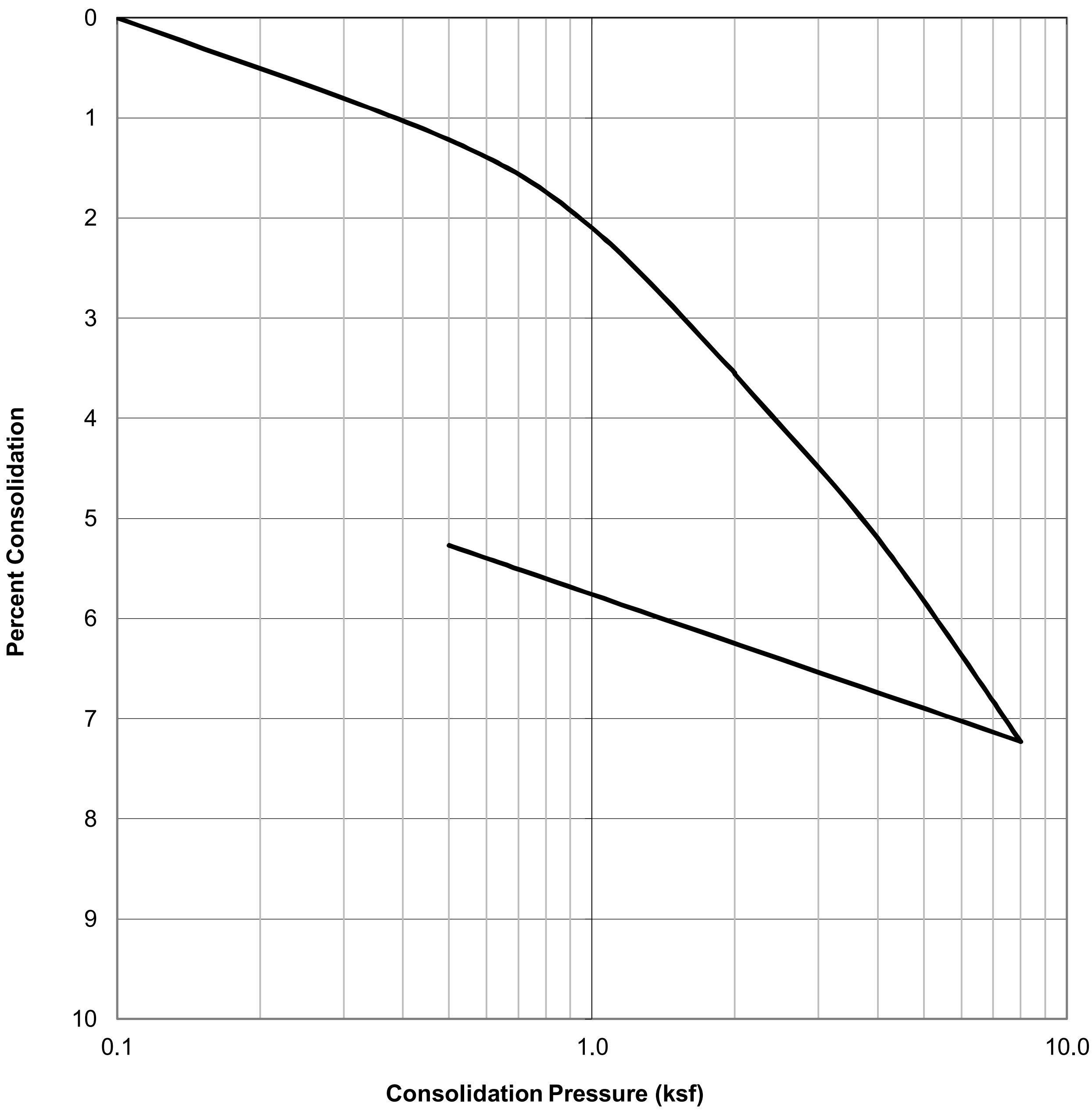
Jan. 2020

Figure B8

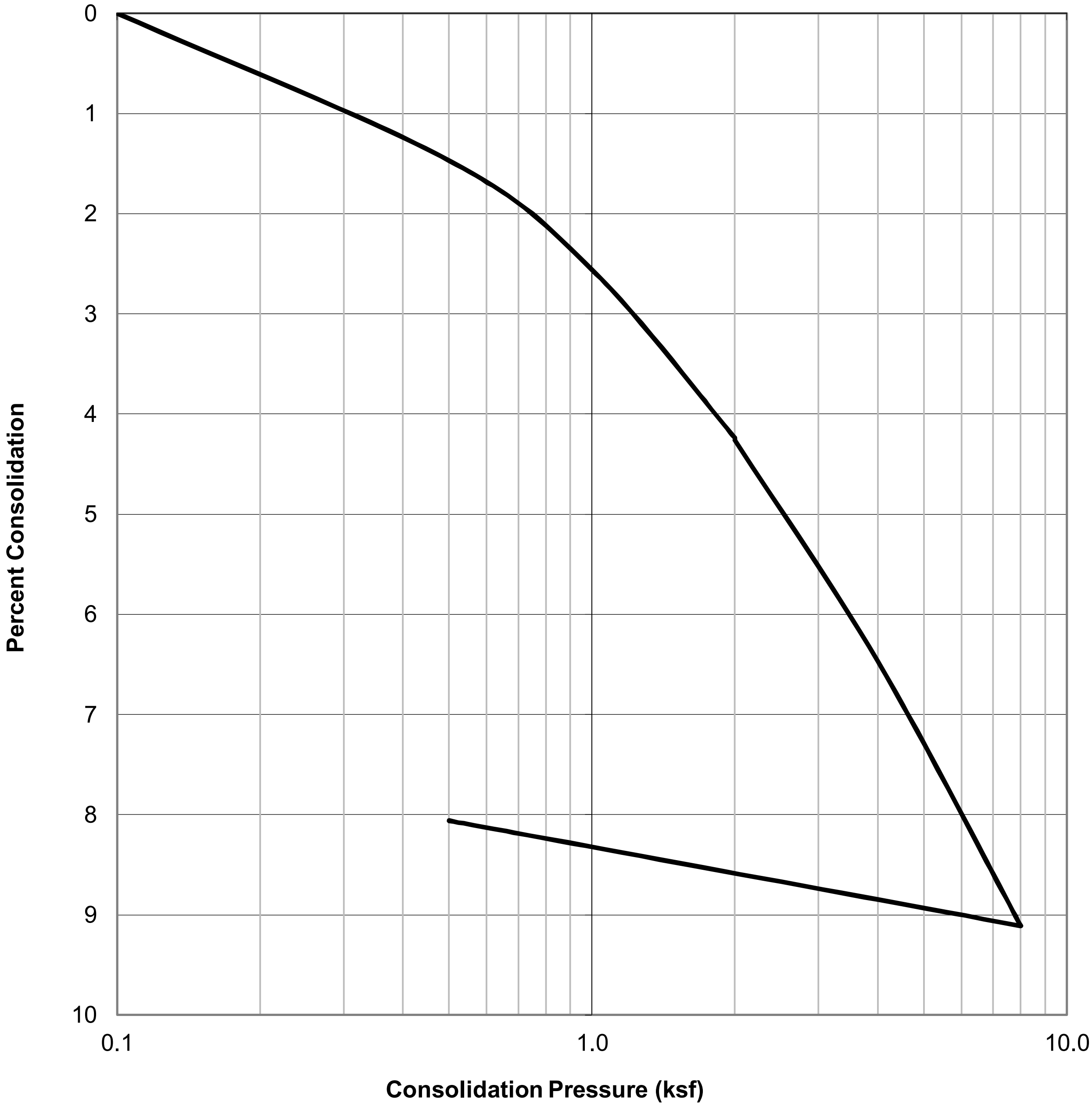
WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@20	Brown Sandy Silt (ML)	88.9	33.8	26.0

GEOCON	**CONSOLIDATION TEST RESULTS** ASTM D-2435	Project No.: W1089-06-01
		21101 West Ventura Boulevard Los Angeles, California
	Checked by: PZ	Jan. 2020 Figure B9

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@25	Brown Sandy Silt (ML)	82.6	38.3	32.2

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

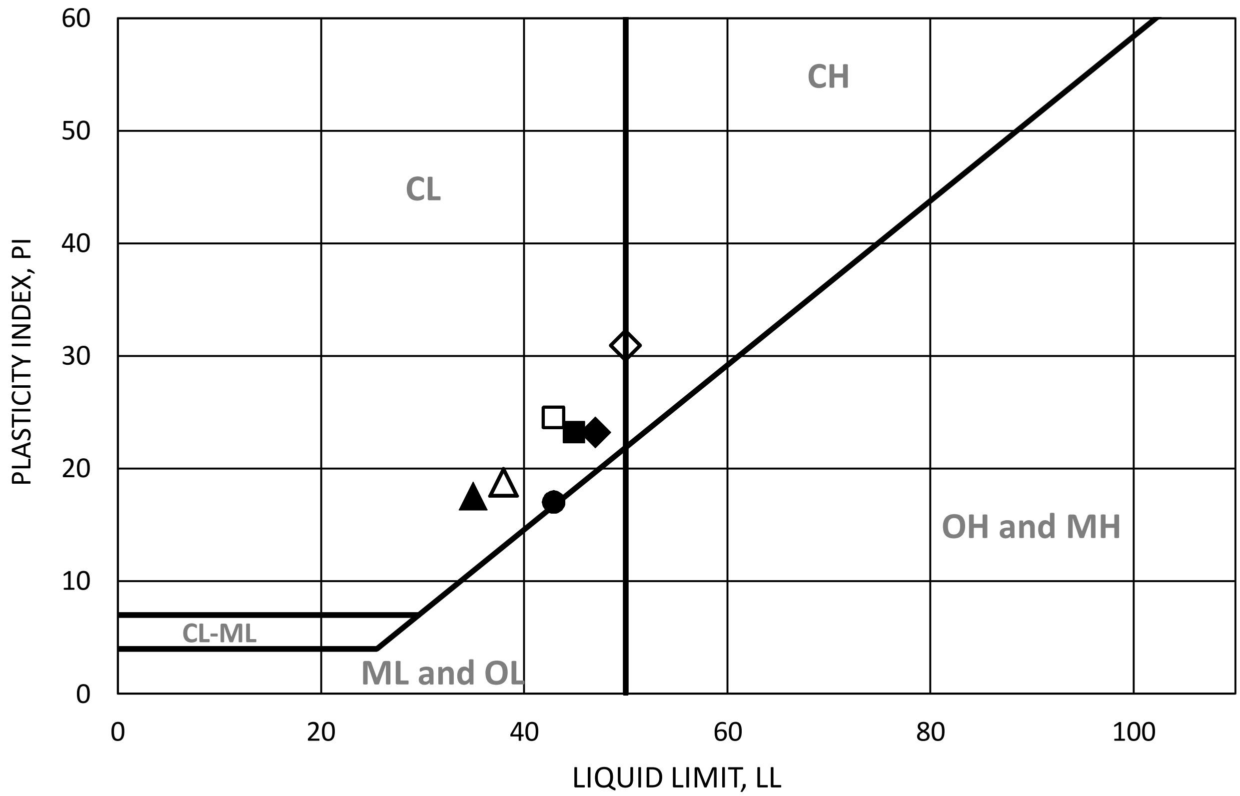
Checked by: PZ

Project No.: W1089-06-01

21101 West Ventura Boulevard
Los Angeles, California

Jan. 2020

Figure B10



SYMBOL	BORING	DEPTH (ft)	LL	PL	PI	MOISTURE CONTENT AT SATURATION	SOIL BEHAVIOR
■	B1	15	45	22	23	--	CL
◆	B1	22.5	47	24	23	--	CL
▲	B1	30	35	17	18	--	CL
●	B1	40	43	26	17	31.7	CL
□	B1	47.5	43	19	24	--	CL
◇	B1	50	50	19	31	--	CL
△	B1	57.5	38	19	19	--	CL
○							

N/P = Non-Plastic

GEOCON

ATTERBERG LIMITS

ASTM D-4318

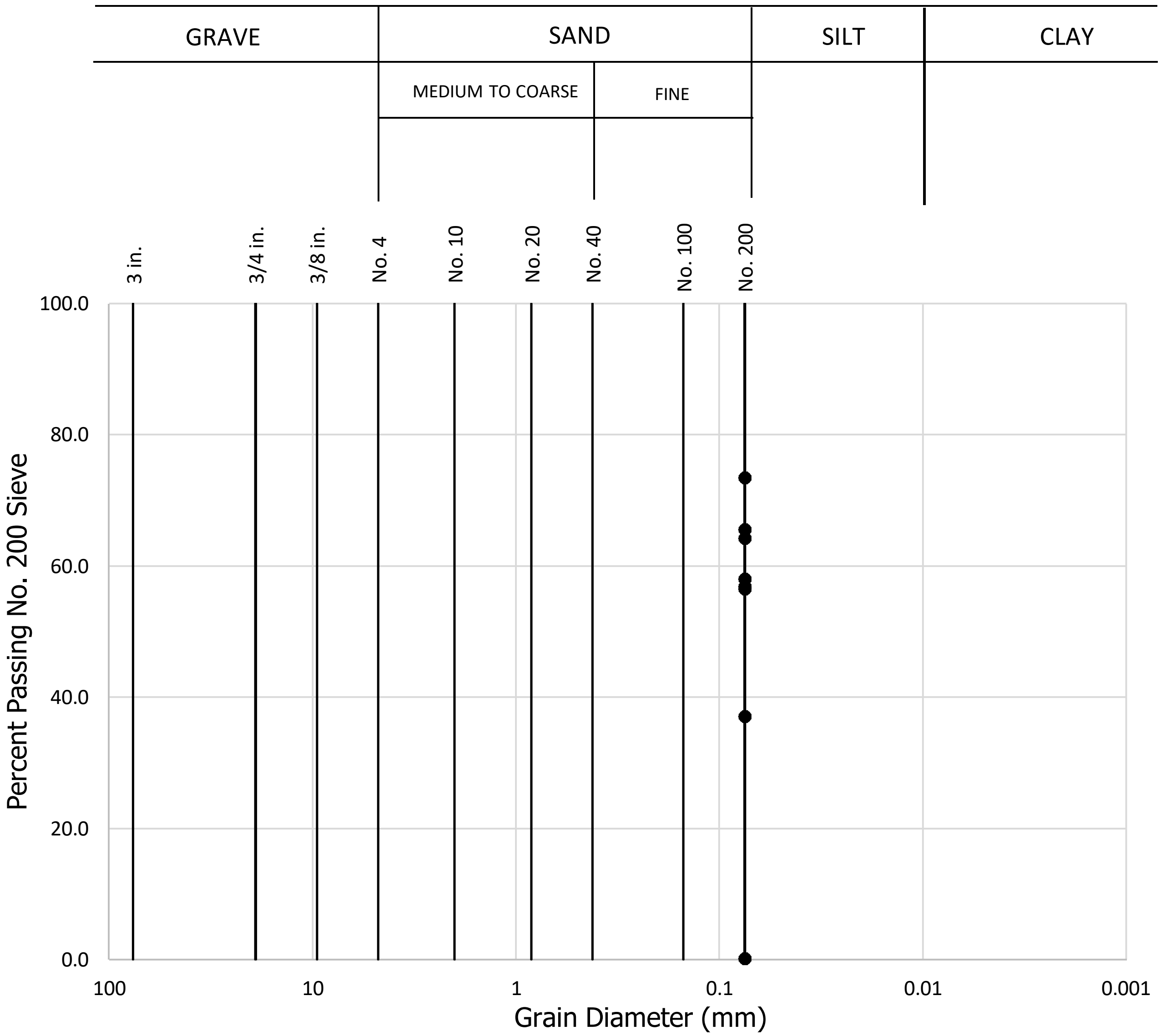
Checked by: PZ

Project No.: W1089-06-01

21101 West Ventura Boulevard
Los Angeles, California

Jan. 2020

Figure B11



Sample No.	Percent Passing No. 200 Sieve
B1 @ 7.5'	65.6
B1 @ 12.5'	64.2
B1 @ 17.5'	56.5
B1 @ 22.5'	73.4
B1 @ 27.5'	58.0
B1 @ 32.5'	0.1
B1 @ 37.5'	37.0
B1 @ 47.5'	56.8

GEOCON

GRAIN SIZE ANALYSIS

ASTM D-1140

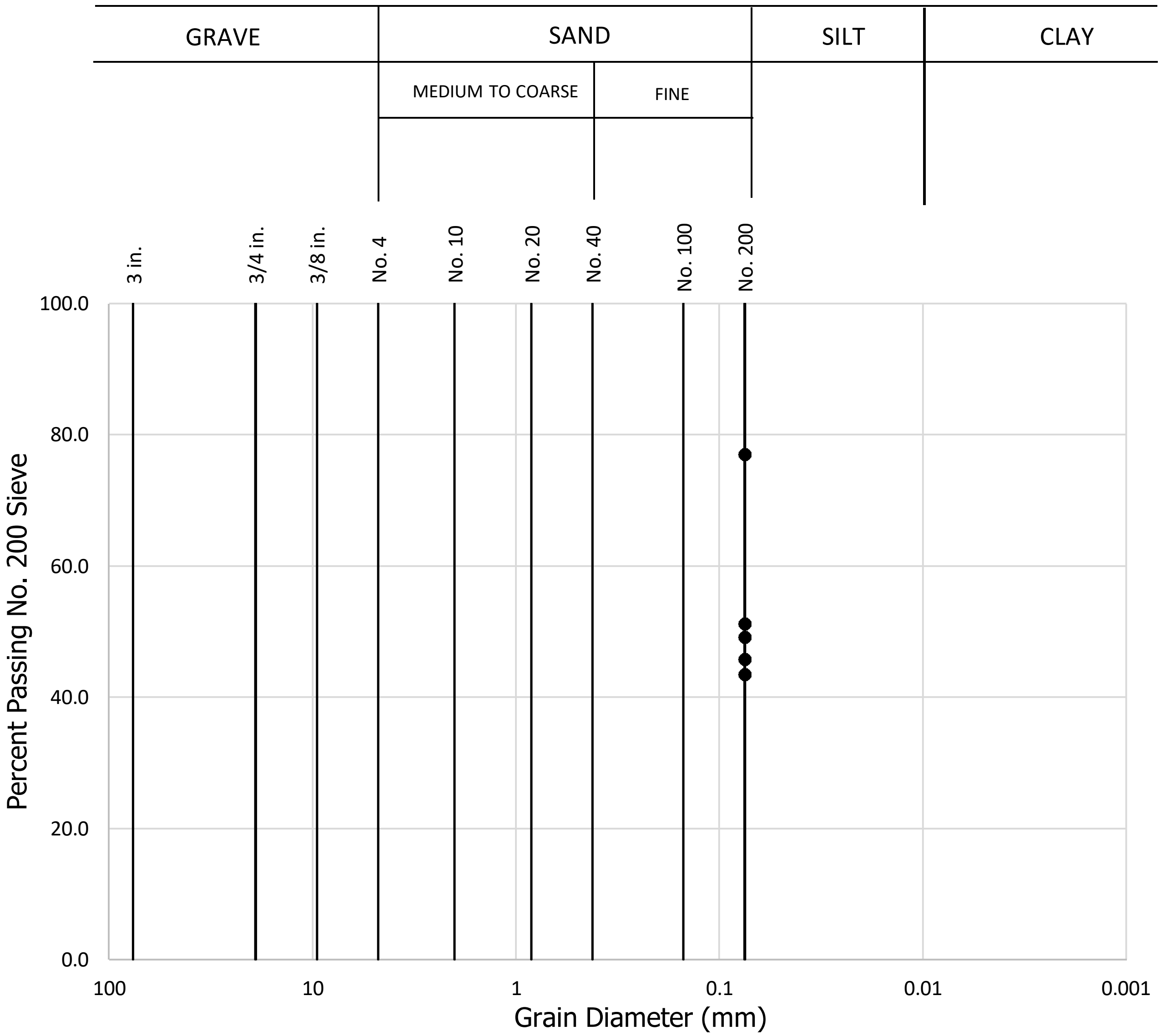
Checked by: PZ

Project No.: W1089-06-01

21101 West Ventura Boulevard
Los Angeles, California

Jan. 2020

Figure B12



Sample No.	Percent Passing No. 200 Sieve
B1 @ 52.5'	49.1
B1 @ 57.5'	51.2
B2 @ 12.5'	45.7
B2 @ 17.5'	43.4
B2 @ 22.5'	77.0

GEOCON

GRAIN SIZE ANALYSIS

ASTM D-1140

Checked by: PZ

Project No.: W1089-06-01

21101 West Ventura Boulevard
Los Angeles, California

Jan. 2020

Figure B13

B1@0-5'

MOLDED SPECIMEN		BEFORE TEST	AFTER TEST
Specimen Diameter	(in.)	4.0	4.0
Specimen Height	(in.)	1.0	0.8
Wt. Comp. Soil + Mold	(gm)	734.7	776.5
Wt. of Mold	(gm)	368.3	368.3
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	494.4	776.5
Dry Wt. of Soil + Cont.	(gm)	457.1	320.8
Wt. of Container	(gm)	194.4	368.3
Moisture Content	(%)	14.2	27.2
Wet Density	(pcf)	110.5	123.0
Dry Density	(pcf)	96.8	96.6
Void Ratio		0.7	0.3
Total Porosity		0.4	0.3
Pore Volume	(cc)	88.2	41.5
Degree of Saturation	(%) [S_{meas}]	52.1	210.5

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
11/11/2019	10:00	1.0	0	0.199
11/11/2019	10:10	1.0	10	0.1985
Add Distilled Water to the Specimen				
11/12/2019	10:00	1.0	1430	0.2745
11/12/2019	11:00	1.0	1490	0.2745

Expansion Index (EI meas) =	76
Expansion Index (Report) =	**76**

Expansion Index, EI_{50}	CBC CLASSIFICATION *	UBC CLASSIFICATION **
0-20	Non-Expansive	Very Low
21-50	Expansive	Low
51-90	Expansive	Medium
91-130	Expansive	High
>130	Expansive	Very High

* Reference: 2016 California Building Code, Section 1803.5.3
** Reference: 1997 Uniform Building Code, Table 18-I-B.

GEOCON

EXPANSION INDEX TEST RESULTS
ASTM D 4829

Checked by: PZ

Project No.: W1089-06-01

21101 West Ventura Boulevard
Los Angeles, California

Jan. 2020

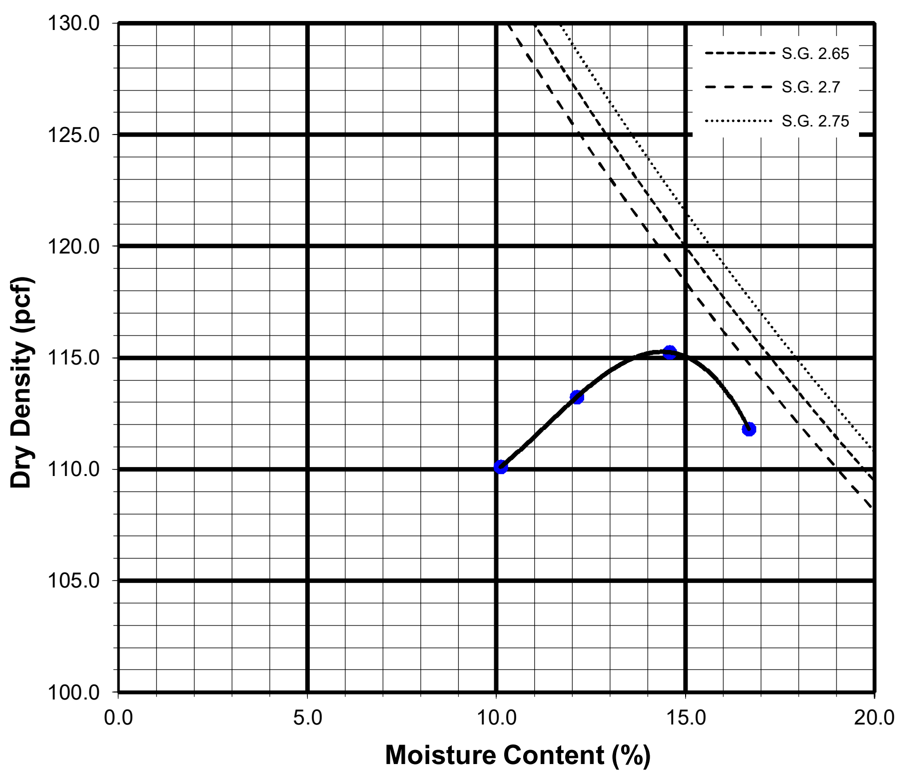
Figure B14

Sample No:

B1@0-5'	Dark Brown Sandy Silt (ML)

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	5983	6069	6146	6122		
Weight of Mold	(g)	4152	4152	4152	4152		
Net Weight of Soil	(g)	1831	1918	1995	1971		
Wet Weight of Soil + Cont.	(g)	674.3	703.5	739.9	733.2		
Dry Weight of Soil + Cont.	(g)	624.9	643.4	664.5	649.4		
Weight of Container	(g)	135.8	147.2	147.3	147.0		
Moisture Content	(%)	10.1	12.1	14.6	16.7		
Wet Density	(pcf)	121.2	126.9	132.1	130.5		
Dry Density	(pcf)	110.1	113.2	115.2	111.8		

Maximum Dry Density (pcf)	**115.0**	**Optimum Moisture Content (%)**	**14.0**



Preparation Method: A

GEOCON	**MODIFIED COMPACTION TEST OF SOILS** ASTM D-1557	Project No.: W1089-06-01
		21101 West Ventura Boulevard Los Angeles, California
	Checked by: PZ	Jan. 2020 Figure B15

SUMMARY OF LABORATORY POTENTIAL
OF HYDROGEN (pH) AND RESISTIVITY TEST RESULTS
CALIFORNIA TEST NO. 643

Sample No.	**pH**	**Resistivity (ohm centimeters)**
B1 @ 0-5'	7.7	930 (Severely Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B1 @ 0-5'	0.012

SUMMARY OF LABORATORY WATER SOLUBLE SULFATE TEST RESULTS
CALIFORNIA TEST NO. 417

Sample No.	**Water Soluble Sulfate (% SQ_4)**	**Sulfate Exposure***
B1 @0-5'	0.000	S0

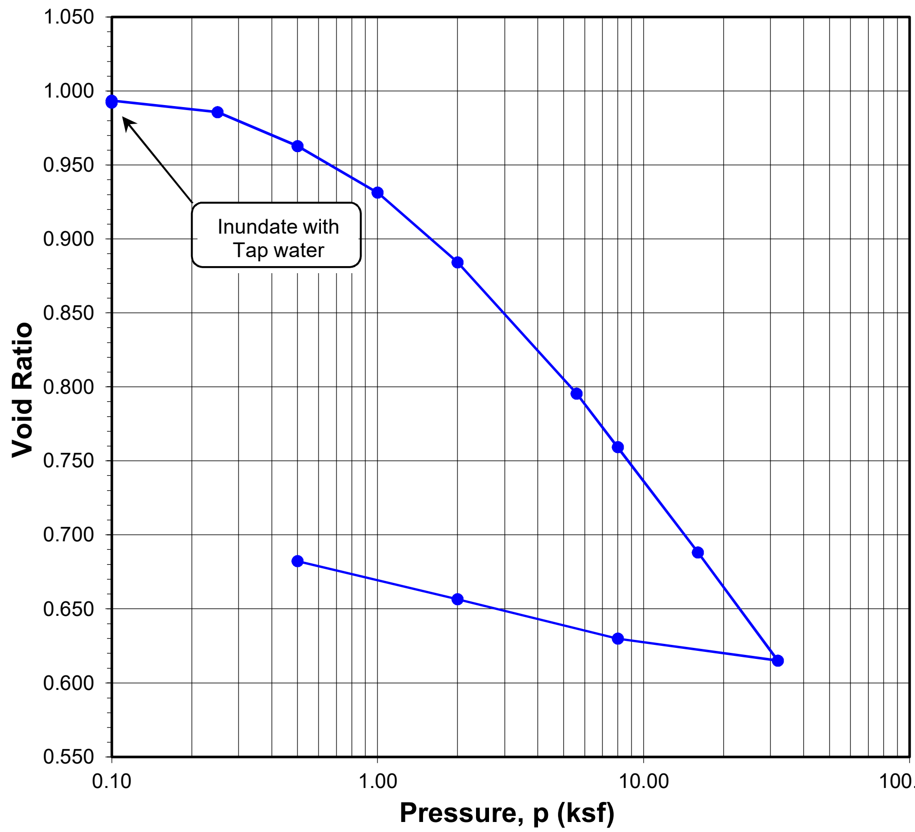
GEOCON	**CORROSIVITY TEST RESULTS**	Project No.: W1089-06-01
		21101 West Ventura Boulevard Los Angeles, California
	Checked by: PZ	Jan. 2020 Figure B16

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name:	JDA Woodland Hills	Tested By: G. Bathala	Date: 07/11/22
Project No.:	13589.002	Checked By: J. Ward	Date: 07/28/22
Boring No.:	LB-1	Depth (ft.): 25.0	
Sample No.:	R-8	Sample Type:	Ring
Soil Identification:	Brown lean clay (CL)		

Sample Diameter (in.)	2.415
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	186.18
Weight of Ring (g)	45.36
Height after consol. (in.)	0.8428
Before Test	
Wt.Wet Sample+Cont. (g)	292.29
Wt.of Dry Sample+Cont. (g)	230.84
Weight of Container (g)	60.93
Initial Moisture Content (%)	36.2
Initial Dry Density (pcf)	86.0
Initial Saturation (%)	100
Initial Vertical Reading (in.)	0.3020
After Test	
Wt.of Wet Sample+Cont. (g)	250.38
Wt. of Dry Sample+Cont. (g)	224.18
Weight of Container (g)	76.75
Final Moisture Content (%)	25.67
Final Dry Density (pcf)	100.7
Final Saturation (%)	100
Final Vertical Reading (in.)	0.1394
Specific Gravity (assumed)	2.75
Water Density (pcf)	62.43

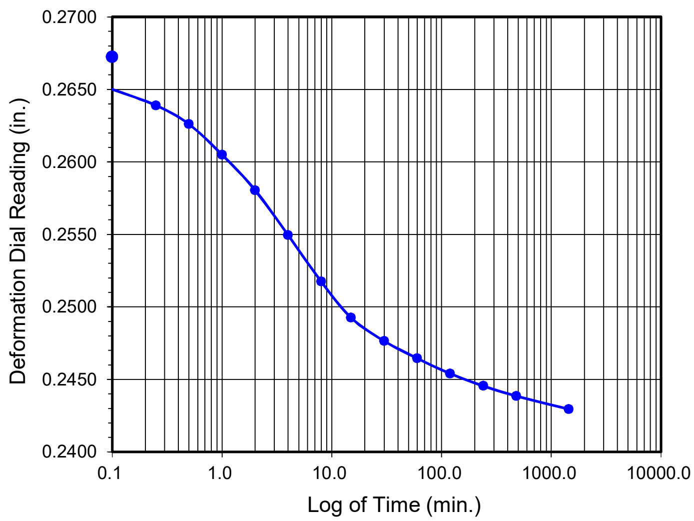


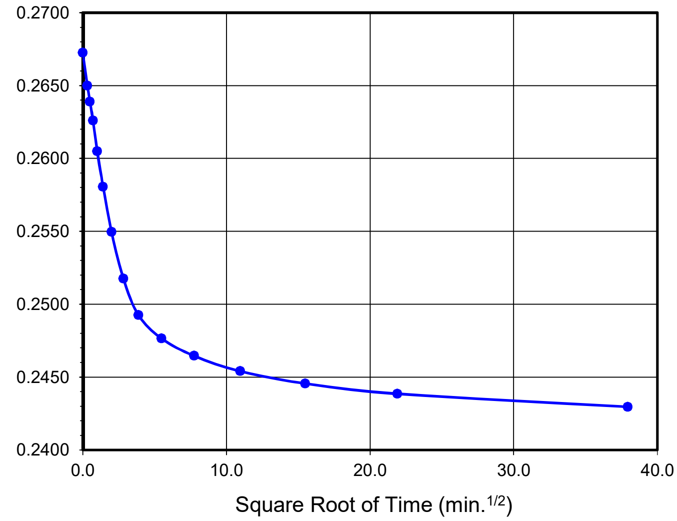
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deforma-tion (%)
0.10	0.3000	0.9980	0.00	0.20	0.992	0.20
0.10	0.3007	0.9987	0.00	0.13	0.993	0.13
0.25	0.2963	0.9943	0.05	0.57	0.986	0.52
0.50	0.2842	0.9822	0.11	1.78	0.963	1.67
1.00	0.2677	0.9657	0.19	3.44	0.931	3.25
2.00	0.2430	0.9410	0.30	5.91	0.884	5.61
5.60	0.1971	0.8951	0.44	10.49	0.796	10.05
8.00	0.1769	0.8749	0.65	12.51	0.759	11.86
16.00	0.1386	0.8366	0.91	16.34	0.688	15.43
32.00	0.0994	0.7974	1.17	20.26	0.615	19.09
8.00	0.1096	0.8076	0.90	19.24	0.630	18.34
2.00	0.1253	0.8233	0.66	17.68	0.656	17.02
0.50	0.1394	0.8374	0.54	16.27	0.682	15.73

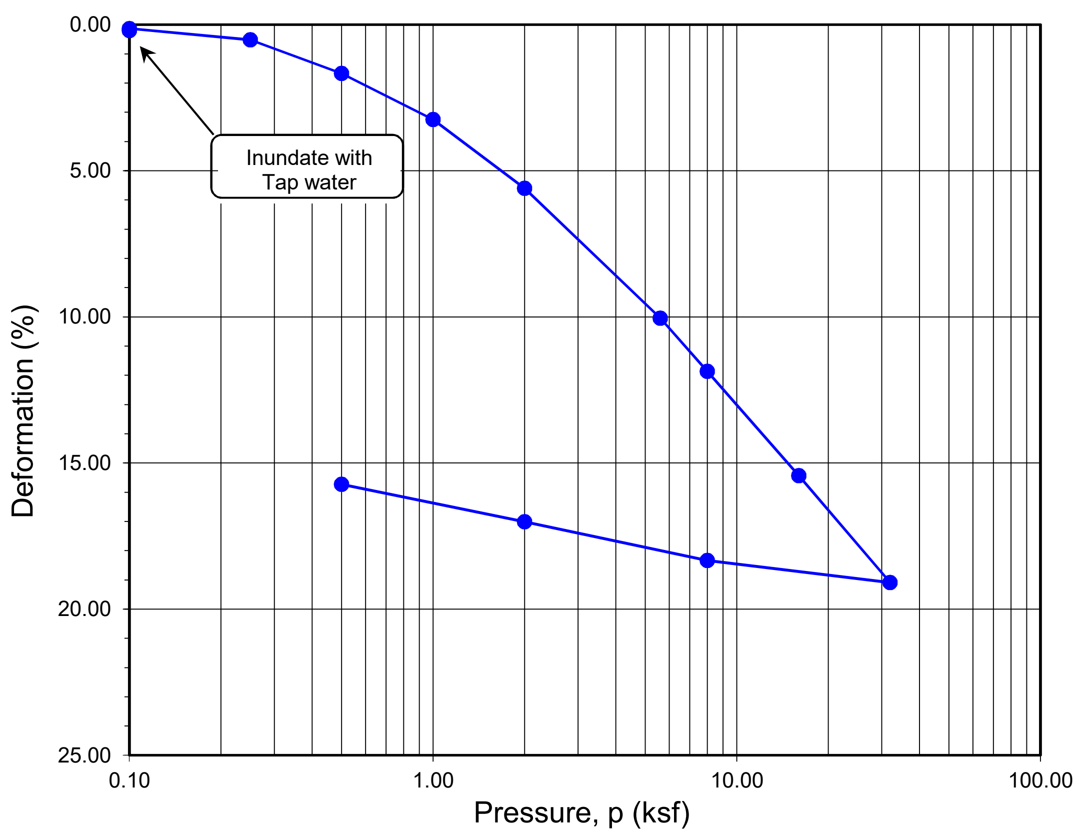
Time Readings @ 2.0 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
7/14/22	7:50:00	0.0	0.0	0.2673
7/14/22	7:50:06	0.1	0.3	0.2650
7/14/22	7:50:15	0.2	0.5	0.2639
7/14/22	7:50:30	0.5	0.7	0.2626
7/14/22	7:51:00	1.0	1.0	0.2605
7/14/22	7:52:00	2.0	1.4	0.2581
7/14/22	7:54:00	4.0	2.0	0.2550
7/14/22	7:58:00	8.0	2.8	0.2518
7/14/22	8:05:00	15.0	3.9	0.2493
7/14/22	8:20:00	30.0	5.5	0.2477
7/14/22	8:50:00	60.0	7.7	0.2465
7/14/22	9:50:00	120.0	11.0	0.2454
7/14/22	11:50:00	240.0	15.5	0.2446
7/14/22	15:50:00	480.0	21.9	0.2439
7/15/22	7:49:00	1439.0	37.9	0.2430

Time Readings @ 2.0 ksf







Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-1	**R-8**	**25.0**	**36.2**	**25.7**	**86.0**	**100.7**	**0.996**	**0.682**	**100**	**100**

Soil Identification: Brown lean clay (CL)

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 13589.002

JDA Woodland Hills

07-22

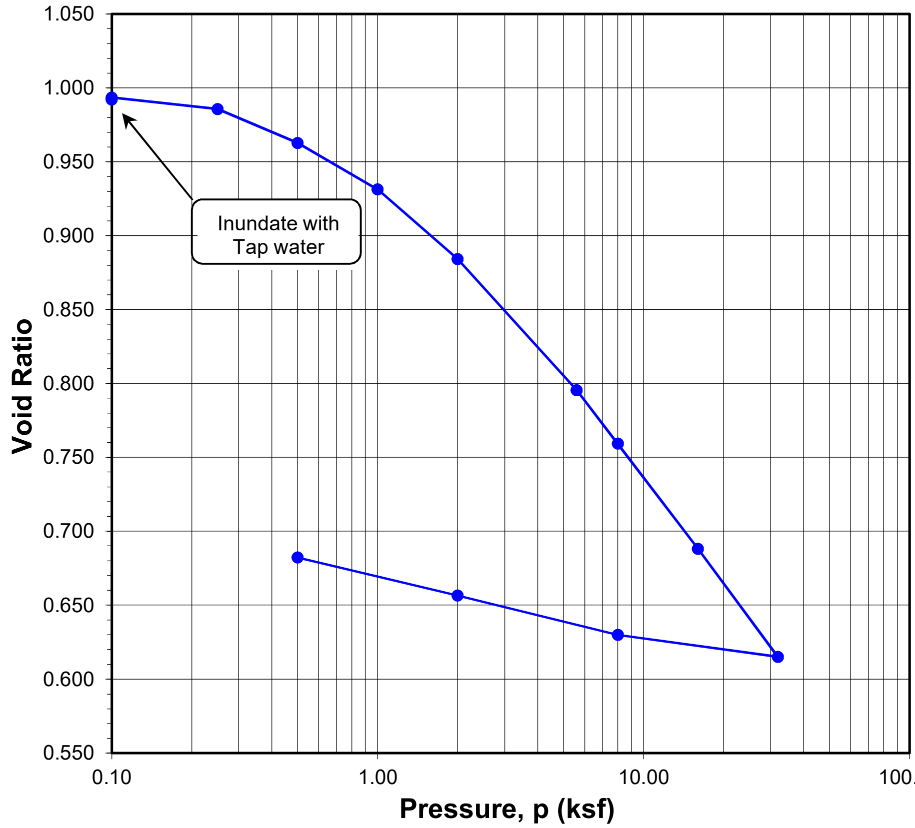
ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name: JDA Woodland Hills
Project No.: 13589.002
Boring No.: LB-1
Sample No.: R-8
Soil Identification: Brown lean clay (CL)

Tested By: G. Bathala Date: 07/11/22
Checked By: J. Ward Date: 07/28/22
Depth (ft.): 25.0
Sample Type: Ring

Sample Diameter (in.)	2.415
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	186.18
Weight of Ring (g)	45.36
Height after consol. (in.)	0.8428
Before Test	
Wt.Wet Sample+Cont. (g)	292.29
Wt.of Dry Sample+Cont. (g)	230.84
Weight of Container (g)	60.93
Initial Moisture Content (%)	36.2
Initial Dry Density (pcf)	86.0
Initial Saturation (%)	100
Initial Vertical Reading (in.)	0.3020
After Test	
Wt.of Wet Sample+Cont. (g)	250.38
Wt. of Dry Sample+Cont. (g)	224.18
Weight of Container (g)	76.75
Final Moisture Content (%)	25.67
Final Dry Density (pcf)	100.7
Final Saturation (%)	100
Final Vertical Reading (in.)	0.1394
Specific Gravity (assumed)	2.75
Water Density (pcf)	62.43

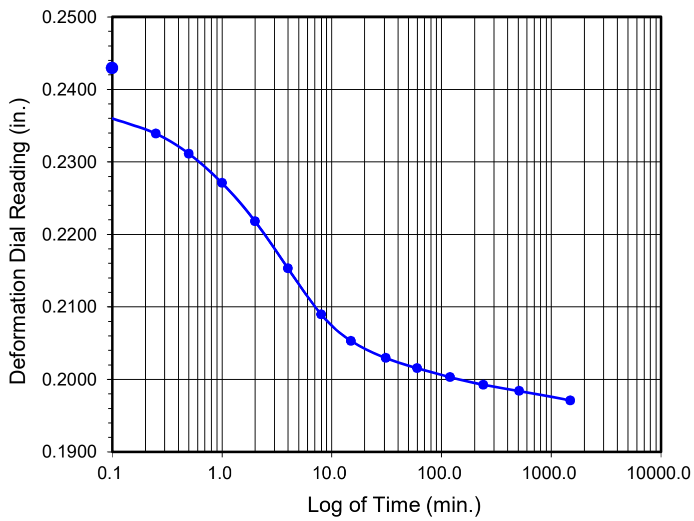


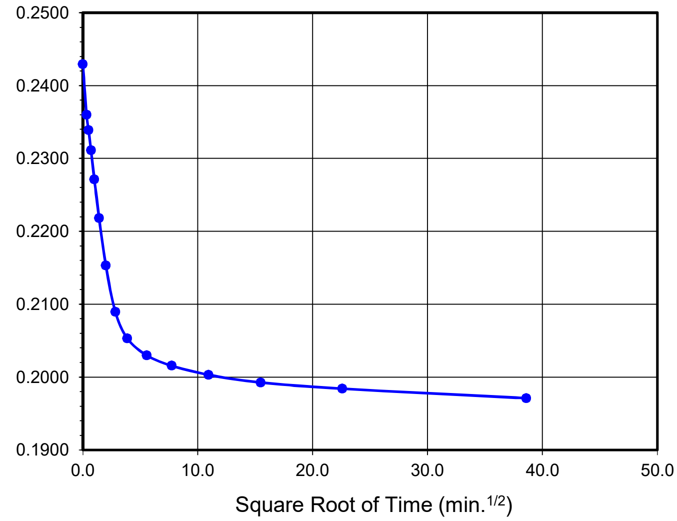
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deforma-tion (%)
0.10	0.3000	0.9980	0.00	0.20	0.992	0.20
0.10	0.3007	0.9987	0.00	0.13	0.993	0.13
0.25	0.2963	0.9943	0.05	0.57	0.986	0.52
0.50	0.2842	0.9822	0.11	1.78	0.963	1.67
1.00	0.2677	0.9657	0.19	3.44	0.931	3.25
2.00	0.2430	0.9410	0.30	5.91	0.884	5.61
5.60	0.1971	0.8951	0.44	10.49	0.796	10.05
8.00	0.1769	0.8749	0.65	12.51	0.759	11.86
16.00	0.1386	0.8366	0.91	16.34	0.688	15.43
32.00	0.0994	0.7974	1.17	20.26	0.615	19.09
8.00	0.1096	0.8076	0.90	19.24	0.630	18.34
2.00	0.1253	0.8233	0.66	17.68	0.656	17.02
0.50	0.1394	0.8374	0.54	16.27	0.682	15.73

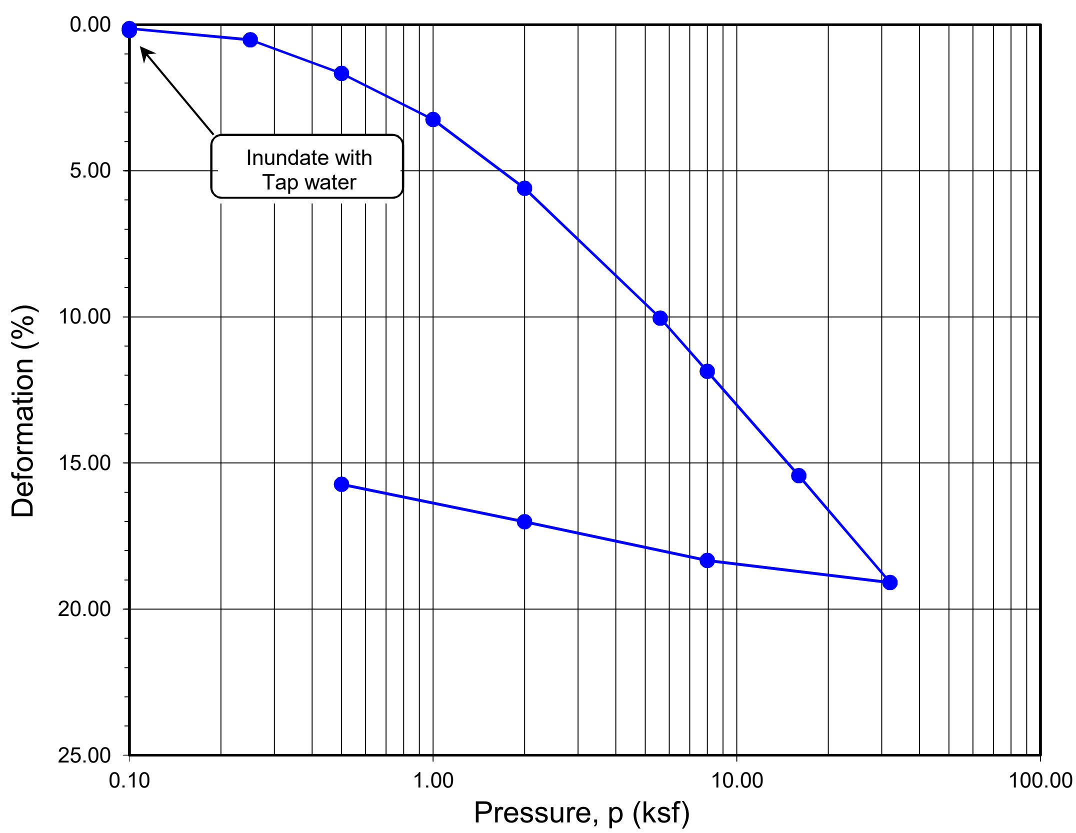
Time Readings @ 5.6 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
7/15/22	7:50:00	0.0	0.0	0.2430
7/15/22	7:50:06	0.1	0.3	0.2360
7/15/22	7:50:15	0.2	0.5	0.2339
7/15/22	7:50:30	0.5	0.7	0.2311
7/15/22	7:51:00	1.0	1.0	0.2271
7/15/22	7:52:00	2.0	1.4	0.2218
7/15/22	7:54:00	4.0	2.0	0.2153
7/15/22	7:58:00	8.0	2.8	0.2090
7/15/22	8:05:00	15.0	3.9	0.2053
7/15/22	8:21:00	31.0	5.6	0.2030
7/15/22	8:50:00	60.0	7.7	0.2016
7/15/22	9:50:00	120.0	11.0	0.2003
7/15/22	11:50:00	240.0	15.5	0.1993
7/15/22	16:20:00	510.0	22.6	0.1984
7/16/22	8:40:00	1490.0	38.6	0.1971

Time Readings @ 5.6 ksf







Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-1	**R-8**	**25.0**	**36.2**	**25.7**	**86.0**	**100.7**	**0.996**	**0.682**	**100**	**100**

Soil Identification: Brown lean clay (CL)

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 13589.002

JDA Woodland Hills

07-22

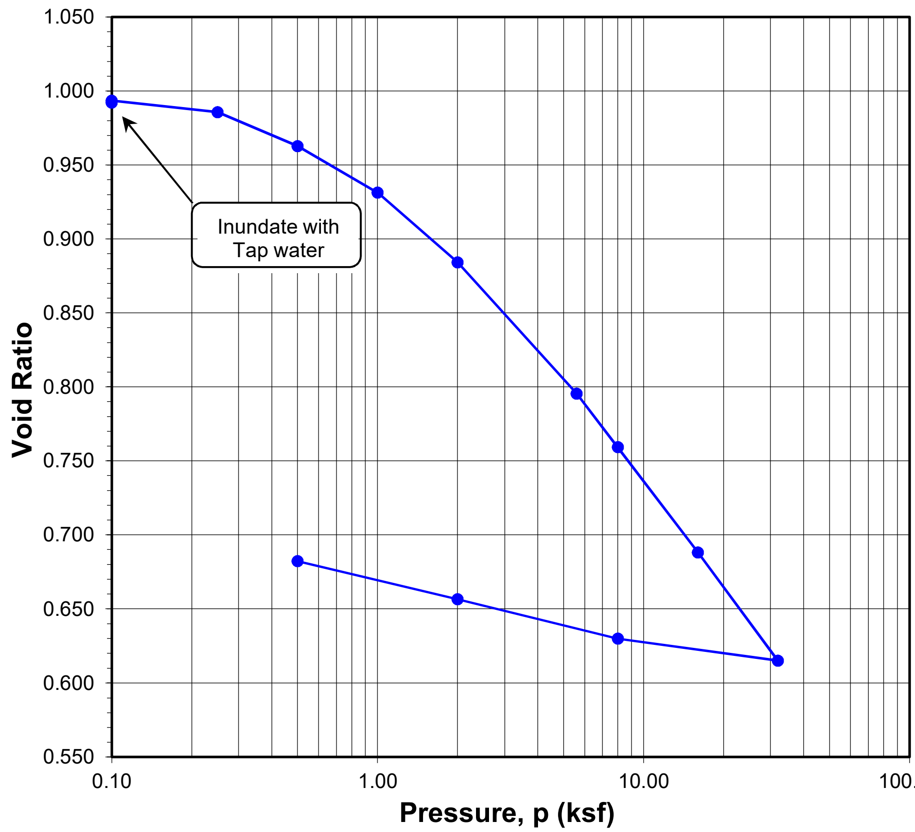
ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name: JDA Woodland Hills
Project No.: 13589.002
Boring No.: LB-1
Sample No.: R-8
Soil Identification: Brown lean clay (CL)

Tested By: G. Bathala Date: 07/11/22
Checked By: J. Ward Date: 07/28/22
Depth (ft.): 25.0
Sample Type: Ring

Sample Diameter (in.)	2.415
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	186.18
Weight of Ring (g)	45.36
Height after consol. (in.)	0.8428
Before Test	
Wt.Wet Sample+Cont. (g)	292.29
Wt.of Dry Sample+Cont. (g)	230.84
Weight of Container (g)	60.93
Initial Moisture Content (%)	36.2
Initial Dry Density (pcf)	86.0
Initial Saturation (%)	100
Initial Vertical Reading (in.)	0.3020
After Test	
Wt.of Wet Sample+Cont. (g)	250.38
Wt. of Dry Sample+Cont. (g)	224.18
Weight of Container (g)	76.75
Final Moisture Content (%)	25.67
Final Dry Density (pcf)	100.7
Final Saturation (%)	100
Final Vertical Reading (in.)	0.1394
Specific Gravity (assumed)	2.75
Water Density (pcf)	62.43

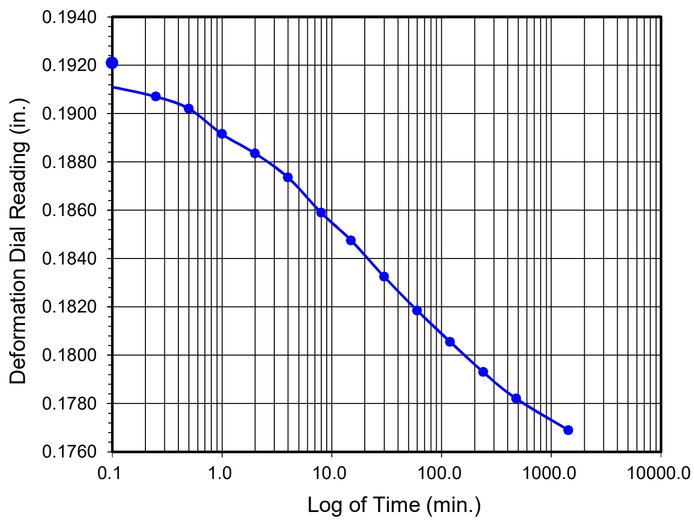


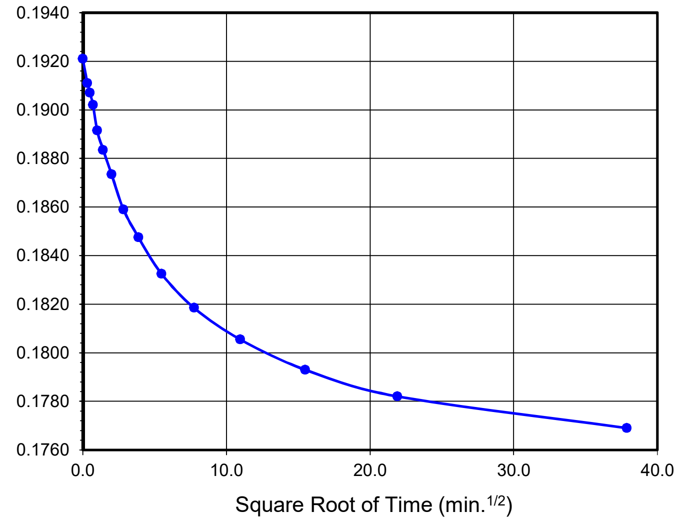
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.3000	0.9980	0.00	0.20	0.992	0.20
0.10	0.3007	0.9987	0.00	0.13	0.993	0.13
0.25	0.2963	0.9943	0.05	0.57	0.986	0.52
0.50	0.2842	0.9822	0.11	1.78	0.963	1.67
1.00	0.2677	0.9657	0.19	3.44	0.931	3.25
2.00	0.2430	0.9410	0.30	5.91	0.884	5.61
5.60	0.1971	0.8951	0.44	10.49	0.796	10.05
8.00	0.1769	0.8749	0.65	12.51	0.759	11.86
16.00	0.1386	0.8366	0.91	16.34	0.688	15.43
32.00	0.0994	0.7974	1.17	20.26	0.615	19.09
8.00	0.1096	0.8076	0.90	19.24	0.630	18.34
2.00	0.1253	0.8233	0.66	17.68	0.656	17.02
0.50	0.1394	0.8374	0.54	16.27	0.682	15.73

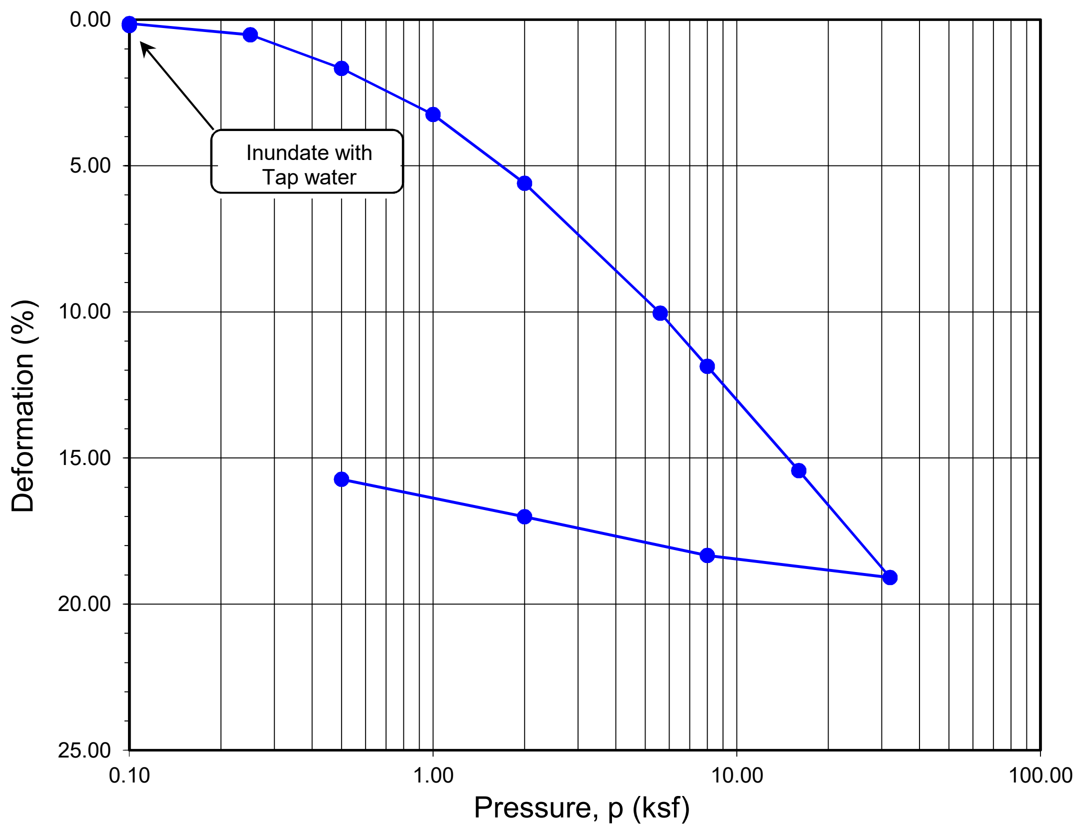
Time Readings @ 8.0 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
7/18/22	7:40:00	0.0	0.0	0.1921
7/18/22	7:40:06	0.1	0.3	0.1911
7/18/22	7:40:15	0.2	0.5	0.1907
7/18/22	7:40:30	0.5	0.7	0.1902
7/18/22	7:41:00	1.0	1.0	0.1892
7/18/22	7:42:00	2.0	1.4	0.1884
7/18/22	7:44:00	4.0	2.0	0.1874
7/18/22	7:48:00	8.0	2.8	0.1859
7/18/22	7:55:00	15.0	3.9	0.1848
7/18/22	8:10:00	30.0	5.5	0.1833
7/18/22	8:40:00	60.0	7.7	0.1819
7/18/22	9:40:00	120.0	11.0	0.1806
7/18/22	11:40:00	240.0	15.5	0.1793
7/18/22	15:40:00	480.0	21.9	0.1782
7/19/22	7:35:00	1435.0	37.9	0.1769

Time Readings @ 8.0 ksf







Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-1	**R-8**	**25.0**	**36.2**	**25.7**	**86.0**	**100.7**	**0.996**	**0.682**	**100**	**100**

Soil Identification: Brown lean clay (CL)

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 13589.002

JDA Woodland Hills

07-22

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

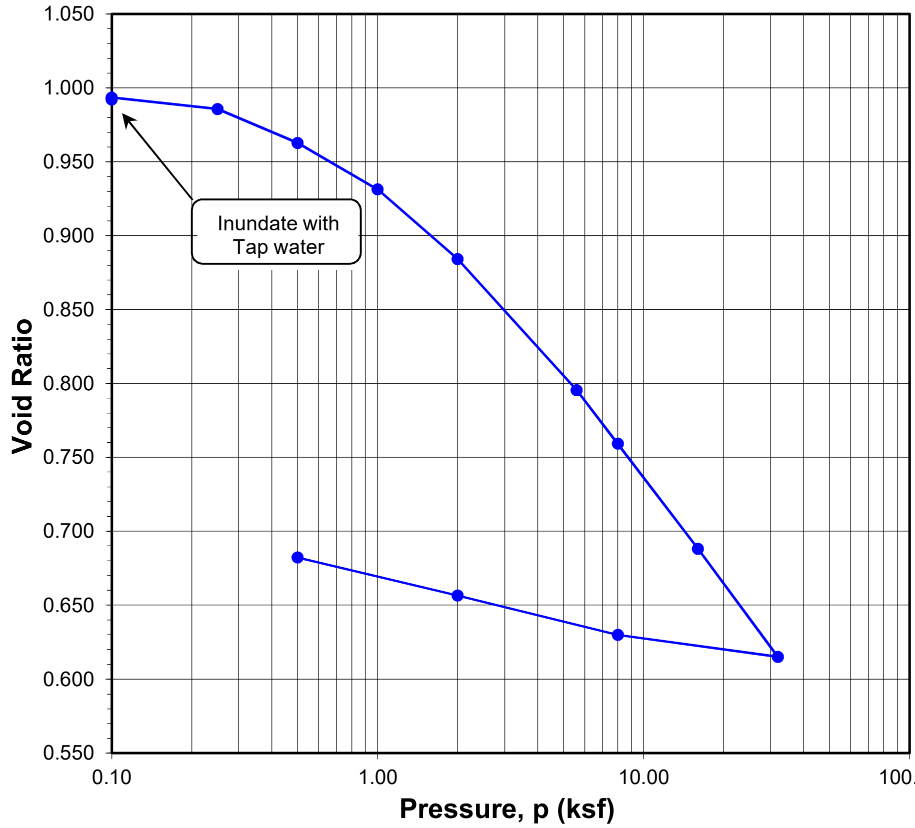
ASTM D 2435

Leighton

Project Name: JDA Woodland Hills
Project No.: 13589.002
Boring No.: LB-1
Sample No.: R-8
Soil Identification: Brown lean clay (CL)

Tested By: G. Bathala Date: 07/11/22
Checked By: J. Ward Date: 07/28/22
Depth (ft.): 25.0
Sample Type: Ring

Sample Diameter (in.)	2.415
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	186.18
Weight of Ring (g)	45.36
Height after consol. (in.)	0.8428
Before Test	
Wt.Wet Sample+Cont. (g)	292.29
Wt.of Dry Sample+Cont. (g)	230.84
Weight of Container (g)	60.93
Initial Moisture Content (%)	36.2
Initial Dry Density (pcf)	86.0
Initial Saturation (%)	100
Initial Vertical Reading (in.)	0.3020
After Test	
Wt.of Wet Sample+Cont. (g)	250.38
Wt. of Dry Sample+Cont. (g)	224.18
Weight of Container (g)	76.75
Final Moisture Content (%)	25.67
Final Dry Density (pcf)	100.7
Final Saturation (%)	100
Final Vertical Reading (in.)	0.1394
Specific Gravity (assumed)	2.75
Water Density (pcf)	62.43

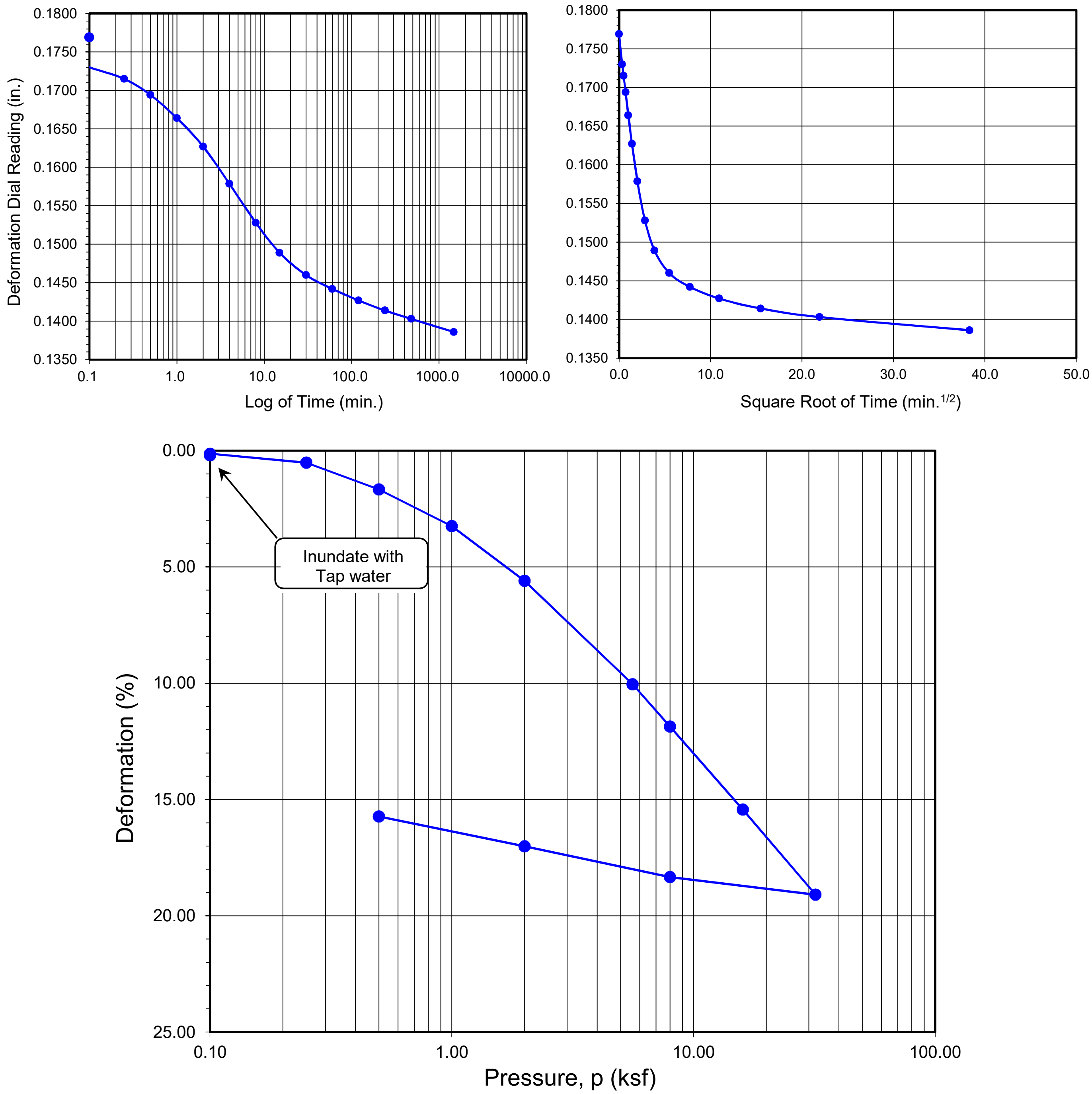


Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deforma-tion (%)
0.10	0.3000	0.9980	0.00	0.20	0.992	0.20
0.10	0.3007	0.9987	0.00	0.13	0.993	0.13
0.25	0.2963	0.9943	0.05	0.57	0.986	0.52
0.50	0.2842	0.9822	0.11	1.78	0.963	1.67
1.00	0.2677	0.9657	0.19	3.44	0.931	3.25
2.00	0.2430	0.9410	0.30	5.91	0.884	5.61
5.60	0.1971	0.8951	0.44	10.49	0.796	10.05
8.00	0.1769	0.8749	0.65	12.51	0.759	11.86
16.00	0.1386	0.8366	0.91	16.34	0.688	15.43
32.00	0.0994	0.7974	1.17	20.26	0.615	19.09
8.00	0.1096	0.8076	0.90	19.24	0.630	18.34
2.00	0.1253	0.8233	0.66	17.68	0.656	17.02
0.50	0.1394	0.8374	0.54	16.27	0.682	15.73

Time Readings @ 16.0 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
7/19/22	7:55:00	0.0	0.0	0.1769
7/19/22	7:55:06	0.1	0.3	0.1730
7/19/22	7:55:15	0.2	0.5	0.1715
7/19/22	7:55:30	0.5	0.7	0.1694
7/19/22	7:56:00	1.0	1.0	0.1664
7/19/22	7:57:00	2.0	1.4	0.1627
7/19/22	7:59:00	4.0	2.0	0.1579
7/19/22	8:03:00	8.0	2.8	0.1528
7/19/22	8:10:00	15.0	3.9	0.1489
7/19/22	8:25:00	30.0	5.5	0.1460
7/19/22	8:55:00	60.0	7.7	0.1442
7/19/22	9:55:00	120.0	11.0	0.1427
7/19/22	11:55:00	240.0	15.5	0.1414
7/19/22	15:55:00	480.0	21.9	0.1403
7/20/22	8:24:00	1469.0	38.3	0.1386

Time Readings @ 16.0 ksf



Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-1	**R-8**	**25.0**	**36.2**	**25.7**	**86.0**	**100.7**	**0.996**	**0.682**	**100**	**100**

Soil Identification: Brown lean clay (CL)

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 13589.002

JDA Woodland Hills

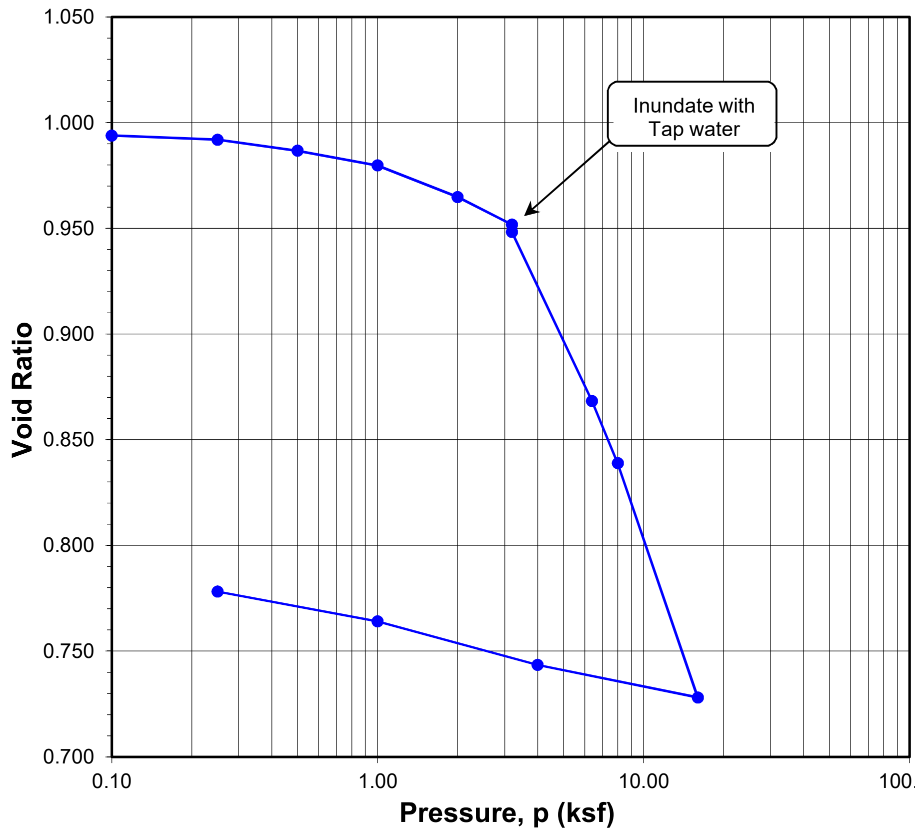
07-22

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Leighton

Project Name:	JDA Woodland Hills	Tested By: G. Bathala	Date: 07/11/22
Project No.:	13589.002	Checked By: J. Ward	Date: 07/28/22
Boring No.:	LB-1	Depth (ft.): 12.5	
Sample No.:	ST-4	Sample Type:	Shelby
Soil Identification:	Dark yellowish brown fat clay (CH)		

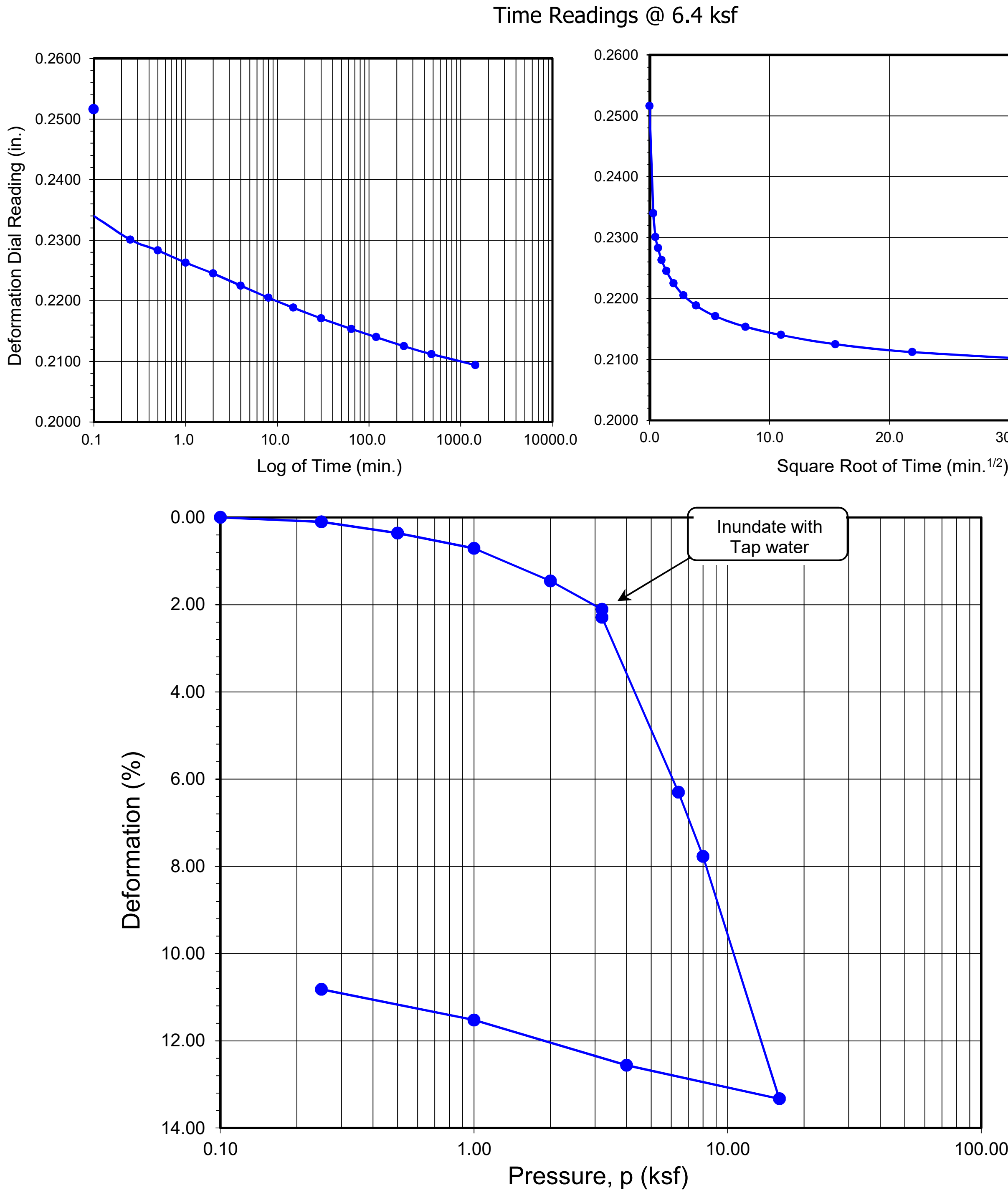
Sample Diameter (in.)	2.865
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	261.36
Weight of Ring (g)	82.67
Height after consol. (in.)	0.8918
Before Test	
Wt.Wet Sample+Cont. (g)	236.08
Wt.of Dry Sample+Cont. (g)	201.92
Weight of Container (g)	64.76
Initial Moisture Content (%)	24.9
Initial Dry Density (pcf)	84.5
Initial Saturation (%)	68
Initial Vertical Reading (in.)	0.2799
After Test	
Wt.of Wet Sample+Cont. (g)	338.97
Wt. of Dry Sample+Cont. (g)	300.40
Weight of Container (g)	75.50
Final Moisture Content (%)	27.12
Final Dry Density (pcf)	94.2
Final Saturation (%)	93
Final Vertical Reading (in.)	0.1685
Specific Gravity (assumed)	2.70
Water Density (pcf)	62.43



Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.2799	1.0000	0.00	0.00	0.994	0.00
0.25	0.2780	0.9981	0.09	0.19	0.992	0.10
0.50	0.2745	0.9946	0.18	0.54	0.987	0.36
1.00	0.2699	0.9900	0.29	1.00	0.980	0.71
2.00	0.2610	0.9811	0.43	1.89	0.965	1.46
3.20	0.2534	0.9735	0.54	2.65	0.952	2.11
3.20	0.2516	0.9717	0.54	2.83	0.948	2.29
6.40	0.2094	0.9295	0.75	7.05	0.868	6.30
8.00	0.1938	0.9139	0.84	8.61	0.839	7.77
16.00	0.1359	0.8560	1.07	14.40	0.728	13.33
4.00	0.1460	0.8661	0.83	13.39	0.743	12.56
1.00	0.1593	0.8794	0.54	12.07	0.764	11.53
0.25	0.1685	0.8886	0.32	11.14	0.778	10.82

Time Readings @ 6.4 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
7/14/22	7:40:00	0.0	0.0	0.2516
7/14/22	7:40:06	0.1	0.3	0.2340
7/14/22	7:40:15	0.2	0.5	0.2301
7/14/22	7:40:30	0.5	0.7	0.2283
7/14/22	7:41:00	1.0	1.0	0.2263
7/14/22	7:42:00	2.0	1.4	0.2245
7/14/22	7:44:00	4.0	2.0	0.2225
7/14/22	7:48:00	8.0	2.8	0.2205
7/14/22	7:55:00	15.0	3.9	0.2189
7/14/22	8:10:00	30.0	5.5	0.2171
7/14/22	8:44:00	64.0	8.0	0.2154
7/14/22	9:40:00	120.0	11.0	0.2140
7/14/22	11:40:00	240.0	15.5	0.2125
7/14/22	15:40:00	480.0	21.9	0.2112
7/15/22	7:40:00	1440.0	37.9	0.2094



Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-1	**ST-4**	**12.5**	**24.9**	**27.1**	**84.5**	**94.2**	**0.994**	**0.778**	**68**	**93**

Soil Identification: Dark yellowish brown fat clay (CH)

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 13589.002

JDA Woodland Hills

07-22

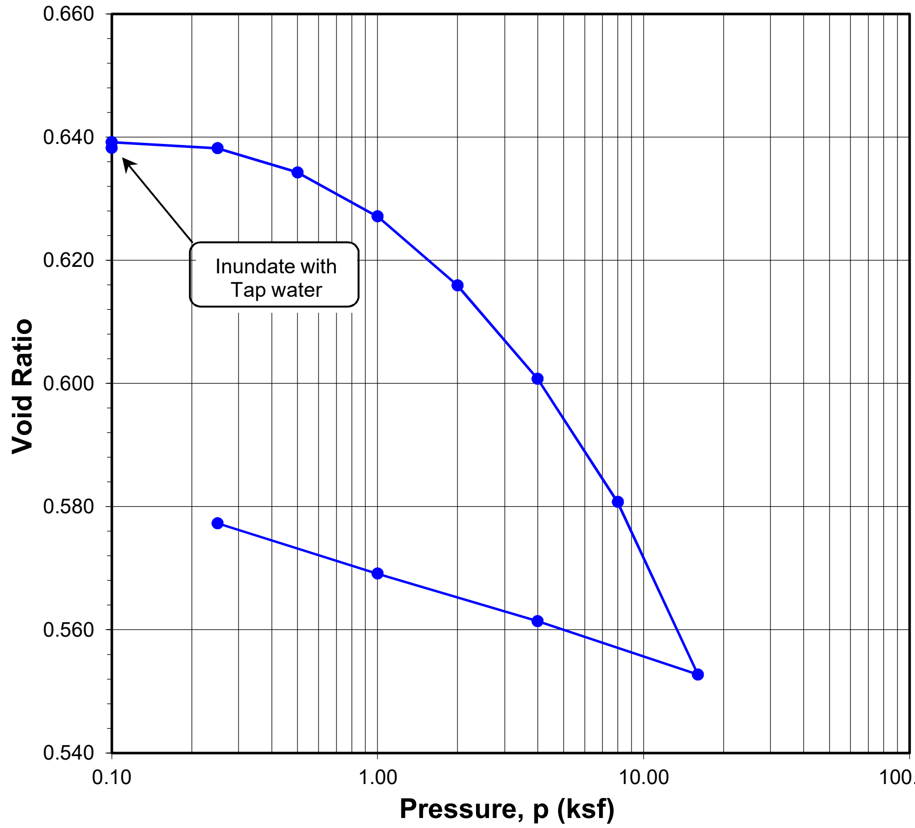
ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Leighton

Project Name:	JDA Woodland Hills	Tested By:	G. Bathala	Date:	07/11/22
Project No.:	13589.002	Checked By:	J. Ward	Date:	07/28/22
Boring No.:	LB-1	Depth (ft.):	35.0		
Sample No.:	ST-10	Sample Type:	Shelby		
Soil Identification:	Yellowish brown lean clay with sand (CL)s				

Sample Diameter (in.)	2.865
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	291.95
Weight of Ring (g)	83.74
Height after consol. (in.)	0.9626
Before Test	
Wt.Wet Sample+Cont. (g)	247.87
Wt.of Dry Sample+Cont. (g)	216.28
Weight of Container (g)	55.15
Initial Moisture Content (%)	19.6
Initial Dry Density (pcf)	102.9
Initial Saturation (%)	83
Initial Vertical Reading (in.)	0.3012
After Test	
Wt.of Wet Sample+Cont. (g)	329.51
Wt. of Dry Sample+Cont. (g)	291.08
Weight of Container (g)	39.02
Final Moisture Content (%)	22.83
Final Dry Density (pcf)	103.3
Final Saturation (%)	98
Final Vertical Reading (in.)	0.2616
Specific Gravity (assumed)	2.70
Water Density (pcf)	62.43

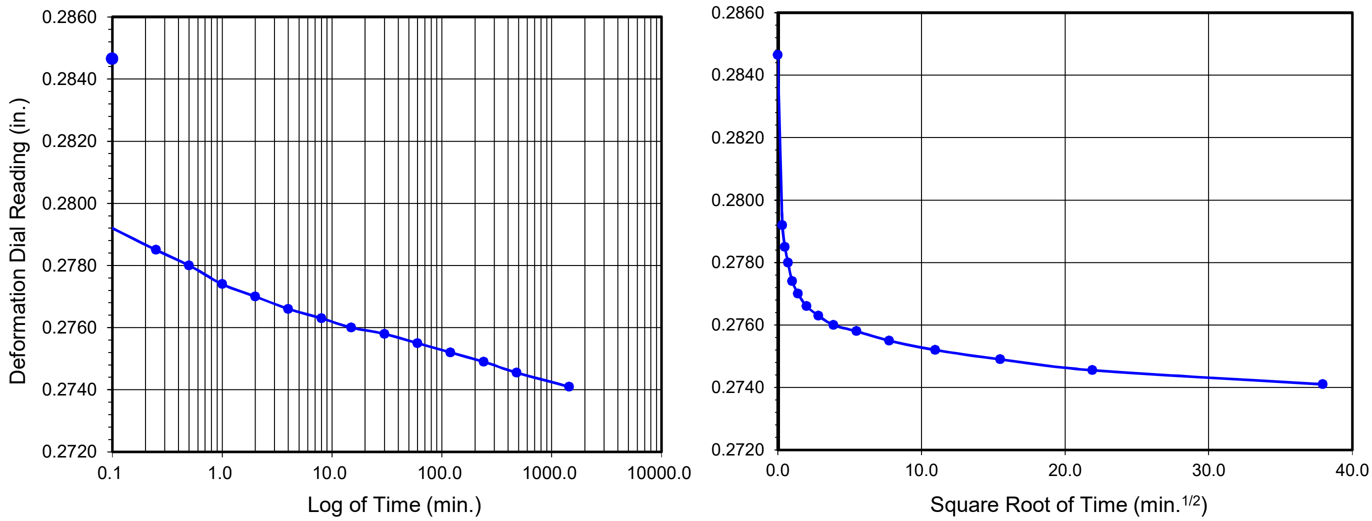


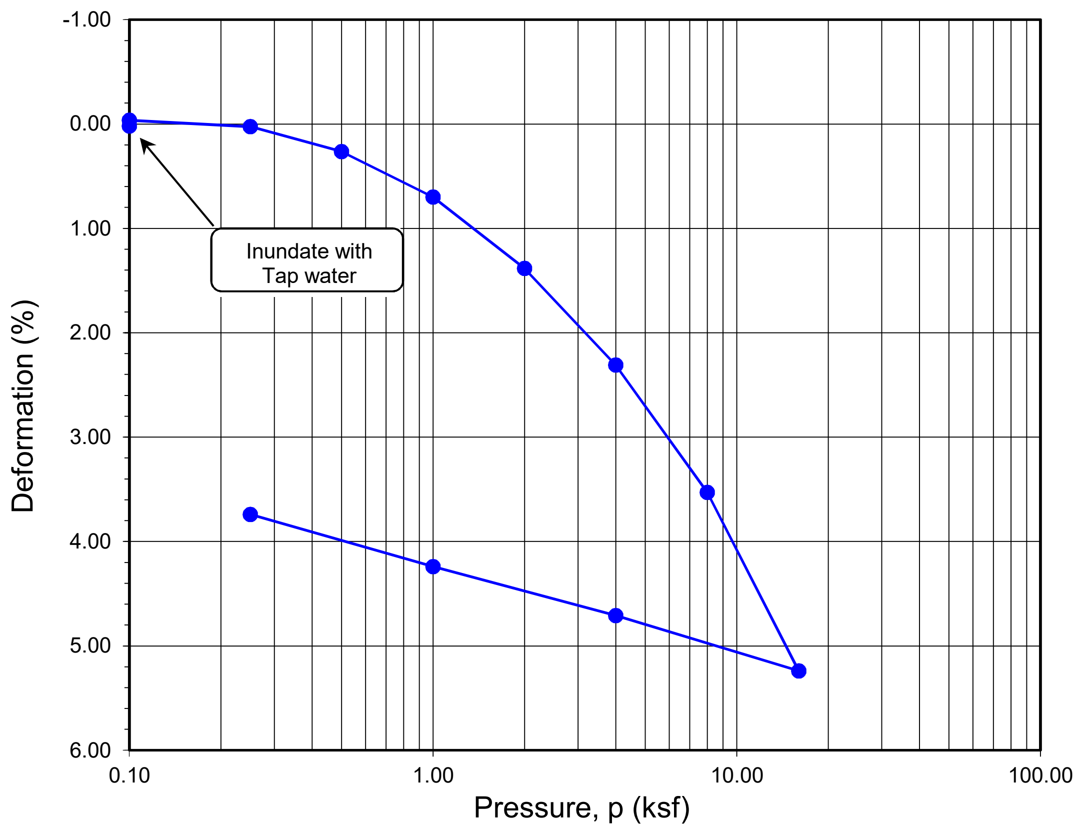
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.3010	0.9998	0.00	0.02	0.638	0.02
0.10	0.3016	1.0004	0.00	-0.04	0.639	-0.04
0.25	0.3005	0.9993	0.05	0.08	0.638	0.03
0.50	0.2976	0.9964	0.10	0.37	0.634	0.27
1.00	0.2924	0.9912	0.18	0.88	0.627	0.70
2.00	0.2847	0.9835	0.27	1.66	0.616	1.39
4.00	0.2741	0.9729	0.40	2.71	0.601	2.31
8.00	0.2603	0.9591	0.56	4.09	0.581	3.53
16.00	0.2411	0.9399	0.77	6.01	0.553	5.24
4.00	0.2487	0.9475	0.54	5.25	0.561	4.71
1.00	0.2552	0.9540	0.36	4.60	0.569	4.24
0.25	0.2616	0.9604	0.22	3.96	0.577	3.74

Time Readings @ 4.0 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
7/14/22	7:45:00	0.0	0.0	0.2847
7/14/22	7:45:06	0.1	0.3	0.2792
7/14/22	7:45:15	0.2	0.5	0.2785
7/14/22	7:45:30	0.5	0.7	0.2780
7/14/22	7:46:00	1.0	1.0	0.2774
7/14/22	7:47:00	2.0	1.4	0.2770
7/14/22	7:49:00	4.0	2.0	0.2766
7/14/22	7:53:00	8.0	2.8	0.2763
7/14/22	8:00:00	15.0	3.9	0.2760
7/14/22	8:15:00	30.0	5.5	0.2758
7/14/22	8:45:00	60.0	7.7	0.2755
7/14/22	9:45:00	120.0	11.0	0.2752
7/14/22	11:45:00	240.0	15.5	0.2749
7/14/22	15:45:00	480.0	21.9	0.2746
7/15/22	7:44:00	1439.0	37.9	0.2741

Time Readings @ 4.0 ksf





Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-1	**ST-10**	**35.0**	**19.6**	**22.8**	**102.9**	**103.3**	**0.639**	**0.577**	**83**	**98**

Soil Identification: Yellowish brown lean clay with sand (CL)s

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 13589.002

JDA Woodland Hills

07-22

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

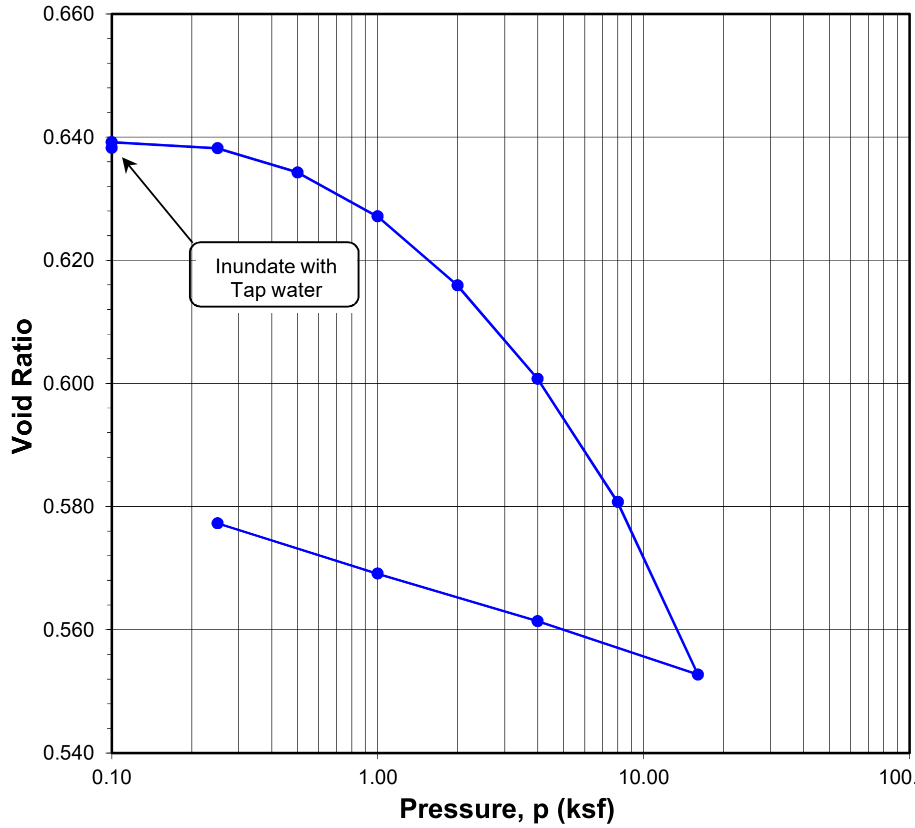
ASTM D 2435

Leighton

Project Name: JDA Woodland Hills
Project No.: 13589.002
Boring No.: LB-1
Sample No.: ST-10
Soil Identification: Yellowish brown lean clay with sand (CL)s

Tested By: G. Bathala Date: 07/11/22
Checked By: J. Ward Date: 07/28/22
Depth (ft.): 35.0
Sample Type: Shelby

Sample Diameter (in.)	2.865
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	291.95
Weight of Ring (g)	83.74
Height after consol. (in.)	0.9626
Before Test	
Wt.Wet Sample+Cont. (g)	247.87
Wt.of Dry Sample+Cont. (g)	216.28
Weight of Container (g)	55.15
Initial Moisture Content (%)	19.6
Initial Dry Density (pcf)	102.9
Initial Saturation (%)	83
Initial Vertical Reading (in.)	0.3012
After Test	
Wt.of Wet Sample+Cont. (g)	329.51
Wt. of Dry Sample+Cont. (g)	291.08
Weight of Container (g)	39.02
Final Moisture Content (%)	22.83
Final Dry Density (pcf)	103.3
Final Saturation (%)	98
Final Vertical Reading (in.)	0.2616
Specific Gravity (assumed)	2.70
Water Density (pcf)	62.43

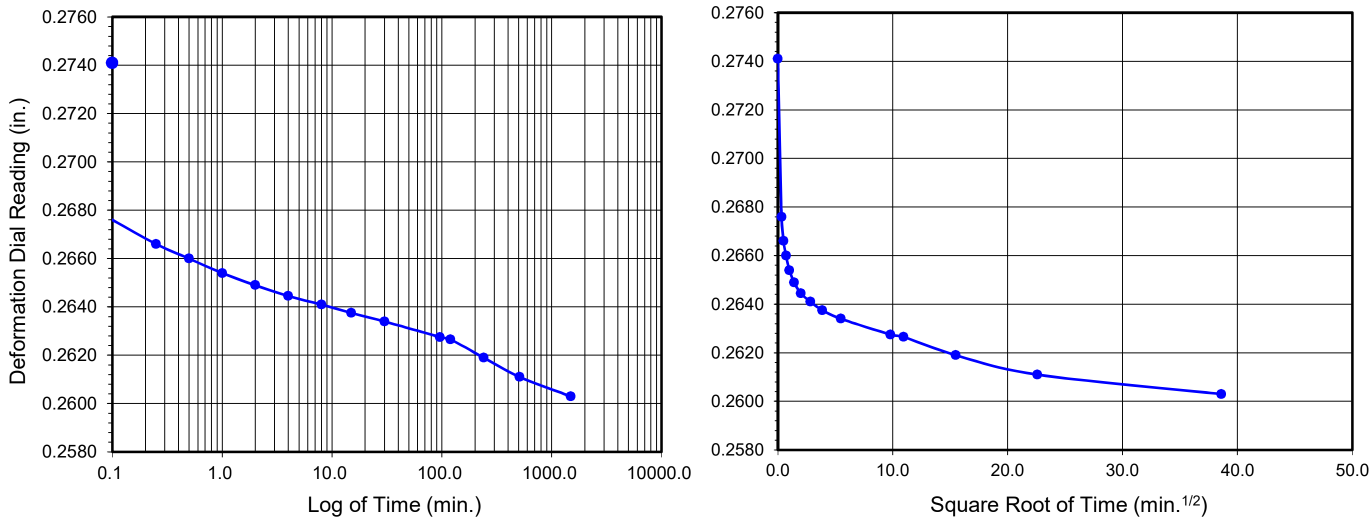


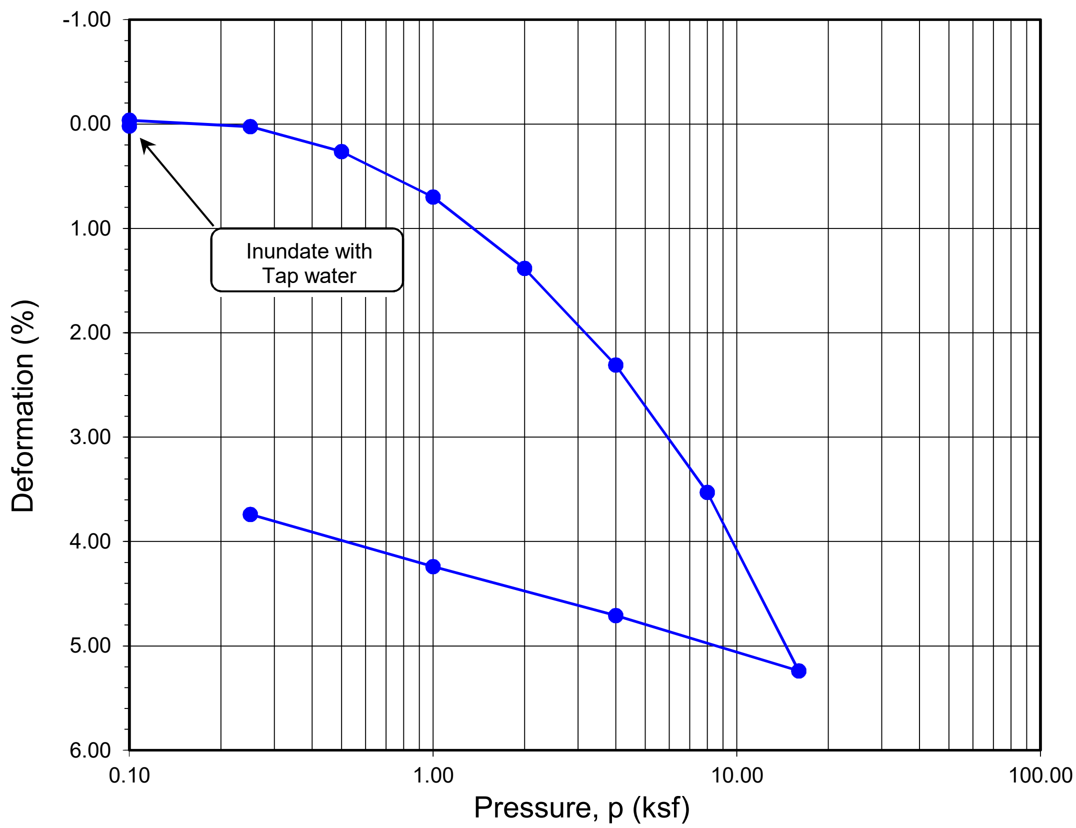
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.3010	0.9998	0.00	0.02	0.638	0.02
0.10	0.3016	1.0004	0.00	-0.04	0.639	-0.04
0.25	0.3005	0.9993	0.05	0.08	0.638	0.03
0.50	0.2976	0.9964	0.10	0.37	0.634	0.27
1.00	0.2924	0.9912	0.18	0.88	0.627	0.70
2.00	0.2847	0.9835	0.27	1.66	0.616	1.39
4.00	0.2741	0.9729	0.40	2.71	0.601	2.31
8.00	0.2603	0.9591	0.56	4.09	0.581	3.53
16.00	0.2411	0.9399	0.77	6.01	0.553	5.24
4.00	0.2487	0.9475	0.54	5.25	0.561	4.71
1.00	0.2552	0.9540	0.36	4.60	0.569	4.24
0.25	0.2616	0.9604	0.22	3.96	0.577	3.74

Time Readings @ 8.0 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
7/15/22	7:45:00	0.0	0.0	0.2741
7/15/22	7:45:06	0.1	0.3	0.2676
7/15/22	7:45:15	0.2	0.5	0.2666
7/15/22	7:45:30	0.5	0.7	0.2660
7/15/22	7:46:00	1.0	1.0	0.2654
7/15/22	7:47:00	2.0	1.4	0.2649
7/15/22	7:49:00	4.0	2.0	0.2645
7/15/22	7:53:00	8.0	2.8	0.2641
7/15/22	8:00:00	15.0	3.9	0.2638
7/15/22	8:15:00	30.0	5.5	0.2634
7/15/22	9:21:00	96.0	9.8	0.2628
7/15/22	9:45:00	120.0	11.0	0.2627
7/15/22	11:45:00	240.0	15.5	0.2619
7/15/22	16:15:00	510.0	22.6	0.2611
7/16/22	8:35:00	1490.0	38.6	0.2603

Time Readings @ 8.0 ksf





Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-1	**ST-10**	**35.0**	**19.6**	**22.8**	**102.9**	**103.3**	**0.639**	**0.577**	**83**	**98**

Soil Identification: Yellowish brown lean clay with sand (CL)s

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 13589.002

JDA Woodland Hills

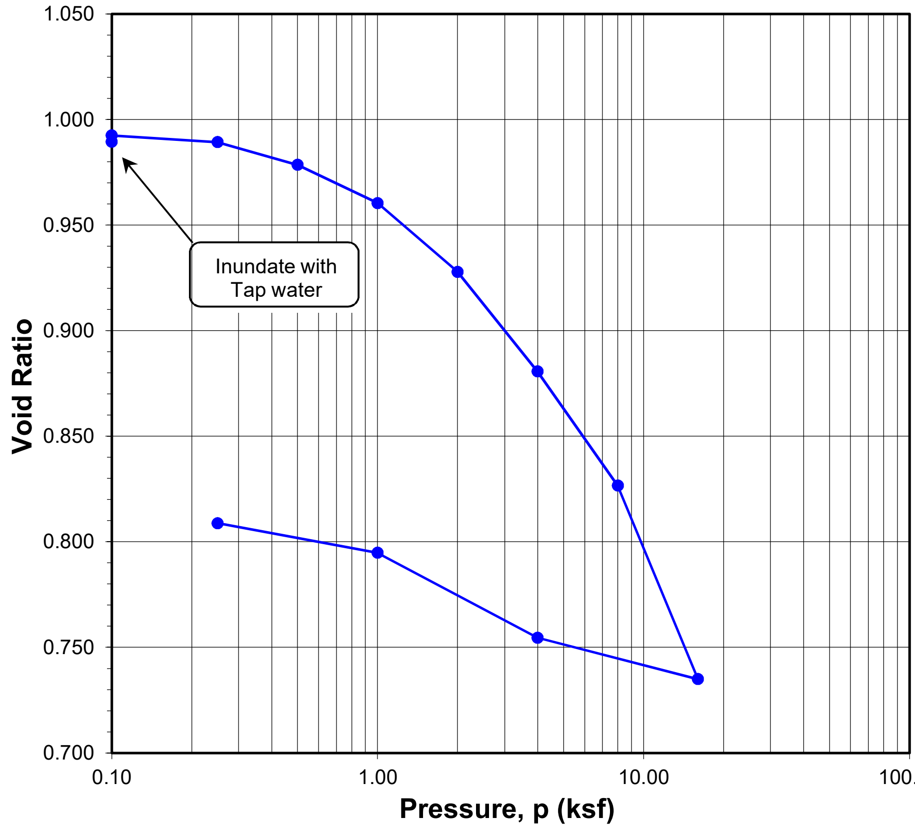
07-22

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name:	JDA Woodland Hills	Tested By: G. Bathala	Date:	07/11/22
Project No.:	13589.002	Checked By: J. Ward	Date:	07/28/22
Boring No.:	LB-1	Depth (ft.): 50.0		
Sample No.:	ST-13	Sample Type:	Shelby	
Soil Identification:	Olive brown fat clay (CH)			

Sample Diameter (in.)	2.865
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	275.00
Weight of Ring (g)	82.54
Height after consol. (in.)	0.9076
Before Test	
Wt.Wet Sample+Cont. (g)	284.49
Wt.of Dry Sample+Cont. (g)	228.07
Weight of Container (g)	59.52
Initial Moisture Content (%)	33.5
Initial Dry Density (pcf)	85.2
Initial Saturation (%)	92
Initial Vertical Reading (in.)	0.3180
After Test	
Wt.of Wet Sample+Cont. (g)	305.59
Wt. of Dry Sample+Cont. (g)	260.70
Weight of Container (g)	39.02
Final Moisture Content (%)	32.26
Final Dry Density (pcf)	90.6
Final Saturation (%)	100
Final Vertical Reading (in.)	0.2244
Specific Gravity (assumed)	2.72
Water Density (pcf)	62.43

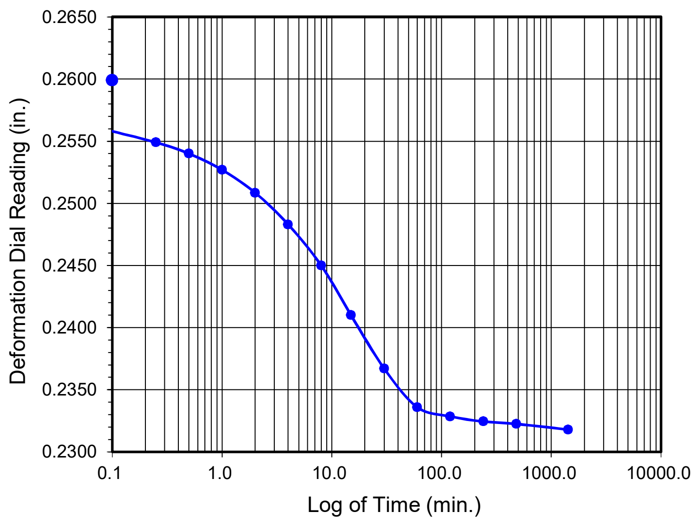


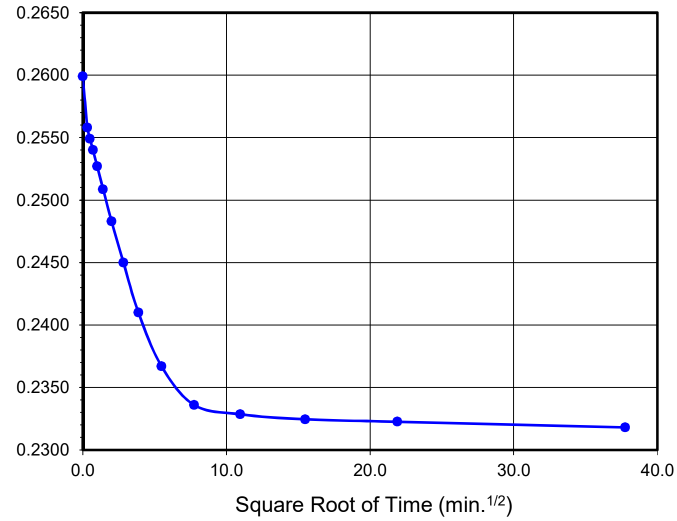
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.3163	0.9983	0.00	0.17	0.990	0.17
0.10	0.3178	0.9998	0.00	0.02	0.993	0.02
0.25	0.3160	0.9980	0.02	0.20	0.989	0.18
0.50	0.3104	0.9924	0.04	0.76	0.979	0.72
1.00	0.3010	0.9830	0.07	1.70	0.960	1.63
2.00	0.2843	0.9663	0.11	3.37	0.928	3.26
4.00	0.2599	0.9419	0.18	5.81	0.881	5.63
8.00	0.2318	0.9138	0.28	8.62	0.827	8.34
16.00	0.1845	0.8665	0.41	13.35	0.735	12.94
4.00	0.1954	0.8774	0.30	12.26	0.755	11.96
1.00	0.2167	0.8987	0.19	10.13	0.795	9.94
0.25	0.2244	0.9064	0.12	9.36	0.809	9.24

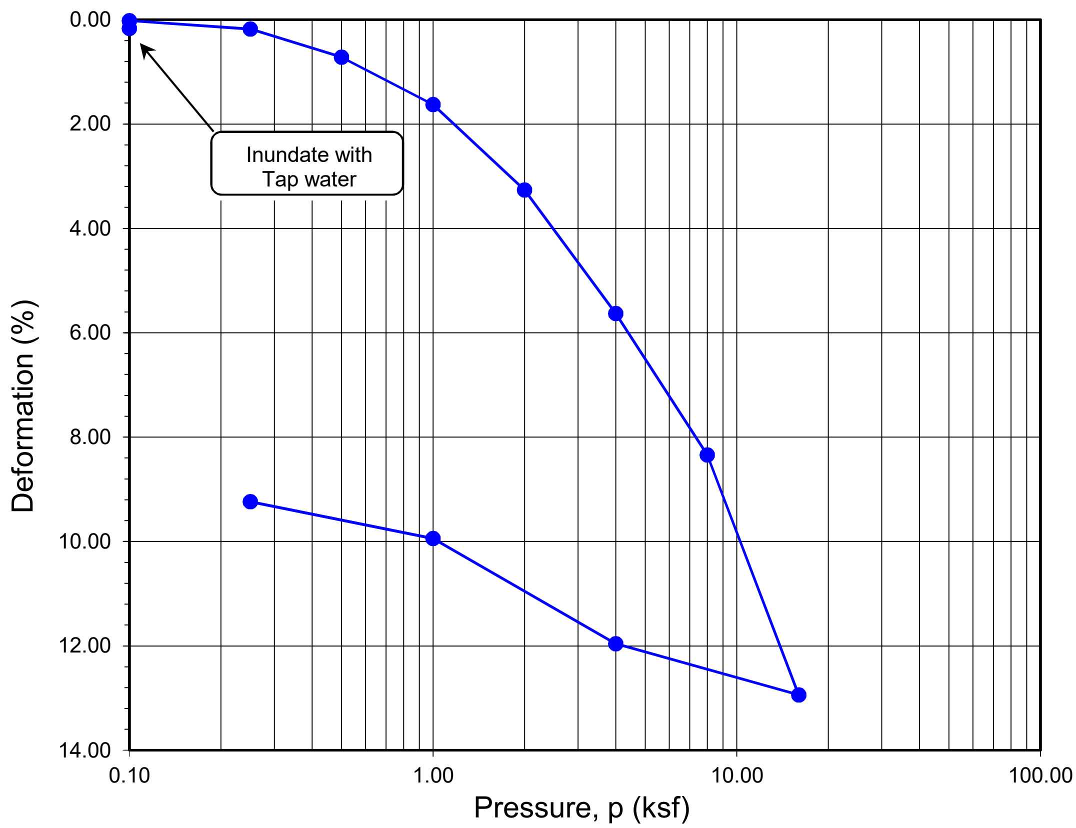
Time Readings @ 8.0 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
7/15/22	7:30:00	0.0	0.0	0.2599
7/15/22	7:30:06	0.1	0.3	0.2558
7/15/22	7:30:15	0.2	0.5	0.2549
7/15/22	7:30:30	0.5	0.7	0.2540
7/15/22	7:31:00	1.0	1.0	0.2527
7/15/22	7:32:00	2.0	1.4	0.2509
7/15/22	7:34:00	4.0	2.0	0.2483
7/15/22	7:38:00	8.0	2.8	0.2450
7/15/22	7:45:00	15.0	3.9	0.2410
7/15/22	8:00:00	30.0	5.5	0.2367
7/15/22	8:30:00	60.0	7.7	0.2336
7/15/22	9:30:00	120.0	11.0	0.2329
7/15/22	11:30:00	240.0	15.5	0.2325
7/15/22	15:30:00	480.0	21.9	0.2323
7/16/22	7:16:00	1426.0	37.8	0.2318

Time Readings @ 8.0 ksf







Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-1	**ST-13**	**50.0**	**33.5**	**32.3**	**85.2**	**90.6**	**0.993**	**0.809**	**92**	**100**

Soil Identification: Olive brown fat clay (CH)

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 13589.002

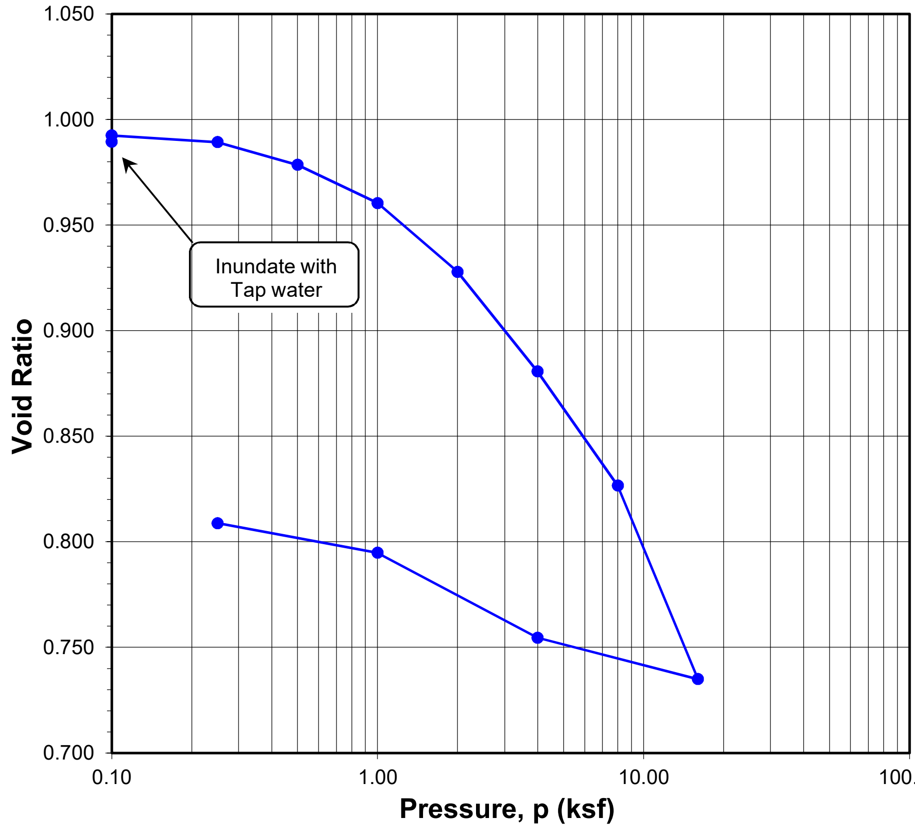
JDA Woodland Hills

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name:	JDA Woodland Hills	Tested By:	G. Bathala	Date:	07/11/22
Project No.:	13589.002	Checked By:	J. Ward	Date:	07/28/22
Boring No.:	LB-1	Depth (ft.):	50.0		
Sample No.:	ST-13	Sample Type:	Shelby		
Soil Identification:	Olive brown fat clay (CH)				

Sample Diameter (in.)	2.865
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	275.00
Weight of Ring (g)	82.54
Height after consol. (in.)	0.9076
Before Test	
Wt.Wet Sample+Cont. (g)	284.49
Wt.of Dry Sample+Cont. (g)	228.07
Weight of Container (g)	59.52
Initial Moisture Content (%)	33.5
Initial Dry Density (pcf)	85.2
Initial Saturation (%)	92
Initial Vertical Reading (in.)	0.3180
After Test	
Wt.of Wet Sample+Cont. (g)	305.59
Wt. of Dry Sample+Cont. (g)	260.70
Weight of Container (g)	39.02
Final Moisture Content (%)	32.26
Final Dry Density (pcf)	90.6
Final Saturation (%)	100
Final Vertical Reading (in.)	0.2244
Specific Gravity (assumed)	2.72
Water Density (pcf)	62.43

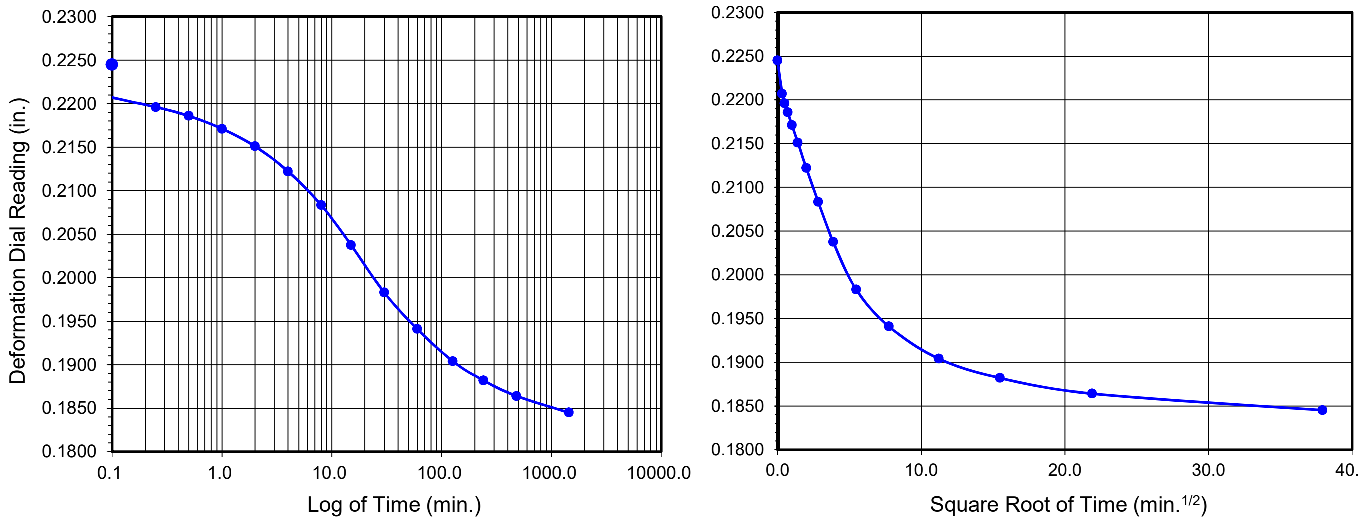


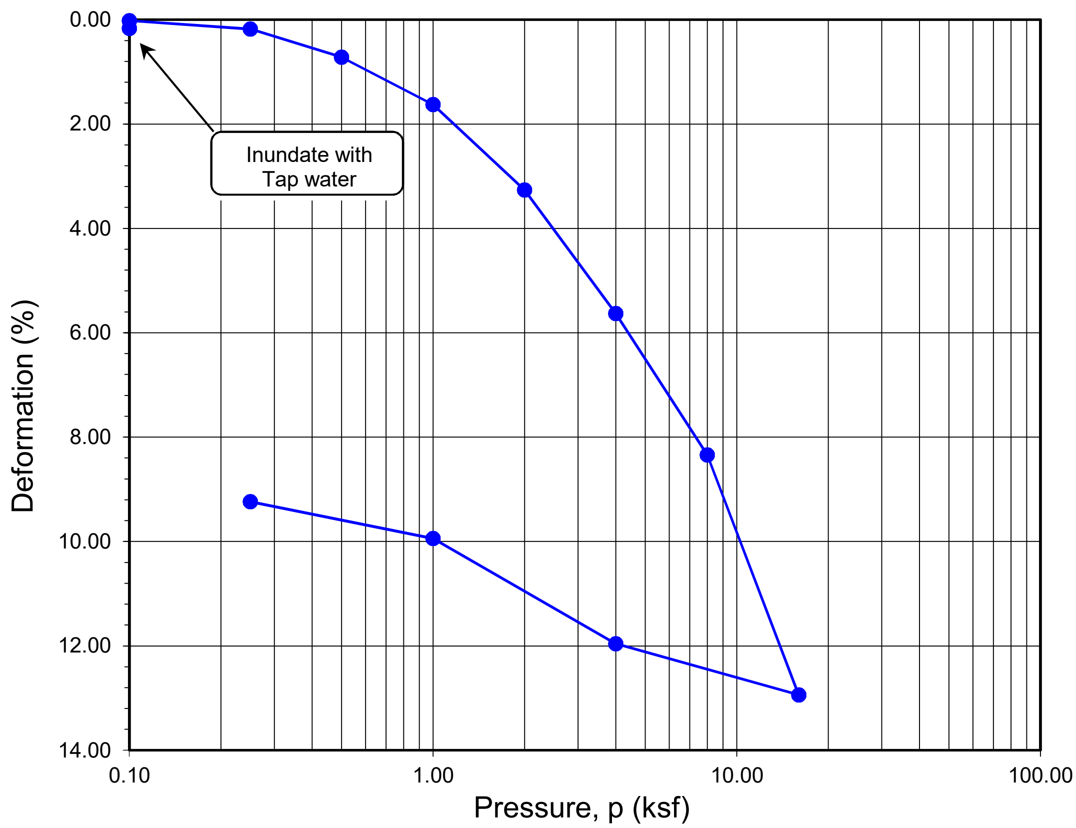
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.3163	0.9983	0.00	0.17	0.990	0.17
0.10	0.3178	0.9998	0.00	0.02	0.993	0.02
0.25	0.3160	0.9980	0.02	0.20	0.989	0.18
0.50	0.3104	0.9924	0.04	0.76	0.979	0.72
1.00	0.3010	0.9830	0.07	1.70	0.960	1.63
2.00	0.2843	0.9663	0.11	3.37	0.928	3.26
4.00	0.2599	0.9419	0.18	5.81	0.881	5.63
8.00	0.2318	0.9138	0.28	8.62	0.827	8.34
16.00	0.1845	0.8665	0.41	13.35	0.735	12.94
4.00	0.1954	0.8774	0.30	12.26	0.755	11.96
1.00	0.2167	0.8987	0.19	10.13	0.795	9.94
0.25	0.2244	0.9064	0.12	9.36	0.809	9.24

Time Readings @ 16.0 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
7/18/22	7:35:00	0.0	0.0	0.2245
7/18/22	7:35:06	0.1	0.3	0.2207
7/18/22	7:35:15	0.2	0.5	0.2196
7/18/22	7:35:30	0.5	0.7	0.2186
7/18/22	7:36:00	1.0	1.0	0.2171
7/18/22	7:37:00	2.0	1.4	0.2151
7/18/22	7:39:00	4.0	2.0	0.2122
7/18/22	7:43:00	8.0	2.8	0.2084
7/18/22	7:50:00	15.0	3.9	0.2038
7/18/22	8:05:00	30.0	5.5	0.1983
7/18/22	8:35:00	60.0	7.7	0.1941
7/18/22	9:41:00	126.0	11.2	0.1904
7/18/22	11:35:00	240.0	15.5	0.1882
7/18/22	15:35:00	480.0	21.9	0.1864
7/19/22	7:35:00	1440.0	37.9	0.1845

Time Readings @ 16.0 ksf





Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-1	**ST-13**	**50.0**	**33.5**	**32.3**	**85.2**	**90.6**	**0.993**	**0.809**	**92**	**100**

Soil Identification: Olive brown fat clay (CH)

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 13589.002

JDA Woodland Hills

07-22

TESTS for SULFATE CONTENT CHLORIDE CONTENT and pH of SOILS

Project Name: JDA Woodland Hills — Tested By : G. Berdy — Date: 07/21/22

Project No. : 13589.002 — Checked By: J. Ward — Date: 08/01/22

Boring No.	LB-2			
Sample No.	B-1			
Sample Depth (ft)	1-5			
Soil Identification:	Gray CL			
Wet Weight of Soil + Container (g)	0.00			
Dry Weight of Soil + Container (g)	0.00			
Weight of Container (g)	1.00			
Moisture Content (%)	0.00			
Weight of Soaked Soil (g)	100.53			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	15			
Crucible No.	21			
Furnace Temperature (°C)	860			
Time In / Time Out	9:00/9:45			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	22.1789			
Wt. of Crucible (g)	22.1759			
Wt. of Residue (g) (A)	0.0030			
PPM of Sulfate (A) x 41150	123.45			
PPM of Sulfate, Dry Weight Basis	**123**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	15			
ml of AgNO3 Soln. Used in Titration (C)	0.7			
PPM of Chloride (C -0.2) * 100 * 30 / B	100			
PPM of Chloride, Dry Wt. Basis	**100**			

pH TEST, DOT California Test 643

pH Value	**7.80**			
Temperature °C	20.8			

SOIL RESISTIVITY TEST

DOT CA TEST 643

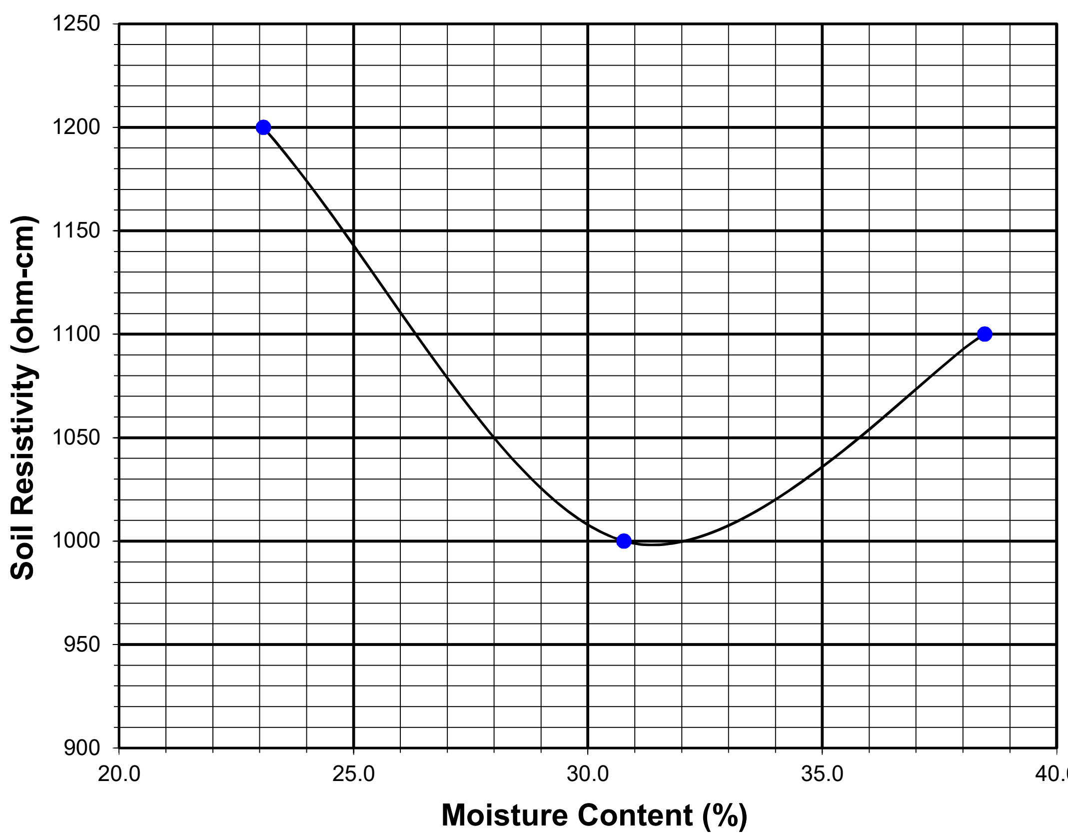
Project Name:	JDA Woodland Hills			
Project No. :	13589.002	Tested By :	J. Domingo	Date: 07/26/22
Boring No.:	LB-2	Checked By:	J. Ward	Date: 08/01/22
Sample No. :	B-1	Depth (ft.) :	1-5	
Soil Identification:*	Gray CL			

*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	30	23.08	1200	1200
2	40	30.77	1000	1000
3	50	38.46	1100	1100
4				
5				

Moisture Content (%) (MCi)	0.00
Wet Wt. of Soil + Cont. (g)	0.00
Dry Wt. of Soil + Cont. (g)	0.00
Wt. of Container (g)	1.00
Container No.	
Initial Soil Wt. (g) (Wt)	130.00
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
1000	**31.4**	**123**	**100**	**7.80**	**20.8**



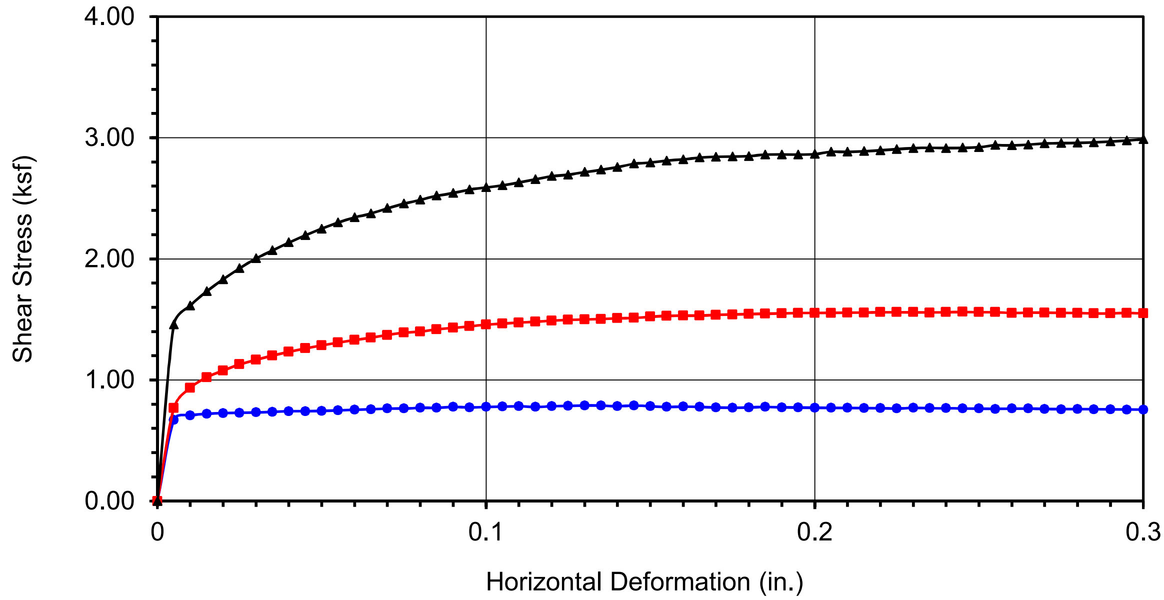
DIRECT SHEAR TEST

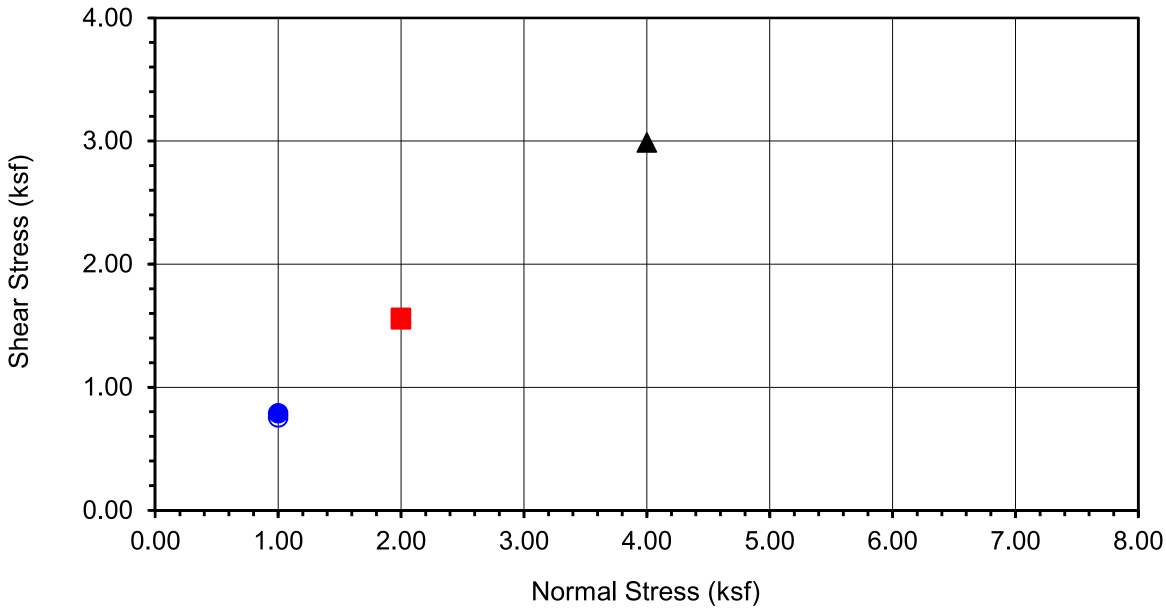
Consolidated Drained - ASTM D 3080

Project Name:	JDA Woodland Hills	Tested By:	GB/JD	Date:	07/25/22
Project No.:	13589.002	Checked By:	J. Ward	Date:	08/01/22
Boring No.:	LB-3	Sample Type:	90% Remold		
Sample No.:	B-1	Depth (ft.):	1-5		

Soil Identification: Dark olive brown lean clay (CL)

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	180.49	180.77	180.89
Weight of Ring(gm):	45.40	45.48	45.43
Before Shearing			
Weight of Wet Sample+Cont.(gm):	176.89	176.89	176.89
Weight of Dry Sample+Cont.(gm):	161.24	161.24	161.24
Weight of Container(gm):	56.65	56.65	56.65
Vertical Rdg.(in): Initial	0.2726	0.2717	0.0000
Vertical Rdg.(in): Final	0.2736	0.2875	-0.0366
After Shearing			
Weight of Wet Sample+Cont.(gm):	203.44	180.94	181.05
Weight of Dry Sample+Cont.(gm):	174.70	153.59	155.91
Weight of Container(gm):	59.16	38.29	40.03
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-3**
Sample No.	**B-1**
Depth (ft)	**1-5**

Sample Type:

90% Remold

Soil Identification:
Dark olive brown lean clay (CL)

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 0.789	■ 1.562	▲ 2.987
Shear Stress @ End of Test (ksf)	○ 0.755	□ 1.550	△ 2.987
Deformation Rate (in./min.)	0.0017	0.0017	0.0017
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	14.96	14.96	14.96
Dry Density (pcf)	97.7	97.9	98.0
Saturation (%)	55.7	55.9	56.1
Soil Height Before Shearing (in.)	0.9990	0.9842	0.9634
Final Moisture Content (%)	24.9	23.7	21.7

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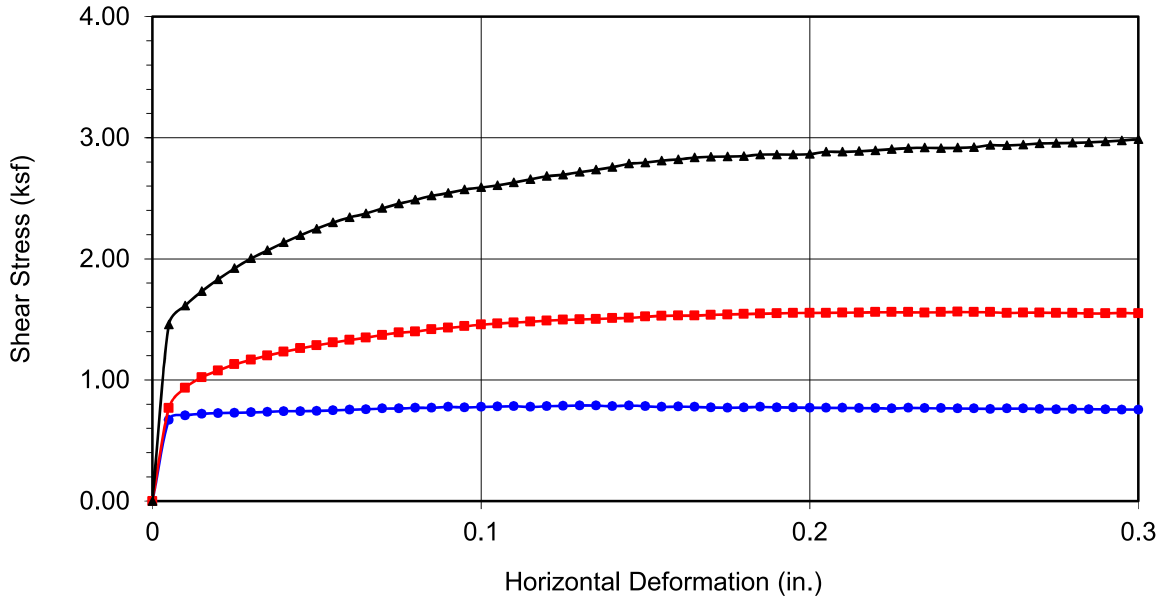
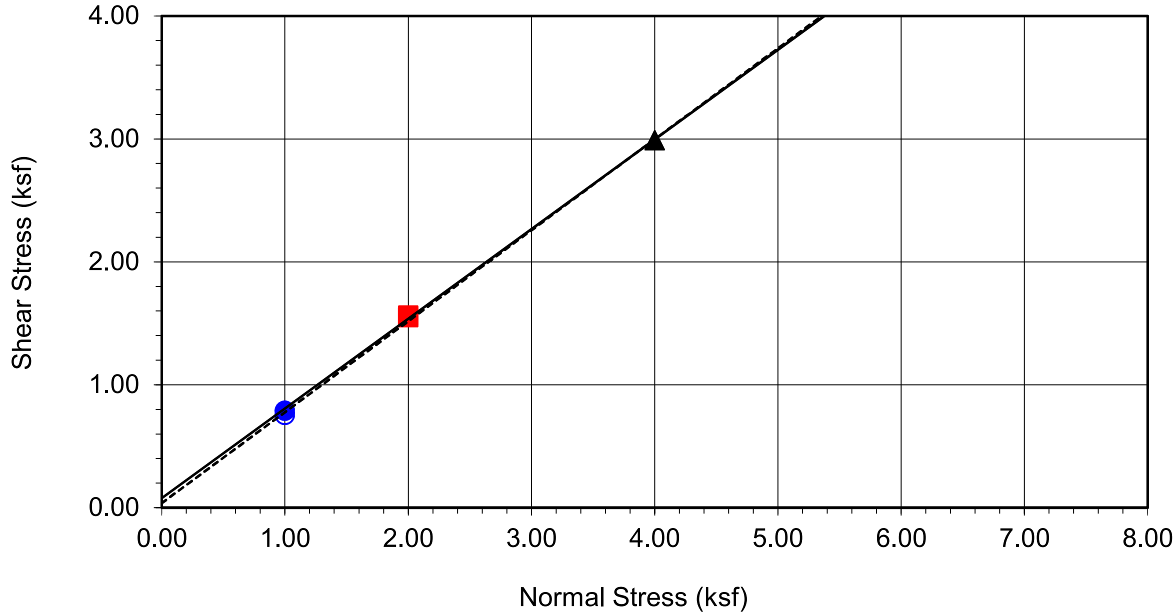
DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 13589.002

JDA Woodland Hills

07-22





Boring No.	**LB-3**
Sample No.	**B-1**
Depth (ft)	**1-5**
Sample Type:	90% Remold

Soil Identification:
Dark olive brown lean clay (CL)

Strength Parameters

	C (psf)	ϕ (°)
Peak	76	36
Ultimate	37	37

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 0.789	■ 1.562	▲ 2.987
Shear Stress @ End of Test (ksf)	○ 0.755	□ 1.550	△ 2.987
Deformation Rate (in./min.)	0.0017	0.0017	0.0017
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	14.96	14.96	14.96
Dry Density (pcf)	97.7	97.9	98.0
Saturation (%)	55.7	55.9	56.1
Soil Height Before Shearing (in.)	0.9990	0.9842	0.9634
Final Moisture Content (%)	24.9	23.7	21.7

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DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 13589.002

JDA Woodland Hills

07-22

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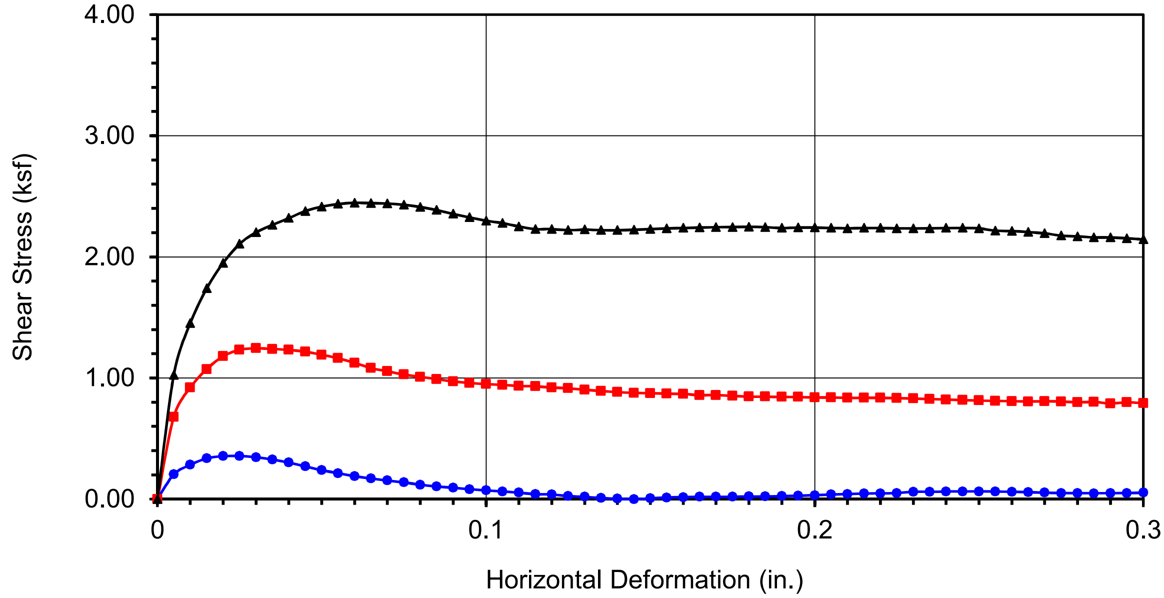
DIRECT SHEAR TEST

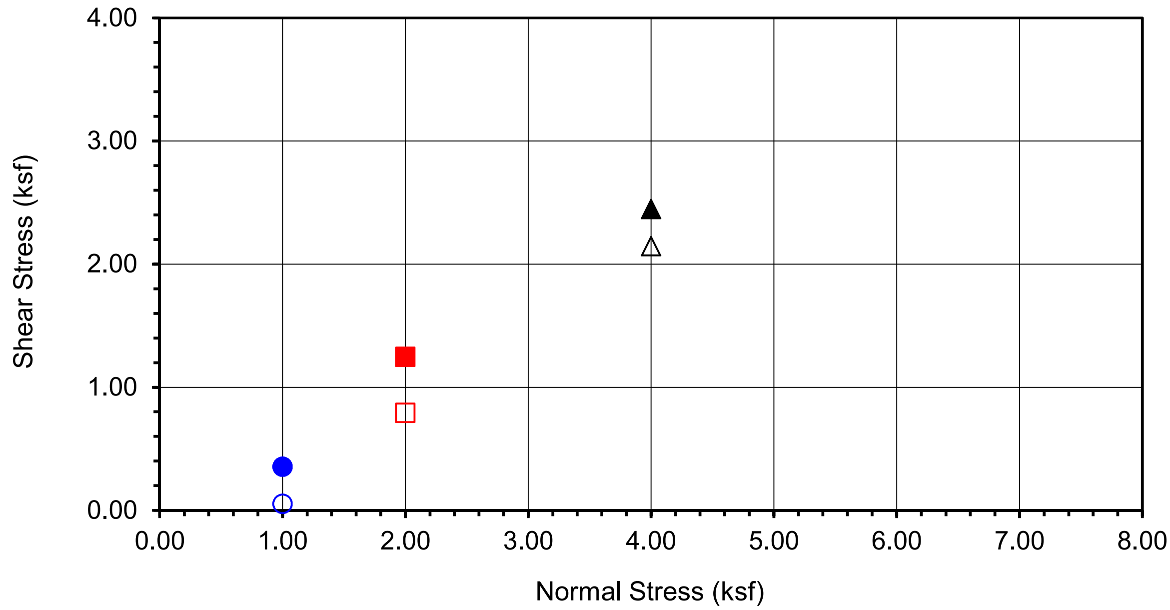
Consolidated Drained - ASTM D 3080

Project Name:	JDA Woodland Hills	Tested By:	GB/JD	Date:	07/26/22
Project No.:	13589.002	Checked By:	J. Ward	Date:	08/01/22
Boring No.:	LB-3	Sample Type:	Ring		
Sample No.:	R-3	Depth (ft.):	10.0		

Soil Identification: Olive lean clay (CL)

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	179.93	186.85	188.47
Weight of Ring(gm):	45.30	45.71	45.87
Before Shearing			
Weight of Wet Sample+Cont.(gm):	187.98	187.98	187.98
Weight of Dry Sample+Cont.(gm):	147.34	147.34	147.34
Weight of Container(gm):	67.11	67.11	67.11
Vertical Rdg.(in): Initial	0.0000	0.2580	0.2587
Vertical Rdg.(in): Final	0.0030	0.2640	0.2705
After Shearing			
Weight of Wet Sample+Cont.(gm):	196.03	213.48	206.97
Weight of Dry Sample+Cont.(gm):	159.97	180.68	175.15
Weight of Container(gm):	59.14	71.40	64.16
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-3**
Sample No.	**R-3**
Depth (ft)	**10**

Sample Type:

Ring

Soil Identification:

Olive lean clay (CL)

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 0.355	■ 1.245	▲ 2.446
Shear Stress @ End of Test (ksf)	○ 0.053	□ 0.792	△ 2.144
Deformation Rate (in./min.)	0.0017	0.0017	0.0017
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	50.65	50.65	50.65
Dry Density (pcf)	74.3	77.9	78.7
Saturation (%)	107.9	117.6	119.8
Soil Height Before Shearing (in.)	1.0030	0.9940	0.9882
Final Moisture Content (%)	35.8	30.0	28.7

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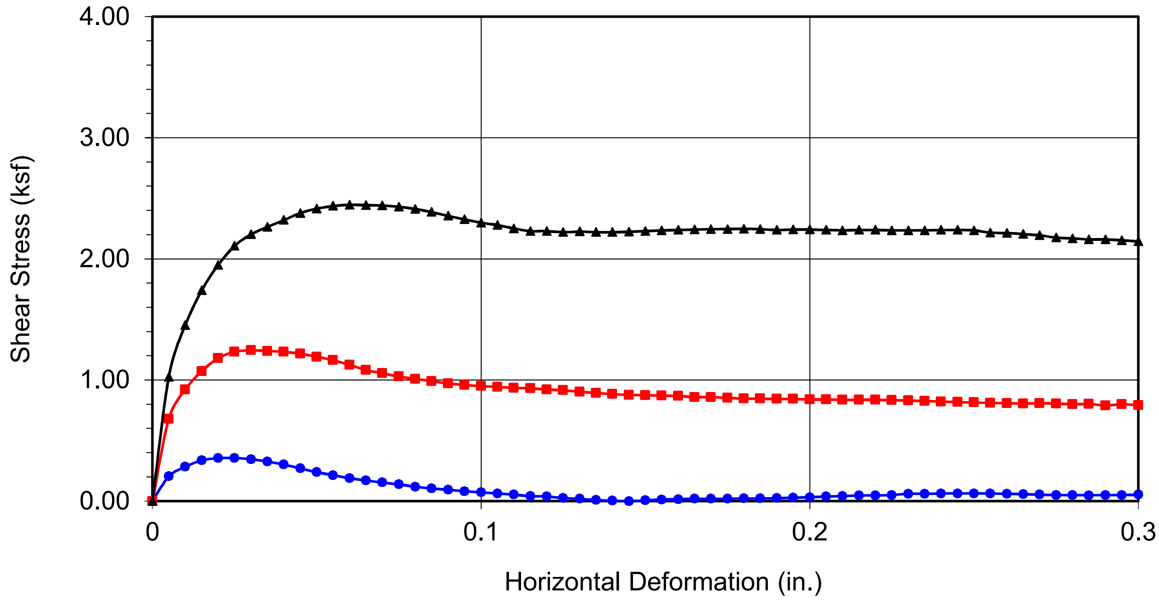
DIRECT SHEAR TEST RESULTS

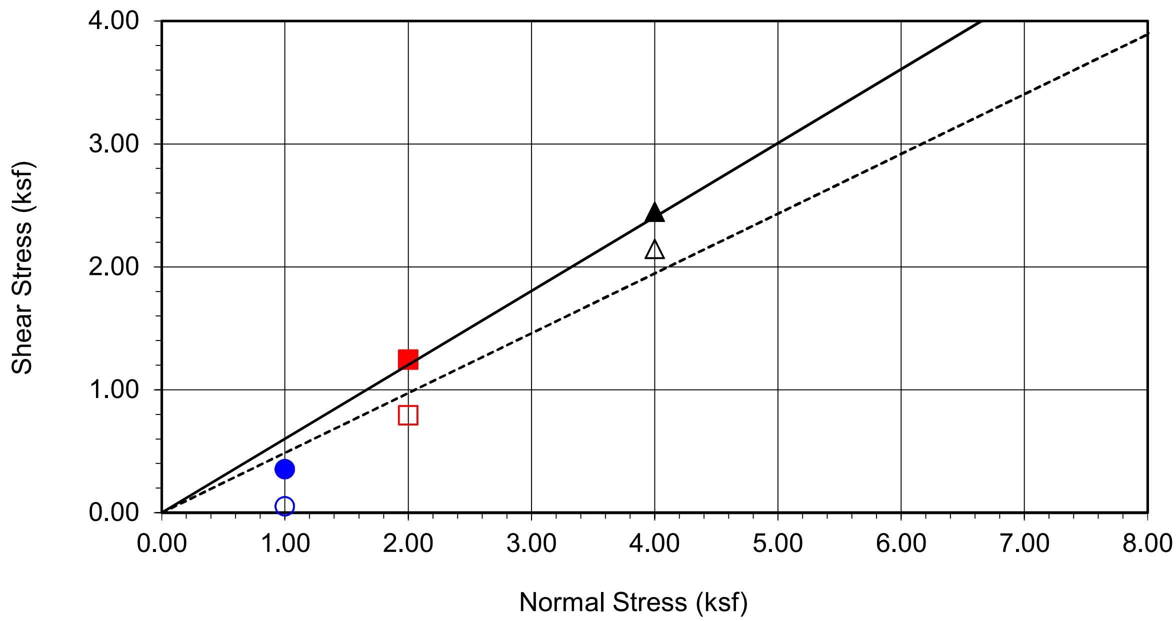
Consolidated Drained - ASTM D 3080

Project No.: 13589.002

JDA Woodland Hills

07-22





Boring No.	**LB-3**
Sample No.	**R-3**
Depth (ft)	**10**
Sample Type:	Ring

Soil Identification:
Olive lean clay (CL)

Strength Parameters

	C (psf)	ϕ (°)
Peak	0	31
Ultimate	0	26

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 0.355	■ 1.245	▲ 2.446
Shear Stress @ End of Test (ksf)	○ 0.053	□ 0.792	△ 2.144
Deformation Rate (in./min.)	0.0017	0.0017	0.0017
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	50.65	50.65	50.65
Dry Density (pcf)	74.3	77.9	78.7
Saturation (%)	107.9	117.6	119.8
Soil Height Before Shearing (in.)	1.0030	0.9940	0.9882
Final Moisture Content (%)	35.8	30.0	28.7

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DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 13589.002

JDA Woodland Hills

07-22

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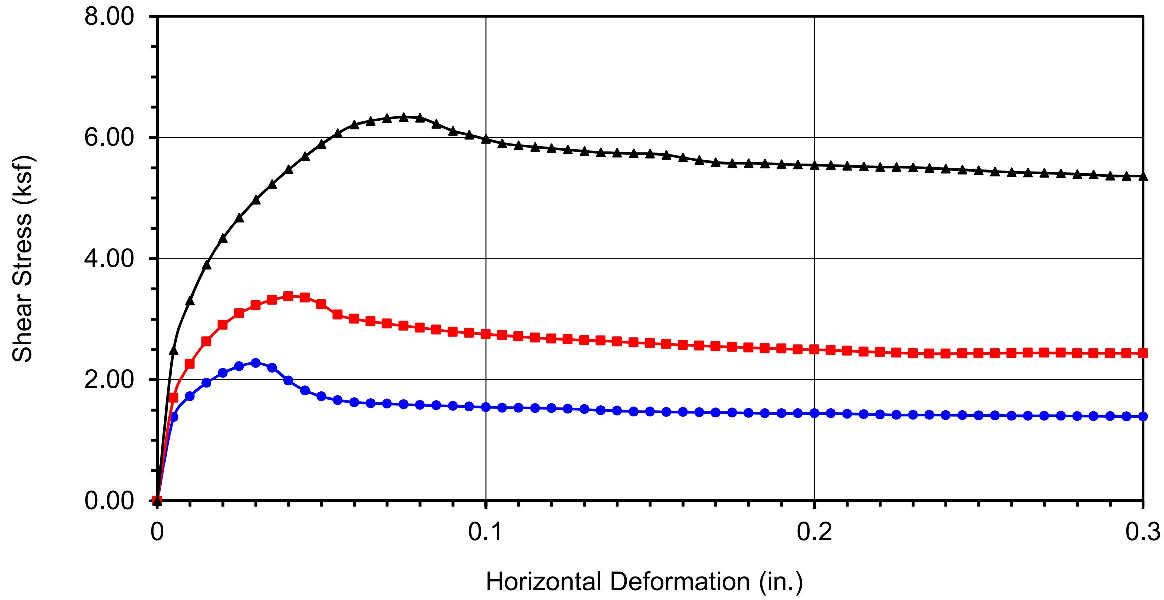
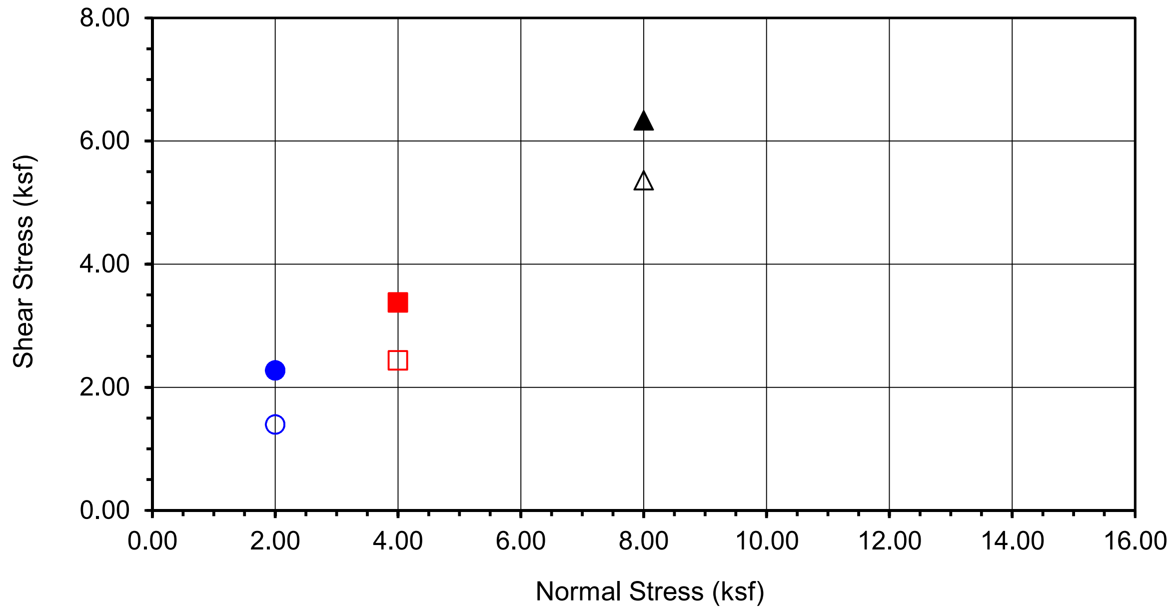
DIRECT SHEAR TEST

Consolidated Drained - ASTM D 3080

Project Name:	JDA Woodland Hills	Tested By:	GB/JD	Date:	07/26/22
Project No.:	13589.002	Checked By:	J. Ward	Date:	08/01/22
Boring No.:	LB-3	Sample Type:	Ring		
Sample No.:	R-6	Depth (ft.):	20.0		
Soil Identification:	Light olive brown lean clay (CL)				

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	188.06	191.87	192.29
Weight of Ring(gm):	44.64	45.19	45.58
Before Shearing			
Weight of Wet Sample+Cont.(gm):	212.10	212.10	212.10
Weight of Dry Sample+Cont.(gm):	183.34	183.34	183.34
Weight of Container(gm):	77.76	77.76	77.76
Vertical Rdg.(in): Initial	0.2595	0.2647	0.0000
Vertical Rdg.(in): Final	0.3668	0.3809	-0.0250
After Shearing			
Weight of Wet Sample+Cont.(gm):	208.14	204.49	186.37
Weight of Dry Sample+Cont.(gm):	175.00	171.01	153.77
Weight of Container(gm):	64.59	59.05	40.03
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-3**
Sample No.	**R-6**
Depth (ft)	**20**
Sample Type: Ring	
Soil Identification: Light olive brown lean clay (CL)	

Normal Stress (kip/ft²)	2.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 2.273	■ 3.373	▲ 6.335
Shear Stress @ End of Test (ksf)	○ 1.393	□ 2.433	△ 5.363
Deformation Rate (in./min.)	0.0017	0.0017	0.0017
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	27.24	27.24	27.24
Dry Density (pcf)	93.7	95.9	95.9
Saturation (%)	92.1	97.0	97.1
Soil Height Before Shearing (in.)	0.8927	0.8838	0.9750
Final Moisture Content (%)	30.0	29.9	28.7

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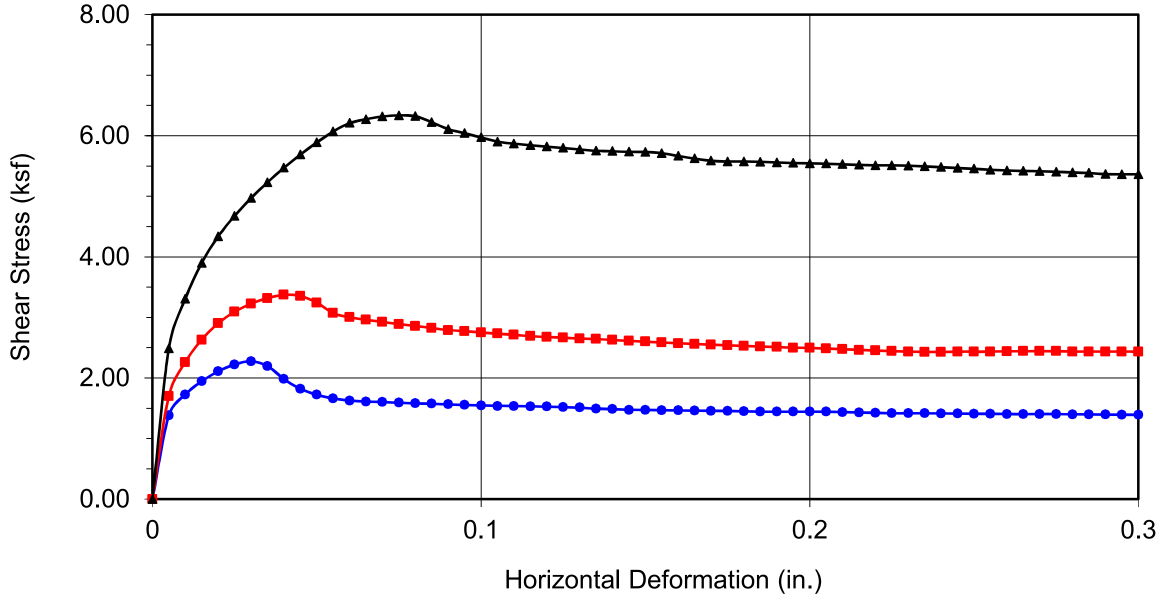
DIRECT SHEAR TEST RESULTS

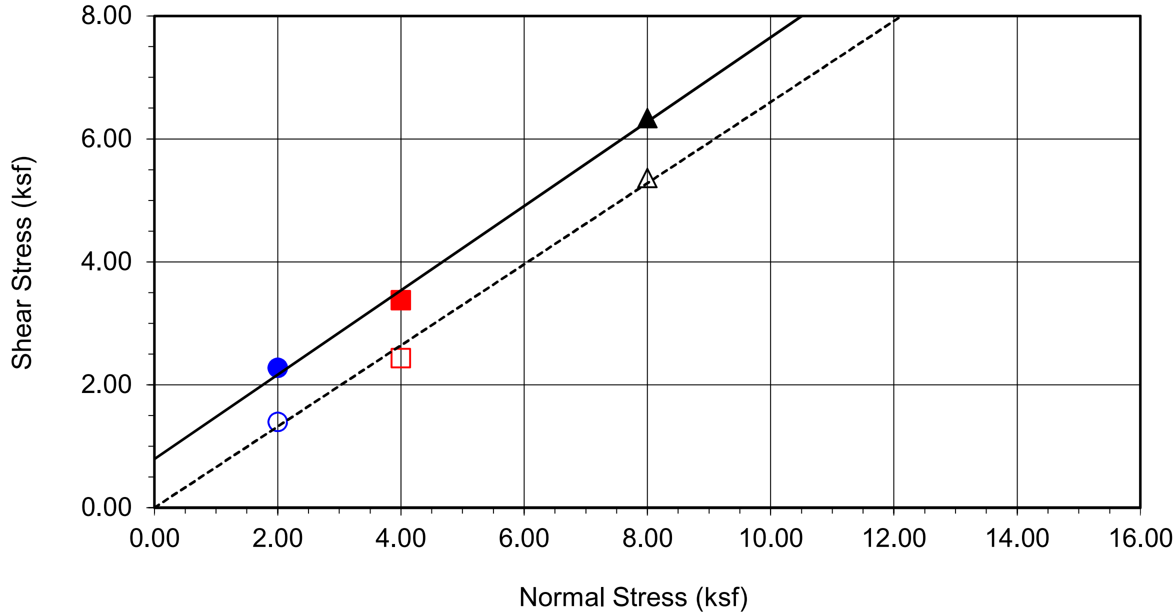
Consolidated Drained - ASTM D 3080

Project No.: 13589.002

JDA Woodland Hills

07-22





Boring No.	**LB-3**
Sample No.	**R-6**
Depth (ft)	**20**
Sample Type:	Ring

Soil Identification:
Light olive brown lean clay (CL)

Strength Parameters

	C (psf)	ϕ (°)
Peak	792	34
Ultimate	0	33

Normal Stress (kip/ft²)	2.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 2.273	■ 3.373	▲ 6.335
Shear Stress @ End of Test (ksf)	○ 1.393	□ 2.433	△ 5.363
Deformation Rate (in./min.)	0.0017	0.0017	0.0017
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	27.24	27.24	27.24
Dry Density (pcf)	93.7	95.9	95.9
Saturation (%)	92.1	97.0	97.1
Soil Height Before Shearing (in.)	0.8927	0.8838	0.9750
Final Moisture Content (%)	30.0	29.9	28.7

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DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 13589.002

JDA Woodland Hills

07-22

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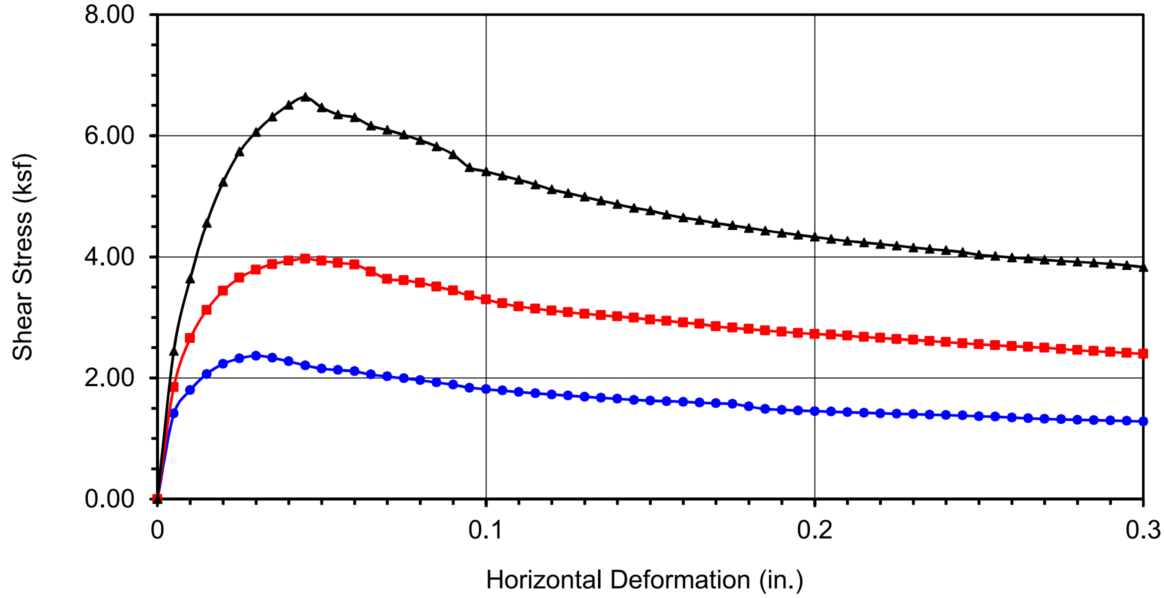
DIRECT SHEAR TEST

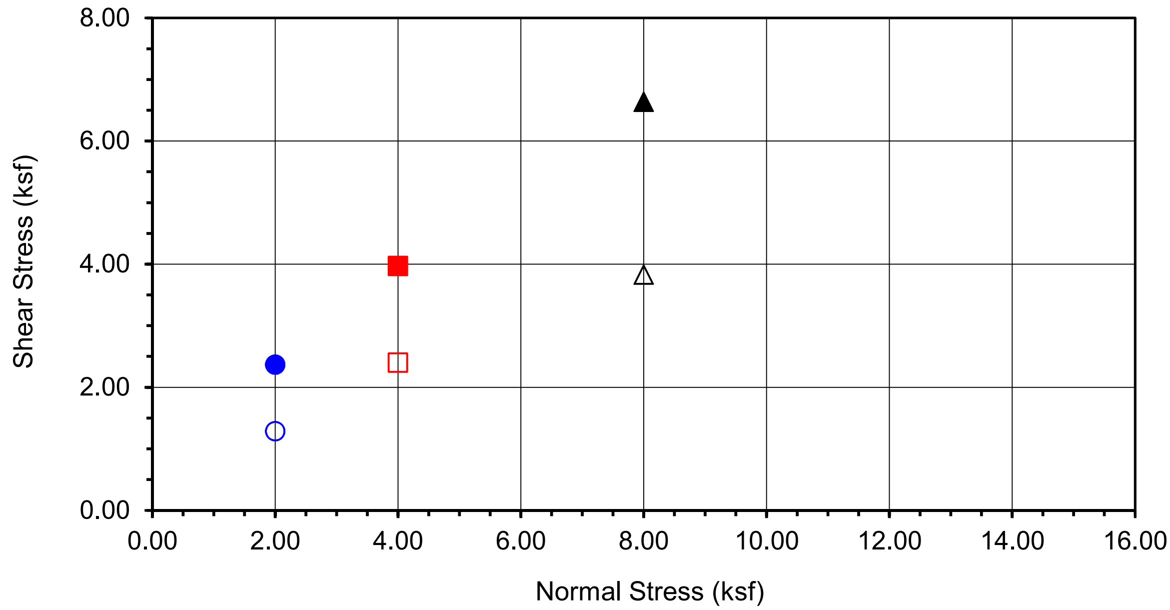
Consolidated Drained - ASTM D 3080

Project Name:	JDA Woodland Hills	Tested By:	GB/JD	Date:	07/25/22
Project No.:	13589.002	Checked By:	J. Ward	Date:	08/01/22
Boring No.:	LB-3	Sample Type:	Ring		
Sample No.:	R-8	Depth (ft.):	30.0		

Soil Identification: Dark olive brown lean clay (CL), claystone noted

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	171.93	173.56	174.81
Weight of Ring(gm):	45.83	45.47	45.37
Before Shearing			
Weight of Wet Sample+Cont.(gm):	177.95	177.95	177.95
Weight of Dry Sample+Cont.(gm):	140.28	140.28	140.28
Weight of Container(gm):	58.01	58.01	58.01
Vertical Rdg.(in): Initial	0.2521	0.2300	0.0000
Vertical Rdg.(in): Final	0.2567	0.2377	-0.0127
After Shearing			
Weight of Wet Sample+Cont.(gm):	167.61	184.68	191.00
Weight of Dry Sample+Cont.(gm):	124.29	141.77	149.49
Weight of Container(gm):	39.49	55.79	60.88
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-3**
Sample No.	**R-8**
Depth (ft)	**30**
Sample Type: Ring	
Soil Identification: Dark olive brown lean clay (CL), claystone noted	

Normal Stress (kip/ft²)	2.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 2.367	■ 3.967	▲ 6.637
Shear Stress @ End of Test (ksf)	○ 1.283	□ 2.399	△ 3.829
Deformation Rate (in./min.)	0.0017	0.0017	0.0017
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	45.79	45.79	45.79
Dry Density (pcf)	71.9	73.1	73.8
Saturation (%)	92.0	94.6	96.4
Soil Height Before Shearing (in.)	0.9954	0.9923	0.9873
Final Moisture Content (%)	51.1	49.9	46.8

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DIRECT SHEAR TEST RESULTS

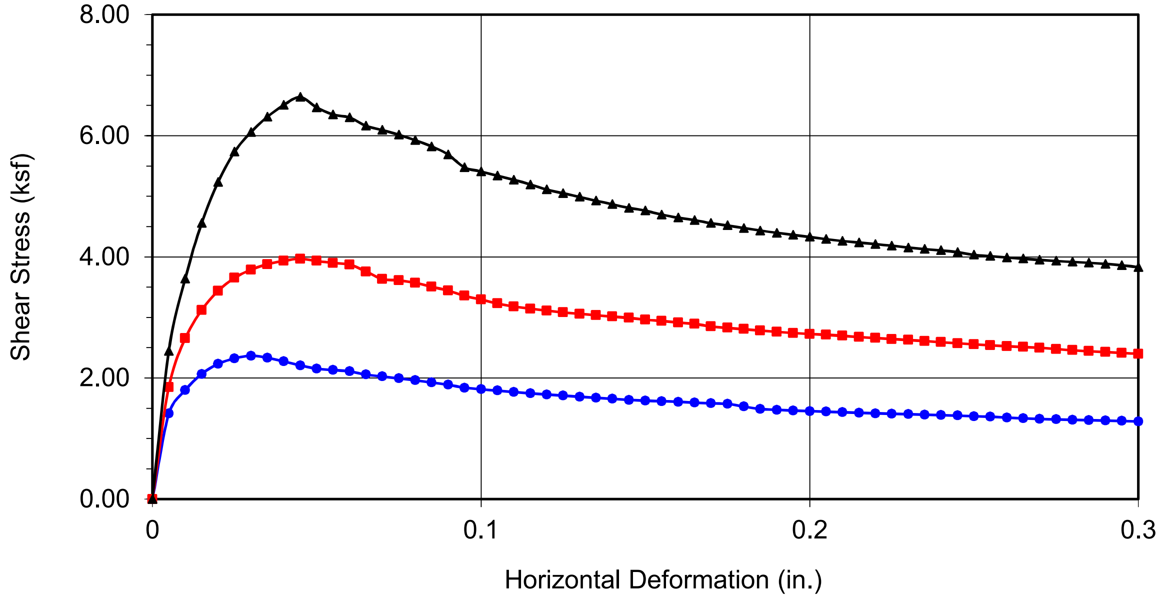
Consolidated Drained - ASTM D 3080

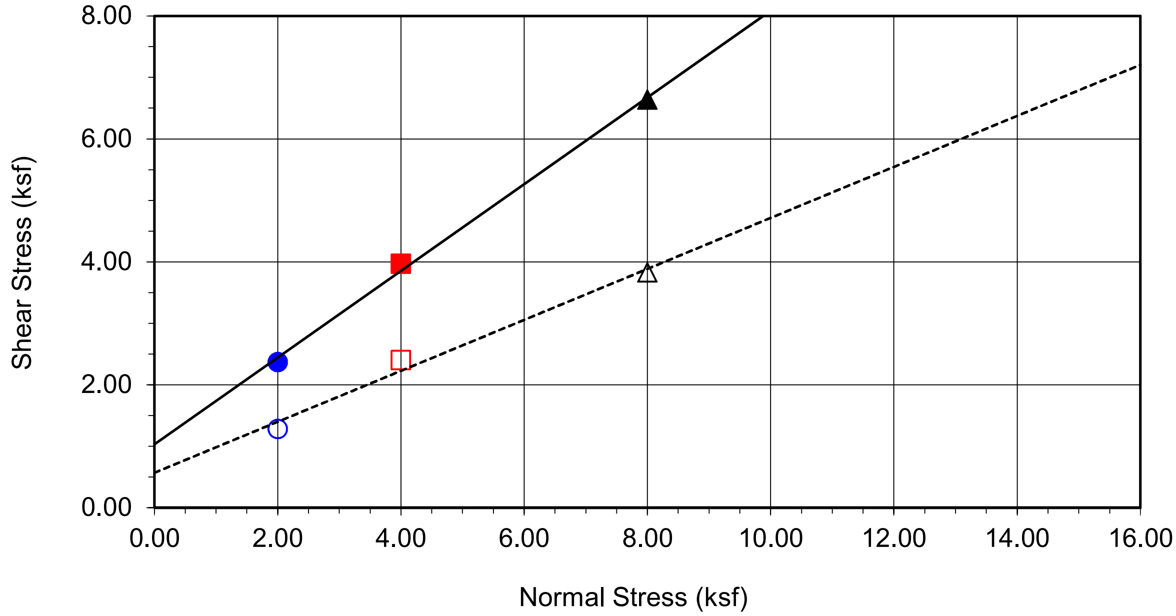
Project No.: 13589.002

JDA Woodland Hills

07-22

DS LB-3, R-8 @ 30





Boring No.	**LB-3**
Sample No.	**R-8**
Depth (ft)	**30**
Sample Type:	Ring

Soil Identification:
Dark olive brown lean clay (CL), claystone noted

Strength Parameters

	C (psf)	ϕ (°)
Peak	1032	35
Ultimate	568	23

Normal Stress (kip/ft²)	2.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 2.367	■ 3.967	▲ 6.637
Shear Stress @ End of Test (ksf)	○ 1.283	□ 2.399	△ 3.829
Deformation Rate (in./min.)	0.0017	0.0017	0.0017
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	45.79	45.79	45.79
Dry Density (pcf)	71.9	73.1	73.8
Saturation (%)	92.0	94.6	96.4
Soil Height Before Shearing (in.)	0.9954	0.9923	0.9873
Final Moisture Content (%)	51.1	49.9	46.8

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DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 13589.002

JDA Woodland Hills

07-22

EXPANSION INDEX of SOILS
ASTM D 4829

Project Name:	JDA Woodland Hills	Tested By:	G. Berdy	Date:	07/26/22
Project No.:	13589.002	Checked By:	J. Ward	Date:	08/01/22
Boring No.:	LB-1	Depth (ft.):	1-5		
Sample No.:	B-1				
Soil Identification:	Gray clayey sand (SC)				

Dry Wt. of Soil + Cont.	(g)	1000.00
Wt. of Container No.	(g)	0.00
Dry Wt. of Soil	(g)	1000.00
Weight Soil Retained on #4 Sieve		0.00
Percent Passing # 4		100.00

MOLDED SPECIMEN		Before Test	After Test
Specimen Diameter	(in.)	4.01	4.01
Specimen Height	(in.)	1.0000	1.0330
Wt. Comp. Soil + Mold	(g)	591.80	443.00
Wt. of Mold	(g)	187.70	0.00
Specific Gravity (Assumed)		2.70	2.70
Container No.		O	O
Wet Wt. of Soil + Cont.	(g)	817.40	630.70
Dry Wt. of Soil + Cont.	(g)	749.20	558.09
Wt. of Container	(g)	0.00	187.70
Moisture Content	(%)	9.10	19.60
Wet Density	(pcf)	121.9	129.4
Dry Density	(pcf)	111.7	108.2
Void Ratio		0.509	0.559
Total Porosity		0.337	0.358
Pore Volume	(cc)	69.8	76.6
Degree of Saturation (%) [S_{meas}]		**48.3**	94.7

SPECIMEN INUNDATION in distilled water for the period of 24 h or expansion rate < 0.0002 in./h

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
07/26/22	13:57	1.0	0	0.5880
07/26/22	14:07	1.0	10	0.5865
Add Distilled Water to the Specimen				
07/26/22	14:32	1.0	25	0.6100
07/27/22	6:02	1.0	955	0.6210
07/27/22	7:13	1.0	1026	0.6210

Expansion Index (EI_{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	**35**

Borehole	Depth	Liquid Limit	Plastic Limit	Plasticity Index	Maximum Size (mm)	%<#200 Sieve	Class-ification	Water Content (%)	Dry Density (pcf)	Satur-ation (%)	Void Ratio
LB-1	0.9							16.7			
LB-1	5.0							18.0	107.9		
LB-1	7.5							19.5	96.0		
LB-1	10.0							17.7	107.5		
LB-1	15.0							25.2	95.9		
LB-1	17.5							24.6	96.1		
LB-1	20.0							25.7	95.7		
LB-1	30.0							32.1	88.9		
LB-1	40.0							26.4	92.6		
LB-1	45.0							24.1	101.5		
LB-3	5.0							19.7	95.7		
LB-3	7.5							11.1	91.2		
LB-3	12.5							20.5	103.0		
LB-3	15.0							23.9	98.5		
LB-3	40.0							34.4	89.1		

US LAB SUMMARY 13589.002 JDA WOODLAND HILLS BORING LOGS.GPJ ROCKLOG2012.GDT 8/2/22

Leighton

Summary of Laboratory Results

Project Name: Proposed Storage

Project Number: 13589.001

Date: 8/2/2022 11:32:13 AM

Figure No. 1

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name:	JDA Woodland Hills	Tested By:	J. Gonzalez	Date:	07/20/22
Project No.:	13589.002	Checked By:	A. Santos	Date:	07/25/22
Boring No.:	LB-3	Depth (ft.):	1-5		
Sample No.:	B-1				
Soil Identification:	Dark olive brown lean clay (CL)				

Preparation Method: [X] Moist [ ] Dry [X] Mechanical Ram [ ] Manual Ram

Mold Volume (ft³) 0.03330 *Ram Weight = 10 lb.; Drop = 18 in.*

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3580	3707	3689			
Weight of Mold (g)	1826	1826	1826			
Net Weight of Soil (g)	1754	1881	1863			
Wet Weight of Soil + Cont. (g)	452.5	414.4	421.8			
Dry Weight of Soil + Cont. (g)	409.6	364.6	364.8			
Weight of Container (g)	39.8	38.1	39.5			
Moisture Content (%)	11.60	15.25	17.52			
Wet Density (pcf)	116.1	124.5	123.3			
Dry Density (pcf)	104.1	108.0	104.9			

Maximum Dry Density (pcf) **108.2** **Optimum Moisture Content (%)** **14.8**

PROCEDURE USED

[X] **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

[ ] **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

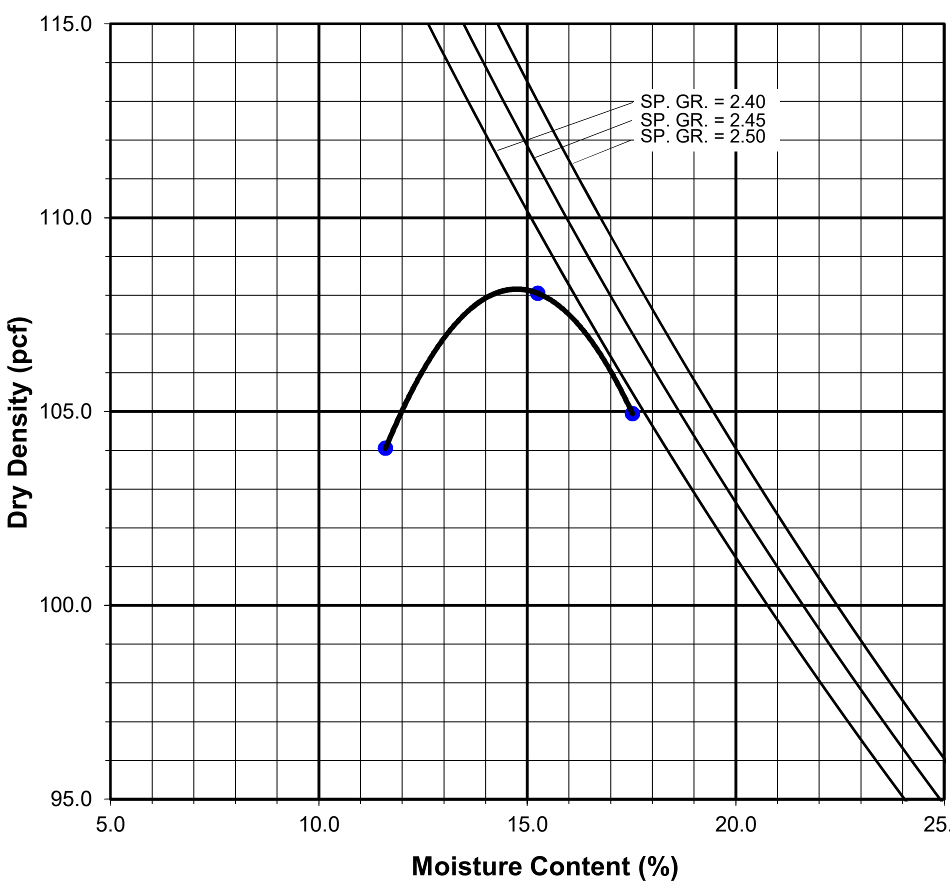
[ ] **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI

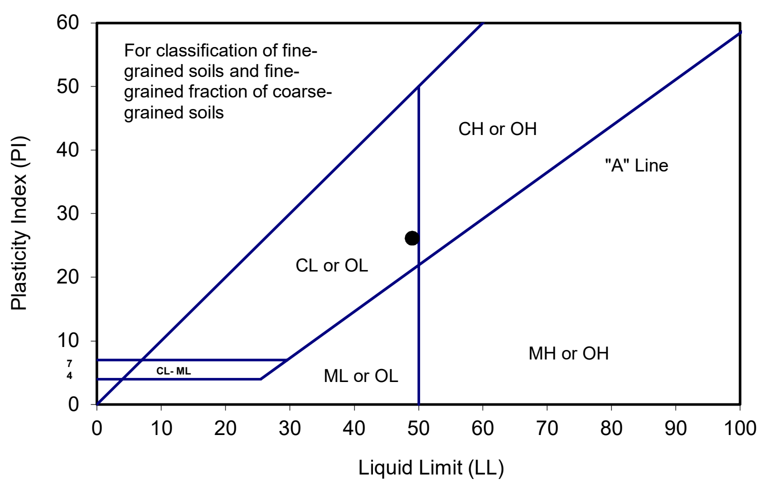


ATTERBERG LIMITS
ASTM D 4318

Project Name:	JDA Woodland Hills	Tested By:	ACS/JD	Date:	07/29/22
Project No. :	13589.002	Input By:	J. Ward	Date:	08/02/22
Boring No.:	LB-1	Checked By:	J. Ward		
Sample No.:	R-8	Depth (ft.)	25.0		
Soil Identification:	Brown lean clay (CL)				

TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			33	24	17	
Wet Wt. of Soil + Cont. (g)	8.45	8.91	19.98	18.16	20.40	
Dry Wt. of Soil + Cont. (g)	7.07	7.45	13.92	12.56	13.82	
Wt. of Container (g)	1.07	1.02	1.04	1.06	1.05	
Moisture Content (%) [Wn]	23.00	22.71	47.05	48.70	51.53	

Liquid Limit	**49**
Plastic Limit	**23**
Plasticity Index	**26**
Classification	**CL**



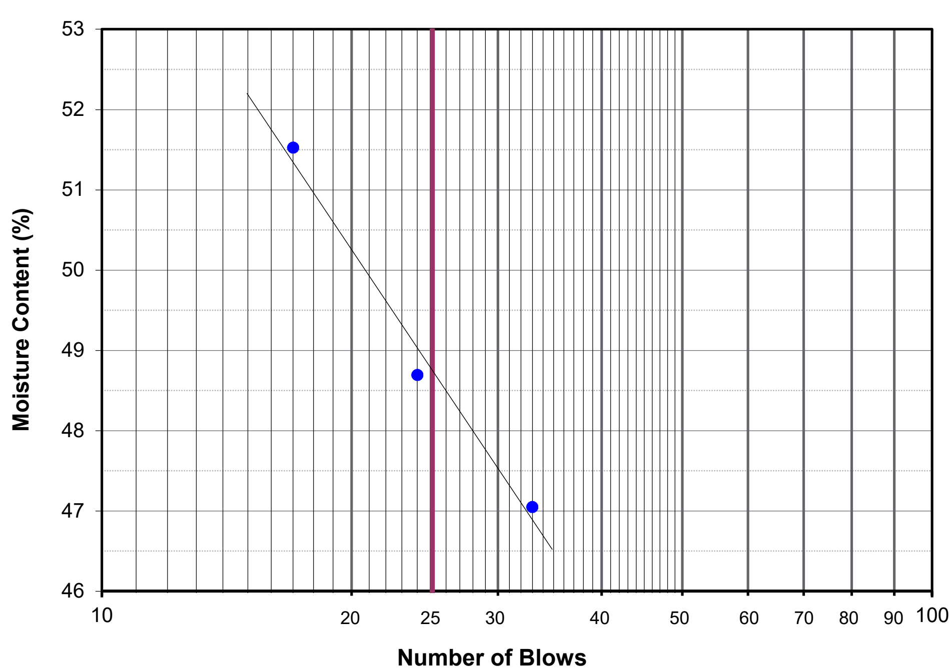
PI at "A" - Line = 0.73(LL-20) 21.17

One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$

PROCEDURES USED

- [ ] Wet Preparation
  Multipoint - Wet
- [x] Dry Preparation
  Multipoint - Dry
- [x] Procedure A
  Multipoint Test
- [ ] Procedure B
  One-point Test



ATTERBERG LIMITS
ASTM D 4318

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Project Name:	JDA Woodland Hills	Tested By:	ACS/JD	Date:	07/29/22
Project No. :	13589.002	Input By:	J. Ward	Date:	08/01/22
Boring No.:	LB-1	Checked By:	J. Ward		
Sample No.:	ST-4	Depth (ft.)	12.5		
Soil Identification:	Dark yellowish brown fat clay (CH)				

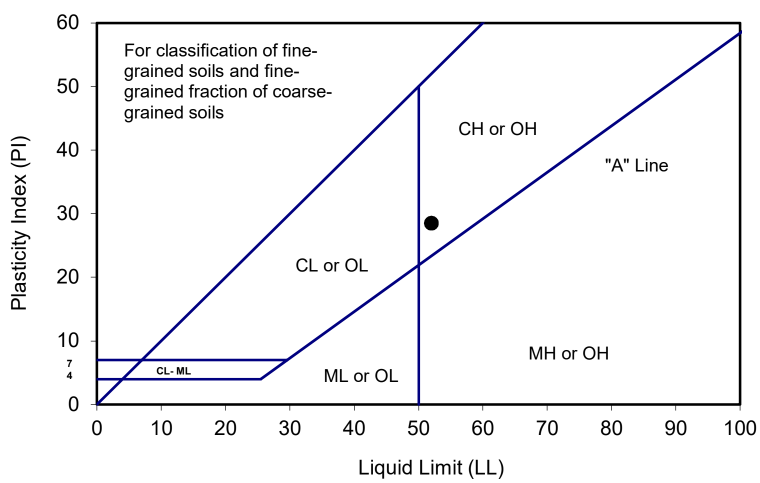
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			29	23	18	
Wet Wt. of Soil + Cont. (g)	9.27	9.35	20.19	19.10	17.35	
Dry Wt. of Soil + Cont. (g)	7.71	7.77	13.72	12.93	11.72	
Wt. of Container (g)	1.09	1.02	1.10	1.09	1.07	
Moisture Content (%) [Wn]	23.56	23.41	51.27	52.11	52.86	

Liquid Limit	**52**
Plastic Limit	**23**
Plasticity Index	**29**
Classification	**CH**

PI at "A" - Line = 0.73(LL-20) | 23.36

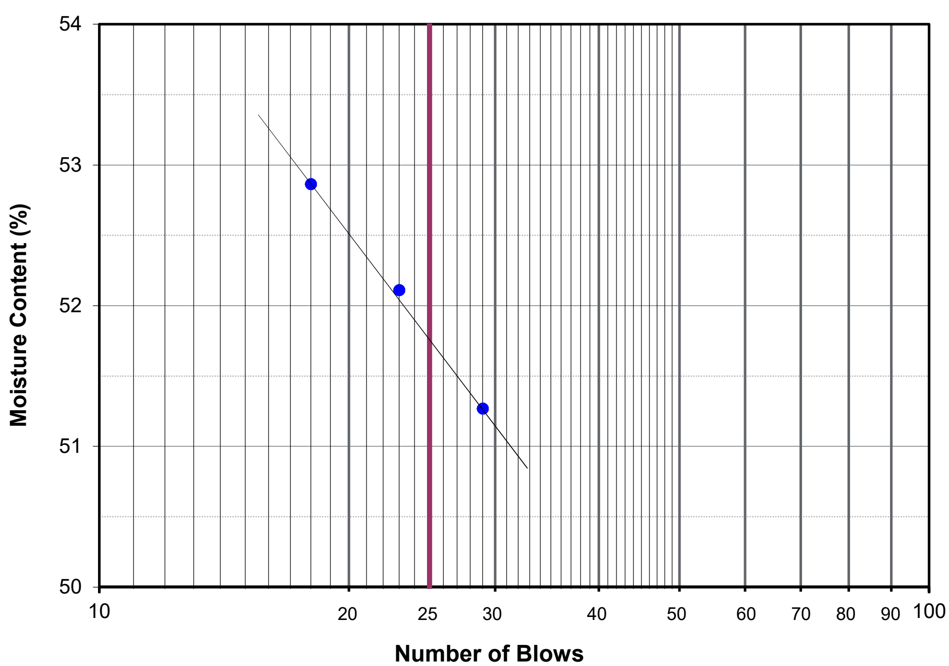
One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$



PROCEDURES USED

- [ ] Wet Preparation
  Multipoint - Wet
- [X] Dry Preparation
  Multipoint - Dry
- [X] Procedure A
  Multipoint Test
- [ ] Procedure B
  One-point Test



ATTERBERG LIMITS
ASTM D 4318

Project Name:	JDA Woodland Hills	Tested By:	J. Domingo	Date:	07/29/22
Project No. :	13589.002	Input By:	A. Santos	Date:	08/01/22
Boring No.:	LB-1	Checked By:	ACS/JHW		
Sample No.:	ST-10	Depth (ft.)	35.0		
Soil Identification:	Yellowish brown lean clay with sand (CL)s				

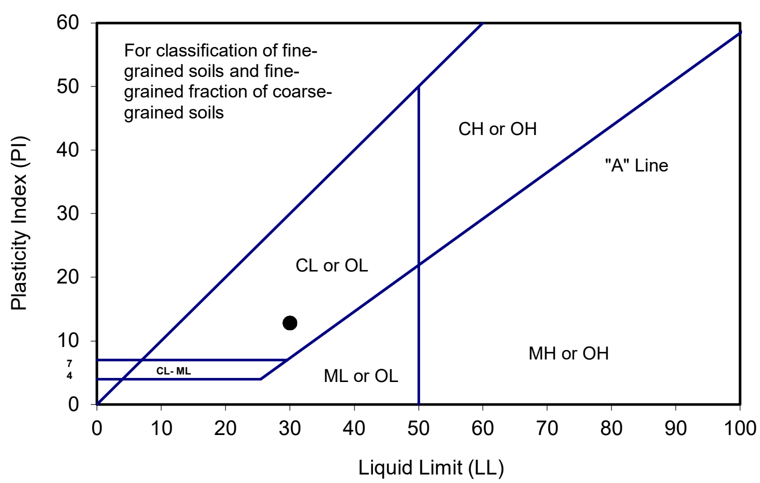
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			29	22	16	
Wet Wt. of Soil + Cont. (g)	9.40	9.32	21.31	21.45	21.65	
Dry Wt. of Soil + Cont. (g)	8.17	8.11	16.84	16.65	16.59	
Wt. of Container (g)	1.03	1.07	1.12	1.04	1.05	
Moisture Content (%) [Wn]	17.23	17.19	28.44	30.75	32.56	

Liquid Limit	**30**
Plastic Limit	**17**
Plasticity Index	**13**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) 7.3

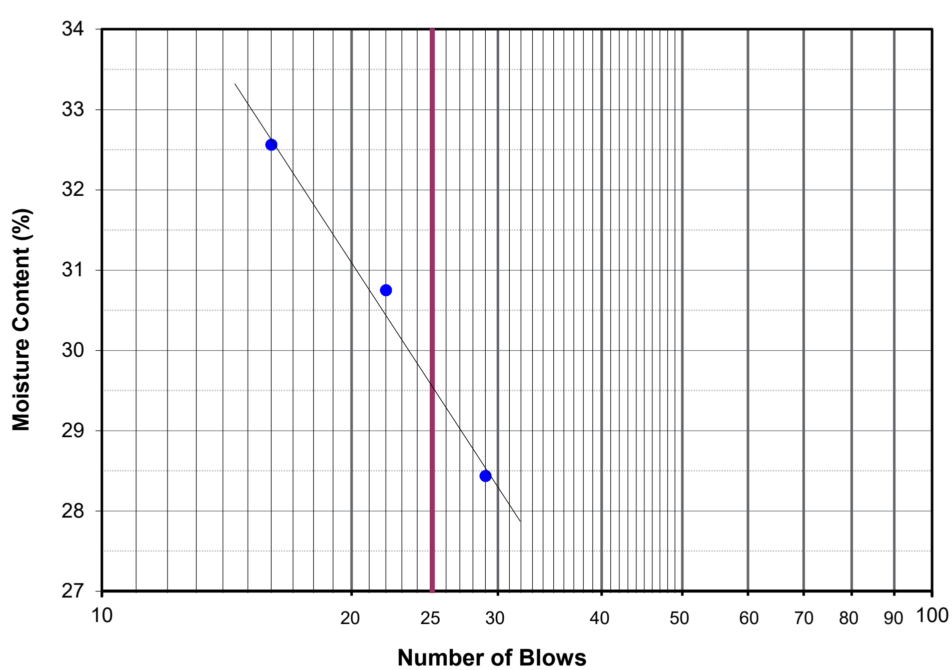
One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$



PROCEDURES USED

- [ ] Wet Preparation Multipoint - Wet
- [X] Dry Preparation Multipoint - Dry
- [X] Procedure A Multipoint Test
- [ ] Procedure B One-point Test



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ATTERBERG LIMITS
ASTM D 4318

Project Name:	JDA Woodland Hills	Tested By:	J. Domingo	Date:	07/29/22
Project No. :	13589.002	Input By:	A. Santos	Date:	08/01/22
Boring No.:	LB-1	Checked By:	ACS/JHW		
Sample No.:	ST-13	Depth (ft.)	50.0		
Soil Identification:	Olive brown fat clay (CH)				

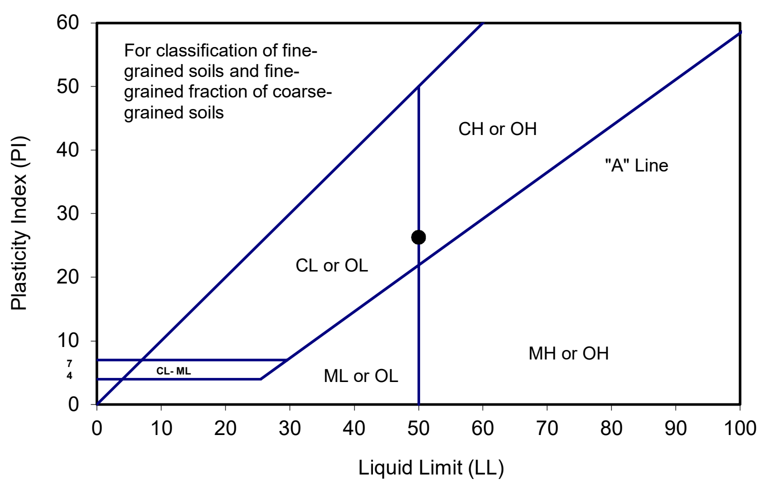
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			32	25	18	
Wet Wt. of Soil + Cont. (g)	9.19	9.31	21.25	21.36	21.14	
Dry Wt. of Soil + Cont. (g)	7.63	7.73	14.65	14.61	14.34	
Wt. of Container (g)	1.08	1.04	1.07	1.06	1.05	
Moisture Content (%) [Wn]	23.82	23.62	48.60	49.82	51.17	

Liquid Limit	**50**
Plastic Limit	**24**
Plasticity Index	**26**
Classification	**CH**

PI at "A" - Line = 0.73(LL-20) 21.9

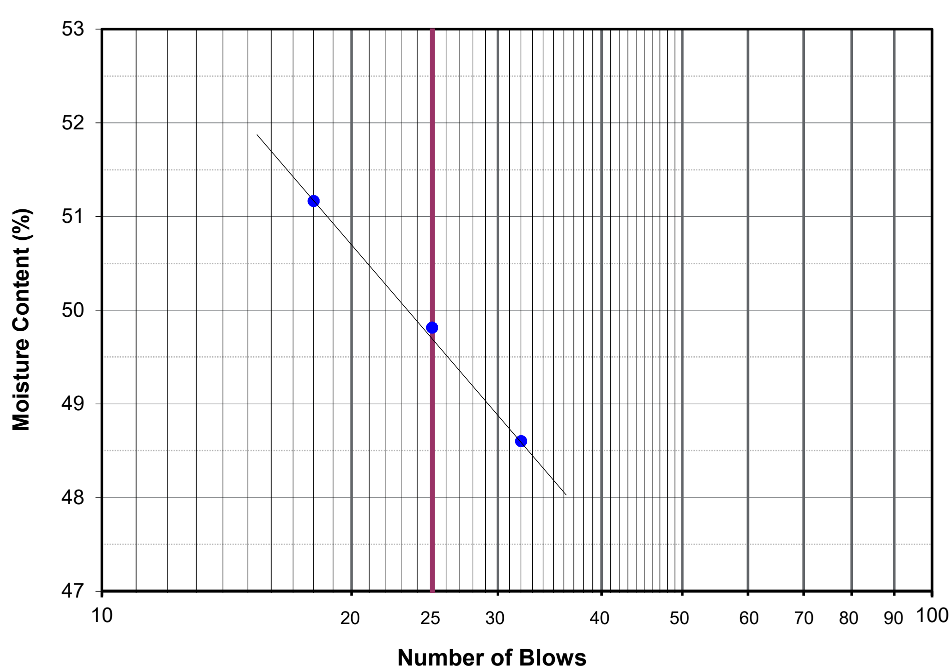
One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$



PROCEDURES USED

- [ ] Wet Preparation — Multipoint - Wet
- [x] Dry Preparation — Multipoint - Dry
- [x] Procedure A — Multipoint Test
- [ ] Procedure B — One-point Test

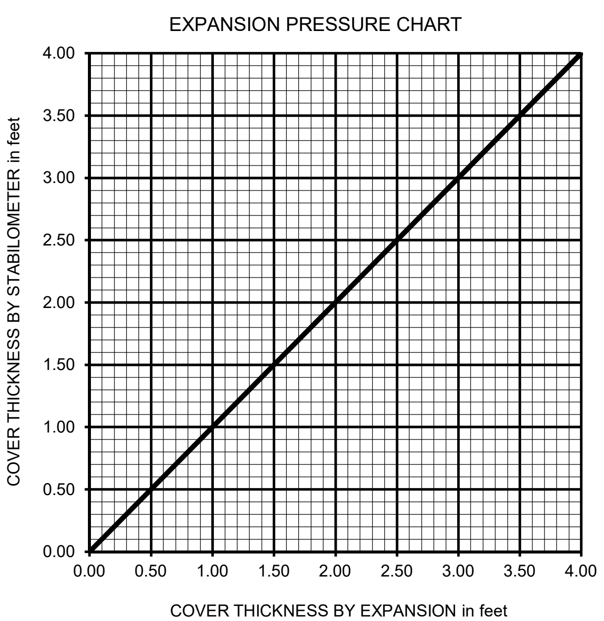


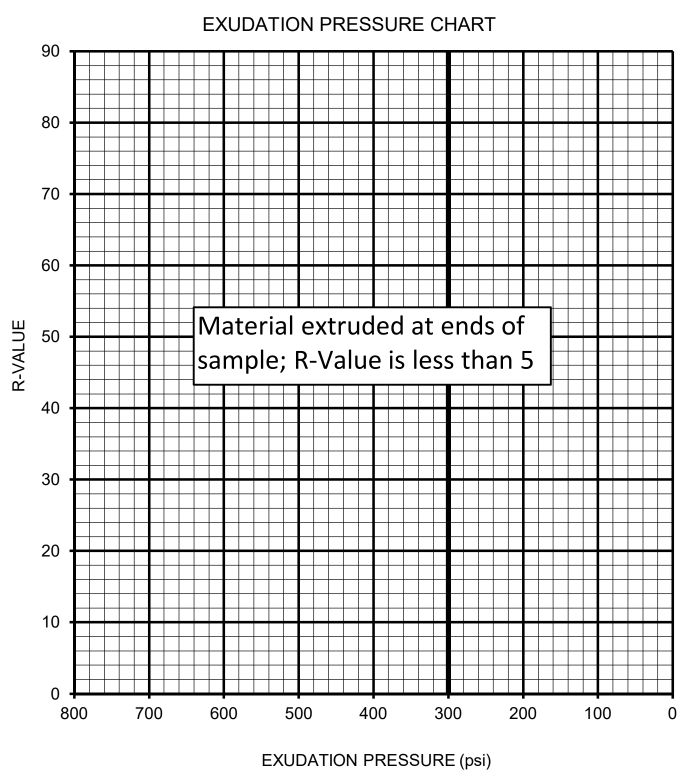
R-VALUE TEST RESULTS
DOT CA Test 301

PROJECT NAME:	JDA Woodland Hills	PROJECT NUMBER:	13589.002
BORING NUMBER:	LB-3	DEPTH (FT.):	1-5
SAMPLE NUMBER:	B-1	TECHNICIAN:	A. Santos
SAMPLE DESCRIPTION:	Dark olive brown lean clay (CL)	DATE COMPLETED:	8/1/2022

TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %			
HEIGHT OF SAMPLE, Inches			
DRY DENSITY, pcf			
COMPACTOR PRESSURE, psi			
EXUDATION PRESSURE, psi			
EXPANSION, Inches x 10exp-4			
STABILITY Ph 2,000 lbs (160 psi)			
TURNS DISPLACEMENT			
R-VALUE UNCORRECTED	N/A	N/A	N/A
R-VALUE CORRECTED	N/A	N/A	N/A

DESIGN CALCULATION DATA	a	b	c
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	N/A	N/A	N/A
EXPANSION PRESSURE THICKNESS, ft.	N/A	N/A	N/A





R-VALUE BY EXPANSION:	N/A
R-VALUE BY EXUDATION:	N/A
EQUILIBRIUM R-VALUE:	< 5

APPENDIX D

ENGINEERING ANALYSIS

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APPENDIX D

Site-Specific Ground Motion Hazard Analysis

Leighton performed a site-specific ground motion study using the computer program OpenSHA (Open-Source Seismic Hazard Analysis), in accordance with the requirements of the 2019 California Building Code (CBC) and ASCE 7-16 documents. Site-specific seismic design parameters were developed for new and existing structures. The details of our study are presented in the following sections.

D.1 Attenuation Relationships

Attenuation relationships (Ground Motion Prediction Equations or GMPEs) describe the relation of ground motion levels with earthquake magnitude and distance (distance between the site and seismic source), site geology, and subsurface characterization. These relationships can be used to describe the variation of peak ground acceleration and response spectral acceleration with earthquake magnitude and distance, and to also incorporate the local geological conditions and near-source effects.

We used four GMPEs: Abrahamson et al. (2014) NGA West 2, Boore et al. (2014) NGA West 2, Campbell and Bozorgnia (2014) NGA West 2, and Chiou and Youngs (2014) NGA West 2. These GMPEs are based on the median rotated direction (RotD50) of horizontal ground motion.

The average shear wave velocity in the upper 30 meters (100 feet), Vs30, was based on measured shear wave velocity from CPT-1 of 260 meters per second and CGS mapped value of 352 meters per second. Therefore, Site Class D and Vs30 of 260 m/sec and 352 m/sec were used with the selected GMPEs. The response spectrums were combined to account for the varying bedrock and alluvium depths across the site.

D.2 Design Criteria

The earthquake ground motions considered include the Risk-Targeted Maximum Considered Earthquake (MCE_R) and the Design Earthquake (DE). The MCE_R is defined as the maximum component of horizontal ground motion with a 2% probability of exceedance in 50 years (2,475-year average return interval) adjusted for targeted risk (ASCE 7-16). The DE ground motions are defined as 2/3 of MCE_R ground motions (ASCE 7-16).

D.3 Methodology

The 2019 CBC requires the procedures of Chapter 21, Site-Specific Ground Motion Procedures for Seismic Design, of ASCE 7-16 be used to determine site-specific seismic response spectra and design parameters. We performed both deterministic and probabilistic seismic hazard analyses (DSHA and PSHA) and process the results in accordance with the procedures in Chapter 21 of ASCE 7-16.

D.4 Probabilistic Seismic Hazard Analysis

A PSHA is a mathematical process based on probability and statistics that is used to estimate the mean number of events per year in which the level of some ground motion parameter, Z (peak ground acceleration and/or spectral response acceleration in this study), exceeds a specified value z at the project site. This mean number of events per year, also referred to as “annual frequency of exceedance,” is designated as “v(Z≥z).” The inverse of this number is called the “average return period (ARP),” which is expressed in terms of years. Having the annual frequency of exceedance of a certain level of acceleration or spectral response acceleration, v(Z≥z), the probability of exceeding that level Pr(Z≥z), within any time period of interest, t, is then obtained assuming a Poisson Distribution as follows:

$$\mathrm{Pr}\ (Z \geq z) = 1 - e^{-\upsilon\,(Z \geq z)\,t}$$

PSHA procedures require the specification of probability functions to describe the uncertainty in both the time and location of future earthquake occurrences and the uncertainty in the ground motion level that will be produced at the site.

The basic key elements of a PSHA are:

- Defining the location, geometry, and characteristics of earthquake sources relative the site;
- Specifying an earthquake recurrence relationship for various magnitudes on each source up to the maximum magnitude;
- Selecting appropriate attenuation relationships, which relate the variation of the earthquake ground motion parameter with earthquake distance, directivity, magnitude, site geology, and subsurface characterization; and
- Determining the probability of exceedance of peak ground accelerations and/or response spectral levels (i.e., seismic hazards) utilizing the above input parameters.

The frequencies of exceedance of peak ground accelerations and spectral response accelerations at the site were calculated by evaluating the following:

- The annual frequency of earthquakes of various magnitudes on a fault obtained from the fault recurrence relationships;
- Given an earthquake of a certain magnitude on a certain fault, the probability distribution of the location of the earthquake on the fault was obtained using the selected rupture area versus magnitude relationship and assuming equal likelihood of rupture along the length and some prescribed probabilities along the depth of the fault; and
- Given an earthquake of a certain magnitude occurring at a certain distance from the site, the probability distribution of ground motion at the site was obtained from the selected attenuation relationships.

The above process is repeated a sufficient number of times to cover all the sources, then summed to obtain the total seismic hazard at the site. This process results in a relationship between ground motion level and the probability of being exceeded.

The computer program OpenSHA (2021) was used to perform the probabilistic seismic hazard analysis (PSHA).

D.5 Deterministic Seismic Hazard Analysis

The DSHA consists of a four-step process (Reiter, 1990):

- Defining the location, geometry, and characteristics of earthquake sources relative to the site;
- Determination of the site-to-source distance for each earthquake source defined relative to the site;
- Selection of the controlling earthquake relative to the site as defined by some ground motion parameter. The controlling earthquake is defined by the seismic scenario based on the above two steps that produces the largest magnitude of the ground motion parameter being used;
- Using the controlling earthquake, the deterministic ground motions at the site is obtained from the selected attenuation relationships; and

- Deterministic ground motions represent the 84th percentile average horizontal component and modified using Shahi and Baker (2014) to represent the maximum component horizontal ground motions.

The NGA-West2 deterministic spreadsheet by the Pacific Earthquake Engineering Research Center (PEER, 2015) was used for the DSHA. The fault distances used are based on the deaggregation results obtained from the USGS Unified Hazard Tool website (https://earthquake.usgs.gov/hazards/interactive/). The fault magnitudes used are based on the Building Seismic Safety Council (BSSC) 2014 scenario catalog website (https://earthquake.usgs.gov/scenarios/catalog/bssc2014/). The input parameters are shown in the NGA-West2 deterministic spreadsheets, attached at the end of this appendix.

D.6 Code-Based General Seismic Response Spectra and Design Parameters

Seismic response spectra and design parameters were computed as determined by Chapter 11 of ASCE 7-16. These values are used to process the site-specific design response spectrum to ensure the site-specific DE and MCE_R response spectra meet or exceed minimum requirements. The code-based seismic design parameters were derived by using the ATC Hazards by Location Tool (https://hazards.atcouncil.org/) to obtain the design values from the United States Geological Survey (USGS).

The code-based parameters determined from the referenced online are attached at the end of this appendix.

D.7 Site-Specific Response Spectra

The site-specific MCE_R and DE response spectra were developed per the methodology prescribed in Chapter 21 of ASCE 7-16. Site-specific response spectra for MCE_R and DE were computed for a structural damping ratio of 5 percent of critical damping. Targeted risk coefficients were determined from mapped values in ASCE 7-16 to calculate MCE_R.

We used the Shahi and Baker (2014) SaRotD100/SaRotD50 factors to develop the maximum component of horizontal ground motion as required in the definition of ground motion in the current building codes (2019 CBC and ASCE 7-16). These factors enabled us to estimate the maximum horizontal component of ground motion.

Figure D.1 presents a graph and table with ordinates of the maximum design response spectra from the two different Vs30 calculations.

Figure D.2A and D.3A presents a graph and table with ordinates of the RotD50 and the maximum component MCE response spectra from the PSHA. The maximum component (MC) factors from Shahi and Baker (2014) are also presented the figures.

Figure D.2B and D.3B presents plots and tables with ordinates of the MCE_R response spectra from the DSHA.

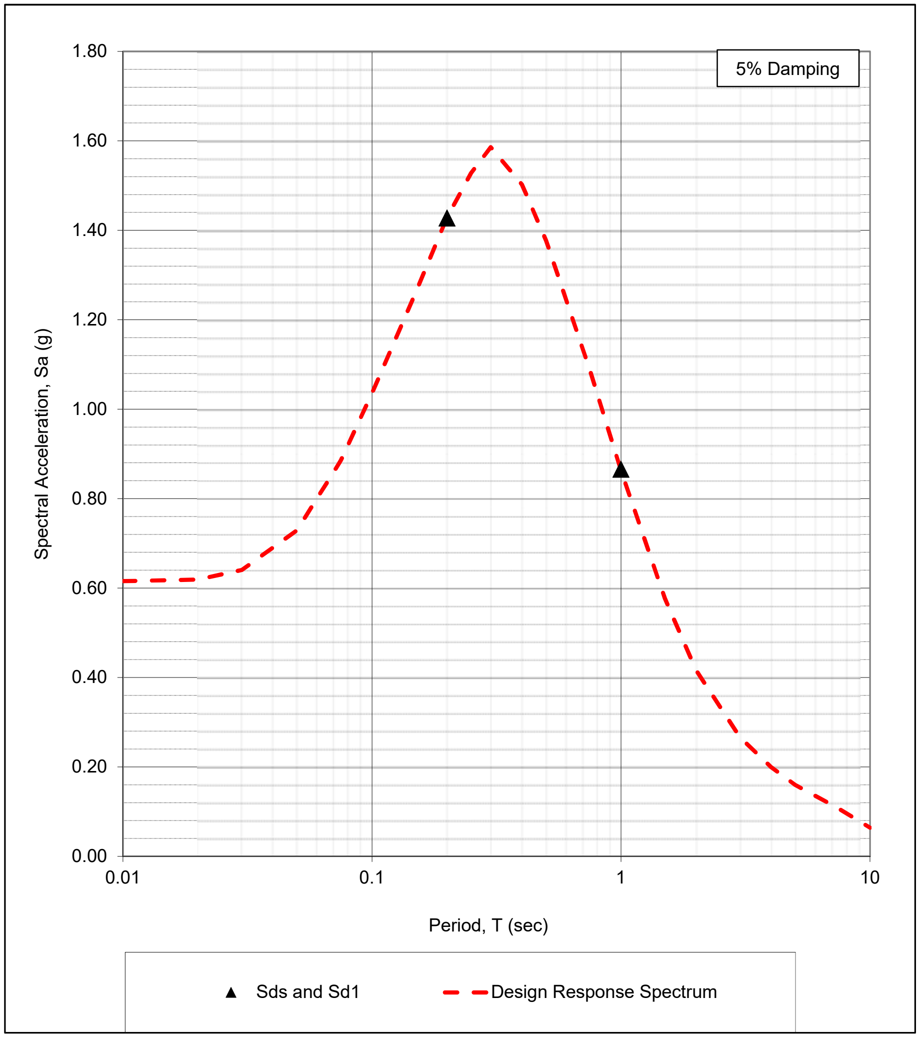
Per Chapter 21.2.3 of ASCE 7-16, the deterministic and probabilistic spectra were compared to establish site-specific maximum component MCE spectra. This step is shown on Figure D.2C and D.3C.

Figure D.2D and D.3D shows a comparison of site-specific vs. general code-based spectra for the MCE_R spectrum.

The DE spectrum is shown on Figure D.2E and D.3E, which also includes the 80% of the general code-based spectrum floor stipulated in Chapter 21.3 of ASCE 7-16.

ASCE 7-16 DESIGN RESPONSE SPECTRUM AND SITE-SPECIFIC S_{DS} AND S_{D1}

Project:	Proposed Self-Storage Facility
Project Number:	13589.001
Location:	21101 Ventura Boulevard



Period T (s)	Design Response Spectrum (g)
0.01	0.615
0.02	0.619
0.03	0.640
0.05	0.729
0.075	0.885
0.10	1.036
0.15	1.262
0.20	1.428
0.25	1.528
0.30	1.586
0.40	1.502
0.50	1.377
0.75	1.087
1.00	0.863
1.50	0.577
2.00	0.417
3.00	0.267
4.00	0.200
5.00	0.160
7.50	0.107
10.00	0.064

SDS =	1.427	g
SD1 =	0.865	g

Note: Based on ASCE 7-16 Section 21.4, the parameter S_{DS} shall be taken as 90% of the maximum spectral acceleration, Sa, obtained from the site-specific spectrum, at any period within the range from 0.2 to 5 s, inclusive. The parameter S_{D1} shall be taken as the maximum value of the product, TSa, for periods from 1 to 2 s for sites with Vs30 > 1,200 ft/s (Vs30 > 365.76 m/s) and for periods from 1 to 5 s for sites with $Vs_{30} \leq$ 1,200 ft/s ($Vs_{30} \leq$ 365.76 m/s). The design Sa shall not be less than 80% of 2/3 of the general procedure (ASCE 7-16 Sec 11.4.6)

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Figure D1

Table 1: Site-Specific Seismic Ground Motion Analysis per ASCE 7-16

Project Name: Proposed Self-Storage Facility
Project Location: 21101 Ventura Boulevard
Project Number: 13589.001
Site Class: D
Return Period: 2475 years (2% probability of exceedance in 50 years)
Percent Damping: 5%

Date: July 2022
Latitude: 34.168161°
Longitude: -118.59298°

Seismic Design Coefficients: Per ASCE 7-16 & 2019 CBC

S_s	1.654	**S_{MS}**	**2.141**		T_0	0.216
S_1	0.6	**S_{M1}**	**1.298**		T_s	0.907
F_a	1	**S_{DS}**	**1.427**		T_L	8
F_v	2.5	**S_{D1}**	**0.865**		**PGA_M**	**0.776**
C_{RS}	0.933	C_{R1}	0.913			

	Sec. 21.2.1.1 Probabilistic				Sec. 21.2.2 Deterministic			Sec. 11.4.6 General Procedure	Sec. 21.3 Design Response Spectrum				Risk Targeted Spectrum
Period (sec)	Spectral Acceleration (g)	Seismic Risk Coefficients	Maximum Response Coefficients	MCE_R Response Spectrum (g)	Spectral Acceleration (g)	Maximum Response Coefficients	MCE_R Response Spectrum (g)	Design Response Spectral Acceleration (g)	Lower Limit of General Procedure - 80% of S_a (g)	MCE_R - S_{aM} (g)	2/3 * S_{aM} (g)	Design Response Spectrum (g)	1.5 * Design Response Spectrum (g)
0.01	0.869	0.933	1.19	0.965	0.776	1.19	0.923	0.472	0.377	0.923	0.615	0.615	0.923
0.02	0.873	0.933	1.19	0.969	0.780	1.19	0.929	0.502	0.402	0.929	0.619	0.619	0.929
0.03	0.918	0.933	1.19	1.019	0.807	1.19	0.960	0.533	0.426	0.960	0.640	0.640	0.960
0.05	1.099	0.933	1.19	1.221	0.919	1.19	1.094	0.594	0.475	1.094	0.729	0.729	1.094
0.075	1.409	0.933	1.19	1.564	1.116	1.19	1.328	0.670	0.536	1.328	0.885	0.885	1.328
0.1	1.662	0.933	1.19	1.845	1.306	1.19	1.554	0.747	0.597	1.554	1.036	1.036	1.554
0.15	1.938	0.933	1.20	2.170	1.577	1.20	1.893	0.900	0.720	1.893	1.262	1.262	1.893
0.2	2.079	0.933	1.21	2.347	1.771	1.21	2.143	1.052	0.842	2.143	1.428	1.428	2.143
0.25	2.135	0.932	1.22	2.427	1.879	1.22	2.292	1.103	0.882	2.292	1.528	1.528	2.292
0.3	2.128	0.931	1.22	2.415	1.950	1.22	2.379	1.103	0.882	2.379	1.586	1.586	2.379
0.4	1.974	0.928	1.23	2.254	1.904	1.23	2.342	1.103	0.882	2.254	1.502	1.502	2.254
0.5	1.815	0.926	1.23	2.066	1.747	1.23	2.149	1.103	0.882	2.066	1.377	1.377	2.066
0.75	1.430	0.919	1.24	1.630	1.390	1.24	1.724	1.103	0.882	1.630	1.087	1.087	1.630
1	1.144	0.913	1.24	1.295	1.118	1.24	1.387	1.000	0.800	1.295	0.863	0.863	1.295
1.5	0.764	0.913	1.24	0.865	0.745	1.24	0.924	0.667	0.533	0.865	0.577	0.577	0.865
2	0.552	0.913	1.24	0.625	0.538	1.24	0.667	0.500	0.400	0.625	0.417	0.417	0.625
3	0.338	0.913	1.25	0.386	0.305	1.25	0.382	0.333	0.267	0.382	0.255	0.267	0.400
4	0.230	0.913	1.26	0.264	0.194	1.26	0.245	0.250	0.200	0.245	0.163	0.200	0.300
5	0.171	0.913	1.26	0.196	0.137	1.26	0.172	0.200	0.160	0.172	0.115	0.160	0.240
7.5	0.098	0.913	1.28	0.114	0.067	1.28	0.086	0.133	0.107	0.086	0.057	0.107	0.160
10	0.063	0.913	1.29	0.074	0.038	1.29	0.049	0.080	0.064	0.049	0.032	0.064	0.096

Table 2: Site-Specific Seismic Ground Motion Analysis per ASCE 7-16

Project Name: Proposed Self-Storage Facility
Project Location: 21101 Ventura Boulevard
Project Number: 13589.001
Site Class: D
Shear Wave Velocity: 260 m/sec
Return Period: 2475 years (2% probability of exceedance in 50 years)
Percent Damping: 5%

Date: July 2022
Latitude: 34.168161°
Longitude: -118.59298°

Seismic Design Coefficients: Per ASCE 7-16 & 2019 CBC

S_s	1.654	$\mathbf{S_{MS}}$	**1.956**	T_0	0.216
S_1	0.6	$\mathbf{S_{M1}}$	**1.298**	T_s	0.907
F_a	1	$\mathbf{S_{DS}}$	**1.304**	T_L	8
F_v	2.5	$\mathbf{S_{D1}}$	**0.865**	$\mathbf{PGA_M}$	**0.700**
C_{RS}	0.933	C_{R1}	0.913		

	Sec. 21.2.1.1 Probabilistic				Sec. 21.2.2 Deterministic			Sec. 11.4.6 General Procedure	Sec. 21.3 Design Response Spectrum				Risk Targeted Spectrum
Period (sec)	Spectral Acceleration (g)	Seismic Risk Coefficients	Maximum Response Coefficients	MCE_R Response Spectrum (g)	Spectral Acceleration (g)	Maximum Response Coefficients	MCE_R Response Spectrum (g)	Design Response Spectral Acceleration (g)	Lower Limit of General Procedure - 80% of S_a (g)	MCE_R - S_{aM} (g)	2/3 * S_{aM} (g)	Design Response Spectrum (g)	1.5 * Design Response Spectrum (g)
0.01	0.818	0.933	1.19	0.908	0.700	1.19	0.832	0.472	0.377	0.832	0.555	0.555	0.832
0.02	0.823	0.933	1.19	0.914	0.700	1.19	0.833	0.502	0.402	0.833	0.555	0.555	0.833
0.03	0.844	0.933	1.19	0.937	0.701	1.19	0.834	0.533	0.426	0.834	0.556	0.556	0.834
0.05	0.976	0.933	1.19	1.083	0.762	1.19	0.907	0.594	0.475	0.907	0.605	0.605	0.907
0.075	1.249	0.933	1.19	1.387	0.905	1.19	1.077	0.670	0.536	1.077	0.718	0.718	1.077
0.1	1.480	0.933	1.19	1.644	1.059	1.19	1.260	0.747	0.597	1.260	0.840	0.840	1.260
0.15	1.711	0.933	1.20	1.916	1.280	1.20	1.536	0.900	0.720	1.536	1.024	1.024	1.536
0.2	1.831	0.933	1.21	2.068	1.462	1.21	1.768	1.052	0.842	1.768	1.179	1.179	1.768
0.25	1.926	0.932	1.22	2.190	1.594	1.22	1.945	1.103	0.882	1.945	1.297	1.297	1.945
0.3	1.993	0.931	1.22	2.262	1.715	1.22	2.092	1.103	0.882	2.092	1.395	1.395	2.092
0.4	1.933	0.928	1.23	2.206	1.767	1.23	2.173	1.103	0.882	2.173	1.449	1.449	2.173
0.5	1.815	0.926	1.23	2.066	1.700	1.23	2.091	1.103	0.882	2.066	1.377	1.377	2.066
0.75	1.430	0.919	1.24	1.630	1.390	1.24	1.724	1.103	0.882	1.630	1.087	1.087	1.630
1	1.144	0.913	1.24	1.295	1.118	1.24	1.387	1.000	0.800	1.295	0.863	0.863	1.295
1.5	0.764	0.913	1.24	0.865	0.745	1.24	0.924	0.667	0.533	0.865	0.577	0.577	0.865
2	0.552	0.913	1.24	0.625	0.538	1.24	0.667	0.500	0.400	0.625	0.417	0.417	0.625
3	0.338	0.913	1.25	0.386	0.305	1.25	0.382	0.333	0.267	0.382	0.255	0.267	0.400
4	0.230	0.913	1.26	0.264	0.194	1.26	0.245	0.250	0.200	0.245	0.163	0.200	0.300
5	0.171	0.913	1.26	0.196	0.137	1.26	0.172	0.200	0.160	0.172	0.115	0.160	0.240
7.5	0.098	0.913	1.28	0.114	0.067	1.28	0.086	0.133	0.107	0.086	0.057	0.107	0.160
10	0.063	0.913	1.29	0.074	0.038	1.29	0.049	0.080	0.064	0.049	0.032	0.064	0.096

X-Axis: Period (sec)
Y-Axis: SA (g)
Number of Data Sets: 1

DATASET #1

Name:
Num Points: 21
Info:
IMR Param List:

IMR = NGAWest2 2014 Averaged No Idriss; IMR Weights = ['Abrahamson, Silva & Kamai (2014)': 0.25, 'Boore, Stewart, Seyhan & Atkinson (2014)': 0.25, 'Campbell & Bozorgnia (2014)': 0.25, 'Chiou & Youngs (2014)': 0.25]; Std Dev Type = Total; Tectonic Region = Active Shallow Crust; Additional Epistemic Uncertainty = null; Component = RotD50; Gaussian Truncation = None

Site Param List:

Longitude = -118.59298; Latitude = 34.168161; Vs30 = 260.0; Vs30 Type = Measured; Depth 2.5 km/sec = 2.3; Depth 1.0 km/sec = 250.0

IML/Prob Param List:

Map Type = IML@Prob; Probability = 0.02

Forecast Param List:

Eqk Rup Forecast = Mean UCERF3; Mean UCERF3 Presets = (POISSON ONLY) Both FM Branch Averaged; Apply Aftershock Filter = false; Aleatory Mag-Area StdDev = 0.0; Background Seismicity = Include; Treat Background Seismicity As = Point Sources; Fault Grid Spacing = 1.0; Probability Model = Poisson; Sect Upper Depth Averaging Tolerance = 100.0; Use Mean Upper Depth = true; Rup Mag Averaging Tolerance = 1.0; Rupture Rake To Use = Def. Model Mean; Fault Model(s) = Both; Ignore Cache = false

TimeSpan Param List:

Duration = 50.0

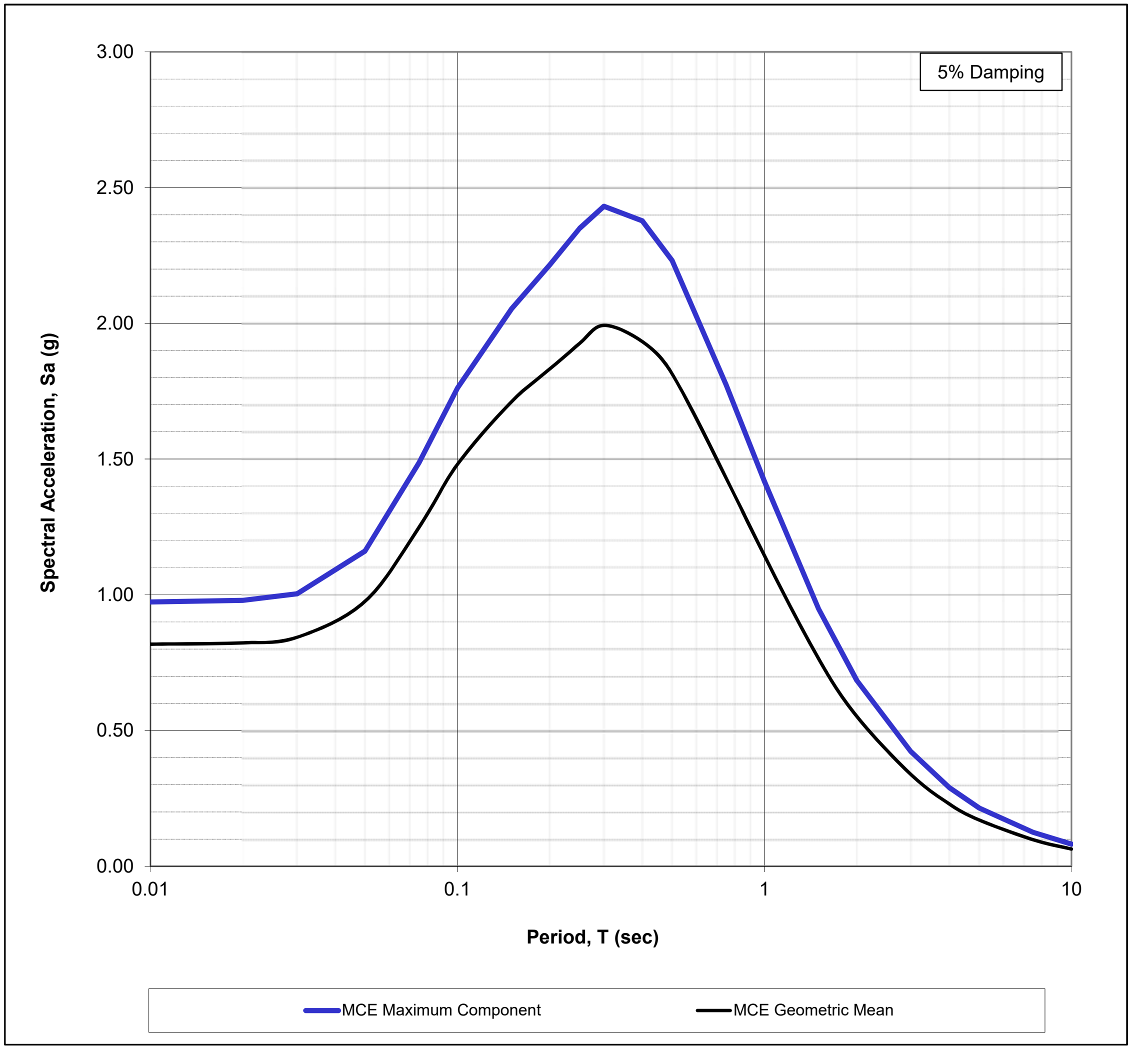
Maximum Distance = 200.0; Pt Src Dist Corr = None

X, Y Data:
0.01 0.81801087
0.02 0.8229622
0.03 0.8435231
0.05 0.975725
0.075 1.2489538
0.1 1.480494
0.15 1.711497
0.2 1.8314142
0.25 1.9261639
0.3 1.9926331
0.4 1.9326103

0.5	1.8146901
0.75	1.4302684
1.0	1.1435851
1.5	0.7644864
2.0	0.55191123
3.0	0.33799905
4.0	0.22960098
5.0	0.1705206
7.5	0.09770661
10.0	0.06314506

MCE PROBABILISTIC SPECTRA (2,475-YEAR AVERAGE RETURN INTERVAL)

Project: Proposed Self-Storage Facility
Project Number: 13589.001
Location: 21101 Ventura Boulevard



Period T (s)	MCE GEOMEAN Sa (g)	Maximum Component Factor	MCE MAX COMP Site-Specific Sa (g)
0.01	0.818	1.19	0.973
0.02	0.823	1.19	0.979
0.03	0.844	1.19	1.004
0.05	0.976	1.19	1.161
0.075	1.249	1.19	1.486
0.10	1.480	1.19	1.762
0.15	1.711	1.20	2.054
0.20	1.831	1.21	2.216
0.25	1.926	1.22	2.350
0.30	1.993	1.22	2.431
0.40	1.933	1.23	2.377
0.50	1.815	1.23	2.232
0.75	1.430	1.24	1.774
1.00	1.144	1.24	1.418
1.50	0.764	1.24	0.948
2.00	0.552	1.24	0.684
3.00	0.338	1.25	0.422
4.00	0.230	1.26	0.289
5.00	0.171	1.26	0.215
7.50	0.098	1.28	0.125
10.00	0.063	1.29	0.081

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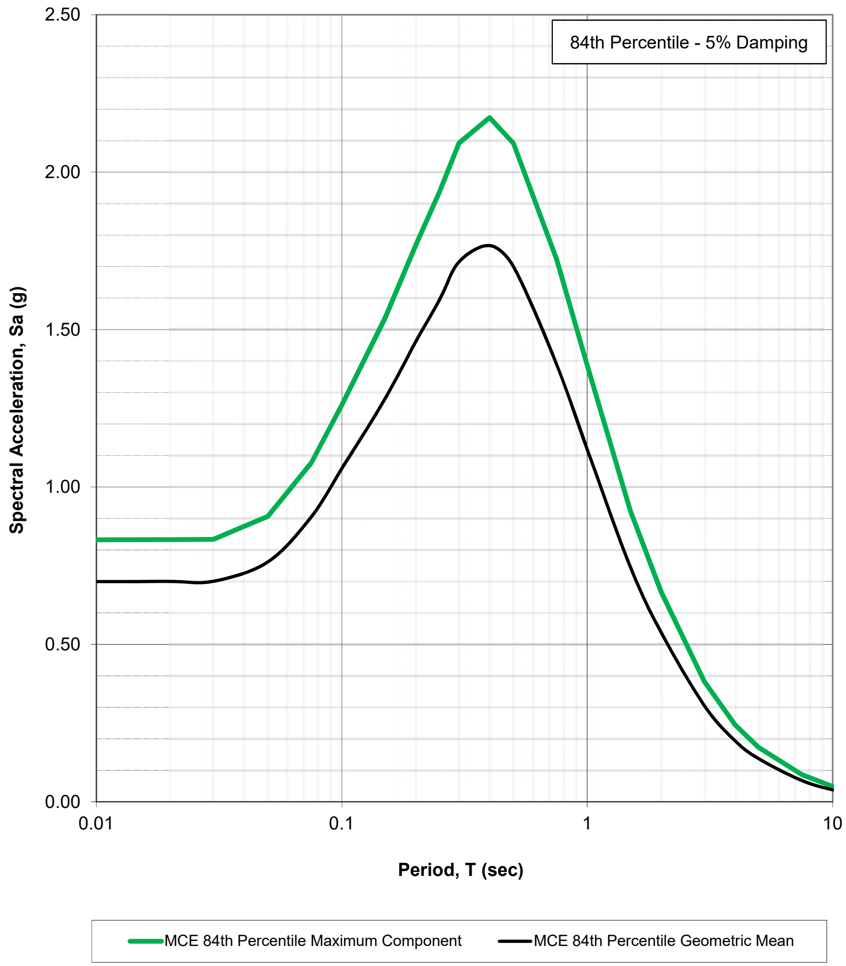
Figure D.2A

MCE DETERMINISTIC SPECTRA

Project: Proposed Self-Storage Facility

Project Number: 13589.001

Location: 21101 Ventura Boulevard



DETERMINISTIC PGA MAGNITUDE				
MC FACTOR		DSHA - 84TH PERCENTILE		
Period T (s)	Maximum Component Factor	Period T (s)	MCE GEOMEAN Sa (g)	MCE MAX COMP Sa (g)
0.01	1.19	0.01	0.700	0.832
0.02	1.19	0.02	0.700	0.833
0.03	1.19	0.03	0.701	0.834
0.05	1.19	0.05	0.762	0.907
0.075	1.19	0.075	0.905	1.077
0.10	1.19	0.10	1.059	1.260
0.15	1.20	0.15	1.280	1.536
0.20	1.21	0.20	1.462	1.768
0.25	1.22	0.25	1.594	1.937
0.30	1.22	0.30	1.715	2.092
0.40	1.23	0.40	1.767	2.173
0.50	1.23	0.50	1.700	2.091
0.75	1.24	0.75	1.390	1.724
1.00	1.24	1.00	1.118	1.387
1.50	1.24	1.50	0.745	0.924
2.00	1.24	2.00	0.538	0.667
3.00	1.25	3.00	0.305	0.382
4.00	1.26	4.00	0.194	0.245
5.00	1.26	5.00	0.137	0.172
7.50	1.28	7.50	0.067	0.086
10.00	1.29	10.00	0.038	0.049

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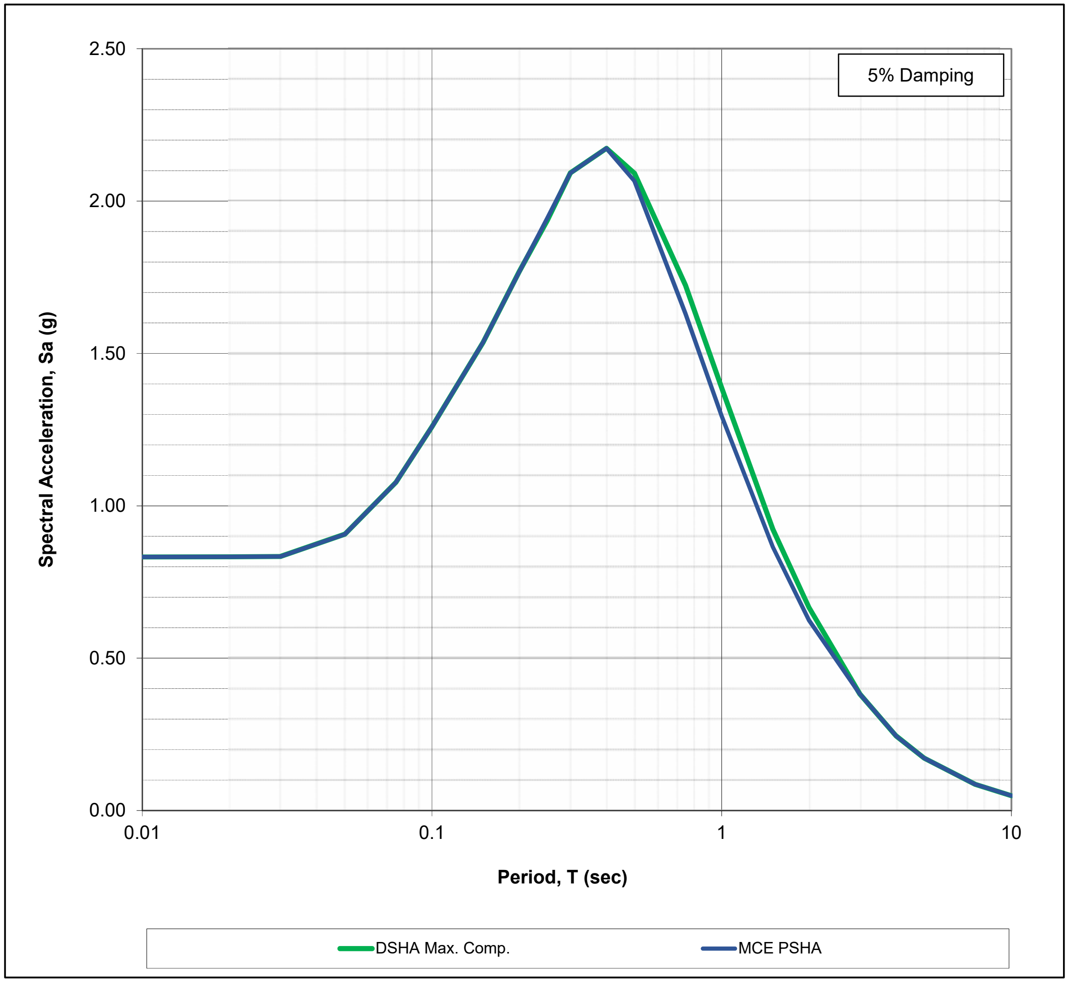
Figure D.2B

MCE SPECTRA COMPARISON - MAXIMUM HORIZONTAL COMPONENT

Project: Proposed Self-Storage Facility

Project Number: 13589.001

Location: 21101 Ventura Boulevard



DSHA		PSHA			
Period T (s)	MAX COMP. Sa (g)	Period T (s)	MCE MAX COMP. Sa (g)	Site Risk Coefficient (Cs)	MCE_R Sa (g)
0.01	0.832	0.01	0.973	0.933	0.832
0.02	0.833	0.02	0.979	0.933	0.833
0.03	0.834	0.03	1.004	0.933	0.834
0.05	0.907	0.05	1.161	0.933	0.907
0.075	1.077	0.075	1.486	0.933	1.077
0.10	1.260	0.10	1.762	0.933	1.260
0.15	1.536	0.15	2.054	0.933	1.536
0.20	1.768	0.20	2.216	0.933	1.768
0.25	1.937	0.25	2.350	0.932	1.945
0.30	2.092	0.30	2.431	0.931	2.092
0.40	2.173	0.40	2.377	0.928	2.173
0.50	2.091	0.50	2.232	0.926	2.066
0.75	1.724	0.75	1.774	0.919	1.630
1.00	1.387	1.00	1.418	0.913	1.295
1.50	0.924	1.50	0.948	0.913	0.865
2.00	0.667	2.00	0.684	0.913	0.625
3.00	0.382	3.00	0.422	0.913	0.382
4.00	0.245	4.00	0.289	0.913	0.245
5.00	0.172	5.00	0.215	0.913	0.172
7.50	0.086	7.50	0.125	0.913	0.086
10.00	0.049	10.00	0.081	0.913	0.049

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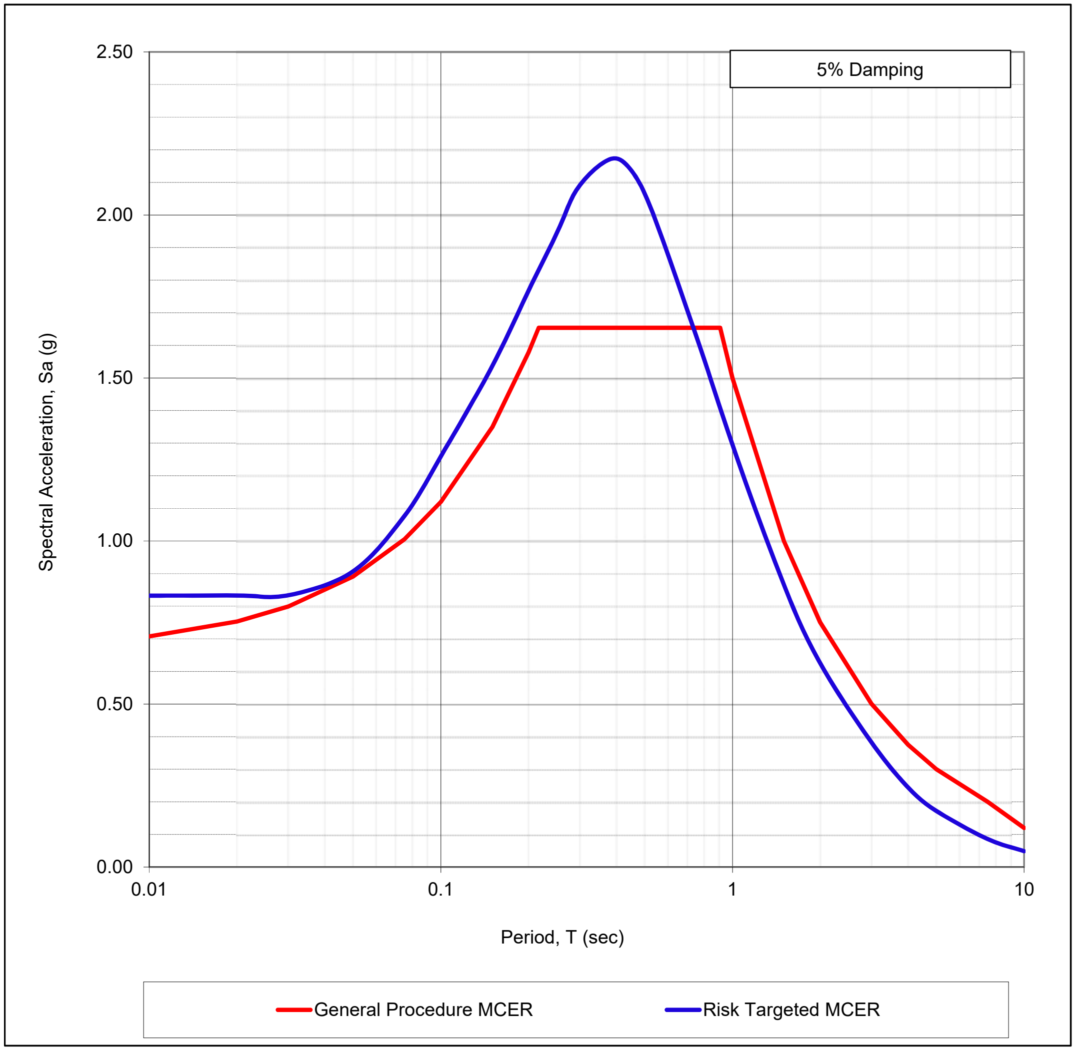
Figure D.2C

RISK TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) RESPONSE SPECTRUM

Project: Proposed Self-Storage Facility
Project Number: 13589.001
Location: 21101 Ventura Boulevard



SITE-SPECIFIC vs. GENERAL CODE-BASED SPECTRA

Period T (s)	DETERM. MCE_R Sa (g)	PROB. MCE_R Sa (g)	**Risk TGT MCE_R Sa (g)**	General Procedure Sa (g)
0.01	0.832	0.908	**0.832**	0.707
0.02	0.833	0.914	**0.833**	0.753
0.03	0.834	0.937	**0.834**	0.799
0.05	0.907	1.083	**0.907**	0.891
0.075	1.077	1.387	**1.077**	1.005
0.10	1.260	1.644	**1.260**	1.120
0.15	1.536	1.916	**1.536**	1.349
0.20	1.768	2.068	**1.768**	1.579
0.25	1.945	2.190	**1.945**	1.654
0.30	2.092	2.262	**2.092**	1.654
0.40	2.173	2.206	**2.173**	1.654
0.50	2.091	2.066	**2.066**	1.654
0.75	1.724	1.630	**1.630**	1.654
1.00	1.387	1.295	**1.295**	1.500
1.50	0.924	0.865	**0.865**	1.000
2.00	0.667	0.625	**0.625**	0.750
3.00	0.382	0.386	**0.382**	0.500
4.00	0.245	0.264	**0.245**	0.375
5.00	0.172	0.196	**0.172**	0.300
7.50	0.086	0.114	**0.086**	0.200
10.00	0.049	0.074	**0.049**	0.120

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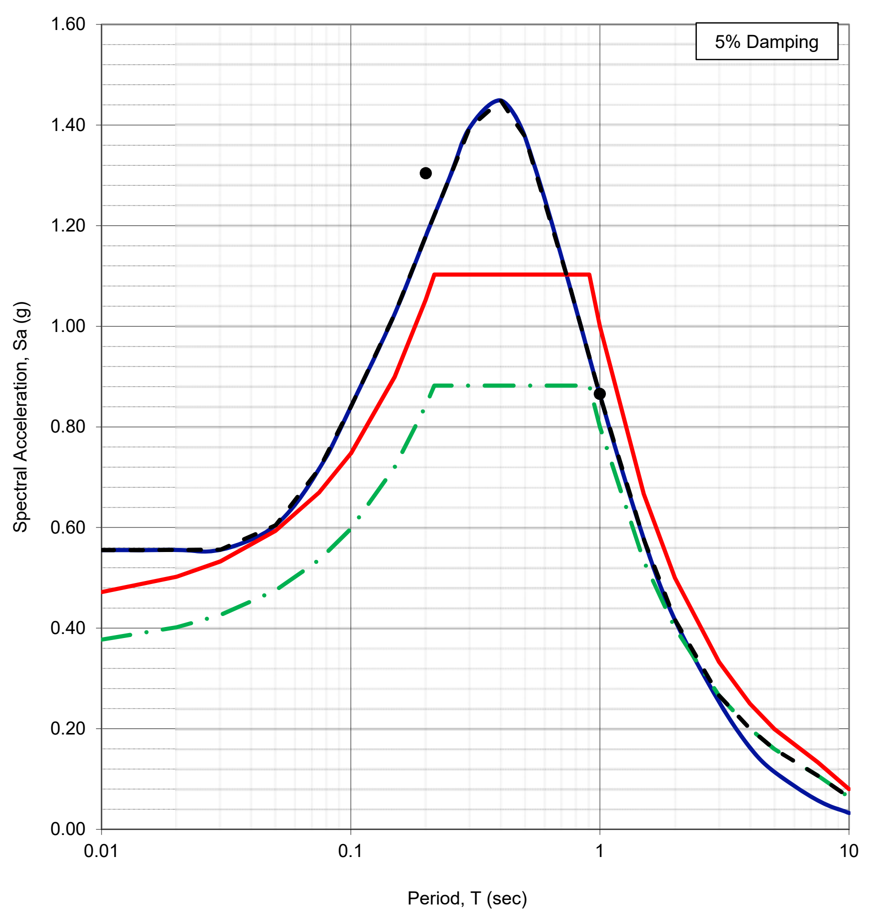
Figure D.2D

ASCE 7-16 DESIGN RESPONSE SPECTRUM AND SITE-SPECIFIC S_{DS} AND S_{D1}

Project: Proposed Self-Storage Facility
Project Number: 13589.001
Location: 21101 Ventura Boulevard



	CODE BASED GENERAL PROCEDURE SPECTRUM			RISK TGT SPECTRUM	DESIGN RESPONSE SPECTRUM
Period T (s)	GENERAL PROC. MCER CURVE Sa (g)	2/3 GENERAL PROC. MCER CURVE Sa (g)	80% * 2/3 GENERAL PROC. MCER CURVE Sa (g)	2/3*MCE_R CURVE Sa (g)	MAX of 2/3 MCE_R and 80% * 2/3 GENERAL PROC. MCER Sa (g)
0.01	0.707	0.472	0.377	0.555	0.555
0.02	0.753	0.502	0.402	0.555	0.555
0.03	0.799	0.533	0.426	0.556	0.556
0.05	0.891	0.594	0.475	0.605	0.605
0.075	1.005	0.670	0.536	0.718	0.718
0.10	1.120	0.747	0.597	0.840	0.840
0.15	1.349	0.900	0.720	1.024	1.024
0.20	1.579	1.052	0.842	1.179	1.179
0.25	1.654	1.103	0.882	1.297	1.297
0.30	1.654	1.103	0.882	1.395	1.395
0.40	1.654	1.103	0.882	1.449	1.449
0.50	1.654	1.103	0.882	1.377	1.377
0.75	1.654	1.103	0.882	1.087	1.087
1.00	1.500	1.000	0.800	0.863	0.863
1.50	1.000	0.667	0.533	0.577	0.577
2.00	0.750	0.500	0.400	0.417	0.417
3.00	0.500	0.333	0.267	0.255	0.267
4.00	0.375	0.250	0.200	0.163	0.200
5.00	0.300	0.200	0.160	0.115	0.160
7.50	0.200	0.133	0.107	0.057	0.107
10.00	0.120	0.080	0.064	0.032	0.064

S_{DS} = 1.304 g
S_{D1} = 0.865 g

Note: Based on ASCE 7-16 Section 21.4, the parameter S_{DS} shall be taken as 90% of the maximum spectral acceleration, Sa, obtained from the site-specific spectrum, at any period within the range from 0.2 to 5 s, inclusive. The parameter S_{D1} shall be taken as the maximum value of the product, TSa, for periods from 1 to 2 s for sites with Vs_{30} > 1,200 ft/s (Vs_{30} > 365.76 m/s) and for periods from 1 to 5 s for sites with Vs_{30} ≤ 1,200 ft/s (Vs_{30} ≤ 365.76 m/s). The design Sa shall not be less than 80% of 2/3 of the general procedure (ASCE 7-16 Sec 11.4.6)

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Figure D.2E

Anacapa-Dume Alt 2 (0) Fault

PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

WEIGHTED AVERAGE of 2014 NGA WEST-2 GMPEs

Last updated: 04 14 15

by Emel Seyhan, PhD, PEER & UCLA -- email: emel.seyhan@gmail.com, peer_center@berkeley.edu

This excel file will be updated as necessary on the PEER website to fix any typos or other errors. Please check the website frequently for new versions at: http://peer.berkeley.edu/ngawest2/databases/

Legend	Pre-defined option	Main input variable	Calculated variable	Input var. flag	Internal variable

GMPE averaging	Geometric	*Weighted average of the natural logarithm of the spectral values*

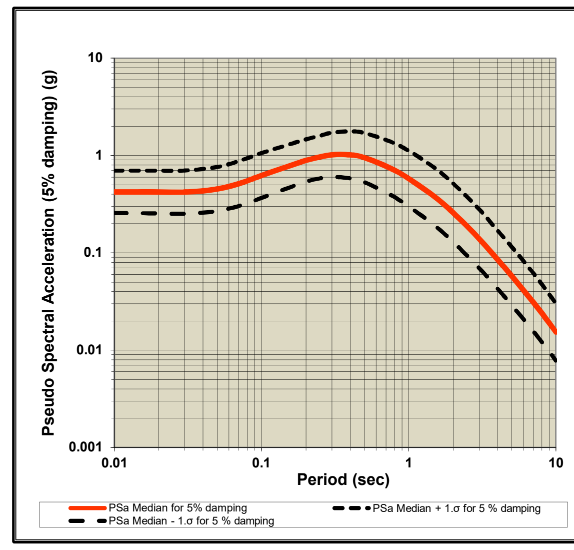
GMPEs	ASK14	BSSA14	CB14	CY14	I14
Weight	0.25	0.25	0.25	0.25	0

# of std. dev.	1	
Damping ratio (%)	5	*Modification factors are calculated in Sheet DSF*

ASK14 Abrahamson & Silva & Kamai 2014 NGA West-2 Model
BSSA14 Boore & Stewart & Seyhan & Atkinson 2014 NGA West-2 Model
CB14 Campbell & Bozorgnia 2014 NGA West-2 Model
CY14 Chiou & Youngs 2014 NGA West-2 Model
I14 Idriss 2014 NGA West-2 Model

RotD50 Horizontal Component of PGA, PGV and *IMs*

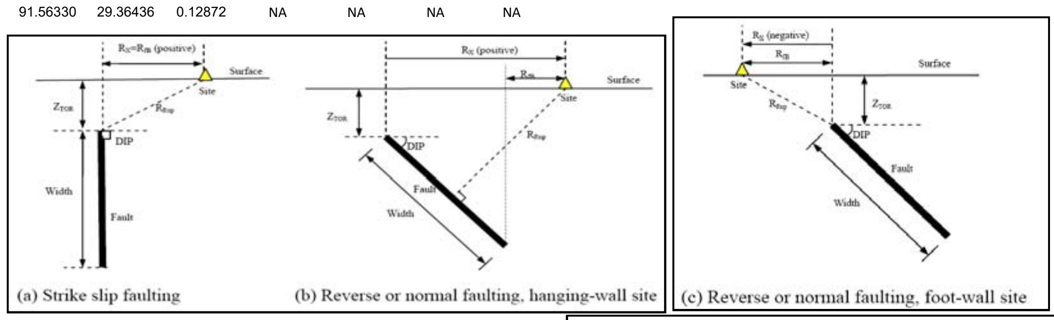
GMP	*T* (s)	Baseline: 5% Damping: PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	User defined: 5% Damping: PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	0.01	0.42364	0.69952	0.25656	0.00105	0.42364	0.69952	0.25656	0.00105
	0.02	0.42158	0.69911	0.25422	0.00419	0.42158	0.69911	0.25422	0.00419
	0.03	0.42078	0.70060	0.25272	0.00940	0.42078	0.70060	0.25272	0.00940
	0.05	0.45232	0.76208	0.26846	0.02807	0.45232	0.76208	0.26846	0.02807
	0.075	0.53081	0.90471	0.31144	0.07412	0.53188	0.90652	0.31206	0.07427
	0.1	0.62240	1.05853	0.36597	0.15450	0.62427	1.06170	0.36706	0.15497
	0.15	0.76838	1.28025	0.46116	0.42916	0.76991	1.28281	0.46208	0.43002
	0.2	0.88685	1.46155	0.53813	0.88060	0.88863	1.46447	0.53921	0.88236
	0.25	0.96298	1.59424	0.58167	1.49404	0.96490	1.59742	0.58284	1.49703
	0.3	1.01662	1.71478	0.60271	2.27127	1.01764	1.71649	0.60332	2.27354
	0.4	1.01368	1.76691	0.58155	4.02612	1.01469	1.76868	0.58213	4.03015
	0.5	0.95076	1.70037	0.53162	5.90034	0.95171	1.70207	0.53215	5.90624
	0.75	0.74155	1.38996	0.39563	10.35457	0.74155	1.38996	0.39563	10.35457
	1	0.57883	1.11837	0.29958	14.36873	0.57883	1.11837	0.29958	14.36873
	1.5	0.37667	0.74419	0.19065	21.03833	0.37705	0.74494	0.19084	21.05937
	2	0.25911	0.51739	0.12976	25.72783	0.25885	0.51687	0.12963	25.70210
	3	0.14070	0.28241	0.07010	31.43410	0.14056	0.28212	0.07003	31.40267
	4	0.08572	0.17038	0.04313	34.04637	0.08563	0.17021	0.04308	34.01233
	5	0.05814	0.11580	0.02919	36.07821	0.05796	0.11545	0.02910	35.96998
	7.5	0.02747	0.05457	0.01383	38.36307	0.02739	0.05441	0.01379	38.24798
	10	0.01541	0.03033	0.00783	38.25766	0.01535	0.03021	0.00780	38.10463
PGA (g)	0	0.42124	0.69507	0.25529	0.00105	0.42124	0.69507	0.25529	0.00105
PGV (cm/s)	-1	51.85265	91.56330	29.36436	0.12872	NA	NA	NA	NA

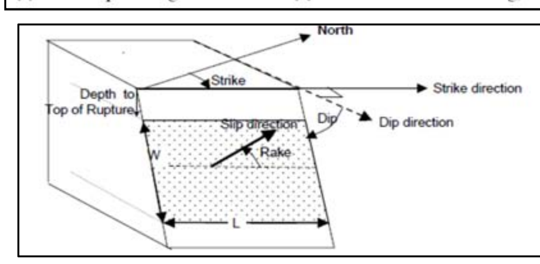


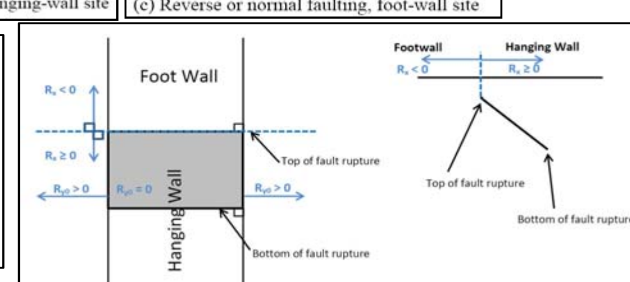
Input variables	Errors and warnings
M_w: 7.16	
R_{RUP} (km): 12.03	
R_{JB} (km): 5.28	
R_X (km): 15.68	
Ry0 (km): 999	*If unknown use 999*
V_{S30} (m/sec): 260	
U (BSSA13): 0	*1: Unspecified fault mech.*
F_{RV}: 1	*1: reverse fault*
F_{NM}: 0	*1: normal fault*
F_{HW}: 1	*1: hanging wall side*
Dip (deg): 41	
Z_{TOR} (km): 1.2	*If unknown use 999*
Z_{HYP} (km): 999	*If unknown use 999*
$Z_{1.0}$ (km): 0.25	*If unknown use 999*
$Z_{2.5}$ (km): 2.3	*If unknown use 999*
W (km): 13.99	*If unknown use 999*
Vs30Flag: measured	*Choose options for V_{S30} from the list*
F_{AS}: no	*Aftershock effect is not applicable.*
Region: California	*Choose region from the list*

Calculated Variables/Flags

ΔDPP: 0	*Always 0 for median calcs.*
PGA_r (g): 0.340	
Z_{BOT} (km) (CB14): 15	*Enter for default W calcs*
SS: 0	*auto calculated*
$V_{s30Flag}$: 1	*measured*
F_{AS}: 0	*Aftershock effect is not applicable.*
Region: 0	*California*
Option for Sa value: 1	*Weighted average of the natural logarithm of the spectral values*







Courtesy: Jennifer Donahue

Definition of Parameters

Damping ratio = Viscous damping ratio (%) See Sanaz et al. (2012) PEER Report
PSA = Pseudo-absolute acceleration response spectrum (g)
PGA = Peak ground acceleration (g)
PGV = Peak ground velocity (cm/s)
S_d = Relative displacement response spectrum (cm)
M_w = Moment magnitude
R_{RUP} = Closest distance to coseismic rupture (km), used in ASK13, CB13 and CY13. See Figures a, b and c for illustation
R_{JB} = Closest distance to surface projection of coseismic rupture (km). See Figures a, b and c for illustation
R_X = Horizontal distance from top of rupture measured perpendicular to fault strike (km). See Figures a, b and c for illustation
R_{y0} = The horizontal distance off the end of the rupture measured parallel to strike (km)
V_{S30} = The average shear-wave velocity (m/s) over a subsurface depth of 30 m
U = Unspecified-mechanism factor: 1 for unspecified; 0 otherwise
F_{RV} = Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
F_{NM} = Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal
F_{HW} = Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise
Dip = Average dip of rupture plane (degrees)
Z_{TOR} = Depth to top of coseismic rupture (km)
Z_{HYP} = Hypocentral depth from the earthquake
$Z_{1.0}$ = Depth to Vs=1 km/sec
$Z_{2.5}$ = Depth to Vs=2.5 km/sec
W = Fault rupture width (km)
$V_{s30flag}$ = 1 for measured, 0 for inferred Vs30
F_{AS} = 0 for mainshock; 1 for aftershock
Region = Specific regions considered in the models, Click on Region to see codes
ΔDPP = Directivity term, direct point parameter; uses 0 for median predictions
PGA_r ***(g)*** = Peak ground acceleration on rock (g), this specific cell is updated in the cell for BSSA14 and CB14, for others it is taken account for in the macros
Z_{BOT} ***(km)*** = The depth to the bottom of the seismogenic crust
Z_{BOR} ***(km)*** = The depth to the bottom of the rupture plane
SS = 1 for strike slip, automatically updated in the cell

Input variables with defaults (If entered 999 as input):

Red colored value: The value is used in the code when input is unknown

DEFAULTs	USER defined	ASK14	BSSA14	CB14	CY14	I14
W (km)	13.99			21.030		
$Z_{1.0}$ (km)	0.250	0.250			0.485	
$\delta Z_{1.0}$ (km)	-0.236		-0.236			
$Z_{2.5}$ (V_{S30}=1100)(km)	2.300			0.398		
$Z_{2.5}$ (V_{S30})(km)	2.300			2.070		
Z_{hyp} (km)	999.00			11.429		
Z_{tor} (km)	1.20			1.203	1.203	
Z_{BOR} (km)	-			15.000		

ACKNOWLEDGEMENTS

Nick Gregor, Bechtel
Silvia Mazzoni, Consultant

All NGA West-2 participants are acknowledged for their constructive comments and feedback.

Compton (4) Fault

PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

WEIGHTED AVERAGE of 2014 NGA WEST-2 GMPEs

Last updated: 04 14 15

by Emel Seyhan, PhD, PEER & UCLA -- email: emel.seyhan@gmail.com, peer_center@berkeley.edu

This excel file will be updated as necessary on the PEER website to fix any typos or other errors. Please check the website frequently for new versions at: http://peer.berkeley.edu/ngawest2/databases/

Legend	Pre-defined option	Main input variable	Calculated variable	Input var. flag	Internal variable

GMPE averaging	Geometric	*Weighted average of the natural logarithm of the spectral values*

GMPEs	ASK14	BSSA14	CB14	CY14	I14
Weight	0.25	0.25	0.25	0.25	0

# of std. dev.	1	
Damping ratio (%)	5	*Modification factors are calculated in Sheet DSF*

ASK14 Abrahamson & Silva & Kamai 2014 NGA West-2 Model
BSSA14 Boore & Stewart & Seyhan & Atkinson 2014 NGA West-2 Model
CB14 Campbell & Bozorgnia 2014 NGA West-2 Model
CY14 Chiou & Youngs 2014 NGA West-2 Model
I14 Idriss 2014 NGA West-2 Model

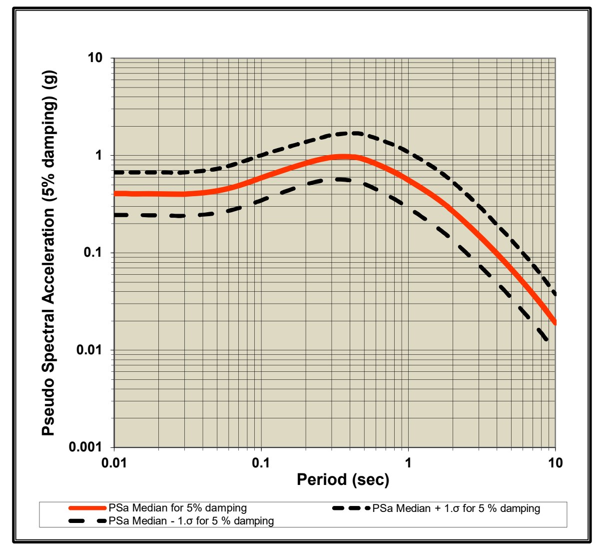
RotD50 Horizontal Component of PGA, PGV and *IMs*

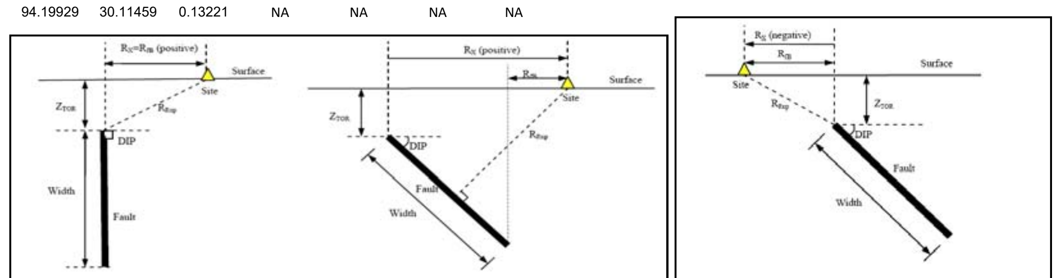
Input variables	Value	Errors and warnings
M_w	7.45	
R_{RUP} (km)	13.41	
R_{JB} (km)	8.06	
R_X (km)	10.48	
Ry0 (km)	999	*If unknown use 999*
V_{S30} (m/sec)	260	
U (BSSA13)	0	*1: Unspecified fault mech.*
F_{RV}	1	*1: reverse fault*
F_{NM}	0	*1: normal fault*
F_{HW}	1	*1: hanging wall side*
Dip (deg)	20	
Z_{TOR} (km)	5.2	*If unknown use 999*
Z_{HYP} (km)	999	*If unknown use 999*
$Z_{1.0}$ (km)	0.25	*If unknown use 999*
$Z_{2.5}$ (km)	2.3	*If unknown use 999*
W (km)	27.37	*If unknown use 999*
Vs30Flag	measured	*Choose options for V_{s30} from the list*
F_{AS}	no	*Aftershock effect is not applicable.*
Region	California	*Choose region from the list*

Calculated Variables/Flags

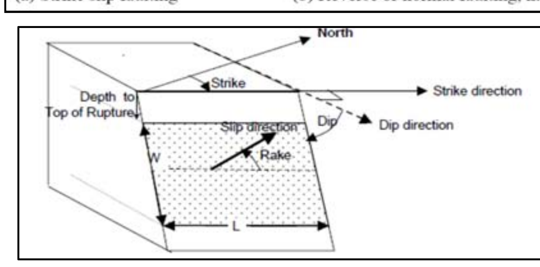
Variable	Value	Note
ΔDPP	0	*Always 0 for median calcs.*
PGA_r (g)	0.301	
Z_{BOT} (km) (CB14)	15	*Enter for default W calcs*
SS	0	*auto calculated*
$V_{s30Flag}$	1	*measured*
F_{AS}	0	*Aftershock effect is not applicable.*
Region	0	*California*
Option for Sa value	1	*Weighted average of the natural logarithm of the spectral values*

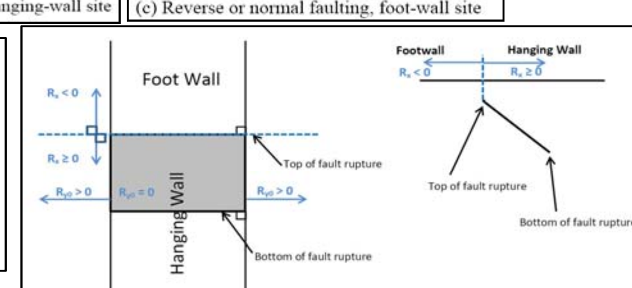
GMP	T (s)	Baseline: 5% Damping				User defined: 5% Damping			
		PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	0.01	0.40592	0.67167	0.24532	0.00101	0.40592	0.67167	0.24532	0.00101
	0.02	0.40358	0.67073	0.24283	0.00401	0.40358	0.67073	0.24283	0.00401
	0.03	0.40142	0.67004	0.24049	0.00897	0.40102	0.66937	0.24025	0.00896
	0.05	0.43201	0.72989	0.25570	0.02681	0.43201	0.72989	0.25570	0.02681
	0.075	0.50542	0.86416	0.29560	0.07057	0.50693	0.86675	0.29649	0.07078
	0.1	0.58949	1.00601	0.34542	0.14633	0.59126	1.00902	0.34646	0.14677
	0.15	0.72464	1.21171	0.43336	0.40474	0.72609	1.21413	0.43423	0.40555
	0.2	0.83114	1.37401	0.50276	0.82528	0.83280	1.37676	0.50376	0.82693
	0.25	0.90629	1.50371	0.54623	1.40610	0.90811	1.50672	0.54732	1.40891
	0.3	0.95659	1.61622	0.56618	2.13716	0.95851	1.61945	0.56731	2.14143
	0.4	0.96913	1.69098	0.55543	3.84920	0.97010	1.69267	0.55599	3.85305
	0.5	0.91517	1.63779	0.51138	5.67948	0.91609	1.63943	0.51189	5.68516
	0.75	0.70951	1.32993	0.37852	9.90710	0.70951	1.32993	0.37852	9.90710
	1	0.56005	1.08183	0.28993	13.90257	0.56005	1.08183	0.28993	13.90257
	1.5	0.37711	0.74485	0.19092	21.06262	0.37748	0.74560	0.19111	21.08368
	2	0.26941	0.53785	0.13495	26.75079	0.26887	0.53677	0.13468	26.69729
	3	0.15219	0.30546	0.07583	34.00168	0.15204	0.30516	0.07575	33.96768
	4	0.09766	0.19412	0.04914	38.79020	0.09757	0.19393	0.04909	38.75141
	5	0.06865	0.13674	0.03447	42.60420	0.06844	0.13633	0.03436	42.47639
	7.5	0.03380	0.06714	0.01702	47.19898	0.03367	0.06688	0.01695	47.01018
	10	0.01918	0.03775	0.00975	47.61795	0.01911	0.03760	0.00971	47.42748
PGA (g)	0	0.40373	0.66755	0.24417	0.00100	0.40373	0.66755	0.24417	0.00100
PGV (cm/s)	-1	53.26137	94.19929	30.11459	0.13221	NA	NA	NA	NA





(a) Strike slip faulting (b) Reverse or normal faulting, hanging-wall site (c) Reverse or normal faulting, foot-wall site





Courtesy: Jennifer Donahue

Definition of Parameters

- ***Damping ratio*** = Viscous damping ratio (%) See Sanaz et al. (2012) PEER Report
- ***PSA*** = Pseudo-absolute acceleration response spectrum (g)
- ***PGA*** = Peak ground acceleration (g)
- ***PGV*** = Peak ground velocity (cm/s)
- S_d = Relative displacement response spectrum (cm)
- M_w = Moment magnitude
- R_{RUP} = Closest distance to coseismic rupture (km), used in ASK13, CB13 and CY13. See Figures a, b and c for illustation
- R_{JB} = Closest distance to surface projection of coseismic rupture (km). See Figures a, b and c for illustation
- R_X = Horizontal distance from top of rupture measured perpendicular to fault strike (km). See Figures a, b and c for illustation
- R_{y0} = The horizontal distance off the end of the rupture measured parallel to strike (km)
- V_{S30} = The average shear-wave velocity (m/s) over a subsurface depth of 30 m
- ***U*** = Unspecified-mechanism factor: 1 for unspecified; 0 otherwise
- F_{RV} = Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
- F_{NM} = Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal
- F_{HW} = Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise
- ***Dip*** = Average dip of rupture plane (degrees)
- Z_{TOR} = Depth to top of coseismic rupture (km)
- Z_{HYP} = Hypocentral depth from the earthquake
- $Z_{1.0}$ = Depth to Vs=1 km/sec
- $Z_{2.5}$ = Depth to Vs=2.5 km/sec
- ***W*** = Fault rupture width (km)
- $V_{s30flag}$ = 1 for measured, 0 for inferred Vs30
- F_{AS} = 0 for mainshock; 1 for aftershock
- ***Region*** = Specific regions considered in the models, Click on Region to see codes
- ***ΔDPP*** = Directivity term, direct point parameter; uses 0 for median predictions
- PGA_r ***(g)*** = Peak ground acceleration on rock (g), this specific cell is updated in the cell for BSSA14 and CB14, for others it is taken account for in the macros
- Z_{BOT} ***(km)*** = The depth to the bottom of the seismogenic crust
- Z_{BOR} ***(km)*** = The depth to the bottom of the rupture plane
- ***SS*** = 1 for strike slip, automatically updated in the cell

Input variables with defaults (If entered 999 as input):

Red colored value: The value is used in the code when input is unknown

DEFAULTs	USER defined	ASK14	BSSA14	CB14	CY14	I14
W (km)	27.37			42.251		
$Z_{1.0}$ (km)	0.250	0.250			0.485	
$\delta Z_{1.0}$ (km)	-0.236		-0.236			
$Z_{2.5}$ (V_{S30}=1100)(km)	2.300			0.398		
$Z_{2.5}$ (V_{S30})(km)	2.300			2.070		
Z_{hyp} (km)	999.00			5.000		
Z_{tor} (km)	5.20			0.549	0.549	
Z_{BOR} (km)	-			15.000		

ACKNOWLEDGEMENTS

Nick Gregor, Bechtel
Silvia Mazzoni, Consultant

All NGA West-2 participants are acknowledged for their constructive comments and feedback.

Hollywood (2) Fault

PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

WEIGHTED AVERAGE of 2014 NGA WEST-2 GMPEs

Last updated: 04 14 15

by Emel Seyhan, PhD, PEER & UCLA -- email: emel.seyhan@gmail.com, peer_center@berkeley.edu

This excel file will be updated as necessary on the PEER website to fix any typos or other errors. Please check the website frequently for new versions at: http://peer.berkeley.edu/ngawest2/databases/

Legend	Pre-defined option	Main input variable	Calculated variable	Input var. flag	Internal variable

GMPE averaging	Geometric	*Weighted average of the natural logarithm of the spectral values*

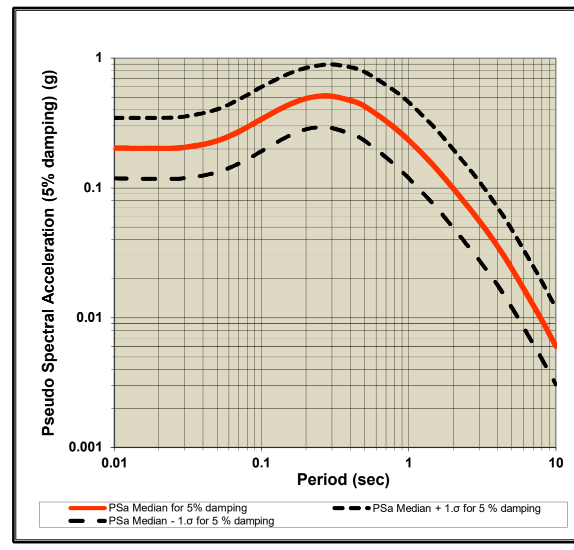
GMPEs	ASK14	BSSA14	CB14	CY14	I14
Weight	0.25	0.25	0.25	0.25	0

# of std. dev.	1	
Damping ratio (%)	5	*Modification factors are calculated in Sheet DSF*

ASK14 Abrahamson & Silva & Kamai 2014 NGA West-2 Model
BSSA14 Boore & Stewart & Seyhan & Atkinson 2014 NGA West-2 Model
CB14 Campbell & Bozorgnia 2014 NGA West-2 Model
CY14 Chiou & Youngs 2014 NGA West-2 Model
I14 Idriss 2014 NGA West-2 Model

RotD50 Horizontal Component of PGA, PGV and *IMs*

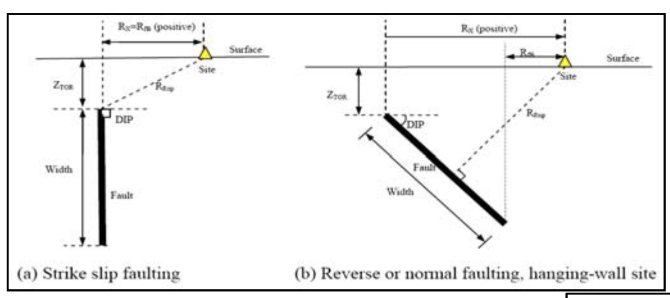
GMP	T (s)	Baseline: 5% Damping PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	User defined: 5% Damping PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	0.01	0.20269	0.34639	0.11860	0.00050	0.20269	0.34639	0.11860	0.00050
	0.02	0.20184	0.34664	0.11752	0.00200	0.20184	0.34664	0.11752	0.00200
	0.03	0.20555	0.35497	0.11902	0.00459	0.20555	0.35497	0.11902	0.00459
	0.05	0.23054	0.40318	0.13182	0.01431	0.23077	0.40358	0.13196	0.01432
	0.075	0.28239	0.49952	0.15964	0.03943	0.28295	0.50052	0.15996	0.03951
	0.1	0.33886	0.59915	0.19165	0.08412	0.33988	0.60095	0.19222	0.08437
	0.15	0.43177	0.75117	0.24818	0.24116	0.43263	0.75267	0.24867	0.24164
	0.2	0.48735	0.84042	0.28261	0.48392	0.48833	0.84210	0.28318	0.48488
	0.25	0.50877	0.88108	0.29378	0.78935	0.50928	0.88196	0.29408	0.79014
	0.3	0.50869	0.89553	0.28895	1.13647	0.50919	0.89643	0.28924	1.13761
	0.4	0.47249	0.85232	0.26193	1.87662	0.47296	0.85317	0.26219	1.87850
	0.5	0.42613	0.78467	0.23142	2.64455	0.42656	0.78545	0.23165	2.64720
	0.75	0.30861	0.59082	0.16120	4.30918	0.30861	0.59082	0.16120	4.30918
	1	0.23287	0.45706	0.11864	5.78063	0.23287	0.45706	0.11864	5.78063
	1.5	0.14556	0.29020	0.07302	8.13022	0.14571	0.29049	0.07309	8.13835
	2	0.09929	0.19922	0.04949	9.85891	0.09919	0.19902	0.04944	9.84905
	3	0.05621	0.11311	0.02794	12.55869	0.05616	0.11300	0.02791	12.54613
	4	0.03576	0.07125	0.01795	14.20463	0.03569	0.07111	0.01792	14.17622
	5	0.02414	0.04819	0.01209	14.97976	0.02404	0.04799	0.01204	14.91984
	7.5	0.01098	0.02186	0.00552	15.33112	0.01094	0.02177	0.00549	15.26980
	10	0.00605	0.01193	0.00307	15.02025	0.00603	0.01189	0.00306	14.96017
PGA (g)	0	0.20167	0.34439	0.11809	0.00050	0.20167	0.34439	0.11809	0.00050
PGV (cm/s)	-1	22.67563	40.38152	12.73315	0.05629	NA	NA	NA	NA



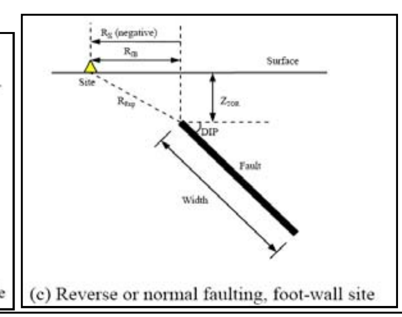
Input variables	Errors and warnings
M_w: 6.7	
R_{RUP} (km): 18.13	
R_{JB} (km): 14.77	
R_X (km): 15.57	
Ry0 (km): 999	*If unknown use 999*
V_{S30} (m/sec): 260	
U (BSSA13): 0	*1: Unspecified fault mech.*
F_{RV}: 0	*1: reverse fault*
F_{NM}: 0	*1: normal fault*
F_{HW}: 0	*1: hanging wall side*
Dip (deg): 70	
Z_{TOR} (km): 0	*If unknown use 999*
Z_{HYP} (km): 999	*If unknown use 999*
$Z_{1.0}$ (km): 0.23	*If unknown use 999*
$Z_{2.5}$ (km): 2.3	*If unknown use 999*
W (km): 16.57	*If unknown use 999*
Vs30Flag: measured	*Choose options for V_{S30} from the list*
F_{AS}: no	*Aftershock effect is not applicable.*
Region: California	*Choose region from the list*

Calculated Variables/Flags

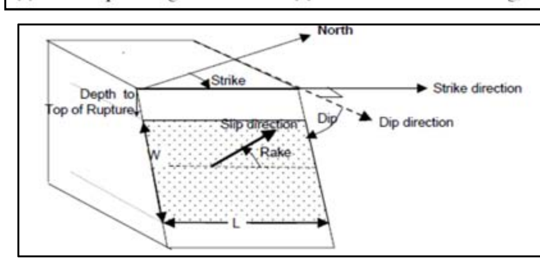
ΔDPP: 0	*Always 0 for median calcs.*
PGA_r (g): 0.169	
Z_{BOT} (km) (CB14): 15	*Enter for default W calcs*
SS: 1	*auto calculated*
$V_{s30Flag}$: 1	*measured*
F_{AS}: 0	*Aftershock effect is not applicable.*
Region: 0	*California*
Option for Sa value: 1	*Weighted average of the natural logarithm of the spectral values*

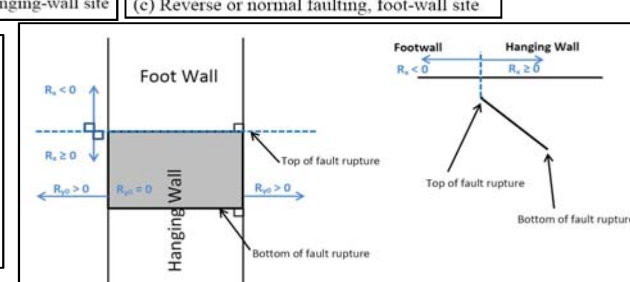


(a) Strike slip faulting (b) Reverse or normal faulting, hanging-wall site



(c) Reverse or normal faulting, foot-wall site





Courtesy: Jennifer Donahue

Definition of Parameters

Damping ratio = Viscous damping ratio (%) See Sanaz et al. (2012) PEER Report
PSA = Pseudo-absolute acceleration response spectrum (g)
PGA = Peak ground acceleration (g)
PGV = Peak ground velocity (cm/s)
S_d = Relative displacement response spectrum (cm)
M_w = Moment magnitude
R_{RUP} = Closest distance to coseismic rupture (km), used in ASK13, CB13 and CY13. See Figures a, b and c for illustation
R_{JB} = Closest distance to surface projection of coseismic rupture (km). See Figures a, b and c for illustation
R_X = Horizontal distance from top of rupture measured perpendicular to fault strike (km). See Figures a, b and c for illustation
R_{y0} = The horizontal distance off the end of the rupture measured parallel to strike (km)
V_{S30} = The average shear-wave velocity (m/s) over a subsurface depth of 30 m
U = Unspecified-mechanism factor: 1 for unspecified; 0 otherwise
F_{RV} = Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
F_{NM} = Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal
F_{HW} = Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise
Dip = Average dip of rupture plane (degrees)
Z_{TOR} = Depth to top of coseismic rupture (km)
Z_{HYP} = Hypocentral depth from the earthquake
$Z_{1.0}$ = Depth to Vs=1 km/sec
$Z_{2.5}$ = Depth to Vs=2.5 km/sec
W = Fault rupture width (km)
$V_{s30flag}$ = 1 for measured, 0 for inferred Vs30
F_{AS} = 0 for mainshock; 1 for aftershock
Region = Specific regions considered in the models, Click on Region to see codes
ΔDPP = Directivity term, direct point parameter; uses 0 for median predictions
PGA_r (g) = Peak ground acceleration on rock (g), this specific cell is updated in the cell for BSSA14 and CB14, for others it is taken account for in the macros
Z_{BOT} (km) = The depth to the bottom of the seismogenic crust
Z_{BOR} (km) = The depth to the bottom of the rupture plane
SS = 1 for strike slip, automatically updated in the cell

Input variables with defaults (If entered 999 as input):

Red colored value: The value is used in the code when input is unknown

DEFAULTs	USER defined	ASK14	BSSA14	CB14	CY14	I14
W (km)	16.57			15.430		
$Z_{1.0}$ (km)	0.230	0.230			0.485	
$\delta Z_{1.0}$ (km)	-0.256		-0.256			
$Z_{2.5}$ (V_{S30}=1100)(km)	2.300			0.398		
$Z_{2.5}$ (V_{S30})(km)	2.300			2.070		
Z_{hyp} (km)	999.00			10.237		
Z_{tor} (km)	0.00			0.501	0.501	
Z_{BOR} (km)	-			15.000		

ACKNOWLEDGEMENTS

Nick Gregor, Bechtel
Silvia Mazzoni, Consultant

All NGA West-2 participants are acknowledged for their constructive comments and feedback.

Malibu Coast Alt 1 (1) Fault

PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

WEIGHTED AVERAGE of 2014 NGA WEST-2 GMPEs

Last updated: 04 14 15

by Emel Seyhan, PhD, PEER & UCLA -- email: emel.seyhan@gmail.com, peer_center@berkeley.edu

This excel file will be updated as necessary on the PEER website to fix any typos or other errors. Please check the website frequently for new versions at: http://peer.berkeley.edu/ngawest2/databases/

Legend	Pre-defined option	Main input variable	Calculated variable	Input var. flag	Internal variable

GMPE averaging	Geometric	Weighted average of the natural logarithm of the spectral values

GMPEs	ASK14	BSSA14	CB14	CY14	I14
Weight	0.25	0.25	0.25	0.25	0

# of std. dev.	1	
Damping ratio (%)	5	Modification factors are calculated in Sheet DSF

ASK14 Abrahamson & Silva & Kamai 2014 NGA West-2 Model
BSSA14 Boore & Stewart & Seyhan & Atkinson 2014 NGA West-2 Model
CB14 Campbell & Bozorgnia 2014 NGA West-2 Model
CY14 Chiou & Youngs 2014 NGA West-2 Model
I14 Idriss 2014 NGA West-2 Model

Input variables	Value	Errors and warnings
M_w	6.64	
R_{RUP} (km)	14.23	
R_{JB} (km)	9.97	
R_X (km)	14.27	
Ry0 (km)	999	If unknown use 999
V_{S30} (m/sec)	260	
U (BSSA13)	0	1: Unspecified fault mech.
F_{RV}	0	1: reverse fault
F_{NM}	0	1: normal fault
F_{HW}	1	1: hanging wall side
Dip (deg)	75	
Z_{TOR} (km)	0	If unknown use 999
Z_{HYP} (km)	999	If unknown use 999
$Z_{1.0}$ (km)	0.25	If unknown use 999
$Z_{2.5}$ (km)	2.3	If unknown use 999
W (km)	15.51	If unknown use 999
Vs30Flag	measured	Choose options for V_{S30} from the list
F_{AS}	no	Aftershock effect is not applicable.
Region	California	Choose region from the list

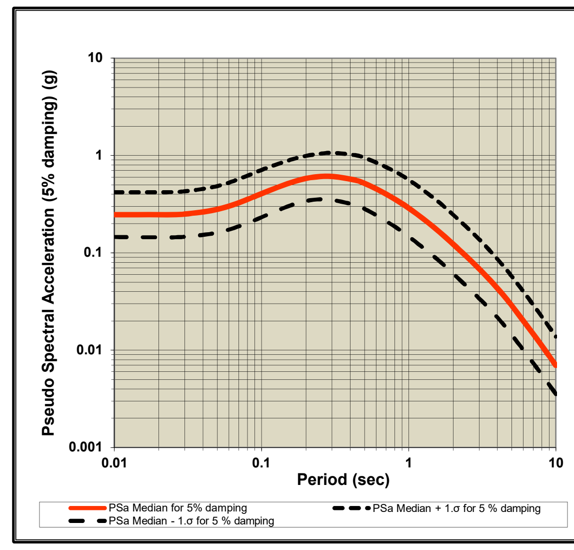
Calculated Variables/Flags

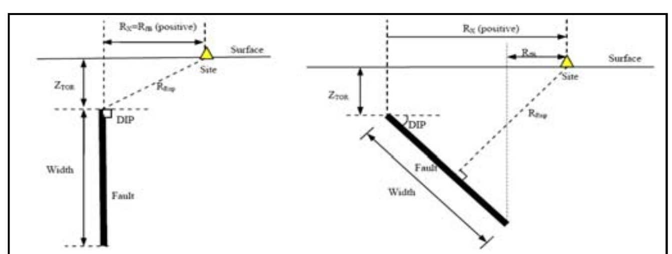
Variable	Value	Comment
ΔDPP	0	Always 0 for median calcs.
PGA_r (g)	0.220	
Z_{BOT} (km) (CB14)	15	Enter for default W calcs
SS	1	auto calculated
$V_{s30Flag}$	1	measured
F_{AS}	0	Aftershock effect is not applicable.
Region	0	California
Option for Sa value	1	Weighted average of the natural logarithm of the spectral values

RotD50 Horizontal Component of PGA, PGV and *IMs*

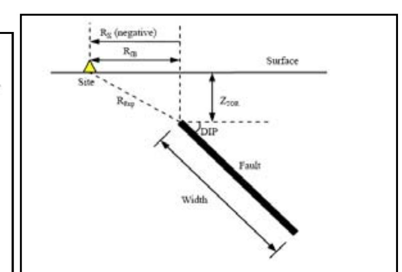
PSa (g), S_d (cm)

GMP T (s)	Baseline: 5% Damping PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	User defined: 5% Damping PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
0.01	0.24683	0.41879	0.14548	0.00061	0.24683	0.41879	0.14548	0.00061
0.02	0.24612	0.41956	0.14437	0.00244	0.24612	0.41956	0.14437	0.00244
0.03	0.25023	0.42865	0.14607	0.00559	0.25023	0.42865	0.14607	0.00559
0.05	0.27940	0.48421	0.16122	0.01734	0.27940	0.48421	0.16122	0.01734
0.075	0.33939	0.59450	0.19375	0.04739	0.34007	0.59568	0.19414	0.04748
0.1	0.40506	0.70871	0.23151	0.10055	0.40627	0.71083	0.23220	0.10085
0.15	0.51304	0.88249	0.29826	0.28655	0.51406	0.88425	0.29885	0.28712
0.2	0.58002	0.98919	0.34010	0.57593	0.58118	0.99116	0.34078	0.57708
0.25	0.60732	1.04115	0.35427	0.94225	0.60793	1.04219	0.35462	0.94319
0.3	0.60981	1.06397	0.34951	1.36240	0.61042	1.06503	0.34986	1.36376
0.4	0.57147	1.02401	0.31892	2.26976	0.57204	1.02504	0.31924	2.27203
0.5	0.51818	0.94906	0.28292	3.21577	0.51870	0.95001	0.28320	3.21899
0.75	0.37919	0.72352	0.19872	5.29470	0.37919	0.72352	0.19872	5.29470
1	0.28765	0.56338	0.14687	7.14052	0.28765	0.56338	0.14687	7.14052
1.5	0.18019	0.35894	0.09046	10.06447	0.18037	0.35930	0.09055	10.07454
2	0.12314	0.24707	0.06138	12.22750	0.12302	0.24682	0.06132	12.21528
3	0.06887	0.13864	0.03422	15.38748	0.06881	0.13851	0.03418	15.37209
4	0.04329	0.08627	0.02172	17.19231	0.04324	0.08619	0.02170	17.17512
5	0.02893	0.05779	0.01449	17.95598	0.02882	0.05756	0.01443	17.88416
7.5	0.01281	0.02552	0.00643	17.89336	0.01278	0.02545	0.00641	17.83968
10	0.00701	0.01382	0.00355	17.39390	0.00698	0.01377	0.00354	17.32432
PGA (g) 0	0.24554	0.41629	0.14482	0.00061	0.24554	0.41629	0.14482	0.00061
PGV (cm/s) -1	27.67701	49.18066	15.57557	0.06870	NA	NA	NA	NA

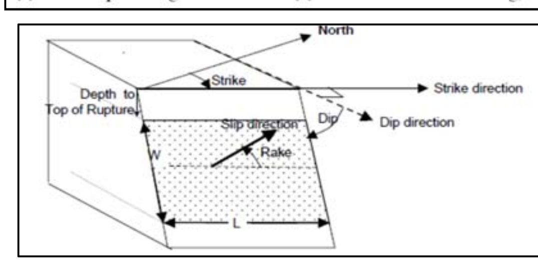


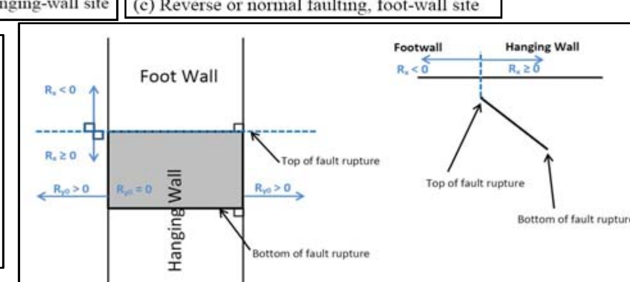


(a) Strike slip faulting (b) Reverse or normal faulting, hanging-wall site



(c) Reverse or normal faulting, foot-wall site





Courtesy: Jennifer Donahue

Definition of Parameters

- **Damping ratio** = Viscous damping ratio (%) See Sanaz et al. (2012) PEER Report
- **PSA** = Pseudo-absolute acceleration response spectrum (g)
- **PGA** = Peak ground acceleration (g)
- **PGV** = Peak ground velocity (cm/s)
- S_d = Relative displacement response spectrum (cm)
- M_w = Moment magnitude
- R_{RUP} = Closest distance to coseismic rupture (km), used in ASK13, CB13 and CY13. See Figures a, b and c for illustation
- R_{JB} = Closest distance to surface projection of coseismic rupture (km). See Figures a, b and c for illustation
- R_X = Horizontal distance from top of rupture measured perpendicular to fault strike (km). See Figures a, b and c for illustation
- R_{y0} = The horizontal distance off the end of the rupture measured parallel to strike (km)
- V_{S30} = The average shear-wave velocity (m/s) over a subsurface depth of 30 m
- **U** = Unspecified-mechanism factor: 1 for unspecified; 0 otherwise
- F_{RV} = Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
- F_{NM} = Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal
- F_{HW} = Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise
- **Dip** = Average dip of rupture plane (degrees)
- Z_{TOR} = Depth to top of coseismic rupture (km)
- Z_{HYP} = Hypocentral depth from the earthquake
- $Z_{1.0}$ = Depth to Vs=1 km/sec
- $Z_{2.5}$ = Depth to Vs=2.5 km/sec
- **W** = Fault rupture width (km)
- $V_{s30flag}$ = 1 for measured, 0 for inferred Vs30
- F_{AS} = 0 for mainshock; 1 for aftershock
- **Region** = Specific regions considered in the models, Click on Region to see codes
- **ΔDPP** = Directivity term, direct point parameter; uses 0 for median predictions
- PGA_r (g) = Peak ground acceleration on rock (g), this specific cell is updated in the cell for BSSA14 and CB14, for others it is taken account for in the macros
- Z_{BOT} (km) = The depth to the bottom of the seismogenic crust
- Z_{BOR} (km) = The depth to the bottom of the rupture plane
- **SS** = 1 for strike slip, automatically updated in the cell

Input variables with defaults (If entered 999 as input):

Red colored value: The value is used in the code when input is unknown

DEFAULTs	USER defined	ASK14	BSSA14	CB14	CY14	I14
W (km)	15.51			14.906		
$Z_{1.0}$ (km)	0.250	0.250			0.485	
$\delta Z_{1.0}$ (km)	-0.236		-0.236			
$Z_{2.5}$ (V_{S30}=1100)(km)	2.300			0.398		
$Z_{2.5}$ (V_{S30})(km)	2.300			2.070		
Z_{hyp} (km)	999.00			9.780		
Z_{tor} (km)	0.00			0.602	0.602	
Z_{BOR} (km)	-			15.000		

ACKNOWLEDGEMENTS

Nick Gregor, Bechtel
Silvia Mazzoni, Consultant

All NGA West-2 participants are acknowledged for their constructive comments and feedback.

Northridge (2) Fault

PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

WEIGHTED AVERAGE of 2014 NGA WEST-2 GMPEs

Last updated: 04 14 15

by Emel Seyhan, PhD, PEER & UCLA -- email: emel.seyhan@gmail.com, peer_center@berkeley.edu

This excel file will be updated as necessary on the PEER website to fix any typos or other errors. Please check the website frequently for new versions at: http://peer.berkeley.edu/ngawest2/databases/

Legend	Pre-defined option	Main input variable	Calculated variable	Input var. flag	Internal variable

GMPE averaging	Geometric	Weighted average of the natural logarithm of the spectral values

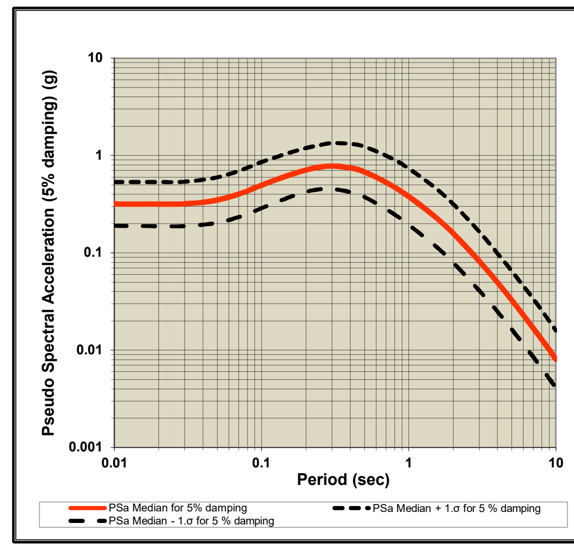
GMPEs	ASK14	BSSA14	CB14	CY14	I14
Weight	0.25	0.25	0.25	0.25	0

# of std. dev.	1	
Damping ratio (%)	5	Modification factors are calculated in Sheet DSF

ASK14 Abrahamson & Silva & Kamai 2014 NGA West-2 Model
BSSA14 Boore & Stewart & Seyhan & Atkinson 2014 NGA West-2 Model
CB14 Campbell & Bozorgnia 2014 NGA West-2 Model
CY14 Chiou & Youngs 2014 NGA West-2 Model
I14 Idriss 2014 NGA West-2 Model

RotD50 Horizontal Component of PGA, PGV and *IMs*

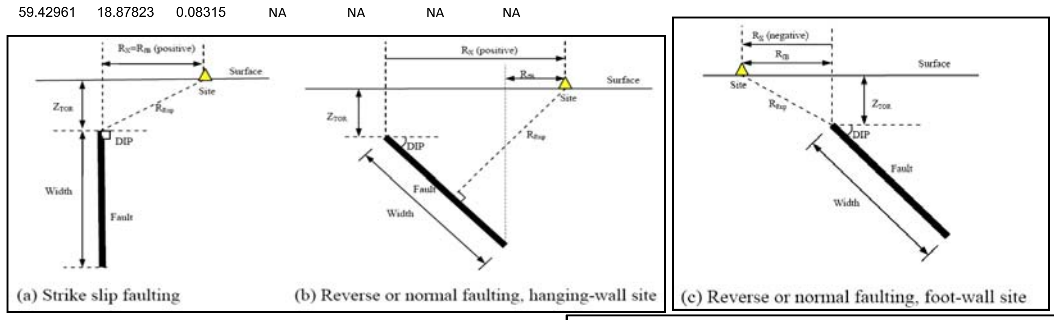
GMP	*T* (s)	Baseline: 5% Damping				User defined: 5% Damping			
		PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	0.01	0.31800	0.53312	0.18968	0.00079	0.31800	0.53312	0.18968	0.00079
	0.02	0.31649	0.53300	0.18793	0.00314	0.31649	0.53300	0.18793	0.00314
	0.03	0.31819	0.53819	0.18812	0.00711	0.31819	0.53819	0.18812	0.00711
	0.05	0.34811	0.59542	0.20352	0.02160	0.34811	0.59542	0.20352	0.02160
	0.075	0.41634	0.71978	0.24082	0.05814	0.41717	0.72122	0.24130	0.05825
	0.1	0.49486	0.85405	0.28673	0.12284	0.49634	0.85662	0.28759	0.12321
	0.15	0.61946	1.04942	0.36566	0.34599	0.62070	1.05152	0.36639	0.34668
	0.2	0.70919	1.18990	0.42268	0.70419	0.71061	1.19228	0.42352	0.70559
	0.25	0.75841	1.27884	0.44977	1.17665	0.75917	1.28012	0.45022	1.17783
	0.3	0.77912	1.33869	0.45344	1.74065	0.77990	1.34003	0.45390	1.74239
	0.4	0.74502	1.31905	0.42079	2.95905	0.74576	1.32037	0.42121	2.96200
	0.5	0.67927	1.23150	0.37467	4.21549	0.67995	1.23273	0.37504	4.21971
	0.75	0.50106	0.94901	0.26455	6.99642	0.50106	0.94901	0.26455	6.99642
	1	0.37739	0.73494	0.19378	9.36810	0.37739	0.73494	0.19378	9.36810
	1.5	0.23646	0.46926	0.11915	13.20709	0.23670	0.46973	0.11927	13.22030
	2	0.15929	0.31876	0.07960	15.81700	0.15913	0.31844	0.07952	15.80118
	3	0.08292	0.16659	0.04127	18.52569	0.08284	0.16643	0.04123	18.50716
	4	0.04982	0.09910	0.02504	19.78660	0.04972	0.09891	0.02499	19.74703
	5	0.03283	0.06544	0.01647	20.37370	0.03273	0.06525	0.01642	20.31257
	7.5	0.01483	0.02948	0.00746	20.70916	0.01477	0.02937	0.00743	20.62632
	10	0.00811	0.01598	0.00412	20.14053	0.00807	0.01590	0.00410	20.03983
PGA (g)	0	0.31629	0.52987	0.18880	0.00079	0.31629	0.52987	0.18880	0.00079
PGV (cm/s)	-1	33.49516	59.42961	18.87823	0.08315	NA	NA	NA	NA

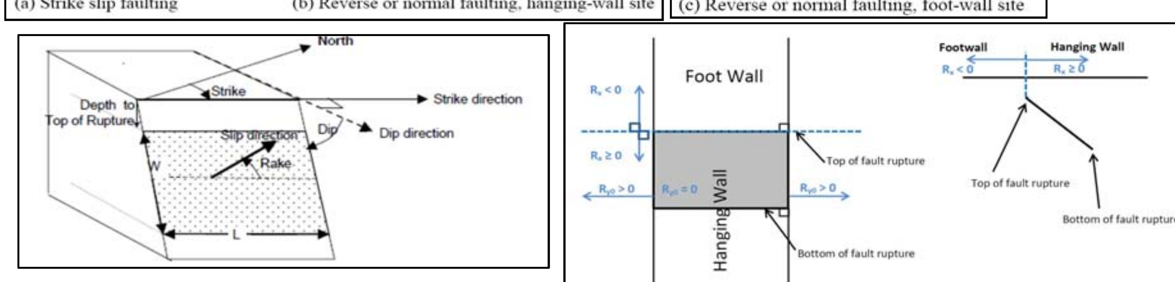


Input variables	Errors and warnings
M_w: 6.89	
R_{RUP} (km): 18.27	
R_{JB} (km): 7.88	
R_X (km): 19.79	
Ry0 (km): 999	If unknown use 999
V_{S30} (m/sec): 260	
U (BSSA13): 0	1: Unspecified fault mech.
F_{RV}: 1	1: reverse fault
F_{NM}: 0	1: normal fault
F_{HW}: 1	1: hanging wall side
Dip (deg): 35	
Z_{TOR} (km): 7.4	If unknown use 999
Z_{HYP} (km): 999	If unknown use 999
$Z_{1.0}$ (km): 0.25	If unknown use 999
$Z_{2.5}$ (km): 2.3	If unknown use 999
W (km): 14.75	If unknown use 999
Vs30Flag: measured	Choose options for V_{s30} from the list
F_{AS}: no	Aftershock effect is not applicable.
Region: California	Choose region from the list

Calculated Variables/Flags

ΔDPP: 0	Always 0 for median calcs.
PGA_r (g): 0.264	
Z_{BOT} (km) (CB14): 15	Enter for default W calcs
SS: 0	auto calculated
$V_{s30Flag}$: 1	measured
F_{AS}: 0	Aftershock effect is not applicable.
Region: 0	California
Option for Sa value: 1	Weighted average of the natural logarithm of the spectral values





Courtesy: Jennifer Donahue

Definition of Parameters

Damping ratio = Viscous damping ratio (%) See Sanaz et al. (2012) PEER Report
PSA = Pseudo-absolute acceleration response spectrum (g)
PGA = Peak ground acceleration (g)
PGV = Peak ground velocity (cm/s)
S_d = Relative displacement response spectrum (cm)
M_w = Moment magnitude
R_{RUP} = Closest distance to coseismic rupture (km), used in ASK13, CB13 and CY13. See Figures a, b and c for illustation
R_{JB} = Closest distance to surface projection of coseismic rupture (km). See Figures a, b and c for illustation
R_X = Horizontal distance from top of rupture measured perpendicular to fault strike (km). See Figures a, b and c for illustation
R_{y0} = The horizontal distance off the end of the rupture measured parallel to strike (km)
V_{S30} = The average shear-wave velocity (m/s) over a subsurface depth of 30 m
U = Unspecified-mechanism factor: 1 for unspecified; 0 otherwise
F_{RV} = Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
F_{NM} = Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal
F_{HW} = Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise
Dip = Average dip of rupture plane (degrees)
Z_{TOR} = Depth to top of coseismic rupture (km)
Z_{HYP} = Hypocentral depth from the earthquake
$Z_{1.0}$ = Depth to Vs=1 km/sec
$Z_{2.5}$ = Depth to Vs=2.5 km/sec
W = Fault rupture width (km)
$V_{s30flag}$ = 1 for measured, 0 for inferred Vs30
F_{AS} = 0 for mainshock; 1 for aftershock
Region = Specific regions considered in the models, Click on Region to see codes
ΔDPP = Directivity term, direct point parameter; uses 0 for median predictions
PGA_r (g) = Peak ground acceleration on rock (g), this specific cell is updated in the cell for BSSA14 and CB14, for others it is taken account for in the macros
Z_{BOT} (km) = The depth to the bottom of the seismogenic crust
Z_{BOR} (km) = The depth to the bottom of the rupture plane
SS = 1 for strike slip, automatically updated in the cell

Input variables with defaults (If entered 999 as input):

Red colored value: The value is used in the code when input is unknown

DEFAULTs	USER defined	ASK14	BSSA14	CB14	CY14	I14
W (km)	14.75			22.598		
$Z_{1.0}$ (km)	0.250	0.250			0.485	
$\delta Z_{1.0}$ (km)	-0.236		-0.236			
$Z_{2.5}$ (V_{S30}=1100)(km)	2.300			0.398		
$Z_{2.5}$ (V_{S30})(km)	2.300			2.070		
Z_{hyp} (km)	999.00			10.225		
Z_{tor} (km)	7.40			2.038	2.038	
Z_{BOR} (km)	-			15.000		

ACKNOWLEDGEMENTS

CEA CALIFORNIA EARTHQUAKE AUTHORITY
Caltrans
PG&E

Nick Gregor, Bechtel
Silvia Mazzoni, Consultant

All NGA West-2 participants are acknowledged for their constructive comments and feedback.

Santa Monica Alt 2 (3) Fault

PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

WEIGHTED AVERAGE of 2014 NGA WEST-2 GMPEs

Last updated: 04 14 15

by Emel Seyhan, PhD, PEER & UCLA -- email: emel.seyhan@gmail.com, peer_center@berkeley.edu

This excel file will be updated as necessary on the PEER website to fix any typos or other errors. Please check the website frequently for new versions at: http://peer.berkeley.edu/ngawest2/databases/

Legend	Pre-defined option	Main input variable	Calculated variable	Input var. flag	Internal variable

GMPE averaging	Geometric

Weighted average of the natural logarithm of the spectral values

GMPEs	ASK14	BSSA14	CB14	CY14	I14
Weight	0.25	0.25	0.25	0.25	0

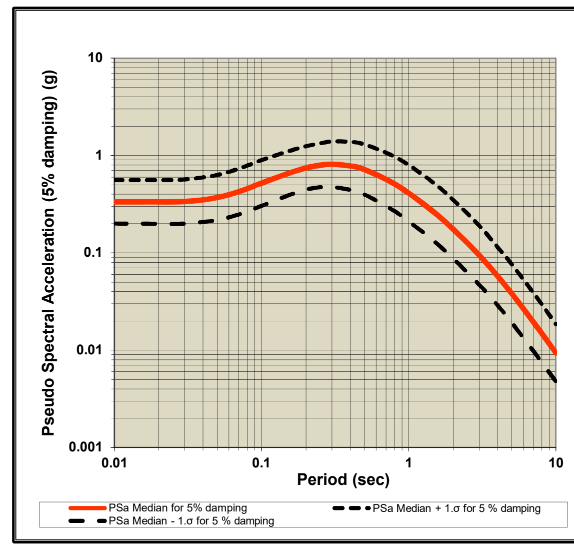
# of std. dev.	1
Damping ratio (%)	5

Modification factors are calculated in Sheet DSF

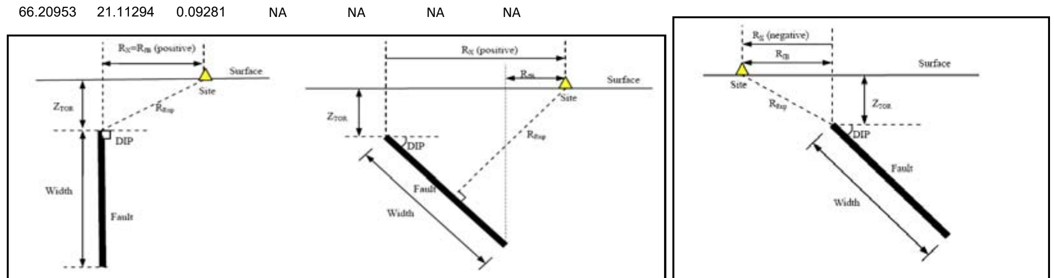
ASK14 Abrahamson & Silva & Kamai 2014 NGA West-2 Model
BSSA14 Boore & Stewart & Seyhan & Atkinson 2014 NGA West-2 Model
CB14 Campbell & Bozorgnia 2014 NGA West-2 Model
CY14 Chiou & Youngs 2014 NGA West-2 Model
I14 Idriss 2014 NGA West-2 Model

RotD50 Horizontal Component of PGA, PGV and *IMs*

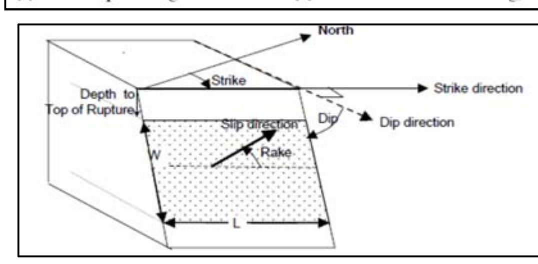
GMP	*T* (s)	Baseline: 5% Damping				User defined: 5% Damping			
		PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	0.01	0.33432	0.55852	0.20012	0.00083	0.33432	0.55852	0.20012	0.00083
	0.02	0.33340	0.55944	0.19869	0.00331	0.33340	0.55944	0.19869	0.00331
	0.03	0.33658	0.56706	0.19978	0.00752	0.33658	0.56706	0.19978	0.00752
	0.05	0.36983	0.63006	0.21709	0.02295	0.36983	0.63006	0.21709	0.02295
	0.075	0.44061	0.75874	0.25586	0.06152	0.44149	0.76026	0.25638	0.06165
	0.1	0.52014	0.89404	0.30261	0.12912	0.52170	0.89672	0.30352	0.12951
	0.15	0.64955	1.09553	0.38512	0.36279	0.65084	1.09772	0.38589	0.36352
	0.2	0.74341	1.24204	0.44496	0.73816	0.74489	1.24453	0.44585	0.73964
	0.25	0.79130	1.32950	0.47097	1.22769	0.79288	1.33216	0.47191	1.23014
	0.3	0.81285	1.39181	0.47473	1.81602	0.81367	1.39320	0.47520	1.81784
	0.4	0.78163	1.38023	0.44263	3.10445	0.78241	1.38162	0.44308	3.10756
	0.5	0.71795	1.29891	0.39684	4.45557	0.71867	1.30021	0.39724	4.46002
	0.75	0.53997	1.02137	0.28547	7.53983	0.53997	1.02137	0.28547	7.53983
	1	0.41029	0.79843	0.21084	10.18499	0.41029	0.79843	0.21084	10.18499
	1.5	0.25826	0.51255	0.13013	14.42482	0.25852	0.51307	0.13026	14.43924
	2	0.17539	0.35118	0.08759	17.41509	0.17521	0.35083	0.08751	17.39767
	3	0.09533	0.19167	0.04741	21.29748	0.09523	0.19148	0.04736	21.27618
	4	0.05795	0.11538	0.02911	23.01720	0.05789	0.11526	0.02908	22.99419
	5	0.03854	0.07688	0.01932	23.91547	0.03838	0.07658	0.01924	23.81981
	7.5	0.01711	0.03405	0.00860	23.89509	0.01706	0.03395	0.00858	23.82341
	10	0.00941	0.01855	0.00477	23.35947	0.00937	0.01847	0.00476	23.26603
PGA (g)	0	0.33248	0.55504	0.19916	0.00083	0.33248	0.55504	0.19916	0.00083
PGV (cm/s)	-1	37.38821	66.20953	21.11294	0.09281	NA	NA	NA	NA

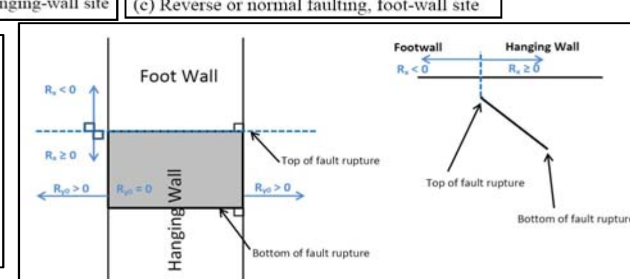


Input variables	Errors and warnings
M_w	
6.78	
R_{RUP} (km)	
12.38	
R_{JB} (km)	
6.44	
R_X (km)	
15.17	
Ry0 (km)	*If unknown use 999*
999	
V_{S30} (m/sec)	
260	
U (BSSA13)	*1: Unspecified fault mech.*
1	
F_{RV}	*1: reverse fault*
0	
F_{NM}	*1: normal fault*
0	
F_{HW}	*1: hanging wall side*
1	
Dip (deg)	
50	
Z_{TOR} (km)	*If unknown use 999*
0	
Z_{HYP} (km)	*If unknown use 999*
999	
$Z_{1.0}$ (km)	*If unknown use 999*
0.25	
$Z_{2.5}$ (km)	*If unknown use 999*
2.3	
W (km)	*If unknown use 999*
13.68	
Vs30Flag	
measured	*Choose options for V_{s30} from the list*
F_{AS}	
no	*Aftershock effect is not applicable.*
Region	
California	*Choose region from the list*



(a) Strike slip faulting (b) Reverse or normal faulting, hanging-wall site (c) Reverse or normal faulting, foot-wall site





Courtesy: Jennifer Donahue

Calculated Variables/Flags

ΔDPP	*Always 0 for median calcs.*
0	
PGA_r (g)	
0.284	
Z_{BOT} (km) (CB14)	*Enter for default W calcs*
15	
SS	
0	*auto calculated*
$V_{s30Flag}$	
1	*measured*
F_{AS}	
0	*Aftershock effect is not applicable.*
Region	
0	*California*
Option for Sa value	
1	*Weighted average of the natural logarithm of the spectral values*

Definition of Parameters

Damping ratio = Viscous damping ratio (%) See Sanaz et al. (2012) PEER Report
PSA = Pseudo-absolute acceleration response spectrum (g)
PGA = Peak ground acceleration (g)
PGV = Peak ground velocity (cm/s)
S_d = Relative displacement response spectrum (cm)
M_w = Moment magnitude
R_{RUP} = Closest distance to coseismic rupture (km), used in ASK13, CB13 and CY13. See Figures a, b and c for illustation
R_{JB} = Closest distance to surface projection of coseismic rupture (km). See Figures a, b and c for illustation
R_X = Horizontal distance from top of rupture measured perpendicular to fault strike (km). See Figures a, b and c for illustation
R_{y0} = The horizontal distance off the end of the rupture measured parallel to strike (km)
V_{S30} = The average shear-wave velocity (m/s) over a subsurface depth of 30 m
U = Unspecified-mechanism factor: 1 for unspecified; 0 otherwise
F_{RV} = Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
F_{NM} = Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal
F_{HW} = Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise
Dip = Average dip of rupture plane (degrees)
Z_{TOR} = Depth to top of coseismic rupture (km)
Z_{HYP} = Hypocentral depth from the earthquake
$Z_{1.0}$ = Depth to Vs=1 km/sec
$Z_{2.5}$ = Depth to Vs=2.5 km/sec
W = Fault rupture width (km)
$V_{s30flag}$ = 1 for measured, 0 for inferred Vs30
F_{AS} = 0 for mainshock; 1 for aftershock
Region = Specific regions considered in the models, Click on Region to see codes
ΔDPP = Directivity term, direct point parameter; uses 0 for median predictions
PGA_r (g) = Peak ground acceleration on rock (g), this specific cell is updated in the cell for BSSA14 and CB14, for others it is taken account for in the macros
Z_{BOT} (km) = The depth to the bottom of the seismogenic crust
Z_{BOR} (km) = The depth to the bottom of the rupture plane
SS = 1 for strike slip, automatically updated in the cell

Input variables with defaults (If entered 999 as input):

Red colored value: The value is used in the code when input is unknown

DEFAULTs	USER defined	ASK14	BSSA14	CB14	CY14	I14
W (km)	13.68			19.084		
$Z_{1.0}$ (km)	0.250	0.250			0.485	
$\delta Z_{1.0}$ (km)	-0.236		-0.236			
$Z_{2.5}$ (V_{S30}=1100)(km)	2.300			0.398		
$Z_{2.5}$ (V_{S30})(km)	2.300			2.070		
Z_{hyp} (km)	999.00			10.607		
Z_{tor} (km)	0.00			0.380	0.380	
Z_{BOR} (km)	-			15.000		

ACKNOWLEDGEMENTS

Nick Gregor, Bechtel
Silvia Mazzoni, Consultant

All NGA West-2 participants are acknowledged for their constructive comments and feedback.

Santa Susan East (Connector) (0) Fault

PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

WEIGHTED AVERAGE of 2014 NGA WEST-2 GMPEs

Last updated: 04 14 15

by Emel Seyhan, PhD, PEER & UCLA -- email: emel.seyhan@gmail.com, peer_center@berkeley.edu

This excel file will be updated as necessary on the PEER website to fix any typos or other errors. Please check the website frequently for new versions at: http://peer.berkeley.edu/ngawest2/databases/

Legend	Pre-defined option	Main input variable	Calculated variable	Input var. flag	Internal variable

GMPE averaging	Geometric	*Weighted average of the natural logarithm of the spectral values*

GMPEs	ASK14	BSSA14	CB14	CY14	I14
Weight	0.25	0.25	0.25	0.25	0

# of std. dev.	1	
Damping ratio (%)	5	*Modification factors are calculated in Sheet DSF*

ASK14 Abrahamson & Silva & Kamai 2014 NGA West-2 Model
BSSA14 Boore & Stewart & Seyhan & Atkinson 2014 NGA West-2 Model
CB14 Campbell & Bozorgnia 2014 NGA West-2 Model
CY14 Chiou & Youngs 2014 NGA West-2 Model
I14 Idriss 2014 NGA West-2 Model

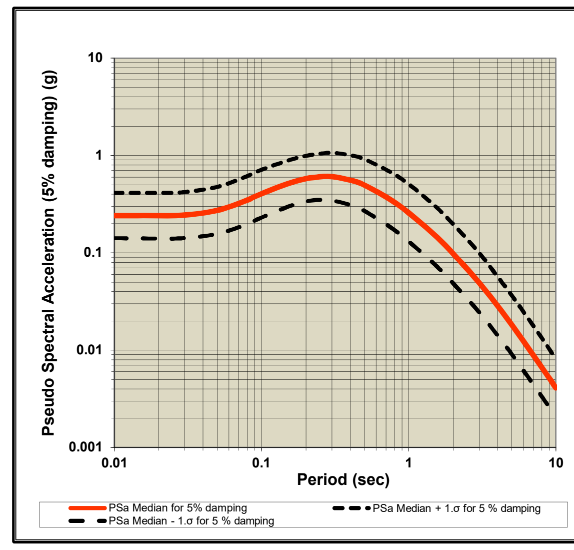
Input variables	Errors and warnings
M_w: 6.38	
R_{RUP} (km): 16.08	
R_{JB} (km): 8.42	
R_X (km): 15.49	
Ry0 (km): 999	*If unknown use 999*
V_{S30} (m/sec): 260	
U (BSSA13): 0	*1: Unspecified fault mech.*
F_{RV}: 1	*1: reverse fault*
F_{NM}: 0	*1: normal fault*
F_{HW}: 1	*1: hanging wall side*
Dip (deg): 55	
Z_{TOR} (km): 0	*If unknown use 999*
Z_{HYP} (km): 999	*If unknown use 999*
$Z_{1.0}$ (km): 0.25	*If unknown use 999*
$Z_{2.5}$ (km): 2.3	*If unknown use 999*
W (km): 17.9	*If unknown use 999*
Vs30Flag: measured	*Choose options for V_{S30} from the list*
F_{AS}: no	*Aftershock effect is not applicable.*
Region: California	*Choose region from the list*

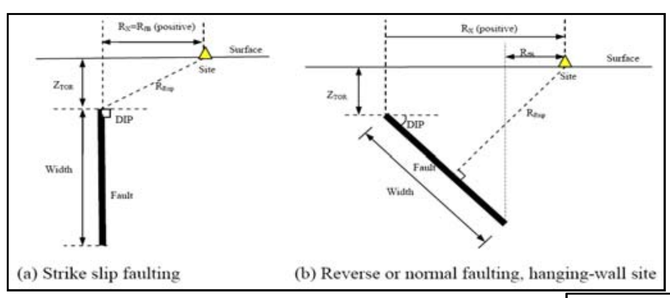
Calculated Variables/Flags

ΔDPP: 0	*Always 0 for median calcs.*
PGA_r (g): 0.222	
Z_{BOT} (km) (CB14): 15	*Enter for default W calcs*
SS: 0	*auto calculated*
$V_{s30Flag}$: 1	*measured*
F_{AS}: 0	*Aftershock effect is not applicable.*
Region: 0	*California*
Option for Sa value: 1	*Weighted average of the natural logarithm of the spectral values*

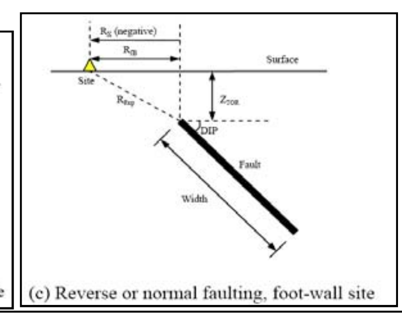
RotD50 Horizontal Component of PGA, PGV and *IMs*

GMP	T (s)	Baseline: 5% Damping				User defined: 5% Damping			
		PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	0.01	0.24117	0.41174	0.14127	0.00060	0.24117	0.41174	0.14127	0.00060
	0.02	0.24050	0.41253	0.14021	0.00239	0.24050	0.41253	0.14021	0.00239
	0.03	0.24427	0.42099	0.14173	0.00546	0.24427	0.42099	0.14173	0.00546
	0.05	0.27249	0.47500	0.15632	0.01691	0.27276	0.47548	0.15647	0.01693
	0.075	0.33436	0.58894	0.18982	0.04669	0.33503	0.59012	0.19020	0.04678
	0.1	0.40417	0.71106	0.22973	0.10033	0.40538	0.71319	0.23042	0.10063
	0.15	0.51166	0.88523	0.29574	0.28578	0.51217	0.88611	0.29603	0.28606
	0.2	0.57758	0.99111	0.33658	0.57350	0.57815	0.99211	0.33692	0.57408
	0.25	0.60336	1.04104	0.34969	0.93610	0.60396	1.04209	0.35004	0.93703
	0.3	0.60673	1.06559	0.34546	1.35551	0.60733	1.06665	0.34581	1.35687
	0.4	0.55809	1.00666	0.30940	2.21662	0.55865	1.00767	0.30971	2.21883
	0.5	0.49649	0.91534	0.26930	3.08115	0.49698	0.91626	0.26957	3.08423
	0.75	0.35253	0.67682	0.18362	4.92245	0.35253	0.67682	0.18362	4.92245
	1	0.25730	0.50670	0.13066	6.38712	0.25730	0.50670	0.13066	6.38712
	1.5	0.15238	0.30482	0.07618	8.51114	0.15254	0.30512	0.07625	8.51965
	2	0.09833	0.19799	0.04883	9.76352	0.09823	0.19779	0.04879	9.75376
	3	0.04975	0.10047	0.02463	11.11475	0.04970	0.10037	0.02461	11.10364
	4	0.02868	0.05736	0.01435	11.39306	0.02863	0.05724	0.01432	11.37027
	5	0.01837	0.03681	0.00917	11.40083	0.01830	0.03666	0.00913	11.35523
	7.5	0.00773	0.01544	0.00387	10.79134	0.00771	0.01540	0.00386	10.75896
	10	0.00411	0.00814	0.00208	10.20512	0.00409	0.00810	0.00207	10.16430
PGA (g)	0	0.23991	0.40929	0.14063	0.00060	0.23991	0.40929	0.14063	0.00060
PGV (cm/s)	-1	22.90513	40.88303	12.83283	0.05686	NA	NA	NA	NA

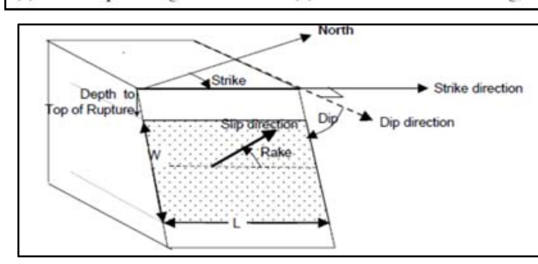


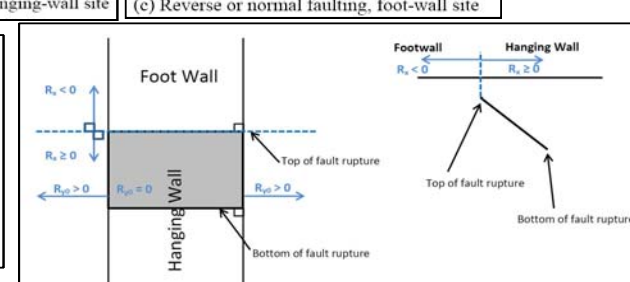


(a) Strike slip faulting (b) Reverse or normal faulting, hanging-wall site



(c) Reverse or normal faulting, foot-wall site





Courtesy: Jennifer Donahue

Definition of Parameters

Damping ratio = Viscous damping ratio (%) See Sanaz et al. (2012) PEER Report
PSA = Pseudo-absolute acceleration response spectrum (g)
PGA = Peak ground acceleration (g)
PGV = Peak ground velocity (cm/s)
S_d = Relative displacement response spectrum (cm)
M_w = Moment magnitude
R_{RUP} = Closest distance to coseismic rupture (km), used in ASK13, CB13 and CY13. See Figures a, b and c for illustation
R_{JB} = Closest distance to surface projection of coseismic rupture (km). See Figures a, b and c for illustation
R_X = Horizontal distance from top of rupture measured perpendicular to fault strike (km). See Figures a, b and c for illustation
R_{y0} = The horizontal distance off the end of the rupture measured parallel to strike (km)
V_{S30} = The average shear-wave velocity (m/s) over a subsurface depth of 30 m
U = Unspecified-mechanism factor: 1 for unspecified; 0 otherwise
F_{RV} = Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
F_{NM} = Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal
F_{HW} = Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise
Dip = Average dip of rupture plane (degrees)
Z_{TOR} = Depth to top of coseismic rupture (km)
Z_{HYP} = Hypocentral depth from the earthquake
$Z_{1.0}$ = Depth to Vs=1 km/sec
$Z_{2.5}$ = Depth to Vs=2.5 km/sec
W = Fault rupture width (km)
$V_{s30flag}$ = 1 for measured, 0 for inferred Vs30
F_{AS} = 0 for mainshock; 1 for aftershock
Region = Specific regions considered in the models, Click on Region to see codes
ΔDPP = Directivity term, direct point parameter; uses 0 for median predictions
PGA_r **(g)** = Peak ground acceleration on rock (g), this specific cell is updated in the cell for BSSA14 and CB14, for others it is taken account for in the macros
Z_{BOT} **(km)** = The depth to the bottom of the seismogenic crust
Z_{BOR} **(km)** = The depth to the bottom of the rupture plane
SS = 1 for strike slip, automatically updated in the cell

Input variables with defaults (If entered 999 as input):

Red colored value: The value is used in the code when input is unknown

DEFAULTs	USER defined	ASK14	BSSA14	CB14	CY14	I14
W (km)	17.90			13.166		
$Z_{1.0}$ (km)	0.250	0.250			0.485	
$\delta Z_{1.0}$ (km)	-0.236		-0.236			
$Z_{2.5}$ (V_{S30}=1100)(km)	2.300			0.398		
$Z_{2.5}$ (V_{S30})(km)	2.300			2.070		
Z_{hyp} (km)	999.00			11.321		
Z_{tor} (km)	0.00			4.215	4.215	
Z_{BOR} (km)	-			15.000		

ACKNOWLEDGEMENTS

Nick Gregor, Bechtel
Silvia Mazzoni, Consultant

All NGA West-2 participants are acknowledged for their constructive comments and feedback.

Santa Susana Alt 1 (0) Fault

PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

WEIGHTED AVERAGE of 2014 NGA WEST-2 GMPEs

Last updated: 04 14 15

by Emel Seyhan, PhD, PEER & UCLA -- email: emel.seyhan@gmail.com, peer_center@berkeley.edu

This excel file will be updated as necessary on the PEER website to fix any typos or other errors. Please check the website frequently for new versions at: http://peer.berkeley.edu/ngawest2/databases/

Legend	Pre-defined option	Main input variable	Calculated variable	Input var. flag	Internal variable

GMPE averaging	Geometric	*Weighted average of the natural logarithm of the spectral values*

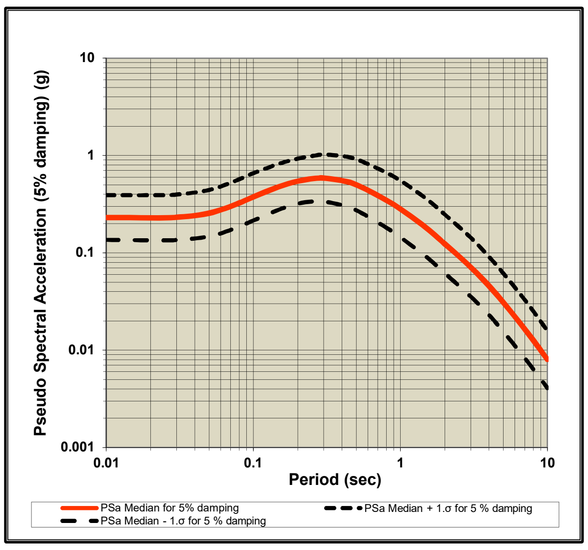
GMPEs	ASK14	BSSA14	CB14	CY14	I14
Weight	0.25	0.25	0.25	0.25	0

# of std. dev.	1	
Damping ratio (%)	5	*Modification factors are calculated in Sheet DSF*

ASK14 Abrahamson & Silva & Kamai 2014 NGA West-2 Model
BSSA14 Boore & Stewart & Seyhan & Atkinson 2014 NGA West-2 Model
CB14 Campbell & Bozorgnia 2014 NGA West-2 Model
CY14 Chiou & Youngs 2014 NGA West-2 Model
I14 Idriss 2014 NGA West-2 Model

RotD50 Horizontal Component of PGA, PGV and *IMs*

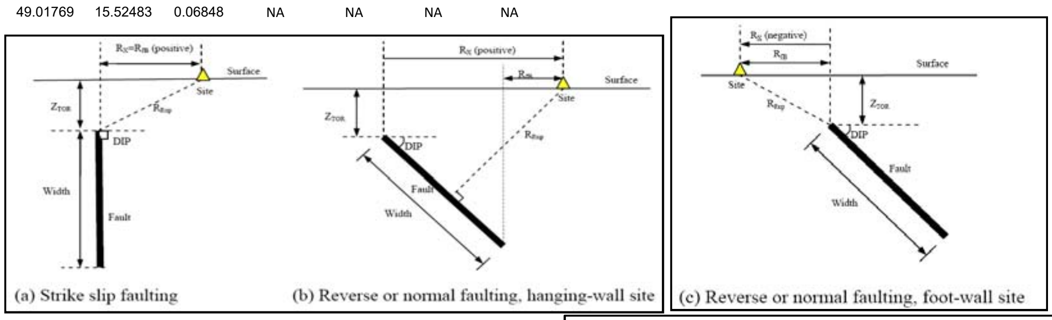
GMP	*T* (s)	Baseline: 5% Damping				User defined: 5% Damping			
		PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	**0.01**	0.23062	0.39097	0.13603	0.00057	0.23062	0.39097	0.13603	0.00057
	0.02	0.22927	0.39056	0.13459	0.00228	0.22927	0.39056	0.13459	0.00228
	0.03	0.23170	0.39670	0.13533	0.00518	0.23170	0.39670	0.13533	0.00518
	0.05	0.25527	0.44229	0.14733	0.01584	0.25527	0.44229	0.14733	0.01584
	0.075	0.31144	0.54551	0.17780	0.04349	0.31206	0.54660	0.17816	0.04357
	0.1	0.37482	0.65584	0.21421	0.09304	0.37594	0.65781	0.21485	0.09332
	0.15	0.47808	0.82234	0.27794	0.26702	0.47904	0.82399	0.27850	0.26756
	0.2	0.54315	0.92594	0.31861	0.53932	0.54424	0.92779	0.31925	0.54040
	0.25	0.57599	0.98655	0.33629	0.89364	0.57657	0.98753	0.33662	0.89453
	0.3	0.58587	1.02088	0.33623	1.30892	0.58646	1.02190	0.33656	1.31022
	0.4	0.55185	0.98710	0.30852	2.19183	0.55240	0.98808	0.30883	2.19402
	0.5	0.50128	0.91626	0.27424	3.11090	0.50178	0.91718	0.27452	3.11401
	0.75	0.36830	0.70114	0.19346	5.14266	0.36830	0.70114	0.19346	5.14266
	1	0.28237	0.55177	0.14451	7.00957	0.28237	0.55177	0.14451	7.00957
	1.5	0.18051	0.35876	0.09082	10.08183	0.18069	0.35912	0.09091	10.09191
	2	0.12411	0.24848	0.06199	12.32317	0.12398	0.24823	0.06192	12.31085
	3	0.07166	0.14396	0.03567	16.00877	0.07158	0.14382	0.03563	15.99276
	4	0.04584	0.09118	0.02304	18.20508	0.04579	0.09109	0.02302	18.18687
	5	0.03108	0.06195	0.01559	19.28592	0.03098	0.06176	0.01554	19.22807
	7.5	0.01451	0.02885	0.00730	20.26016	0.01445	0.02873	0.00727	20.17912
	10	0.00804	0.01584	0.00408	19.96221	0.00801	0.01577	0.00407	19.88236
PGA (g)	**0**	0.22944	0.38869	0.13544	0.00057	0.22944	0.38869	0.13544	0.00057
PGV (cm/s)	**-1**	27.58607	49.01769	15.52483	0.06848	NA	NA	NA	NA

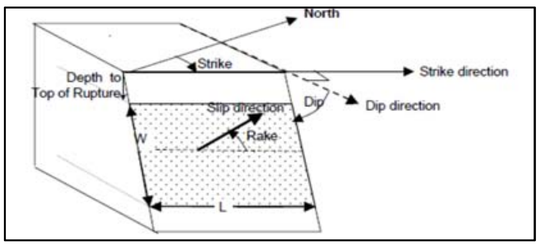


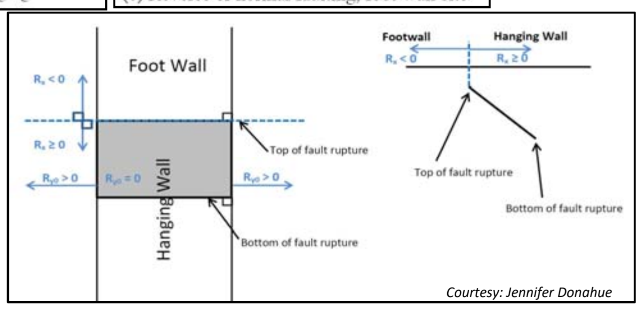
Input variables	Errors and warnings
M_w 6.89	
R_{RUP} (km) 16.54	
R_{JB} (km) 16.46	
R_X (km) -16.46	
Ry0 (km) 999	*If unknown use 999*
V_{S30} (m/sec) 260	
U (BSSA13) 0	*1: Unspecified fault mech.*
F_{RV} 1	*1: reverse fault*
F_{NM} 0	*1: normal fault*
F_{HW} 0	*1: hanging wall side*
Dip (deg) 55	
Z_{TOR} (km) 0	*If unknown use 999*
Z_{HYP} (km) 999	*If unknown use 999*
$Z_{1.0}$ (km) 0.25	*If unknown use 999*
$Z_{2.5}$ (km) 2.3	*If unknown use 999*
W (km) 17.9	*If unknown use 999*
Vs30Flag measured	*Choose options for V_{s30} from the list*
F_{AS} no	*Aftershock effect is not applicable.*
Region California	*Choose region from the list*

Calculated Variables/Flags

ΔDPP 0	*Always 0 for median calcs.*
PGA_r (g) 0.161	
Z_{BOT} (km) (CB14) 15	*Enter for default W calcs*
SS 0	*auto calculated*
$V_{s30Flag}$ 1	*measured*
F_{AS} 0	*Aftershock effect is not applicable.*
Region 0	*California*
Option for Sa value 1	*Weighted average of the natural logarithm of the spectral values*







Courtesy: Jennifer Donahue

Definition of Parameters

Damping ratio = Viscous damping ratio (%) See Sanaz et al. (2012) PEER Report
PSA = Pseudo-absolute acceleration response spectrum (g)
PGA = Peak ground acceleration (g)
PGV = Peak ground velocity (cm/s)
S_d = Relative displacement response spectrum (cm)
M_w = Moment magnitude
R_{RUP} = Closest distance to coseismic rupture (km), used in ASK13, CB13 and CY13. See Figures a, b and c for illustation
R_{JB} = Closest distance to surface projection of coseismic rupture (km). See Figures a, b and c for illustation
R_X = Horizontal distance from top of rupture measured perpendicular to fault strike (km). See Figures a, b and c for illustation
R_{y0} = The horizontal distance off the end of the rupture measured parallel to strike (km)
V_{S30} = The average shear-wave velocity (m/s) over a subsurface depth of 30 m
U = Unspecified-mechanism factor: 1 for unspecified; 0 otherwise
F_{RV} = Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
F_{NM} = Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal
F_{HW} = Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise
Dip = Average dip of rupture plane (degrees)
Z_{TOR} = Depth to top of coseismic rupture (km)
Z_{HYP} = Hypocentral depth from the earthquake
$Z_{1.0}$ = Depth to Vs=1 km/sec
$Z_{2.5}$ = Depth to Vs=2.5 km/sec
W = Fault rupture width (km)
$V_{s30flag}$ = 1 for measured, 0 for inferred Vs30
F_{AS} = 0 for mainshock; 1 for aftershock
Region = Specific regions considered in the models, Click on Region to see codes
ΔDPP = Directivity term, direct point parameter; uses 0 for median predictions
PGA_r (g) = Peak ground acceleration on rock (g), this specific cell is updated in the cell for BSSA14 and CB14, for others it is taken account for in the macros
Z_{BOT} (km) = The depth to the bottom of the seismogenic crust
Z_{BOR} (km) = The depth to the bottom of the rupture plane
SS = 1 for strike slip, automatically updated in the cell

Input variables with defaults (If entered 999 as input):

Red colored value: The value is used in the code when input is unknown

DEFAULTs	USER defined	ASK14	BSSA14	CB14	CY14	I14
W (km)	17.90			15.823		
$Z_{1.0}$ (km)	0.250	0.250			0.485	
$\delta Z_{1.0}$ (km)	-0.236		-0.236			
$Z_{2.5}$ (V_{S30}=1100)(km)	2.300			0.398		
$Z_{2.5}$ (V_{S30})(km)	2.300			2.070		
Z_{hyp} (km)	999.00			12.265		
Z_{tor} (km)	0.00			2.038	2.038	
Z_{BOR} (km)	-			15.000		

ACKNOWLEDGEMENTS

Nick Gregor, Bechtel
Silvia Mazzoni, Consultant

All NGA West-2 participants are acknowledged for their constructive comments and feedback.

Santa Susana Alt 2 (3) Fault

PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

WEIGHTED AVERAGE of 2014 NGA WEST-2 GMPEs

Last updated: 04 14 15

by Emel Seyhan, PhD, PEER & UCLA -- email: emel.seyhan@gmail.com, peer_center@berkeley.edu

This excel file will be updated as necessary on the PEER website to fix any typos or other errors. Please check the website frequently for new versions at: http://peer.berkeley.edu/ngawest2/databases/

Legend	Pre-defined option	Main input variable	Calculated variable	Input var. flag	Internal variable

GMPE averaging	Geometric	*Weighted average of the natural logarithm of the spectral values*

GMPEs	ASK14	BSSA14	CB14	CY14	I14
Weight	0.25	0.25	0.25	0.25	0

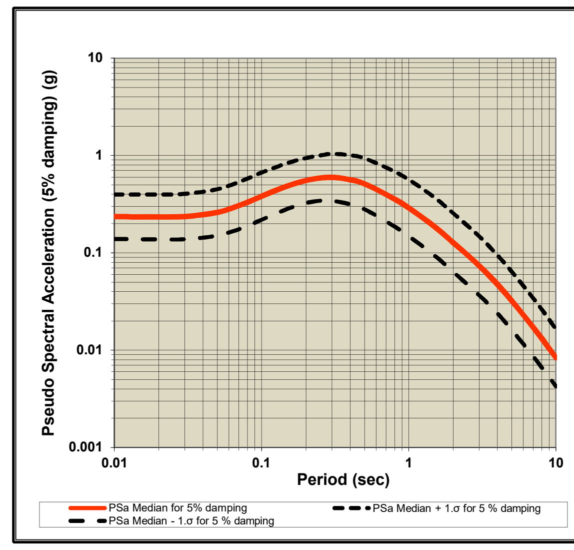
# of std. dev.	1	
Damping ratio (%)	5	*Modification factors are calculated in Sheet DSF*

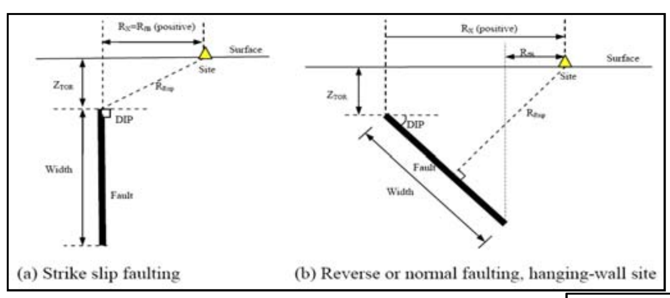
ASK14 Abrahamson & Silva & Kamai 2014 NGA West-2 Model
BSSA14 Boore & Stewart & Seyhan & Atkinson 2014 NGA West-2 Model
CB14 Campbell & Bozorgnia 2014 NGA West-2 Model
CY14 Chiou & Youngs 2014 NGA West-2 Model
I14 Idriss 2014 NGA West-2 Model

RotD50 Horizontal Component of PGA, PGV and *IMs*

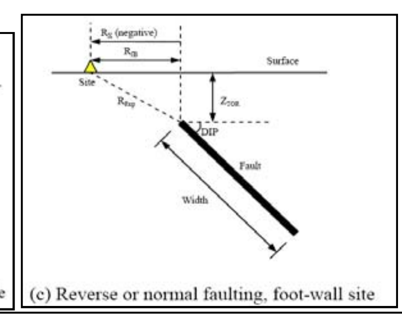
Input variables	Value	Errors and warnings
M_w	6.91	
R_{RUP} (km)	16.21	
R_{JB} (km)	16.17	
R_X (km)	-16.17	
Ry0 (km)	999	*If unknown use 999*
V_{S30} (m/sec)	260	
U (BSSA13)	0	*1: Unspecified fault mech.*
F_{RV}	1	*1: reverse fault*
F_{NM}	0	*1: normal fault*
F_{HW}	0	*1: hanging wall side*
Dip (deg)	53	
Z_{TOR} (km)	0	*If unknown use 999*
Z_{HYP} (km)	999	*If unknown use 999*
$Z_{1.0}$ (km)	0.25	*If unknown use 999*
$Z_{2.5}$ (km)	2.3	*If unknown use 999*
W (km)	11.95	*If unknown use 999*
Vs30Flag	measured	*Choose options for V_{S30} from the list*
F_{AS}	no	*Aftershock effect is not applicable.*
Region	California	*Choose region from the list*

GMP	T (s)	Baseline: 5% Damping				User defined: 5% Damping			
		PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	0.01	0.23517	0.39833	0.13884	0.00058	0.23517	0.39833	0.13884	0.00058
	0.02	0.23380	0.39791	0.13738	0.00232	0.23380	0.39791	0.13738	0.00232
	0.03	0.23618	0.40398	0.13808	0.00528	0.23618	0.40398	0.13808	0.00528
	0.05	0.25996	0.44993	0.15019	0.01613	0.25996	0.44993	0.15019	0.01613
	0.075	0.31674	0.55419	0.18102	0.04423	0.31737	0.55530	0.18139	0.04432
	0.1	0.38085	0.66563	0.21791	0.09454	0.38199	0.66763	0.21856	0.09482
	0.15	0.48544	0.83395	0.28257	0.27113	0.48641	0.83562	0.28314	0.27167
	0.2	0.55170	0.93931	0.32404	0.54781	0.55281	0.94119	0.32469	0.54891
	0.25	0.58564	1.00184	0.34235	0.90861	0.58623	1.00284	0.34269	0.90952
	0.3	0.59634	1.03792	0.34263	1.33230	0.59693	1.03896	0.34297	1.33363
	0.4	0.56288	1.00587	0.31498	2.23563	0.56344	1.00688	0.31529	2.23786
	0.5	0.51202	0.93514	0.28035	3.17758	0.51254	0.93608	0.28063	3.18076
	0.75	0.37705	0.71735	0.19818	5.26485	0.37705	0.71735	0.19818	5.26485
	1	0.28963	0.56567	0.14829	7.18964	0.28963	0.56567	0.14829	7.18964
	1.5	0.18568	0.36892	0.09345	10.37067	0.18586	0.36929	0.09355	10.38104
	2	0.12796	0.25614	0.06393	12.70582	0.12783	0.25589	0.06386	12.69311
	3	0.07410	0.14885	0.03689	16.55491	0.07403	0.14870	0.03685	16.53835
	4	0.04759	0.09465	0.02392	18.90034	0.04754	0.09456	0.02390	18.88144
	5	0.03232	0.06443	0.01622	20.06021	0.03223	0.06423	0.01617	20.00003
	7.5	0.01512	0.03006	0.00761	21.11835	0.01506	0.02994	0.00758	21.03388
	10	0.00839	0.01652	0.00426	20.83076	0.00836	0.01646	0.00424	20.74744
PGA (g)	0	0.23397	0.39601	0.13824	0.00058	0.23397	0.39601	0.13824	0.00058
PGV (cm/s)	-1	28.37220	50.40298	15.97092	0.07043	NA	NA	NA	NA

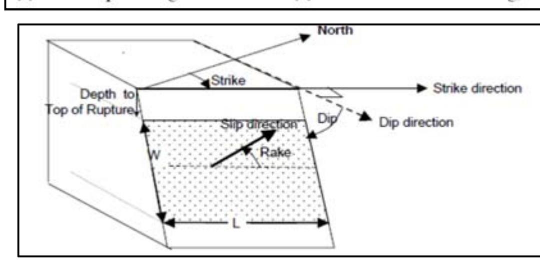


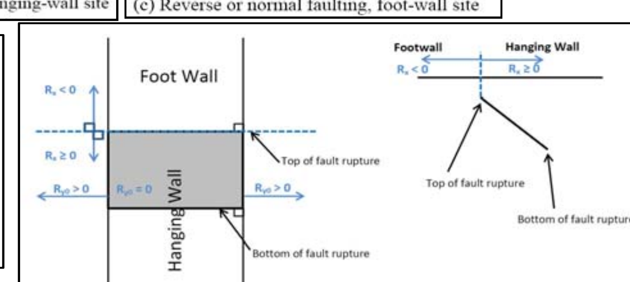


(a) Strike slip faulting (b) Reverse or normal faulting, hanging-wall site



(c) Reverse or normal faulting, foot-wall site





Courtesy: Jennifer Donahue

Calculated Variables/Flags

Variable	Value	
ΔDPP	0	*Always 0 for median calcs.*
PGA_r (g)	0.165	
Z_{BOT} (km) (CB14)	15	*Enter for default W calcs*
SS	0	*auto calculated*
$V_{s30Flag}$	1	*measured*
F_{AS}	0	*Aftershock effect is not applicable.*
Region	0	*California*
Option for Sa value	1	*Weighted average of the natural logarithm of the spectral values*

Definition of Parameters

- **Damping ratio** = Viscous damping ratio (%) See Sanaz et al. (2012) PEER Report
- **PSA** = Pseudo-absolute acceleration response spectrum (g)
- **PGA** = Peak ground acceleration (g)
- **PGV** = Peak ground velocity (cm/s)
- **S_d** = Relative displacement response spectrum (cm)
- **M_w** = Moment magnitude
- **R_{RUP}** = Closest distance to coseismic rupture (km), used in ASK13, CB13 and CY13. See Figures a, b and c for illustration
- **R_{JB}** = Closest distance to surface projection of coseismic rupture (km). See Figures a, b and c for illustration
- **R_X** = Horizontal distance from top of rupture measured perpendicular to fault strike (km). See Figures a, b and c for illustation
- **R_{y0}** = The horizontal distance off the end of the rupture measured parallel to strike (km)
- **V_{S30}** = The average shear-wave velocity (m/s) over a subsurface depth of 30 m
- **U** = Unspecified-mechanism factor: 1 for unspecified; 0 otherwise
- **F_{RV}** = Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
- **F_{NM}** = Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal
- **F_{HW}** = Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise
- **Dip** = Average dip of rupture plane (degrees)
- **Z_{TOR}** = Depth to top of coseismic rupture (km)
- **Z_{HYP}** = Hypocentral depth from the earthquake
- **$Z_{1.0}$** = Depth to Vs=1 km/sec
- **$Z_{2.5}$** = Depth to Vs=2.5 km/sec
- **W** = Fault rupture width (km)
- **$V_{s30flag}$** = 1 for measured, 0 for inferred Vs30
- **F_{AS}** = 0 for mainshock; 1 for aftershock
- **Region** = Specific regions considered in the models, Click on Region to see codes
- **ΔDPP** = Directivity term, direct point parameter; uses 0 for median predictions
- **PGA_r (g)** = Peak ground acceleration on rock (g), this specific cell is updated in the cell for BSSA14 and CB14, for others it is taken account for in the macros
- **Z_{BOT} (km)** = The depth to the bottom of the seismogenic crust
- **Z_{BOR} (km)** = The depth to the bottom of the rupture plane
- **SS** = 1 for strike slip, automatically updated in the cell

Input variables with defaults (If entered 999 as input):

Red colored value: The value is used in the code when input is unknown

DEFAULTs	USER defined	ASK14	BSSA14	CB14	CY14	I14
W (km)	11.95			16.317		
$Z_{1.0}$ (km)	0.250	0.250			0.485	
$\delta Z_{1.0}$ (km)	-0.236		-0.236			
$Z_{2.5}$ (V_{S30}=1100)(km)	2.300			0.398		
$Z_{2.5}$ (V_{S30})(km)	2.300			2.070		
Z_{hyp} (km)	999.00			12.196		
Z_{tor} (km)	0.00			1.969	1.969	
Z_{BOR} (km)	-			15.000		

ACKNOWLEDGEMENTS

Nick Gregor, Bechtel
Silvia Mazzoni, Consultant

All NGA West-2 participants are acknowledged for their constructive comments and feedback.

Table 3: Site-Specific Seismic Ground Motion Analysis per ASCE 7-16

Project Name: Proposed Self-Storage Facility
Project Location: 21101 Ventura Boulevard
Project Number: 13589.001
Site Class: D
Shear Wave Velocity: 352 m/sec
Return Period: 2475 years (2% probability of exceedance in 50 years)
Percent Damping: 5%

Date: July 2022
Latitude: 34.168161°
Longitude: -118.59298°

Seismic Design Coefficients: Per ASCE 7-16 & 2019 CBC

S_s	1.654	$\mathbf{S_{MS}}$	**2.141**	T_0	0.216
S_1	0.6	$\mathbf{S_{M1}}$	**1.200**	T_s	0.907
F_a	1	$\mathbf{S_{DS}}$	**1.427**	T_L	8
F_v	2.5	$\mathbf{S_{D1}}$	**0.800**	$\mathbf{PGA_M}$	**0.776**
C_{RS}	0.933	C_{R1}	0.913		

	Sec. 21.2.1.1 Probabilistic				Sec. 21.2.2 Deterministic			Sec. 11.4.6 General Procedure	Sec. 21.3 Design Response Spectrum				Risk Targeted Spectrum
Period (sec)	Spectral Acceleration (g)	Seismic Risk Coefficients	Maximum Response Coefficients	MCE_R Response Spectrum (g)	Spectral Acceleration (g)	Maximum Response Coefficients	MCE_R Response Spectrum (g)	Design Response Spectral Acceleration (g)	Lower Limit of General Procedure - 80% of S_a (g)	MCE_R - S_{aM} (g)	2/3 * S_{aM} (g)	Design Response Spectrum (g)	1.5 * Design Response Spectrum (g)
0.01	0.869	0.933	1.19	0.965	0.776	1.19	0.923	0.472	0.377	0.923	0.615	0.615	0.923
0.02	0.873	0.933	1.19	0.969	0.780	1.19	0.929	0.502	0.402	0.929	0.619	0.619	0.929
0.03	0.918	0.933	1.19	1.019	0.807	1.19	0.960	0.533	0.426	0.960	0.640	0.640	0.960
0.05	1.099	0.933	1.19	1.221	0.919	1.19	1.094	0.594	0.475	1.094	0.729	0.729	1.094
0.075	1.409	0.933	1.19	1.564	1.116	1.19	1.328	0.670	0.536	1.328	0.885	0.885	1.328
0.1	1.662	0.933	1.19	1.845	1.306	1.19	1.554	0.747	0.597	1.554	1.036	1.036	1.554
0.15	1.938	0.933	1.20	2.170	1.577	1.20	1.893	0.900	0.720	1.893	1.262	1.262	1.893
0.2	2.079	0.933	1.21	2.347	1.771	1.21	2.143	1.052	0.842	2.143	1.428	1.428	2.143
0.25	2.135	0.932	1.22	2.427	1.879	1.22	2.292	1.103	0.882	2.292	1.528	1.528	2.292
0.3	2.128	0.931	1.22	2.415	1.950	1.22	2.379	1.103	0.882	2.379	1.586	1.586	2.379
0.4	1.974	0.928	1.23	2.254	1.904	1.23	2.342	1.103	0.882	2.254	1.502	1.502	2.254
0.5	1.778	0.926	1.23	2.024	1.747	1.23	2.149	1.103	0.882	2.024	1.349	1.349	2.024
0.75	1.329	0.919	1.24	1.515	1.336	1.24	1.656	1.103	0.882	1.515	1.010	1.010	1.515
1	1.014	0.913	1.24	1.148	1.016	1.24	1.260	1.000	0.800	1.148	0.765	0.800	1.200
1.5	0.643	0.913	1.24	0.728	0.629	1.24	0.780	0.667	0.533	0.728	0.485	0.533	0.800
2	0.450	0.913	1.24	0.510	0.421	1.24	0.521	0.500	0.400	0.510	0.340	0.400	0.600
3	0.274	0.913	1.25	0.313	0.229	1.25	0.286	0.333	0.267	0.286	0.191	0.267	0.400
4	0.190	0.913	1.26	0.218	0.140	1.26	0.177	0.250	0.200	0.177	0.118	0.200	0.300
5	0.143	0.913	1.26	0.165	0.097	1.26	0.122	0.200	0.160	0.122	0.082	0.160	0.240
7.5	0.085	0.913	1.28	0.099	0.047	1.28	0.060	0.133	0.107	0.060	0.040	0.107	0.160
10	0.055	0.913	1.29	0.065	0.027	1.29	0.035	0.080	0.064	0.035	0.023	0.064	0.096

X-Axis: Period (sec)
Y-Axis: SA (g)
Number of Data Sets: 1

DATASET #1

Name:
Num Points: 21
Info:
IMR Param List:

IMR = NGAWest2 2014 Averaged No Idriss; IMR Weights = ['Abrahamson, Silva & Kamai (2014)': 0.25, 'Boore, Stewart, Seyhan & Atkinson (2014)': 0.25, 'Campbell & Bozorgnia (2014)': 0.25, 'Chiou & Youngs (2014)': 0.25]; Std Dev Type = Total; Tectonic Region = Active Shallow Crust; Additional Epistemic Uncertainty = null; Component = RotD50; Gaussian Truncation = None

Site Param List:

Longitude = -118.59298; Latitude = 34.168161; Vs30 = 352.0; Vs30 Type = Inferred; Depth 2.5 km/sec = 2.3; Depth 1.0 km/sec = 250.0

IML/Prob Param List:

Map Type = IML@Prob; Probability = 0.02

Forecast Param List:

Eqk Rup Forecast = Mean UCERF3; Mean UCERF3 Presets = (POISSON ONLY) Both FM Branch Averaged; Apply Aftershock Filter = false; Aleatory Mag-Area StdDev = 0.0; Background Seismicity = Include; Treat Background Seismicity As = Point Sources; Fault Grid Spacing = 1.0; Probability Model = Poisson; Sect Upper Depth Averaging Tolerance = 100.0; Use Mean Upper Depth = true; Rup Mag Averaging Tolerance = 1.0; Rupture Rake To Use = Def. Model Mean; Fault Model(s) = Both; Ignore Cache = false

TimeSpan Param List:

Duration = 50.0

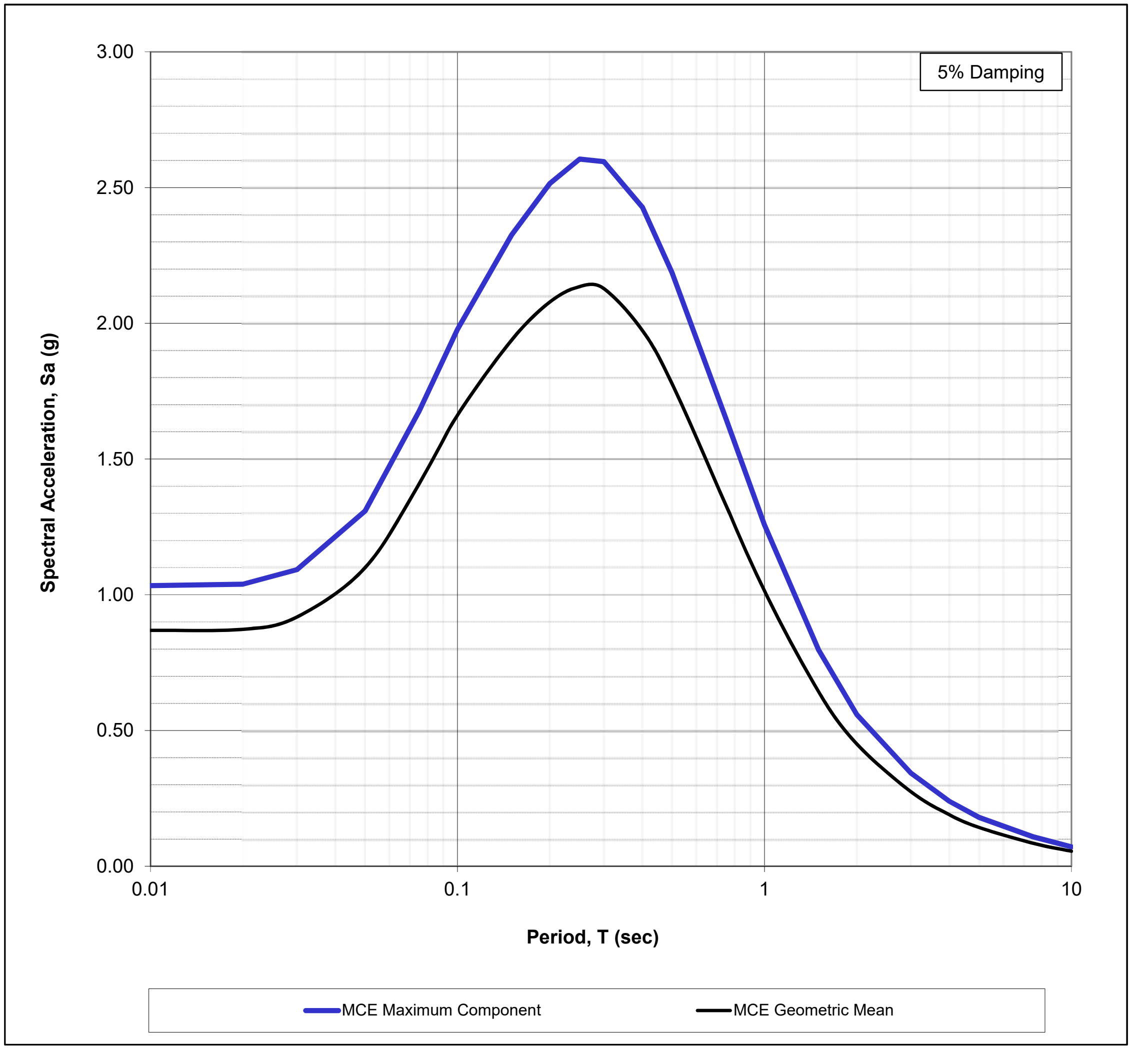
Maximum Distance = 200.0; Pt Src Dist Corr = None

X, Y Data:
0.01 0.8688695
0.02 0.8728592
0.03 0.9182419
0.05 1.0994864
0.075 1.4086219
0.1 1.6617287
0.15 1.9377686
0.2 2.078652
0.25 2.1353185
0.3 2.127628
0.4 1.9742804

0.5	1.7781378
0.75	1.328835
1.0	1.0141371
1.5	0.6430323
2.0	0.45026365
3.0	0.27449322
4.0	0.1898775
5.0	0.1430005
7.5	0.0846105
10.0	0.05530784

MCE PROBABILISTIC SPECTRA (2,475-YEAR AVERAGE RETURN INTERVAL)

Project: Proposed Self-Storage Facility
Project Number: 13589.001
Location: 21101 Ventura Boulevard



Period T (s)	MCE GEOMEAN Sa (g)	Maximum Component Factor	MCE MAX COMP Site-Specific Sa (g)
0.01	0.869	1.19	1.034
0.02	0.873	1.19	1.039
0.03	0.918	1.19	1.093
0.05	1.099	1.19	1.308
0.075	1.409	1.19	1.676
0.10	1.662	1.19	1.977
0.15	1.938	1.20	2.325
0.20	2.079	1.21	2.515
0.25	2.135	1.22	2.605
0.30	2.128	1.22	2.596
0.40	1.974	1.23	2.428
0.50	1.778	1.23	2.187
0.75	1.329	1.24	1.648
1.00	1.014	1.24	1.258
1.50	0.643	1.24	0.797
2.00	0.450	1.24	0.558
3.00	0.274	1.25	0.343
4.00	0.190	1.26	0.239
5.00	0.143	1.26	0.180
7.50	0.085	1.28	0.108
10.00	0.055	1.29	0.071

Leighton

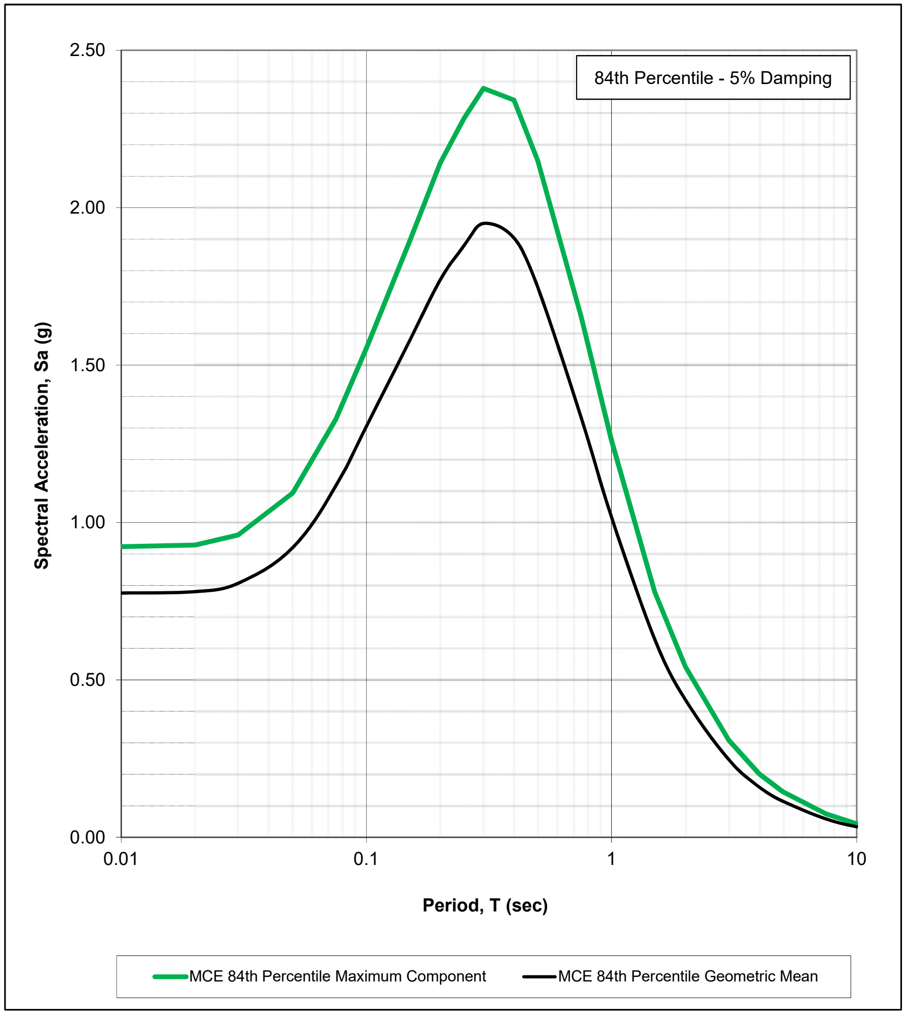
Figure D.3A

MCE DETERMINISTIC SPECTRA

Project: Proposed Self-Storage Facility

Project Number: 13589.001

Location: 21101 Ventura Boulevard



DETERMINISTIC PGA MAGNITUDE				
MC FACTOR		DSHA - 84TH PERCENTILE		
Period T (s)	Maximum Component Factor	Period T (s)	MCE GEOMEAN Sa (g)	MCE MAX COMP Sa (g)
0.01	1.19	0.01	0.776	0.923
0.02	1.19	0.02	0.780	0.929
0.03	1.19	0.03	0.807	0.960
0.05	1.19	0.05	0.919	1.094
0.075	1.19	0.075	1.116	1.328
0.10	1.19	0.10	1.306	1.554
0.15	1.20	0.15	1.577	1.893
0.20	1.21	0.20	1.771	2.143
0.25	1.22	0.25	1.879	2.283
0.30	1.22	0.30	1.950	2.379
0.40	1.23	0.40	1.904	2.342
0.50	1.23	0.50	1.747	2.149
0.75	1.24	0.75	1.336	1.656
1.00	1.24	1.00	1.016	1.260
1.50	1.24	1.50	0.629	0.780
2.00	1.24	2.00	0.437	0.542
3.00	1.25	3.00	0.247	0.309
4.00	1.26	4.00	0.160	0.202
5.00	1.26	5.00	0.115	0.144
7.50	1.28	7.50	0.058	0.074
10.00	1.29	10.00	0.033	0.043

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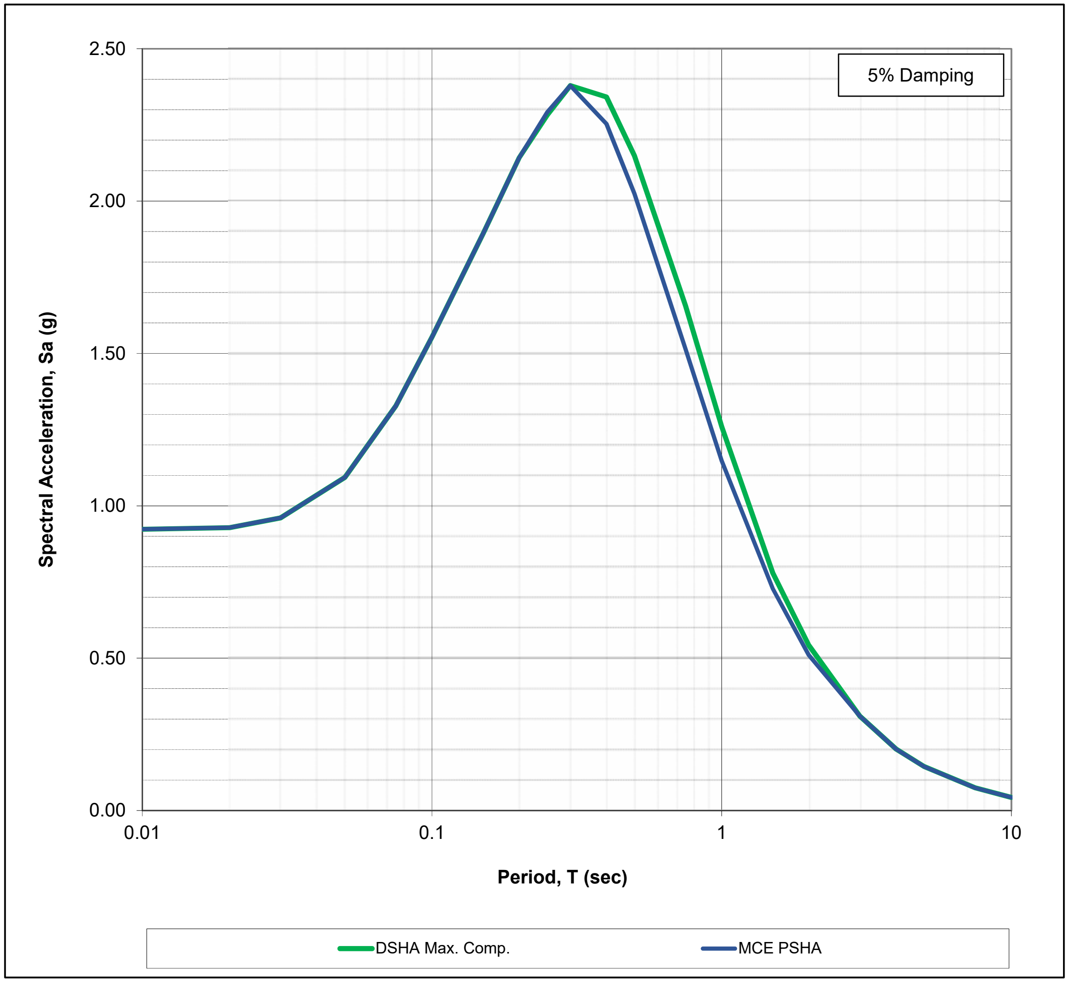
Figure D.3B

MCE SPECTRA COMPARISON - MAXIMUM HORIZONTAL COMPONENT

Project: Proposed Self-Storage Facility

Project Number: 13589.001

Location: 21101 Ventura Boulevard



DSHA		PSHA			
Period T (s)	MAX COMP. Sa (g)	Period T (s)	MCE MAX COMP. Sa (g)	Site Risk Coefficient (Cs)	MCE_R Sa (g)
0.01	0.923	0.01	1.034	0.933	0.923
0.02	0.929	0.02	1.039	0.933	0.929
0.03	0.960	0.03	1.093	0.933	0.960
0.05	1.094	0.05	1.308	0.933	1.094
0.075	1.328	0.075	1.676	0.933	1.328
0.10	1.554	0.10	1.977	0.933	1.554
0.15	1.893	0.15	2.325	0.933	1.893
0.20	2.143	0.20	2.515	0.933	2.143
0.25	2.283	0.25	2.605	0.932	2.292
0.30	2.379	0.30	2.596	0.931	2.379
0.40	2.342	0.40	2.428	0.928	2.254
0.50	2.149	0.50	2.187	0.926	2.024
0.75	1.656	0.75	1.648	0.919	1.515
1.00	1.260	1.00	1.258	0.913	1.148
1.50	0.780	1.50	0.797	0.913	0.728
2.00	0.542	2.00	0.558	0.913	0.510
3.00	0.309	3.00	0.343	0.913	0.309
4.00	0.202	4.00	0.239	0.913	0.202
5.00	0.144	5.00	0.180	0.913	0.144
7.50	0.074	7.50	0.108	0.913	0.074
10.00	0.043	10.00	0.071	0.913	0.043

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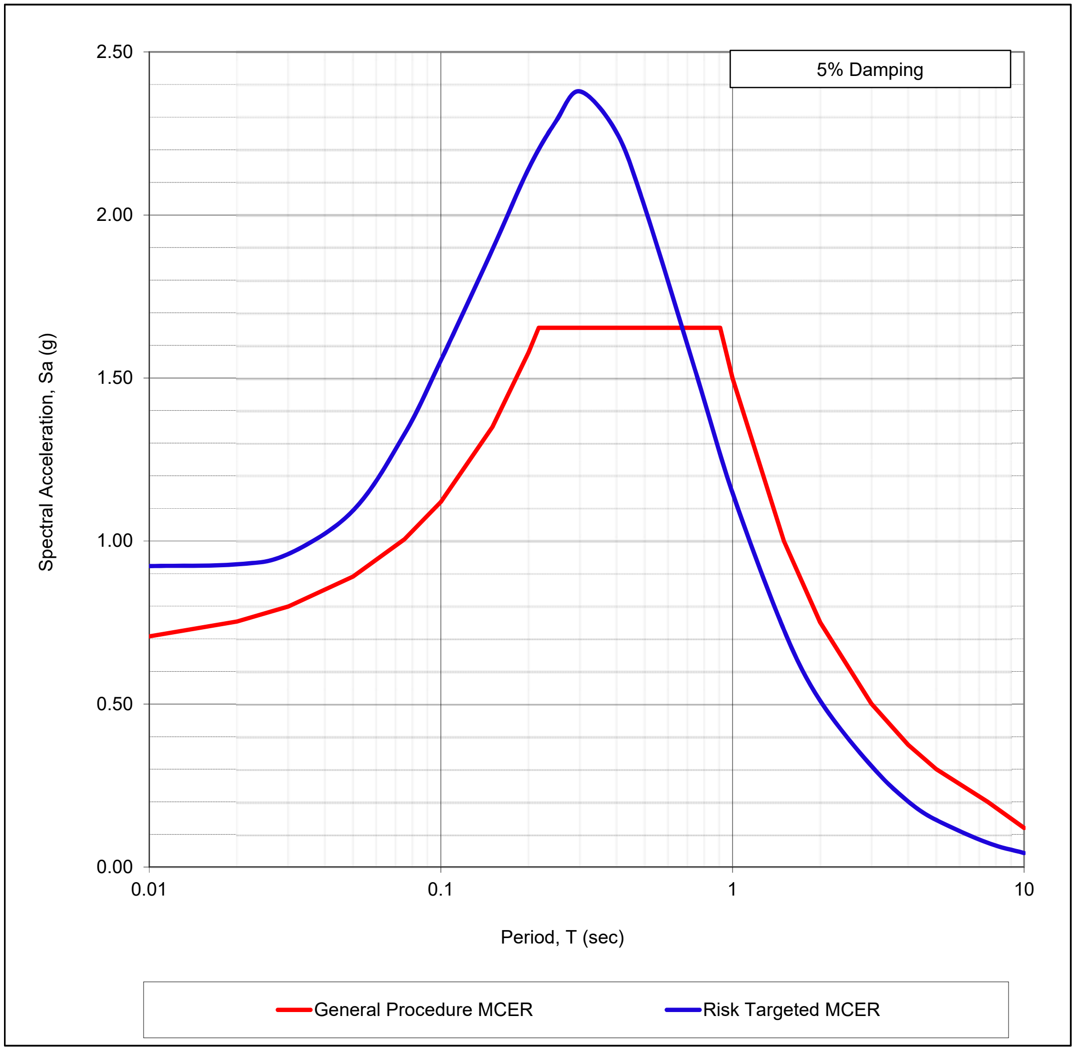
Figure D.3C

RISK TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) RESPONSE SPECTRUM

Project: Proposed Self-Storage Facility
Project Number: 13589.001
Location: 21101 Ventura Boulevard



SITE-SPECIFIC vs. GENERAL CODE-BASED SPECTRA

Period T (s)	DETERM. MCE_R Sa (g)	PROB. MCE_R Sa (g)	**Risk TGT MCE_R Sa (g)**	General Procedure Sa (g)
0.01	0.923	0.965	**0.923**	0.707
0.02	0.929	0.969	**0.929**	0.753
0.03	0.960	1.019	**0.960**	0.799
0.05	1.094	1.221	**1.094**	0.891
0.075	1.328	1.564	**1.328**	1.005
0.10	1.554	1.845	**1.554**	1.120
0.15	1.893	2.170	**1.893**	1.349
0.20	2.143	2.347	**2.143**	1.579
0.25	2.292	2.427	**2.292**	1.654
0.30	2.379	2.415	**2.379**	1.654
0.40	2.342	2.254	**2.254**	1.654
0.50	2.149	2.024	**2.024**	1.654
0.75	1.656	1.515	**1.515**	1.654
1.00	1.260	1.148	**1.148**	1.500
1.50	0.780	0.728	**0.728**	1.000
2.00	0.542	0.510	**0.510**	0.750
3.00	0.309	0.313	**0.309**	0.500
4.00	0.202	0.218	**0.202**	0.375
5.00	0.144	0.165	**0.144**	0.300
7.50	0.074	0.099	**0.074**	0.200
10.00	0.043	0.065	**0.043**	0.120

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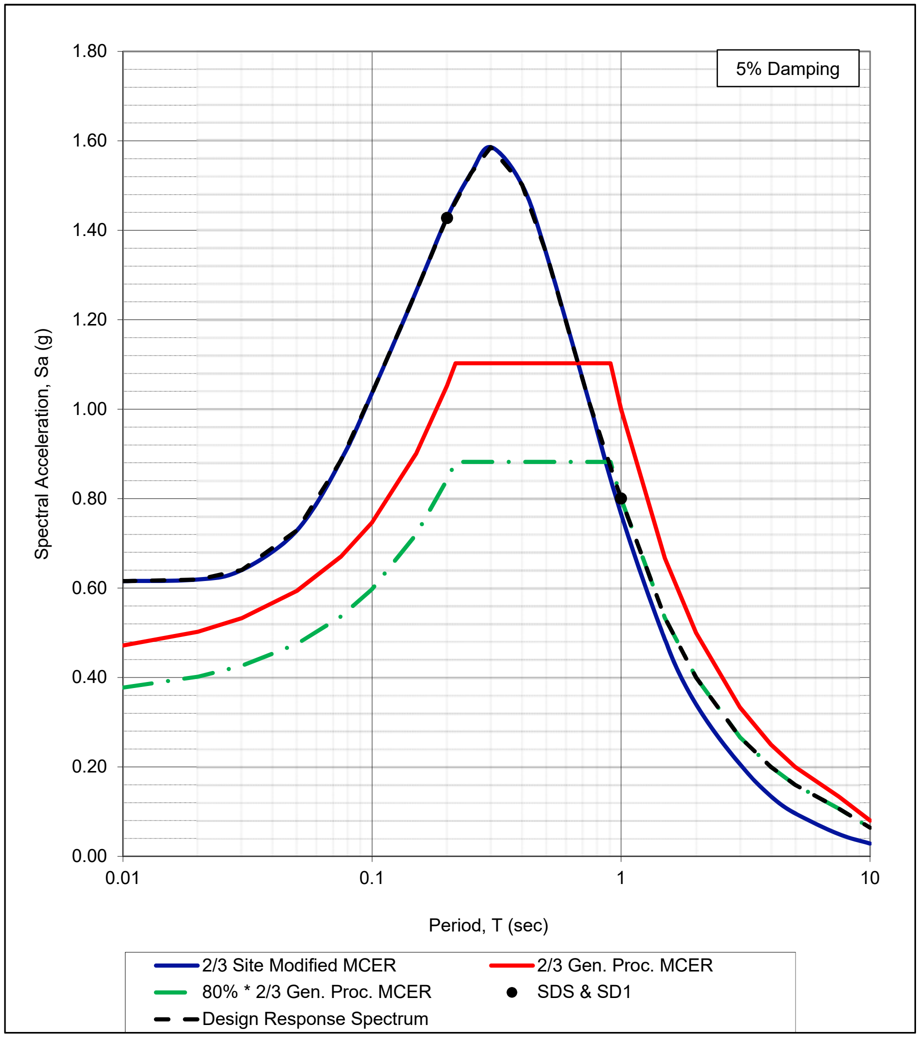
Figure D.3D

ASCE 7-16 DESIGN RESPONSE SPECTRUM AND SITE-SPECIFIC S_{DS} AND S_{D1}

Project:	Proposed Self-Storage Facility
Project Number:	13589.001
Location:	21101 Ventura Boulevard



	CODE BASED GENERAL PROCEDURE SPECTRUM			RISK TGT SPECTRUM	DESIGN RESPONSE SPECTRUM
Period T (s)	GENERAL PROC. MCER CURVE Sa (g)	2/3 GENERAL PROC. MCER CURVE Sa (g)	80% * 2/3 GENERAL PROC. MCER CURVE Sa (g)	2/3*MCE_R CURVE Sa (g)	MAX of 2/3 MCE_R and 80% * 2/3 GENERAL PROC. MCER Sa (g)
0.01	0.707	0.472	0.377	0.615	0.615
0.02	0.753	0.502	0.402	0.619	0.619
0.03	0.799	0.533	0.426	0.640	0.640
0.05	0.891	0.594	0.475	0.729	0.729
0.075	1.005	0.670	0.536	0.885	0.885
0.10	1.120	0.747	0.597	1.036	1.036
0.15	1.349	0.900	0.720	1.262	1.262
0.20	1.579	1.052	0.842	1.428	1.428
0.25	1.654	1.103	0.882	1.528	1.528
0.30	1.654	1.103	0.882	1.586	1.586
0.40	1.654	1.103	0.882	1.502	1.502
0.50	1.654	1.103	0.882	1.349	1.349
0.75	1.654	1.103	0.882	1.010	1.010
1.00	1.500	1.000	0.800	0.765	0.800
1.50	1.000	0.667	0.533	0.485	0.533
2.00	0.750	0.500	0.400	0.340	0.400
3.00	0.500	0.333	0.267	0.206	0.267
4.00	0.375	0.250	0.200	0.134	0.200
5.00	0.300	0.200	0.160	0.096	0.160
7.50	0.200	0.133	0.107	0.050	0.107
10.00	0.120	0.080	0.064	0.029	0.064

S_{DS} = 1.427 g

S_{D1} = 0.800 g

Note: Based on ASCE 7-16 Section 21.4, the parameter S_{DS} shall be taken as 90% of the maximum spectral acceleration, Sa, obtained from the site-specific spectrum, at any period within the range from 0.2 to 5 s, inclusive. The parameter S_{D1} shall be taken as the maximum value of the product, TSa, for periods from 1 to 2 s for sites with Vs_{30} > 1,200 ft/s (Vs_{30} > 365.76 m/s) and for periods from 1 to 5 s for sites with Vs_{30} ≤ 1,200 ft/s (Vs_{30} ≤ 365.76 m/s). The design Sa shall not be less than 80% of 2/3 of the general procedure (ASCE 7-16 Sec 11.4.6)

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Figure D.3E

Anacapa-Dume Alt 2 (0) Fault

PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

WEIGHTED AVERAGE of 2014 NGA WEST-2 GMPEs

Last updated: 04 14 15

by Emel Seyhan, PhD, PEER & UCLA -- email: emel.seyhan@gmail.com, peer_center@berkeley.edu

This excel file will be updated as necessary on the PEER website to fix any typos or other errors. Please check the website frequently for new versions at: http://peer.berkeley.edu/ngawest2/databases/

Legend	Pre-defined option	Main input variable	Calculated variable	Input var. flag	Internal variable

GMPE averaging	Geometric	*Weighted average of the natural logarithm of the spectral values*

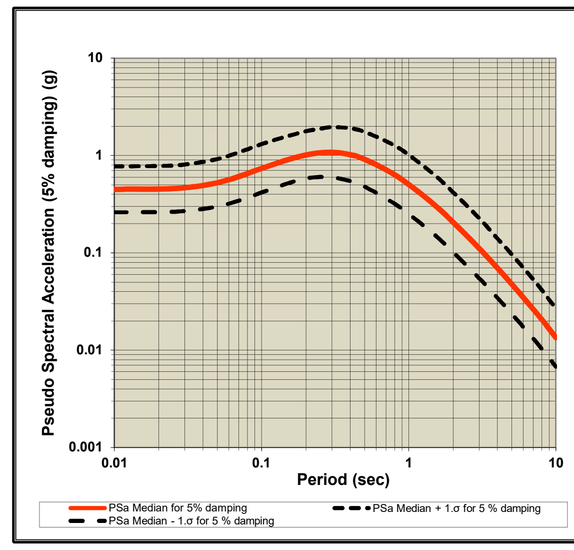
GMPEs	ASK14	BSSA14	CB14	CY14	I14
Weight	0.25	0.25	0.25	0.25	0

# of std. dev.	1	
Damping ratio (%)	5	*Modification factors are calculated in Sheet DSF*

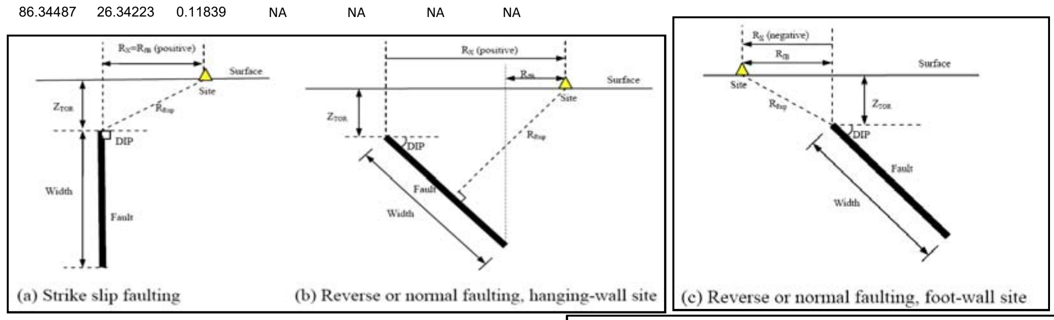
ASK14 Abrahamson & Silva & Kamai 2014 NGA West-2 Model
BSSA14 Boore & Stewart & Seyhan & Atkinson 2014 NGA West-2 Model
CB14 Campbell & Bozorgnia 2014 NGA West-2 Model
CY14 Chiou & Youngs 2014 NGA West-2 Model
I14 Idriss 2014 NGA West-2 Model

RotD50 Horizontal Component of PGA, PGV and *IMs*

GMP	T (s)	Baseline: 5% Damping				User defined: 5% Damping			
		PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	0.01	0.44995	0.77575	0.26098	0.00112	0.44995	0.77575	0.26098	0.00112
	0.02	0.45230	0.78030	0.26218	0.00449	0.45230	0.78030	0.26218	0.00449
	0.03	0.46572	0.80711	0.26873	0.01040	0.46572	0.80711	0.26873	0.01040
	0.05	0.52406	0.91893	0.29887	0.03252	0.52406	0.91893	0.29887	0.03252
	0.075	0.62908	1.11583	0.35466	0.08784	0.63034	1.11807	0.35537	0.08802
	0.1	0.73630	1.30576	0.41519	0.18278	0.73851	1.30968	0.41643	0.18332
	0.15	0.90110	1.57728	0.51480	0.50329	0.90290	1.58043	0.51583	0.50430
	0.2	1.01263	1.77070	0.57910	1.00548	1.01465	1.77425	0.58026	1.00750
	0.25	1.06269	1.87867	0.60113	1.64875	1.06482	1.88242	0.60233	1.65205
	0.3	1.07912	1.94990	0.59721	2.41090	1.08020	1.95185	0.59781	2.41331
	0.4	1.01965	1.90370	0.54614	4.04983	1.02067	1.90560	0.54668	4.05388
	0.5	0.91498	1.74695	0.47923	5.67828	0.91589	1.74869	0.47971	5.68396
	0.75	0.67426	1.33579	0.34034	9.41492	0.67426	1.33579	0.34034	9.41492
	1	0.50531	1.01620	0.25127	12.54371	0.50531	1.01620	0.25127	12.54371
	1.5	0.30997	0.62882	0.15280	17.31303	0.31028	0.62945	0.15295	17.33035
	2	0.20677	0.42052	0.10167	20.53076	0.20656	0.42010	0.10156	20.51023
	3	0.11233	0.22864	0.05519	25.09550	0.11222	0.22841	0.05513	25.07041
	4	0.06967	0.14043	0.03457	27.67199	0.06960	0.14029	0.03453	27.64432
	5	0.04807	0.09703	0.02382	29.83276	0.04793	0.09674	0.02374	29.74326
	7.5	0.02349	0.04722	0.01169	32.80644	0.02342	0.04708	0.01165	32.70802
	10	0.01350	0.02683	0.00679	33.51384	0.01345	0.02672	0.00677	33.37978
PGA (g)	0	0.44765	0.77119	0.25985	0.00111	0.44765	0.77119	0.25985	0.00111
PGV (cm/s)	-1	47.69189	86.34487	26.34223	0.11839	NA	NA	NA	NA

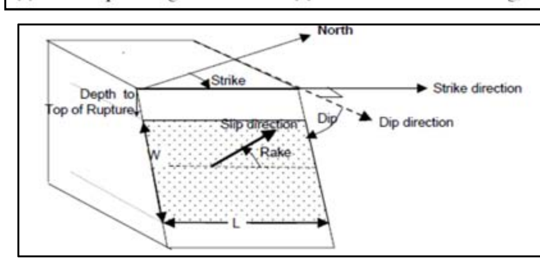


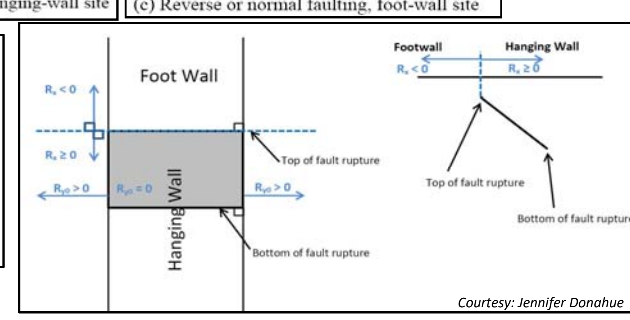
Input variables	Errors and warnings
M_w: 7.16	
R_{RUP} (km): 12.03	
R_{JB} (km): 5.28	
R_X (km): 15.68	
Ry0 (km): 999	*If unknown use 999*
V_{S30} (m/sec): 352	
U (BSSA13): 0	*1: Unspecified fault mech.*
F_{RV}: 1	*1: reverse fault*
F_{NM}: 0	*1: normal fault*
F_{HW}: 1	*1: hanging wall side*
Dip (deg): 41	
Z_{TOR} (km): 1.2	*If unknown use 999*
Z_{HYP} (km): 999	*If unknown use 999*
$Z_{1.0}$ (km): 0.25	*If unknown use 999*
$Z_{2.5}$ (km): 2.3	*If unknown use 999*
W (km): 13.99	*If unknown use 999*
Vs30Flag: inferred	*Choose options for V_{S30} from the list*
F_{AS}: no	*Aftershock effect is not applicable.*
Region: California	*Choose region from the list*



Calculated Variables/Flags

ΔDPP	0	*Always 0 for median calcs.*
PGA_r (g)	0.340	
Z_{BOT} (km) (CB14)	15	*Enter for default W calcs*
SS	0	*auto calculated*
$V_{s30Flag}$	0	*inferred*
F_{AS}	0	*Aftershock effect is not applicable.*
Region	0	*California*
Option for Sa value	1	*Weighted average of the natural logarithm of the spectral values*





Definition of Parameters

Damping ratio = Viscous damping ratio (%) See Sanaz et al. (2012) PEER Report
PSA = Pseudo-absolute acceleration response spectrum (g)
PGA = Peak ground acceleration (g)
PGV = Peak ground velocity (cm/s)
S_d = Relative displacement response spectrum (cm)
M_w = Moment magnitude
R_{RUP} = Closest distance to coseismic rupture (km), used in ASK13, CB13 and CY13. See Figures a, b and c for illustration
R_{JB} = Closest distance to surface projection of coseismic rupture (km). See Figures a, b and c for illustation
R_X = Horizontal distance from top of rupture measured perpendicular to fault strike (km). See Figures a, b and c for illustation
R_{y0} = The horizontal distance off the end of the rupture measured parallel to strike (km)
V_{S30} = The average shear-wave velocity (m/s) over a subsurface depth of 30 m
U = Unspecified-mechanism factor: 1 for unspecified; 0 otherwise
F_{RV} = Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
F_{NM} = Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal
F_{HW} = Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise
Dip = Average dip of rupture plane (degrees)
Z_{TOR} = Depth to top of coseismic rupture (km)
Z_{HYP} = Hypocentral depth from the earthquake
$Z_{1.0}$ = Depth to Vs=1 km/sec
$Z_{2.5}$ = Depth to Vs=2.5 km/sec
W = Fault rupture width (km)
$V_{s30flag}$ = 1 for measured, 0 for inferred Vs30
F_{AS} = 0 for mainshock; 1 for aftershock
Region = Specific regions considered in the models, Click on Region to see codes
ΔDPP = Directivity term, direct point parameter; uses 0 for median predictions
PGA_r (g) = Peak ground acceleration on rock (g), this specific cell is updated in the cell for BSSA14 and CB14, for others it is taken account for in the macros
Z_{BOT} (km) = The depth to the bottom of the seismogenic crust
Z_{BOR} (km) = The depth to the bottom of the rupture plane
SS = 1 for strike slip, automatically updated in the cell

Input variables with defaults (If entered 999 as input):

Red colored value: The value is used in the code when input is unknown

DEFAULTs	USER defined	ASK14	BSSA14	CB14	CY14	I14
W (km)	13.99			21.030		
$Z_{1.0}$ (km)	0.250	0.250			0.411	
$\delta Z_{1.0}$ (km)	-0.161		-0.161			
$Z_{2.5}$ (V_{S30}=1100)(km)	2.300			0.398		
$Z_{2.5}$ (V_{S30})(km)	2.300			1.464		
Z_{hyp} (km)	999.00			11.429		
Z_{tor} (km)	1.20			1.203	1.203	
Z_{BOR} (km)	-			15.000		

ACKNOWLEDGEMENTS

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Silvia Mazzoni, Consultant

All NGA West-2 participants are acknowledged for their constructive comments and feedback.

Compton (4) Fault

PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

WEIGHTED AVERAGE of 2014 NGA WEST-2 GMPEs

Last updated: 04 14 15

by Emel Seyhan, PhD, PEER & UCLA -- email: emel.seyhan@gmail.com, peer_center@berkeley.edu

This excel file will be updated as necessary on the PEER website to fix any typos or other errors. Please check the website frequently for new versions at: http://peer.berkeley.edu/ngawest2/databases/

Legend	Pre-defined option	Main input variable	Calculated variable	Input var. flag	Internal variable

GMPE averaging	Geometric

Weighted average of the natural logarithm of the spectral values

GMPEs	ASK14	BSSA14	CB14	CY14	I14
Weight	0.25	0.25	0.25	0.25	0

# of std. dev.	1
Damping ratio (%)	5

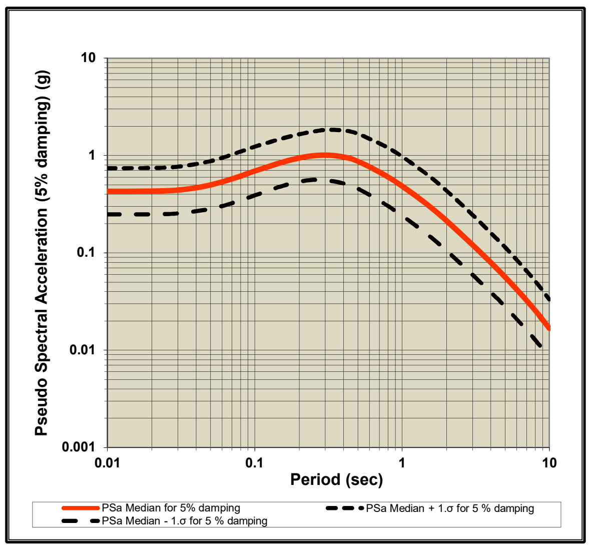
Modification factors are calculated in Sheet DSF

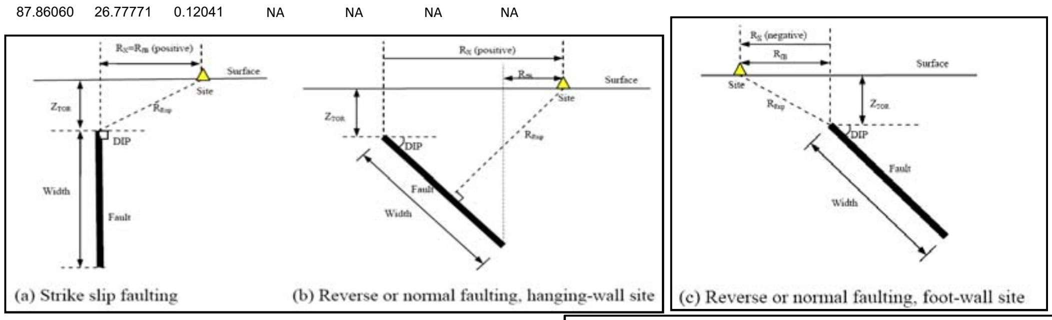
ASK14 Abrahamson & Silva & Kamai 2014 NGA West-2 Model
BSSA14 Boore & Stewart & Seyhan & Atkinson 2014 NGA West-2 Model
CB14 Campbell & Bozorgnia 2014 NGA West-2 Model
CY14 Chiou & Youngs 2014 NGA West-2 Model
I14 Idriss 2014 NGA West-2 Model

RotD50 Horizontal Component of PGA, PGV and *IMs*

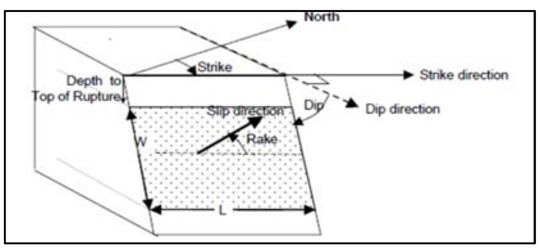
Input variables	Errors and warnings
M_w: 7.45	
R_{RUP} (km): 13.41	
R_{JB} (km): 8.06	
R_X (km): 10.48	
Ry0 (km): 999	*If unknown use 999*
V_{S30} (m/sec): 352	
U (BSSA13): 0	*1: Unspecified fault mech.*
F_{RV}: 1	*1: reverse fault*
F_{NM}: 0	*1: normal fault*
F_{HW}: 1	*1: hanging wall side*
Dip (deg): 20	
Z_{TOR} (km): 5.2	*If unknown use 999*
Z_{HYP} (km): 999	*If unknown use 999*
$Z_{1.0}$ (km): 0.25	*If unknown use 999*
$Z_{2.5}$ (km): 2.3	*If unknown use 999*
W (km): 27.37	*If unknown use 999*
Vs30Flag: inferred	*Choose options for V_{S30} from the list*
F_{AS}: no	*Aftershock effect is not applicable.*
Region: California	*Choose region from the list*

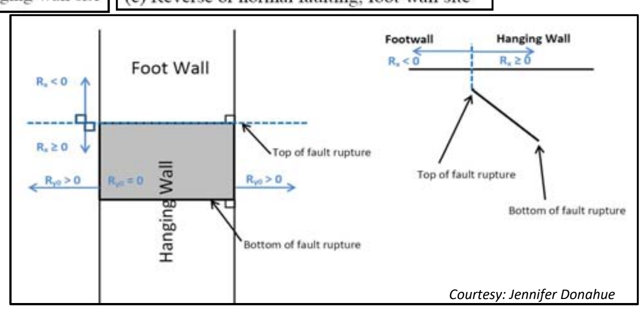
GMP	T (s)	Baseline: 5% Damping PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	User defined: 5% Damping PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	0.01	0.42921	0.74152	0.24844	0.00107	0.42921	0.74152	0.24844	0.00107
	0.02	0.43090	0.74496	0.24925	0.00428	0.43090	0.74496	0.24925	0.00428
	0.03	0.44185	0.76748	0.25438	0.00987	0.44141	0.76671	0.25412	0.00986
	0.05	0.49704	0.87387	0.28271	0.03085	0.49704	0.87387	0.28271	0.03085
	0.075	0.59380	1.05654	0.33373	0.08291	0.59558	1.05971	0.33473	0.08316
	0.1	0.69083	1.22925	0.38824	0.17149	0.69290	1.23294	0.38940	0.17200
	0.15	0.84197	1.47862	0.47944	0.47027	0.84365	1.48157	0.48040	0.47121
	0.2	0.94146	1.65072	0.53694	0.93482	0.94334	1.65403	0.53801	0.93669
	0.25	0.99323	1.75961	0.56064	1.54098	0.99522	1.76313	0.56177	1.54406
	0.3	1.00815	1.82545	0.55678	2.25234	1.01017	1.82910	0.55789	2.25685
	0.4	0.96693	1.80906	0.51682	3.84044	0.96790	1.81087	0.51733	3.84428
	0.5	0.87283	1.66897	0.45647	5.41674	0.87371	1.67064	0.45693	5.42215
	0.75	0.63939	1.26742	0.32256	8.92806	0.63939	1.26742	0.32256	8.92806
	1	0.48513	0.97588	0.24117	12.04274	0.48513	0.97588	0.24117	12.04274
	1.5	0.30908	0.62704	0.15235	17.26317	0.30939	0.62767	0.15250	17.28043
	2	0.21479	0.43680	0.10562	21.32703	0.21436	0.43592	0.10540	21.28437
	3	0.12150	0.24731	0.05970	27.14582	0.12138	0.24706	0.05964	27.11867
	4	0.07938	0.15999	0.03938	31.52753	0.07930	0.15983	0.03934	31.49600
	5	0.05677	0.11458	0.02812	35.22900	0.05660	0.11424	0.02804	35.12332
	7.5	0.02891	0.05810	0.01438	40.36252	0.02879	0.05787	0.01432	40.20107
	10	0.01680	0.03339	0.00846	41.71348	0.01674	0.03325	0.00842	41.54663
PGA (g)	0	0.42711	0.73732	0.24742	0.00106	0.42711	0.73732	0.24742	0.00106
PGV (cm/s)	-1	48.50470	87.86060	26.77771	0.12041	NA	NA	NA	NA





(a) Strike slip faulting (b) Reverse or normal faulting, hanging-wall site (c) Reverse or normal faulting, foot-wall site





Courtesy: Jennifer Donahue

Calculated Variables/Flags

ΔDPP: 0	*Always 0 for median calcs.*
PGA_r (g): 0.301	
Z_{BOT} (km) (CB14): 15	*Enter for default W calcs*
SS: 0	*auto calculated*
$V_{s30Flag}$: 0	*inferred*
F_{AS}: 0	*Aftershock effect is not applicable.*
Region: 0	*California*
Option for Sa value: 1	*Weighted average of the natural logarithm of the spectral values*

Definition of Parameters

Damping ratio = Viscous damping ratio (%) See Sanaz et al. (2012) PEER Report
PSA = Pseudo-absolute acceleration response spectrum (g)
PGA = Peak ground acceleration (g)
PGV = Peak ground velocity (cm/s)
S_d = Relative displacement response spectrum (cm)
M_w = Moment magnitude
R_{RUP} = Closest distance to coseismic rupture (km), used in ASK13, CB13 and CY13. See Figures a, b and c for illustation
R_{JB} = Closest distance to surface projection of coseismic rupture (km). See Figures a, b and c for illustation
R_X = Horizontal distance from top of rupture measured perpendicular to fault strike (km). See Figures a, b and c for illustation
R_{y0} = The horizontal distance off the end of the rupture measured parallel to strike (km)
V_{S30} = The average shear-wave velocity (m/s) over a subsurface depth of 30 m
U = Unspecified-mechanism factor: 1 for unspecified; 0 otherwise
F_{RV} = Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
F_{NM} = Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal
F_{HW} = Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise
Dip = Average dip of rupture plane (degrees)
Z_{TOR} = Depth to top of coseismic rupture (km)
Z_{HYP} = Hypocentral depth from the earthquake
$Z_{1.0}$ = Depth to Vs=1 km/sec
$Z_{2.5}$ = Depth to Vs=2.5 km/sec
W = Fault rupture width (km)
$V_{s30flag}$ = 1 for measured, 0 for inferred Vs30
F_{AS} = 0 for mainshock; 1 for aftershock
Region = Specific regions considered in the models, Click on Region to see codes
ΔDPP = Directivity term, direct point parameter; uses 0 for median predictions
PGA_r (g) = Peak ground acceleration on rock (g), this specific cell is updated in the cell for BSSA14 and CB14, for others it is taken account for in the macros
Z_{BOT} (km) = The depth to the bottom of the seismogenic crust
Z_{BOR} (km) = The depth to the bottom of the rupture plane
SS = 1 for strike slip, automatically updated in the cell

Input variables with defaults (If entered 999 as input):

Red colored value: The value is used in the code when input is unknown

DEFAULTs	USER defined	ASK14	BSSA14	CB14	CY14	I14
W (km)	27.37			42.251		
$Z_{1.0}$ (km)	0.250	0.250			0.411	
$\delta Z_{1.0}$ (km)	-0.161		-0.161			
$Z_{2.5}$ (V_{S30}=1100)(km)	2.300			0.398		
$Z_{2.5}$ (V_{S30})(km)	2.300			1.464		
Z_{hyp} (km)	999.00			5.000		
Z_{tor} (km)	5.20			0.549	0.549	
Z_{BOR} (km)	-			15.000		

ACKNOWLEDGEMENTS

Nick Gregor, Bechtel
Silvia Mazzoni, Consultant

All NGA West-2 participants are acknowledged for their constructive comments and feedback.

Hazards by Location

Search Information

Coordinates: 34.168161, -118.59298

Elevation: 898 ft

Timestamp: 2022-07-01T19:39:58.959Z

Hazard Type: Seismic

Reference Document: ASCE7-16

Risk Category: II

Site Class: D

Basic Parameters

Name	Value	Description
S_S	1.654	MCE_R ground motion (period=0.2s)
S_1	0.6	MCE_R ground motion (period=1.0s)
S_{MS}	1.654	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	1.103	Numeric seismic design value at 0.2s SA
S_{D1}	* null	Numeric seismic design value at 1.0s SA

* See Section 11.4.8

▾Additional Information

Name	Value	Description
SDC	* null	Seismic design category
F_a	1	Site amplification factor at 0.2s
F_v	* null	Site amplification factor at 1.0s
CR_S	0.933	Coefficient of risk (0.2s)
CR_1	0.913	Coefficient of risk (1.0s)
PGA	0.679	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.747	Site modified peak ground acceleration
T_L	8	Long-period transition period (s)
SsRT	1.758	Probabilistic risk-targeted ground motion (0.2s)
SsUH	1.884	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.654	Factored deterministic acceleration value (0.2s)

S1RT	0.619	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.678	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.6	Factored deterministic acceleration value (1.0s)
PGAd	0.679	Factored deterministic acceleration value (PGA)

* See Section 11.4.8

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.

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U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Input

Edition

Dynamic: Conterminous U.S. 2014 (update…

Spectral Period

Peak Ground Acceleration

Latitude

Decimal degrees

34.168161

Time Horizon

Return period in years

2475

Longitude

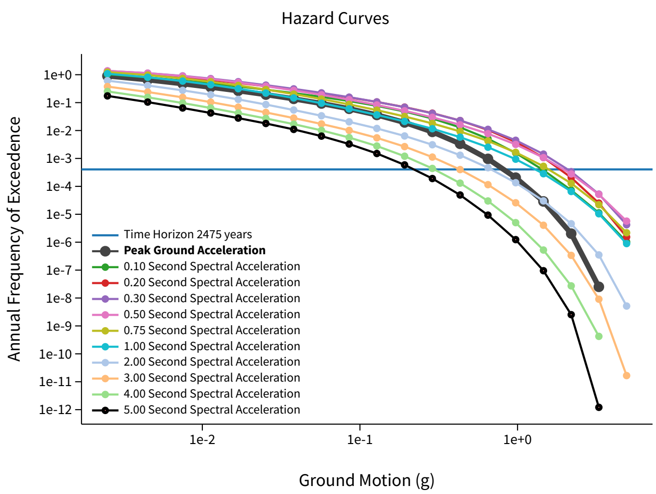
Decimal degrees, negative values for western longitudes

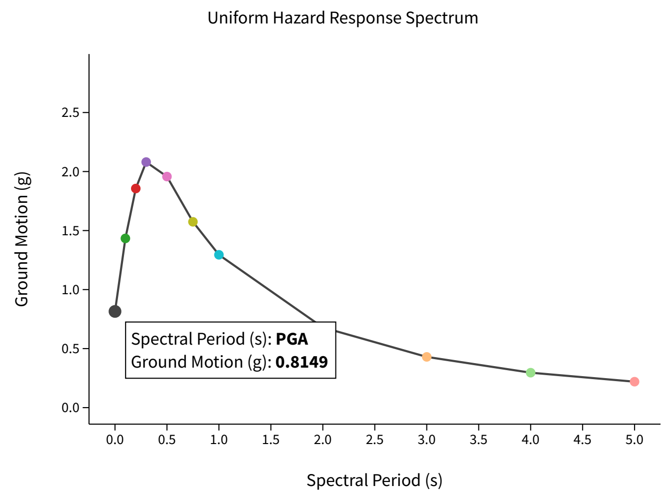
-118.59298

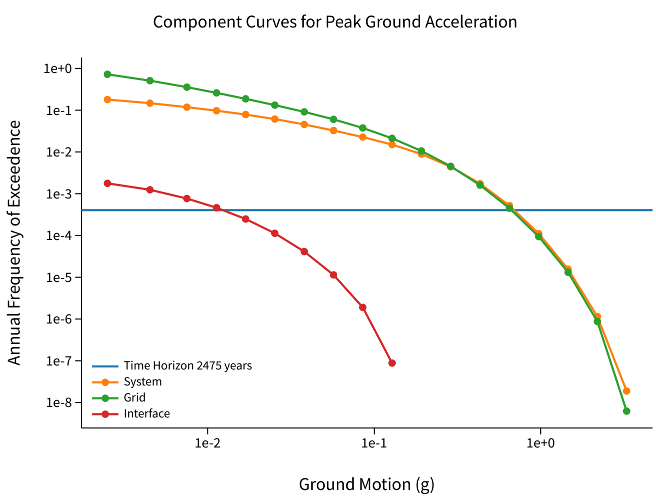
Site Class

259 m/s (Site class D)

Hazard Curve





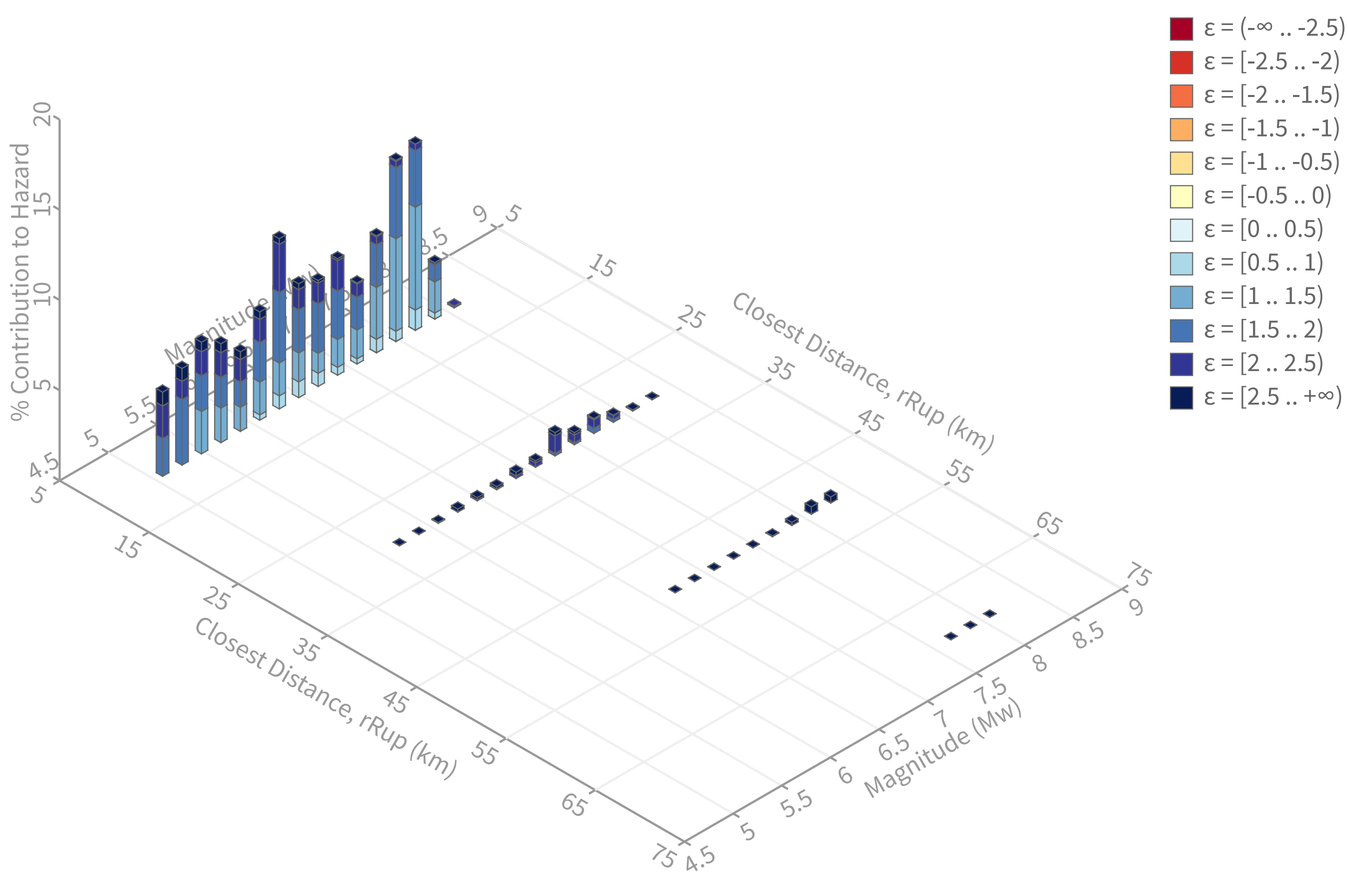


View Raw Data

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.8149071 g

Recovered targets

Return period: 3070.6025 yrs
Exceedance rate: 0.00032566899 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.08 %

Mean (over all sources)

m: 6.63
r: 12.69 km
ε_0: 1.71 σ

Mode (largest m-r bin)

m: 7.69
r: 14.9 km
ε_0: 1.37 σ
Contribution: 10.31 %

Mode (largest m-r-ε_0 bin)

m: 7.69
r: 14.85 km
ε_0: 1.26 σ
Contribution: 5.7 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM32	System							28.24
Santa Susana East (connector) [0]		16.08	6.72	1.80	118.499°W	34.314°N	27.82	4.87
Compton [4]		13.41	7.57	1.22	118.608°W	34.022°N	184.85	3.33
Santa Susana alt 2 [3]		16.21	7.41	1.78	118.564°W	34.312°N	9.39	3.14
Anacapa-Dume alt 2 [0]		12.03	7.33	1.16	118.554°W	34.031°N	166.71	2.97
Northridge [2]		18.27	7.52	1.49	118.550°W	34.343°N	11.35	1.93
Northridge Hills [0]		13.05	7.67	1.52	118.554°W	34.280°N	16.20	1.89
Santa Monica alt 2 [3]		12.38	7.11	1.40	118.540°W	34.039°N	161.09	1.89
Mission Hills 2011 [1]		13.29	7.20	1.76	118.556°W	34.283°N	14.91	1.72
Hollywood [2]		18.13	7.04	2.06	118.422°W	34.084°N	120.73	1.24
UC33brAvg_FM31	System							24.53
Santa Susana East (connector) [0]		16.08	7.12	1.65	118.499°W	34.314°N	27.82	5.06
Northridge [2]		18.27	7.37	1.51	118.550°W	34.343°N	11.35	3.58
Compton [4]		13.41	7.41	1.27	118.608°W	34.022°N	184.85	3.08
Northridge Hills [0]		13.05	7.68	1.52	118.554°W	34.280°N	16.20	1.87
Malibu Coast alt 1 [1]		14.23	7.35	1.65	118.606°W	34.038°N	184.90	1.80
Santa Susana alt 1 [0]		16.54	7.56	1.63	118.544°W	34.310°N	15.94	1.49
Mission Hills 2011 [1]		13.29	6.58	2.09	118.556°W	34.283°N	14.91	1.47
UC33brAvg_FM32 (opt)	Grid							23.73
PointSourceFinite: -118.593, 34.200		5.98	5.78	1.52	118.593°W	34.200°N	0.00	6.49
PointSourceFinite: -118.593, 34.200		5.98	5.78	1.52	118.593°W	34.200°N	0.00	6.49
PointSourceFinite: -118.593, 34.254		9.63	5.97	1.97	118.593°W	34.254°N	0.00	1.41
PointSourceFinite: -118.593, 34.254		9.63	5.97	1.97	118.593°W	34.254°N	0.00	1.41
PointSourceFinite: -118.593, 34.236		8.43	5.85	1.87	118.593°W	34.236°N	0.00	1.09
PointSourceFinite: -118.593, 34.236		8.43	5.85	1.87	118.593°W	34.236°N	0.00	1.09
UC33brAvg_FM31 (opt)	Grid							23.50
PointSourceFinite: -118.593, 34.200		5.98	5.78	1.52	118.593°W	34.200°N	0.00	6.48
PointSourceFinite: -118.593, 34.200		5.98	5.78	1.52	118.593°W	34.200°N	0.00	6.48
PointSourceFinite: -118.593, 34.254		9.64	5.96	1.98	118.593°W	34.254°N	0.00	1.43
PointSourceFinite: -118.593, 34.254		9.64	5.96	1.98	118.593°W	34.254°N	0.00	1.43

Leighton Consulting

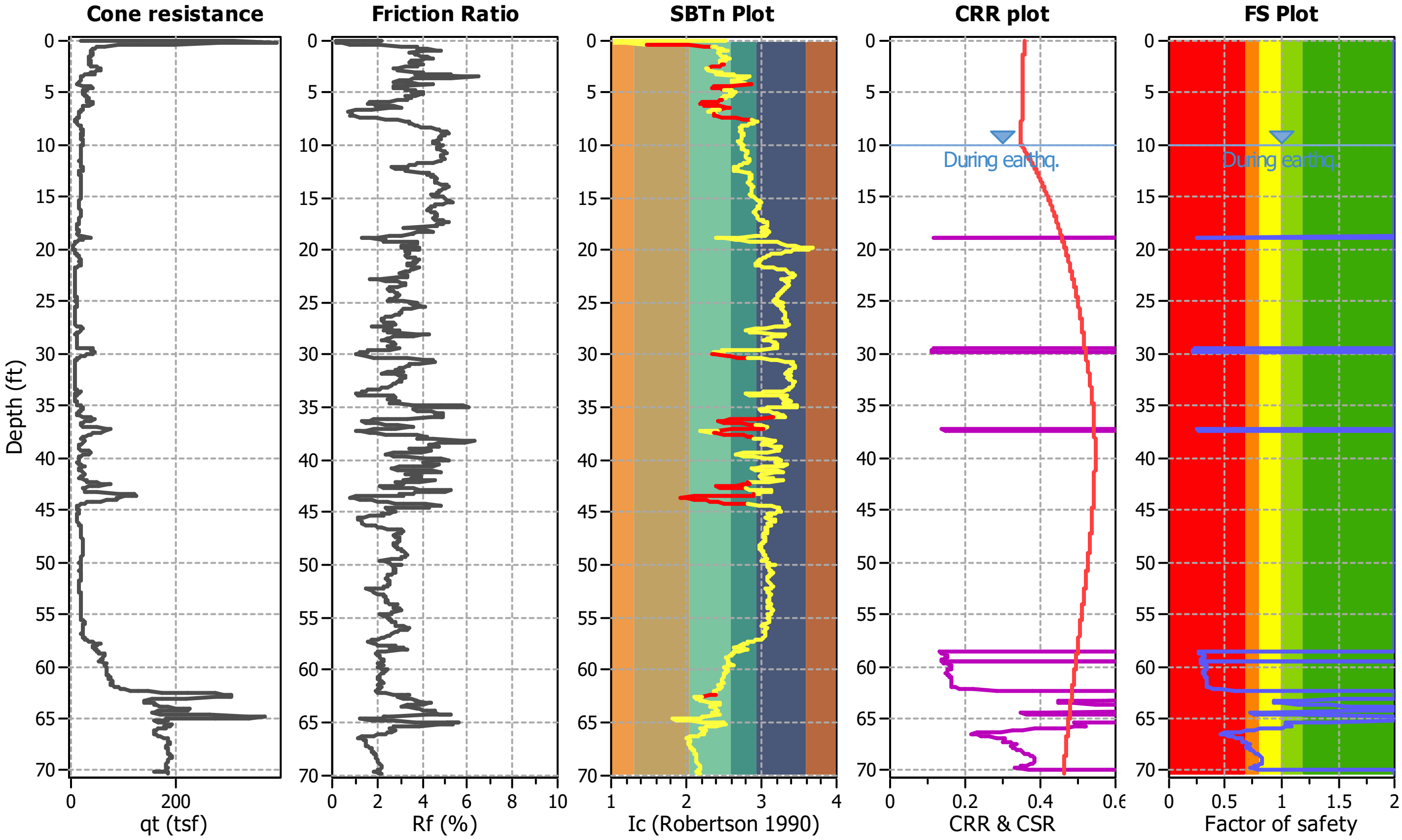
LIQUEFACTION ANALYSIS REPORT

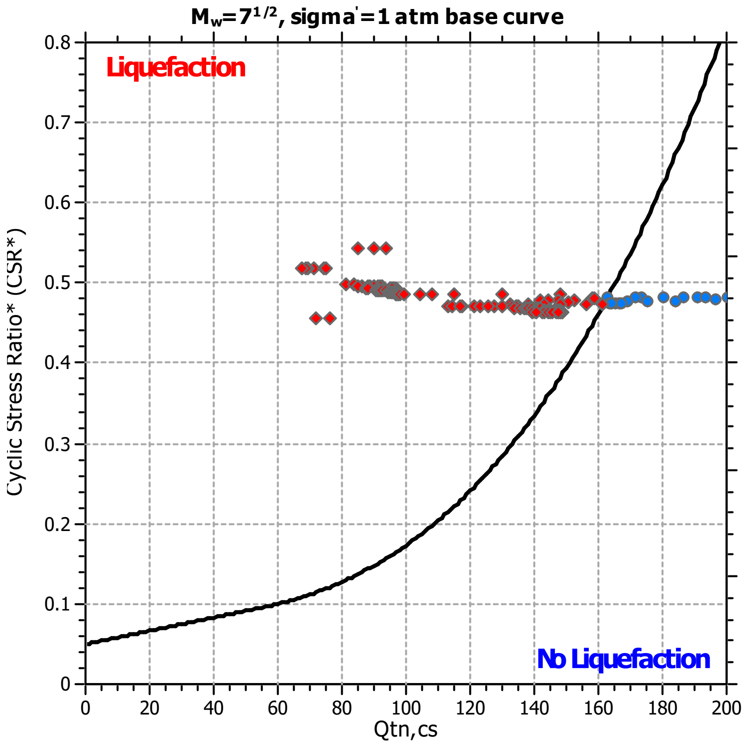
Project title : JDA Woodland Hills **Location : 21101 Ventura Blvd, Woodland Hills, CA**

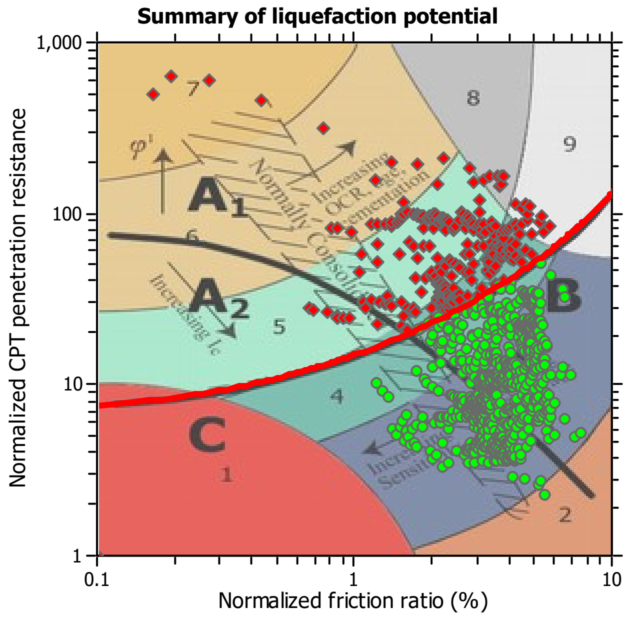
CPT file : CPT-1

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	22.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.63	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.75	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







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

Leighton Consulting

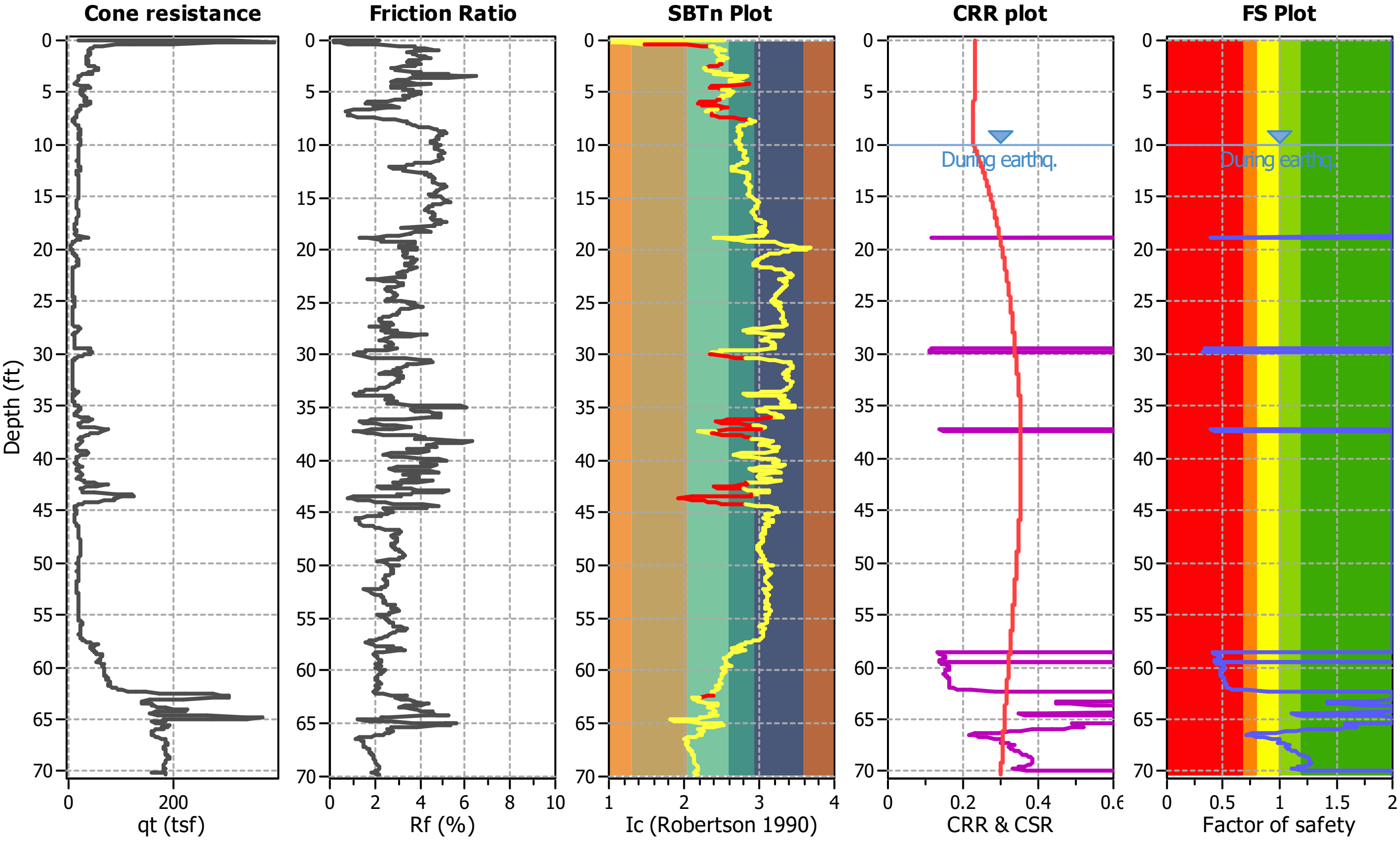
LIQUEFACTION ANALYSIS REPORT

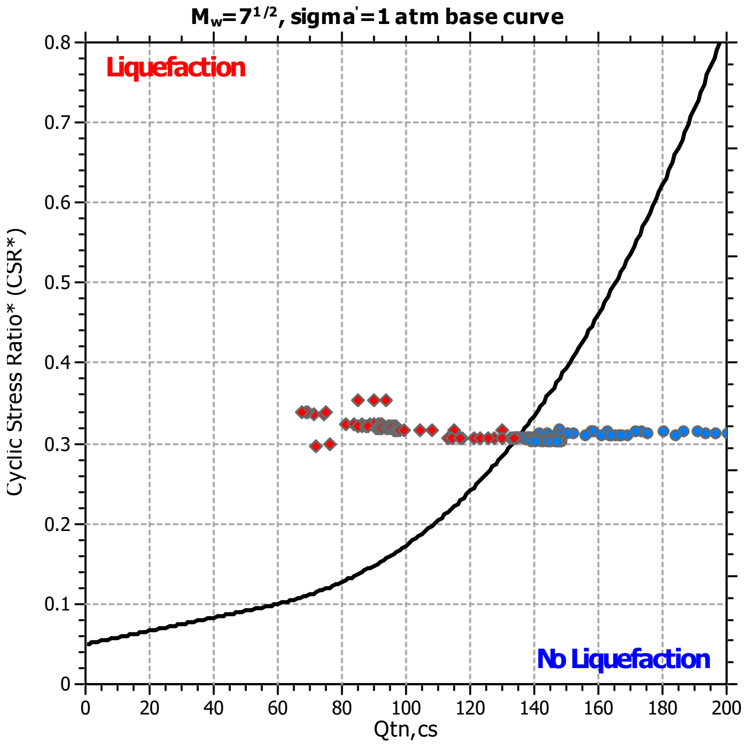
Project title : JDA Woodland Hills **Location : 21101 Ventura Blvd, Woodland Hills, CA**

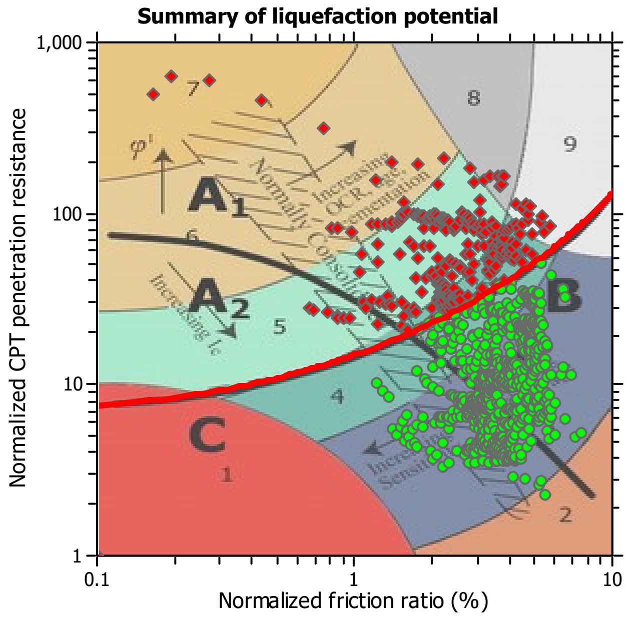
CPT file : CPT-1

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	22.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	6.57	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based







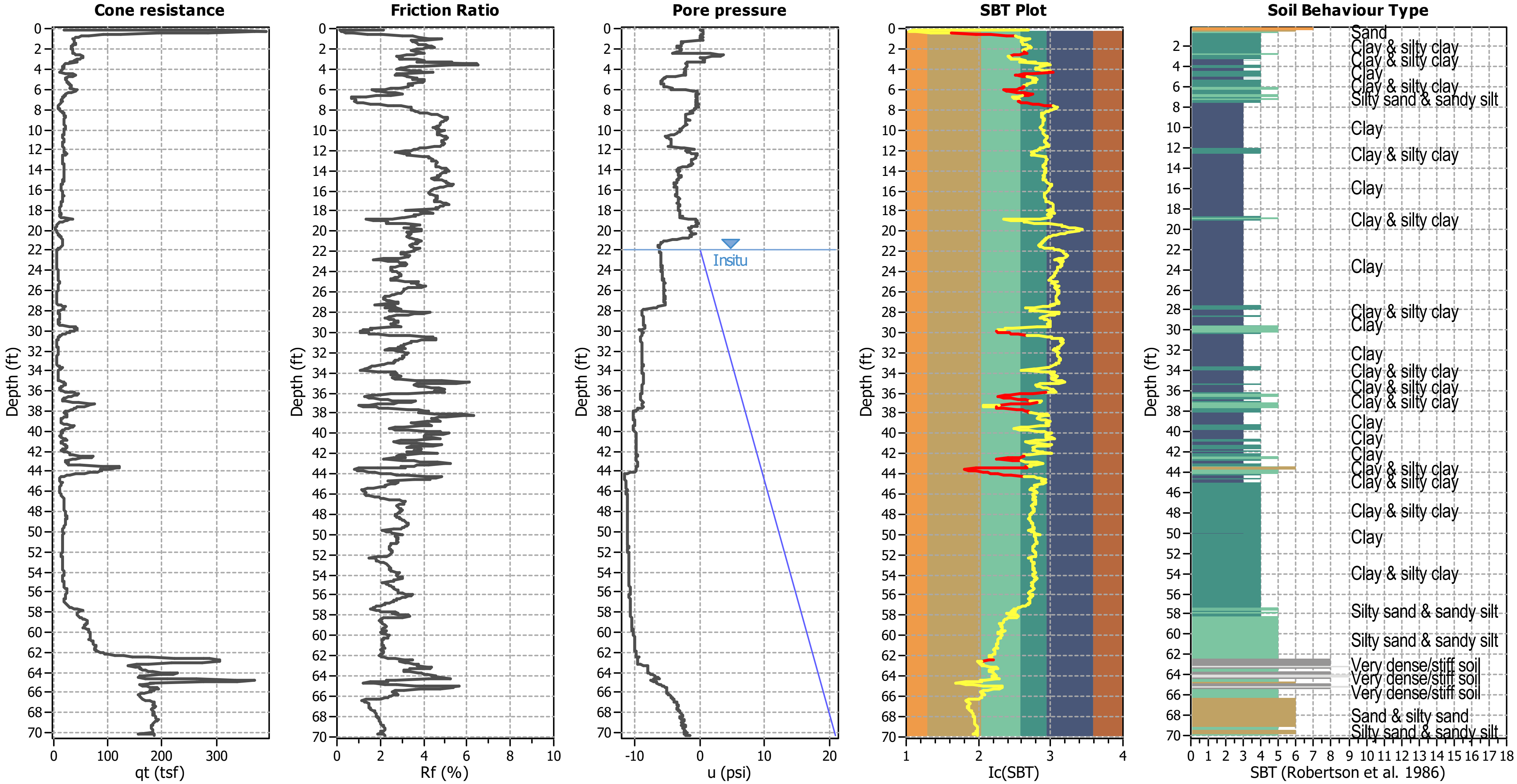
Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading

Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry

Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening

Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



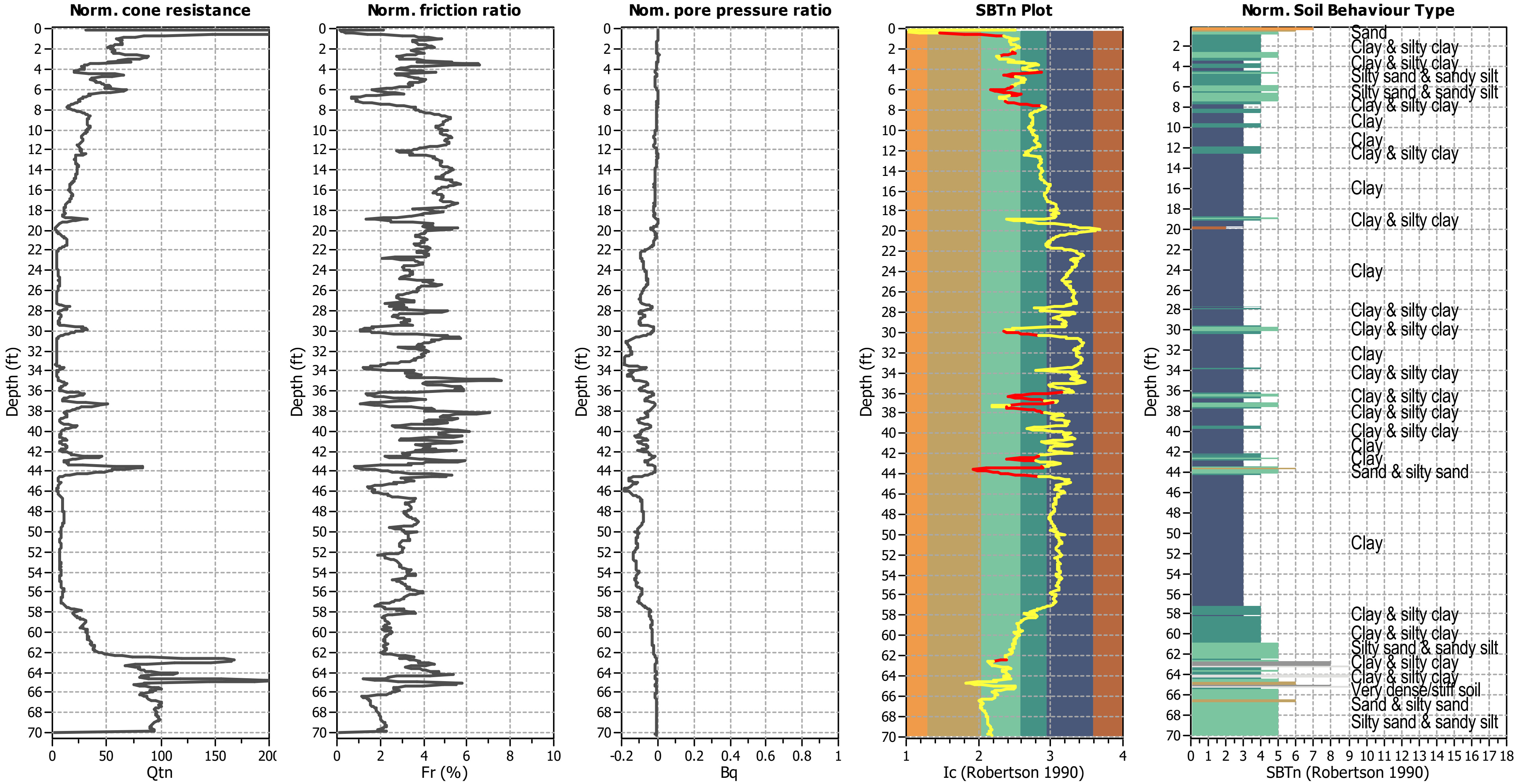
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.57	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	22.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots (normalized)



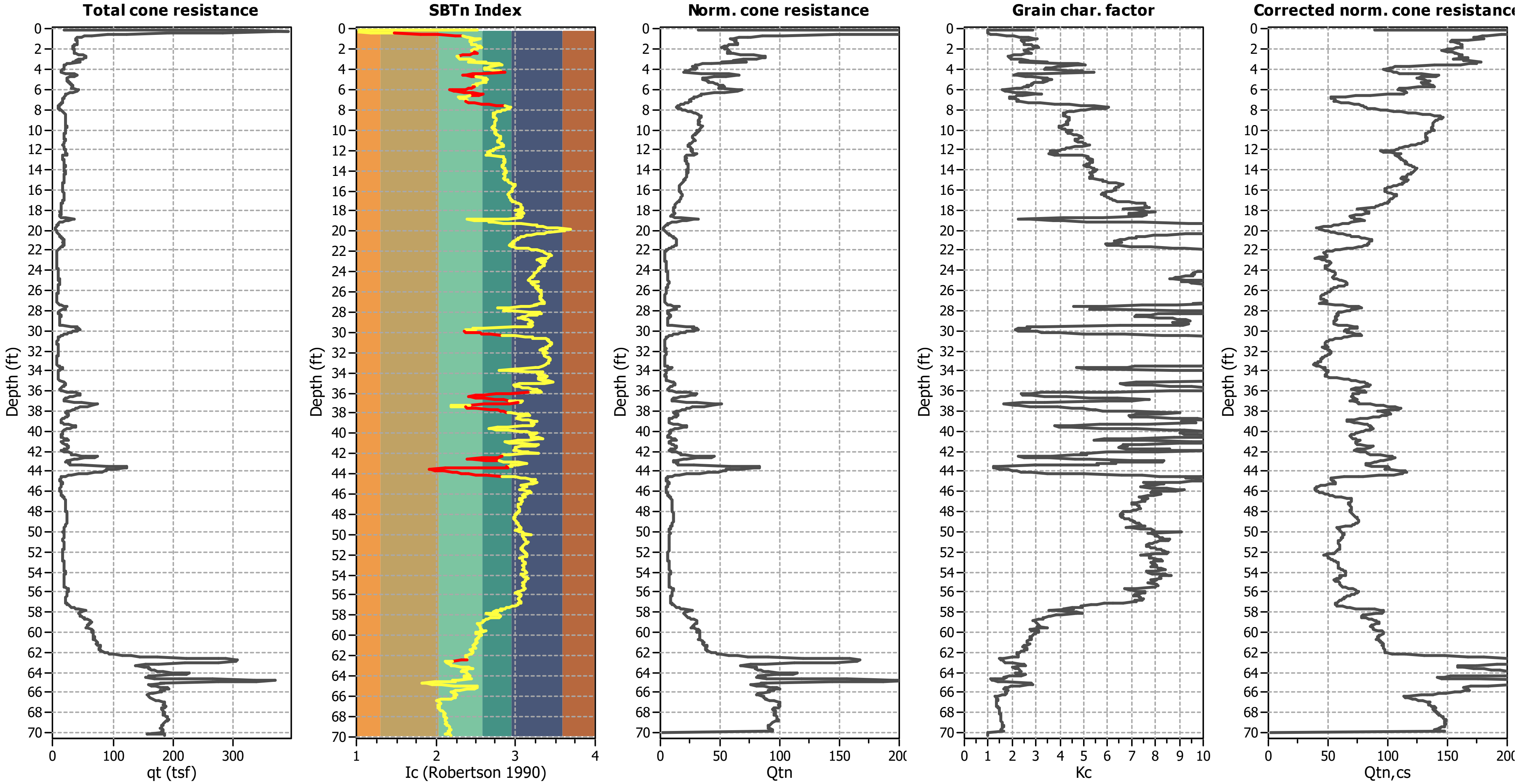
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.57	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	22.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

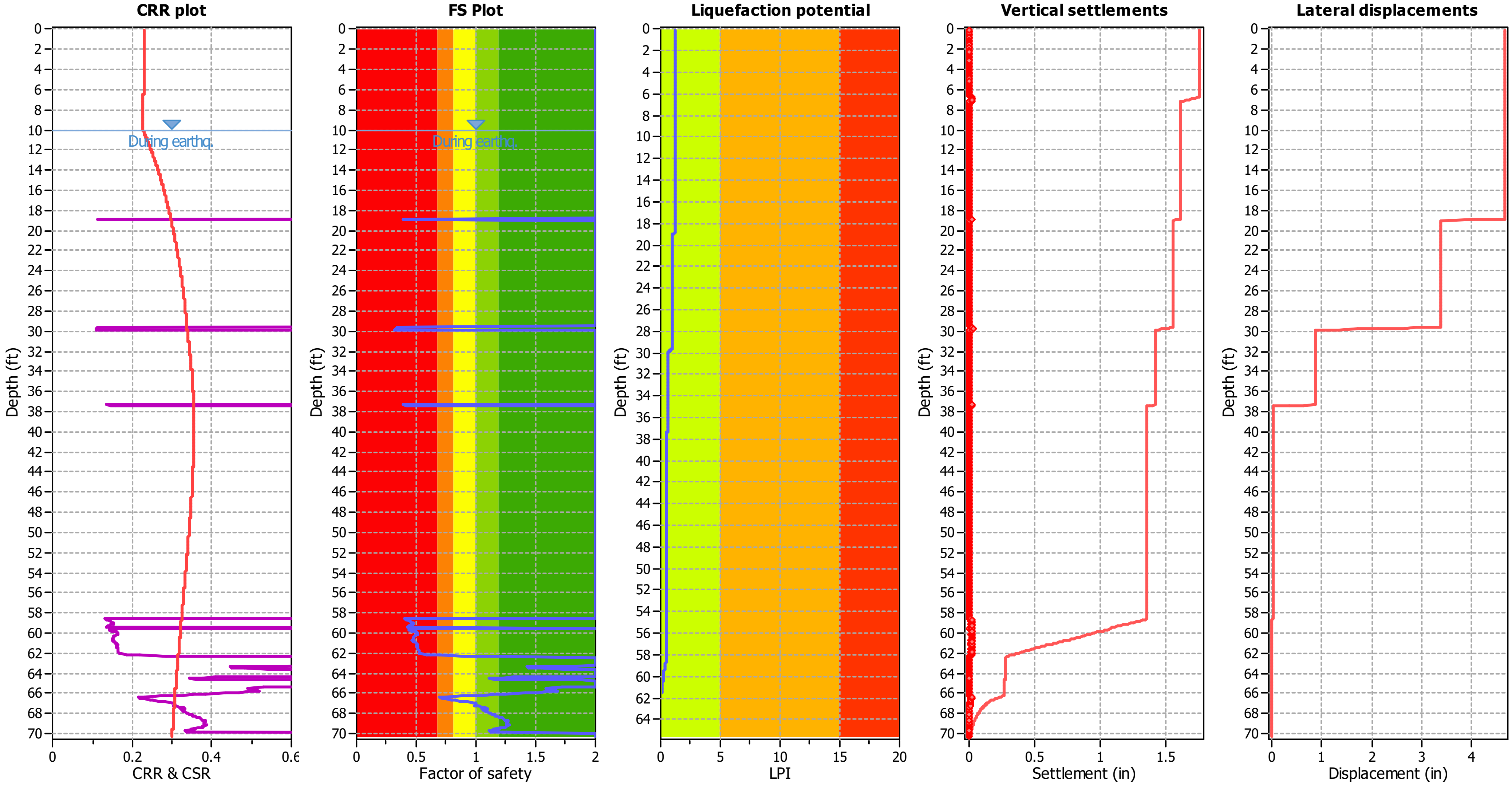
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.57	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	22.00 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.57	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	22.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

Leighton Consulting

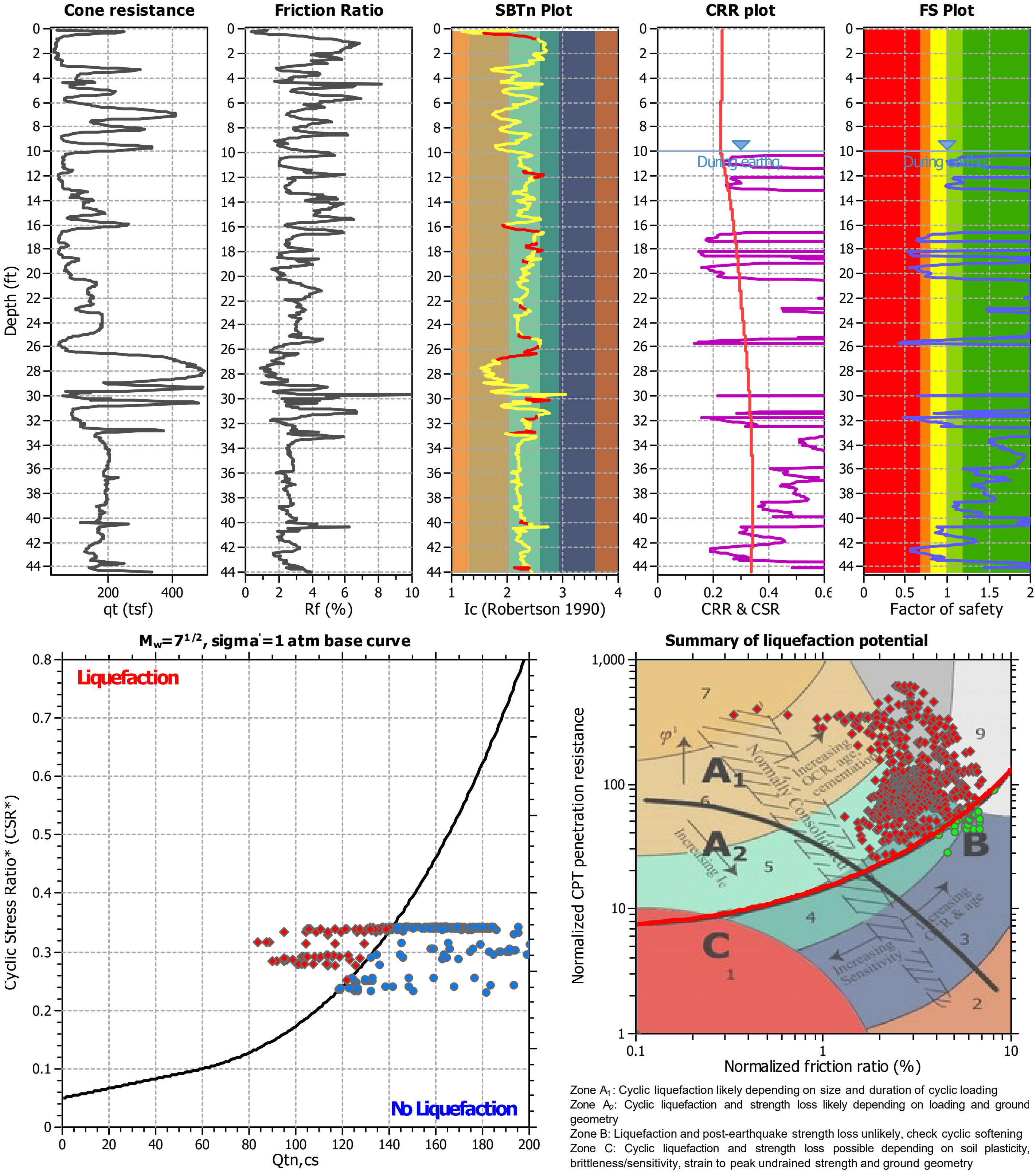
LIQUEFACTION ANALYSIS REPORT

Project title : JDA Woodland Hills **Location : 21101 Ventura Blvd, Woodland Hills, CA**

CPT file : CPT-3

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	45.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.57	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



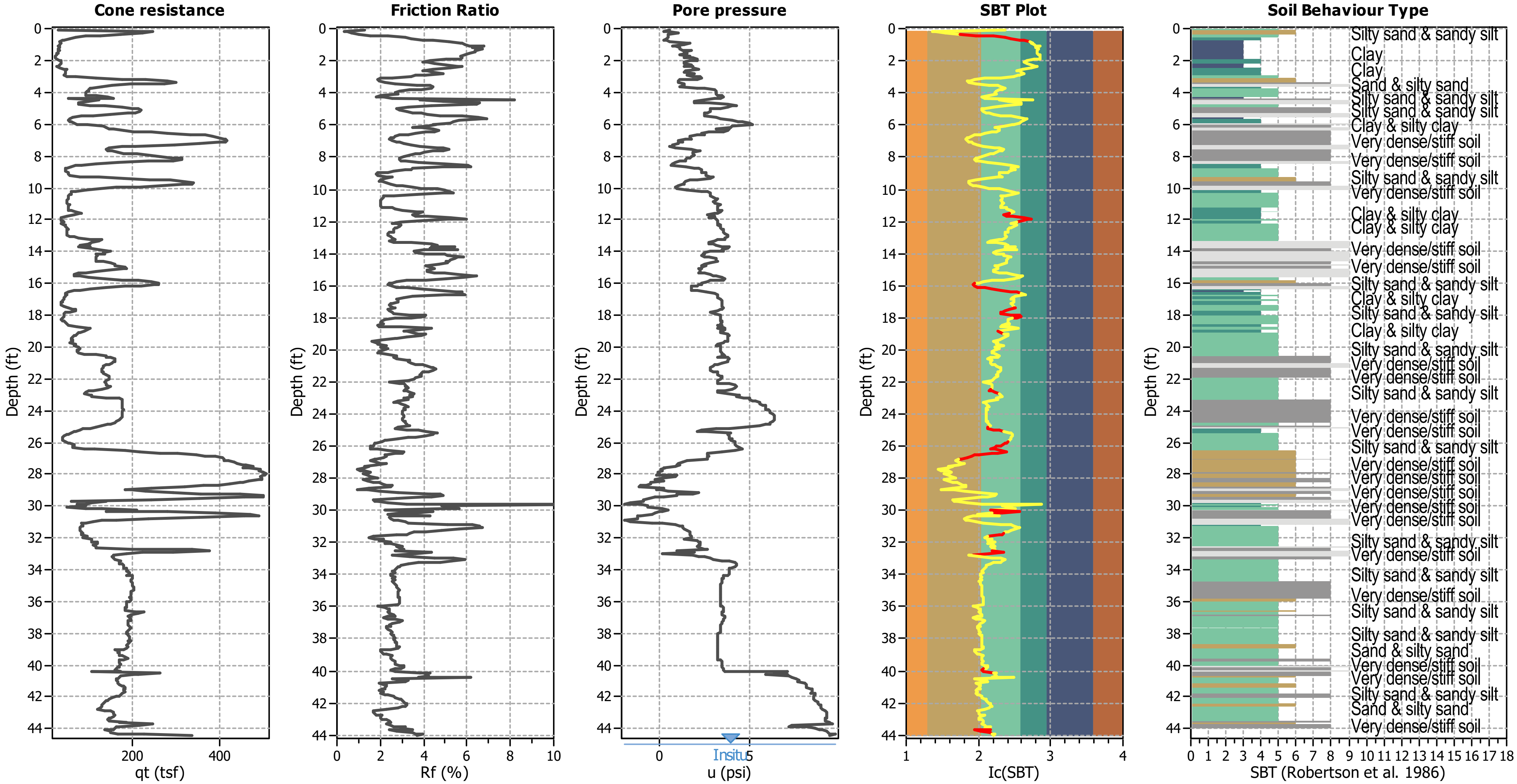
Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading

Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry

Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening

Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



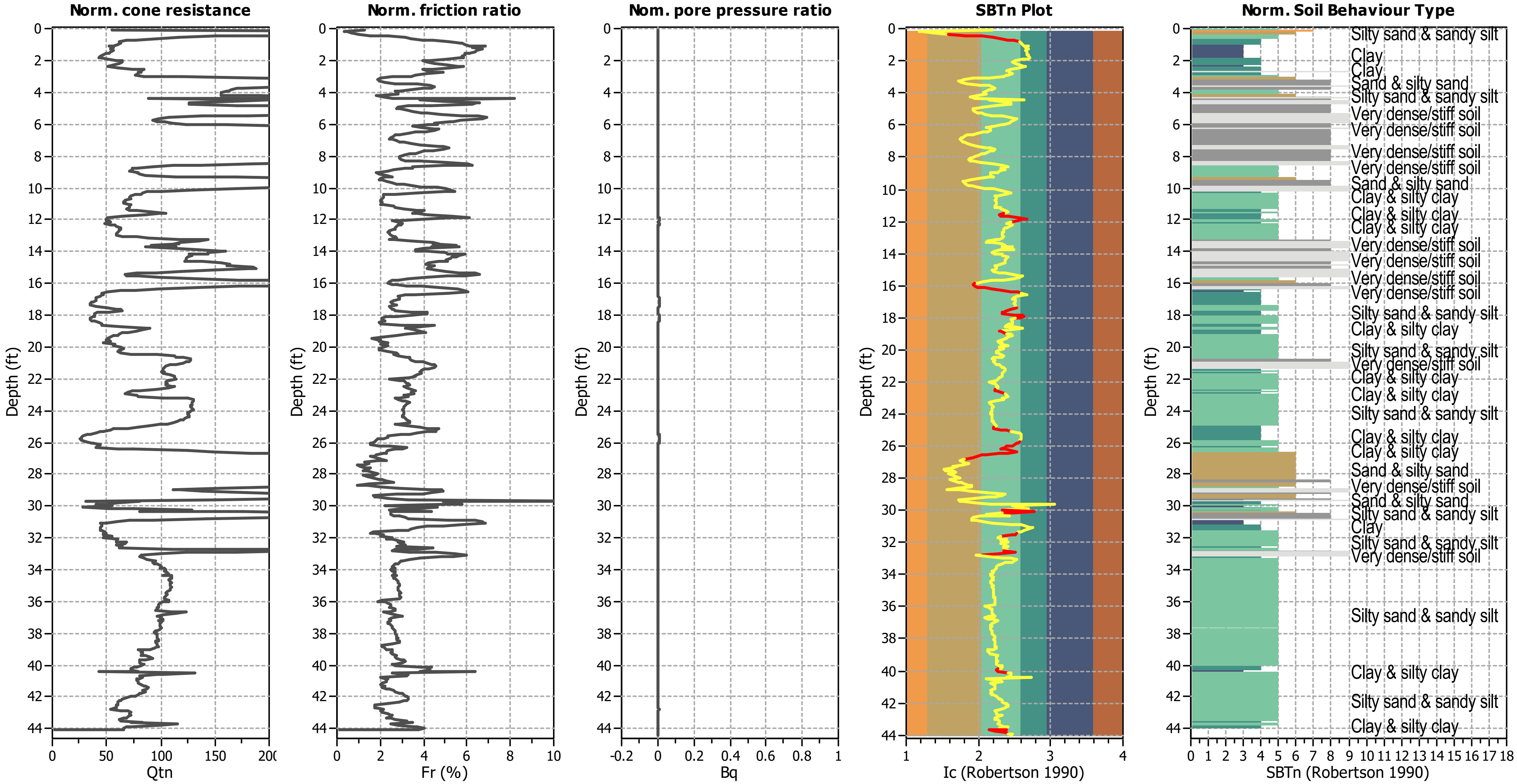
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.57	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots (normalized)



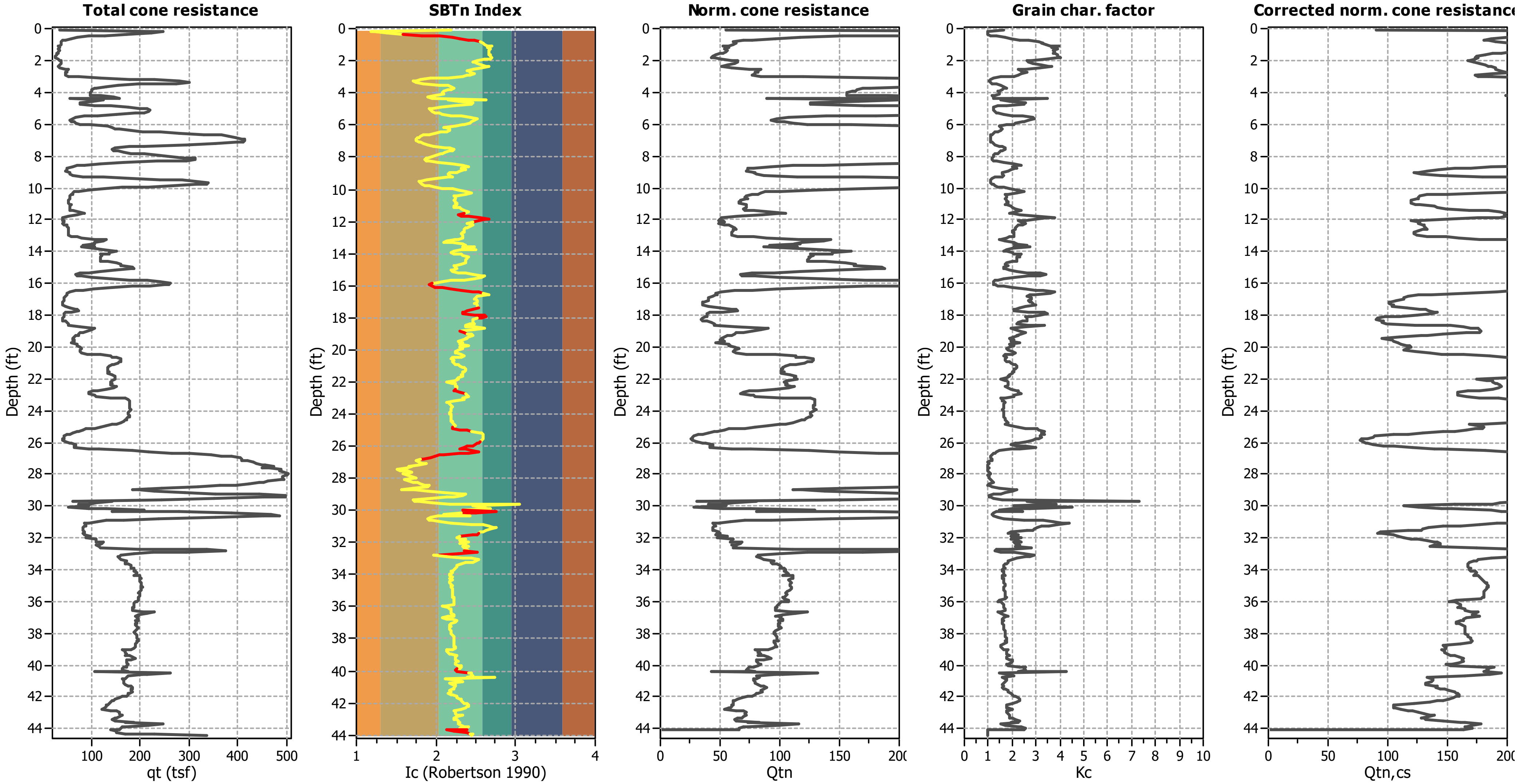
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.57	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

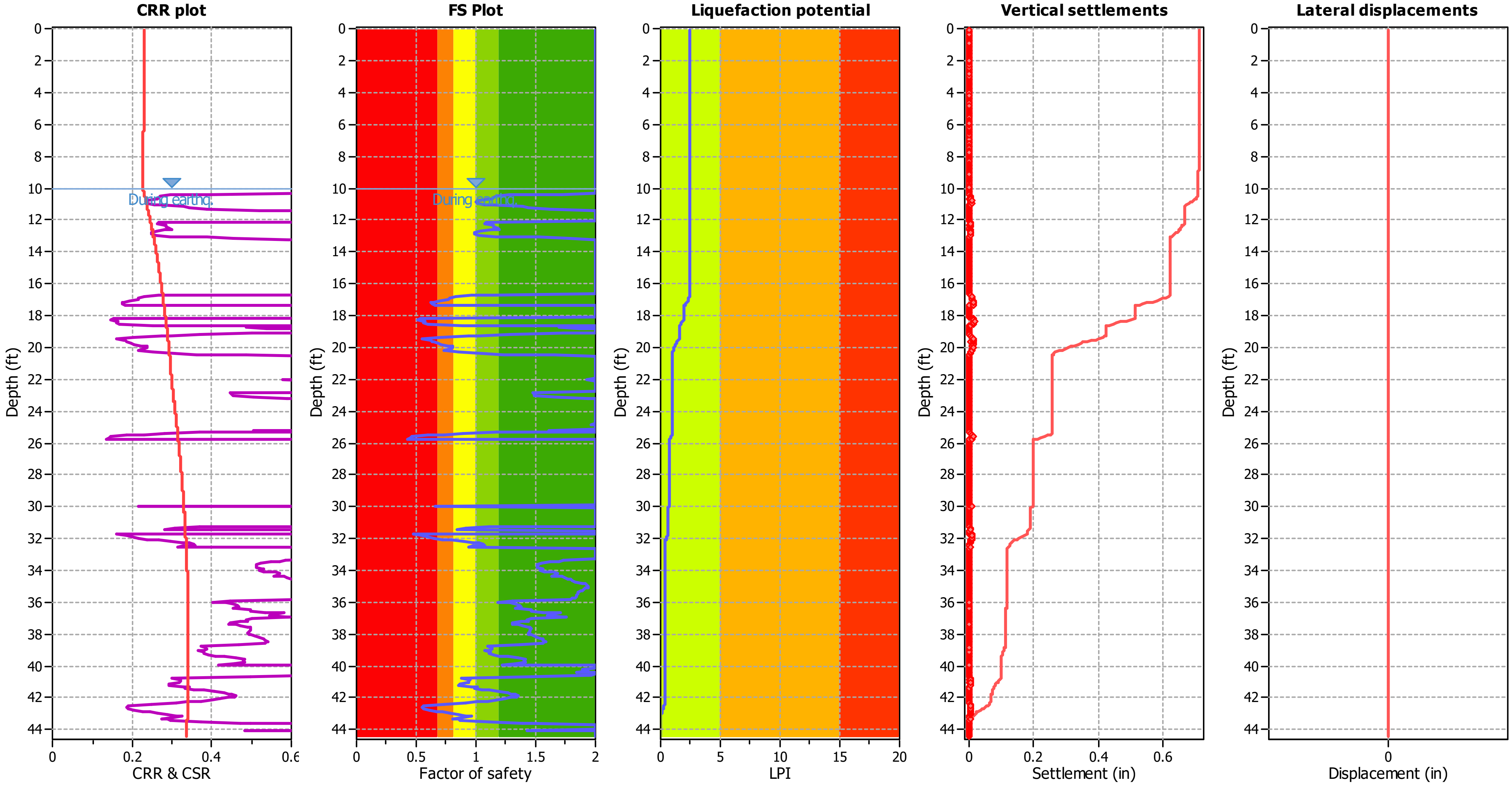
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.57	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.57	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	N/A

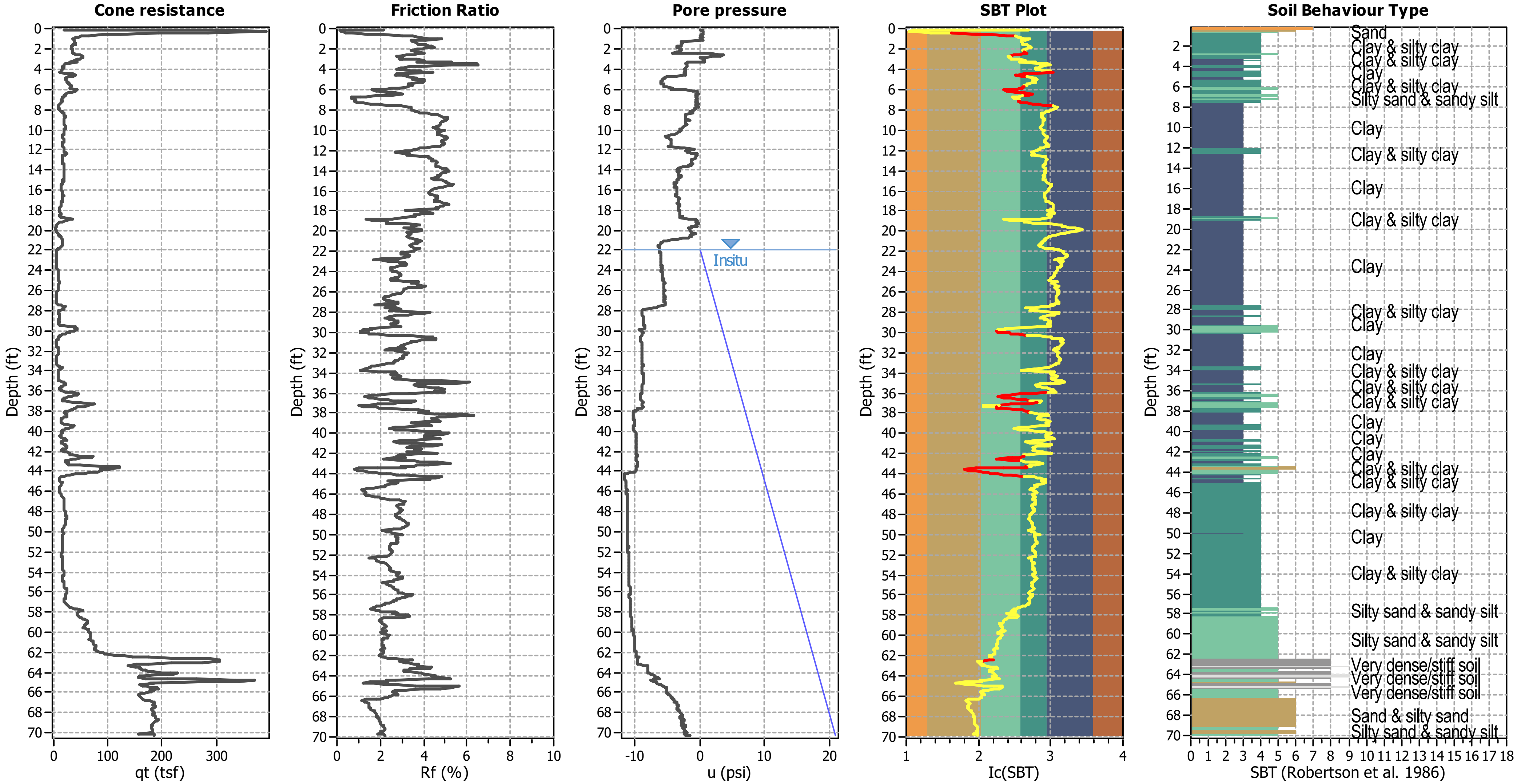
F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

CPT basic interpretation plots



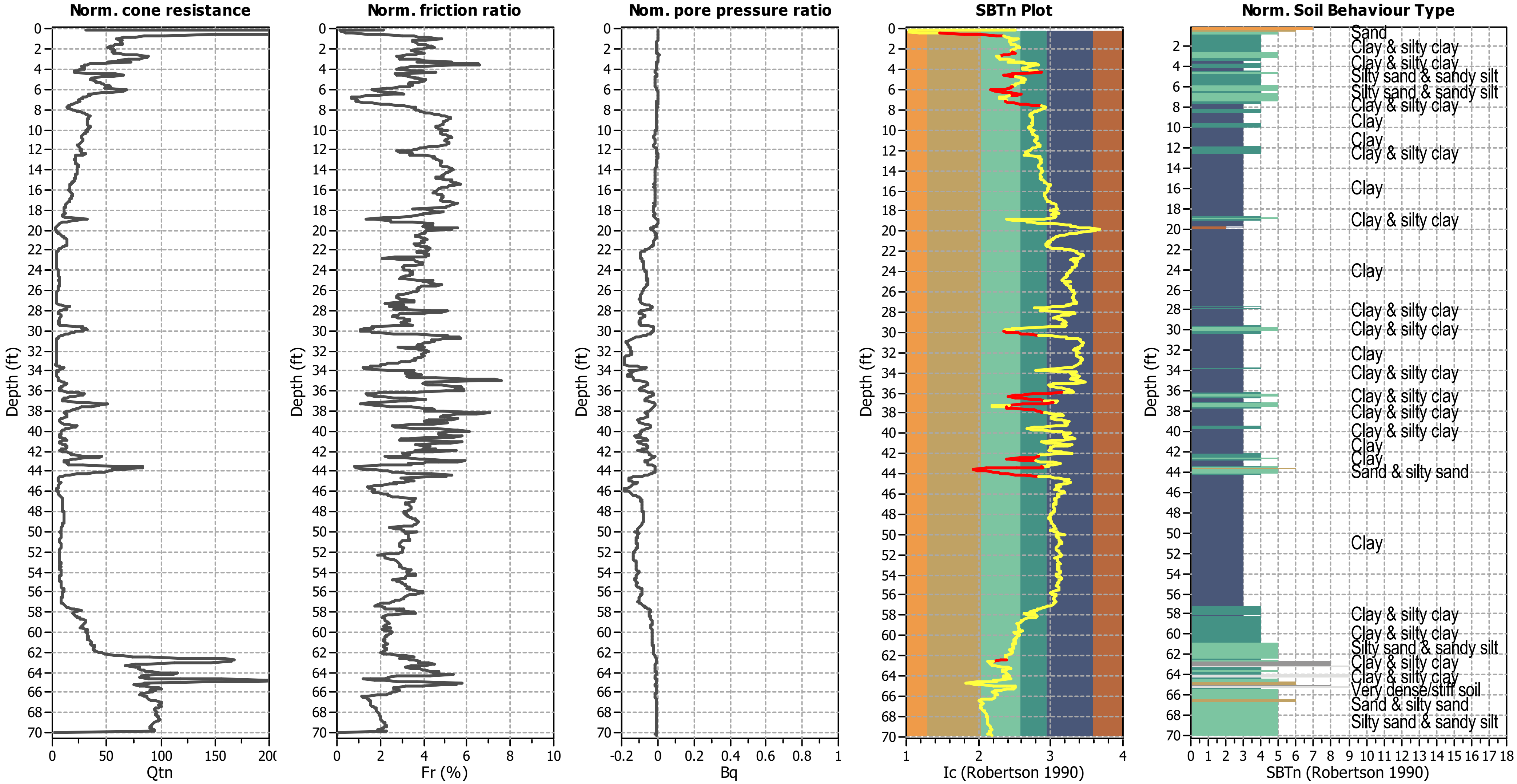
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.63	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.75	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	22.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots (normalized)



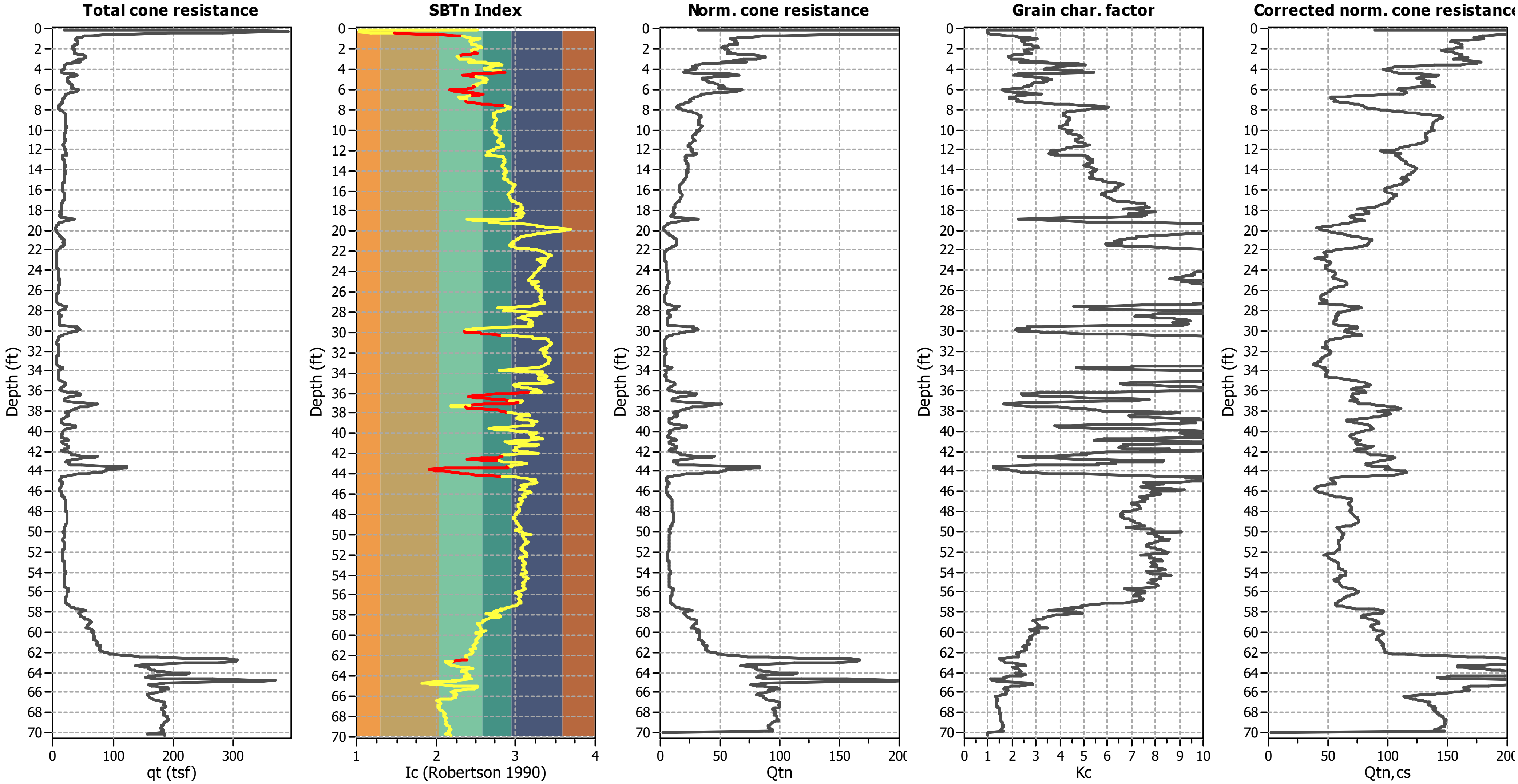
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.63	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.75	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	22.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

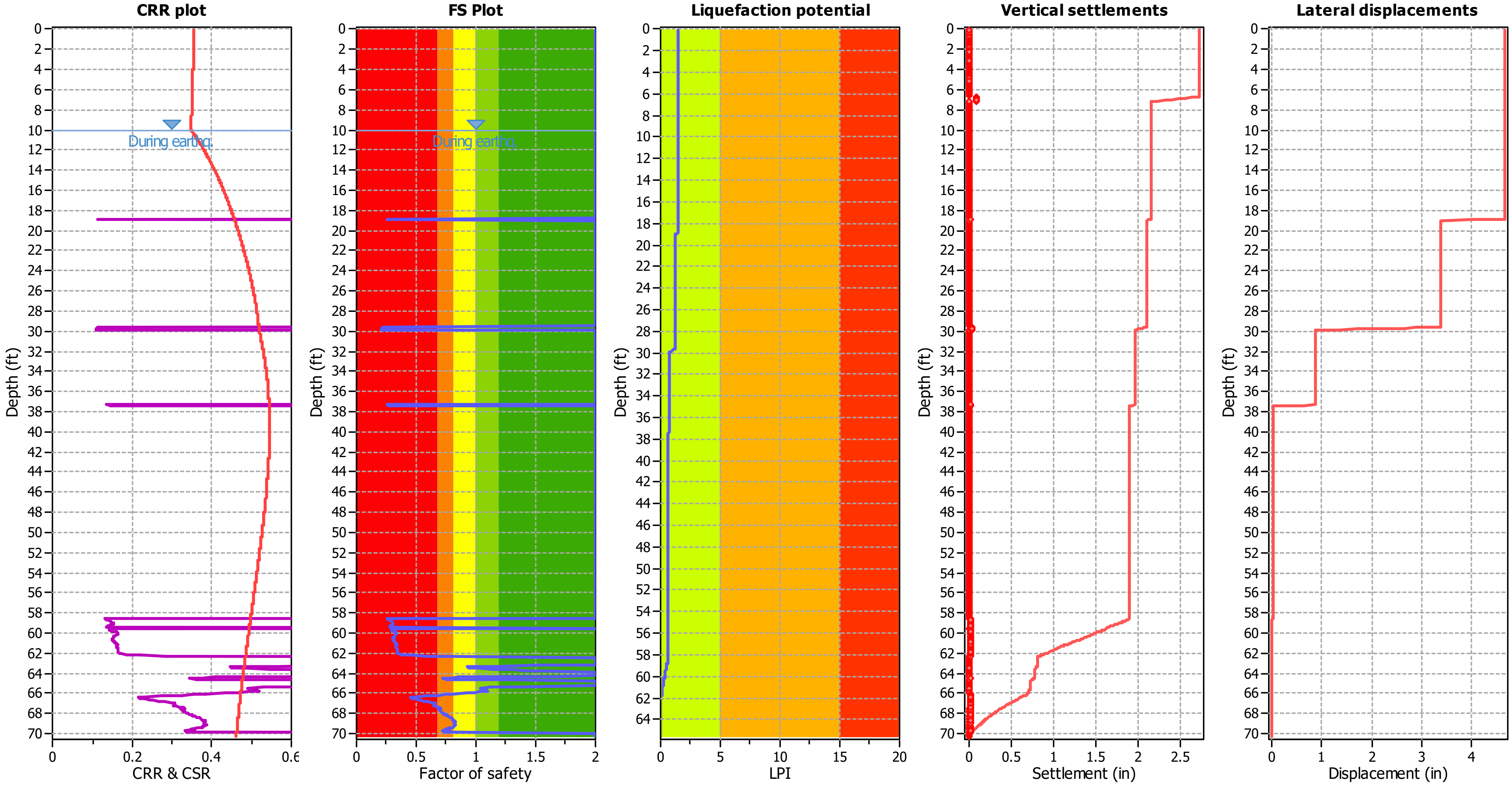
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.63	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.75	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	22.00 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.63	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.75	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	22.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

Leighton Consulting

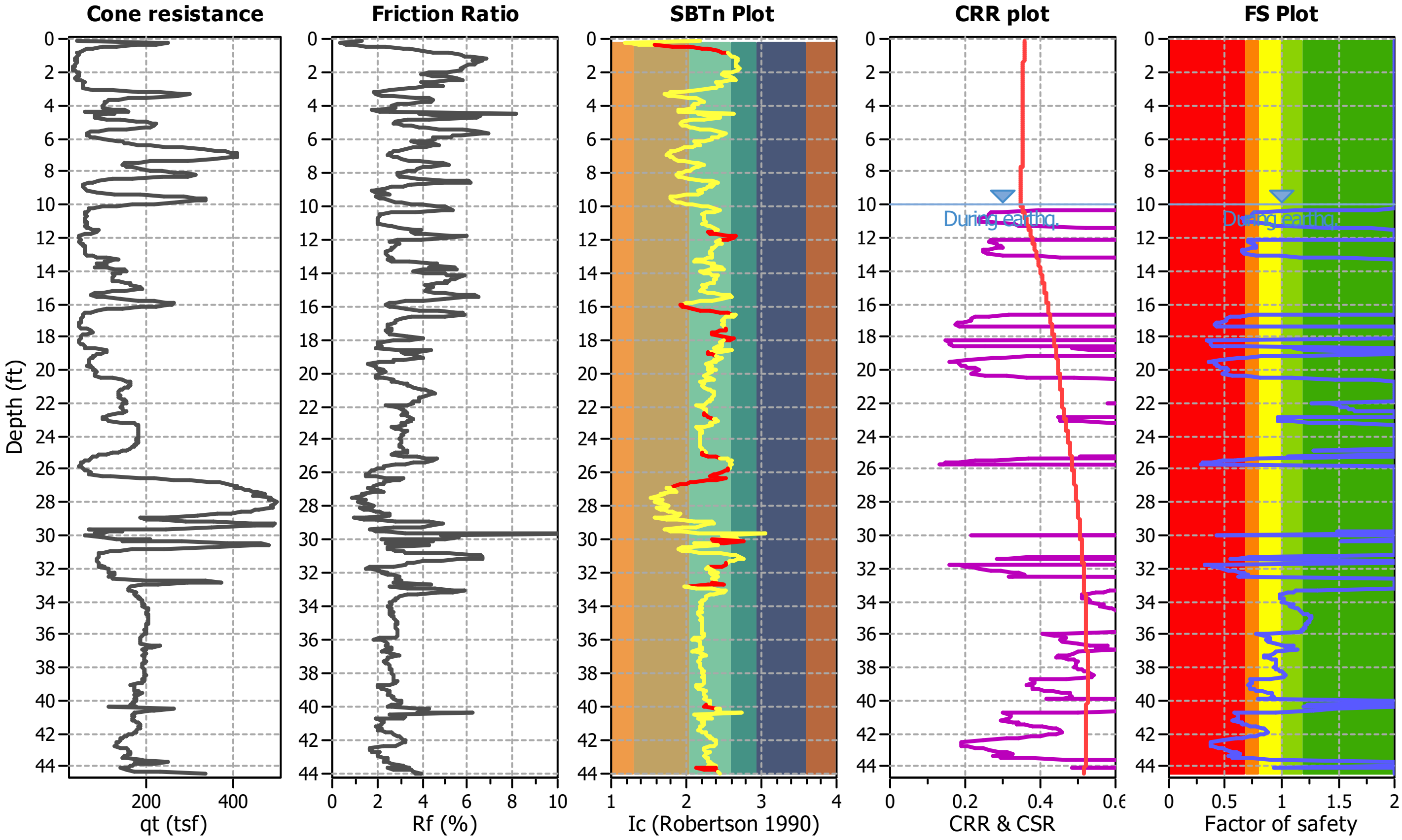
LIQUEFACTION ANALYSIS REPORT

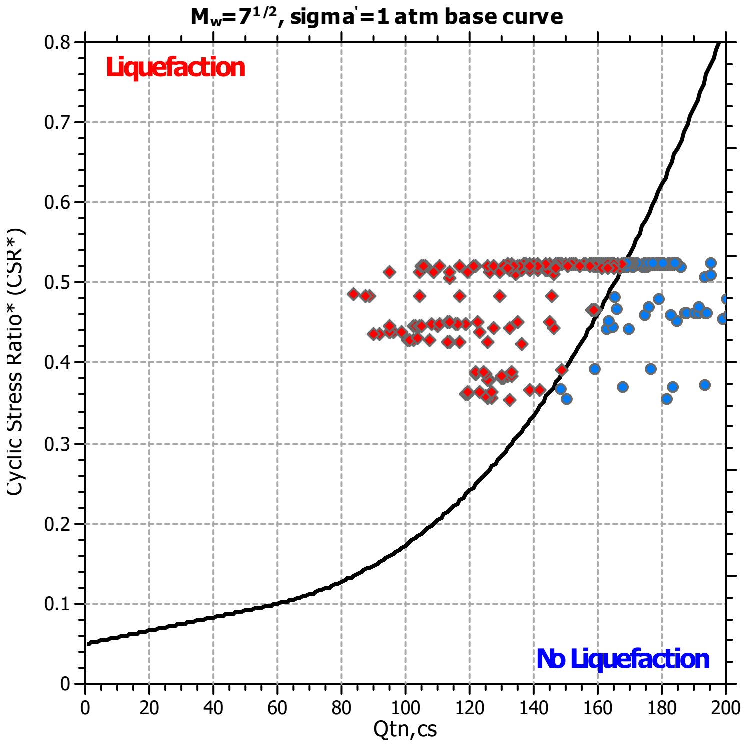
Project title : JDA Woodland Hills **Location : 21101 Ventura Blvd, Woodland Hills, CA**

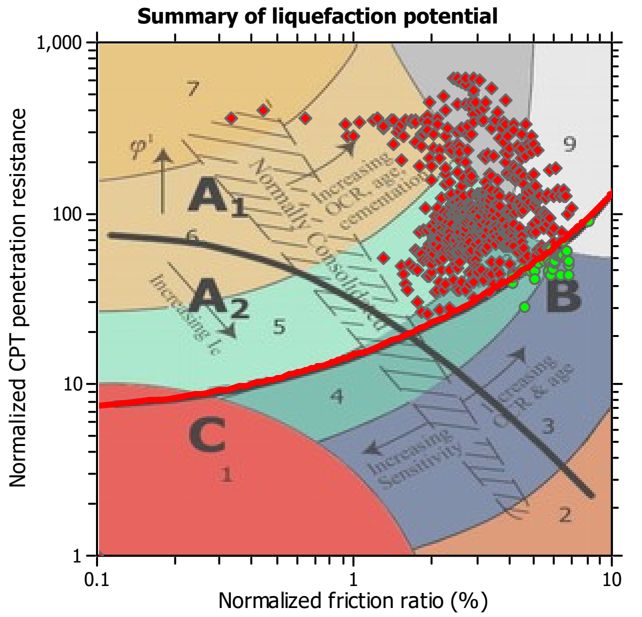
CPT file : CPT-3

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	45.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.63	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.75	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

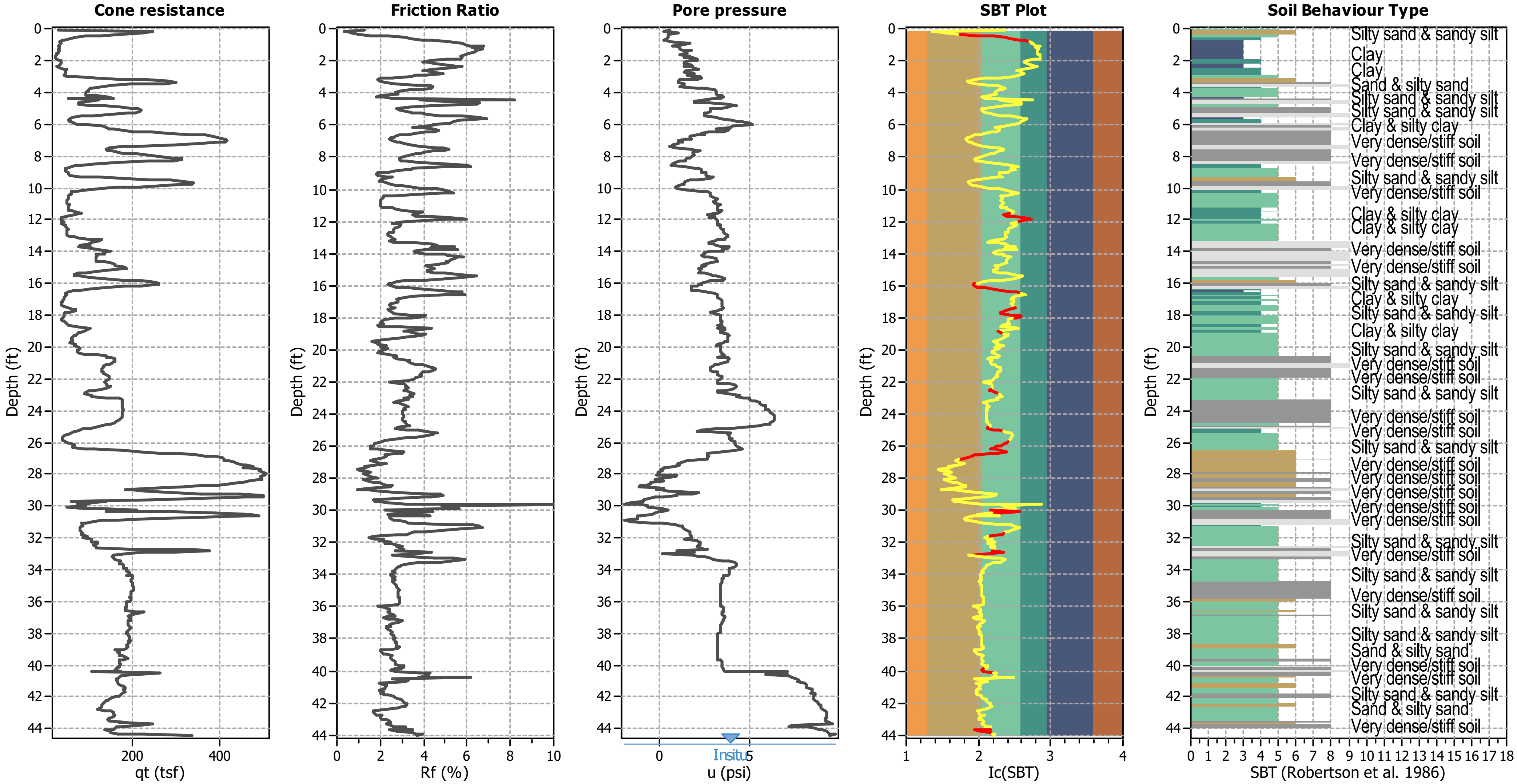






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



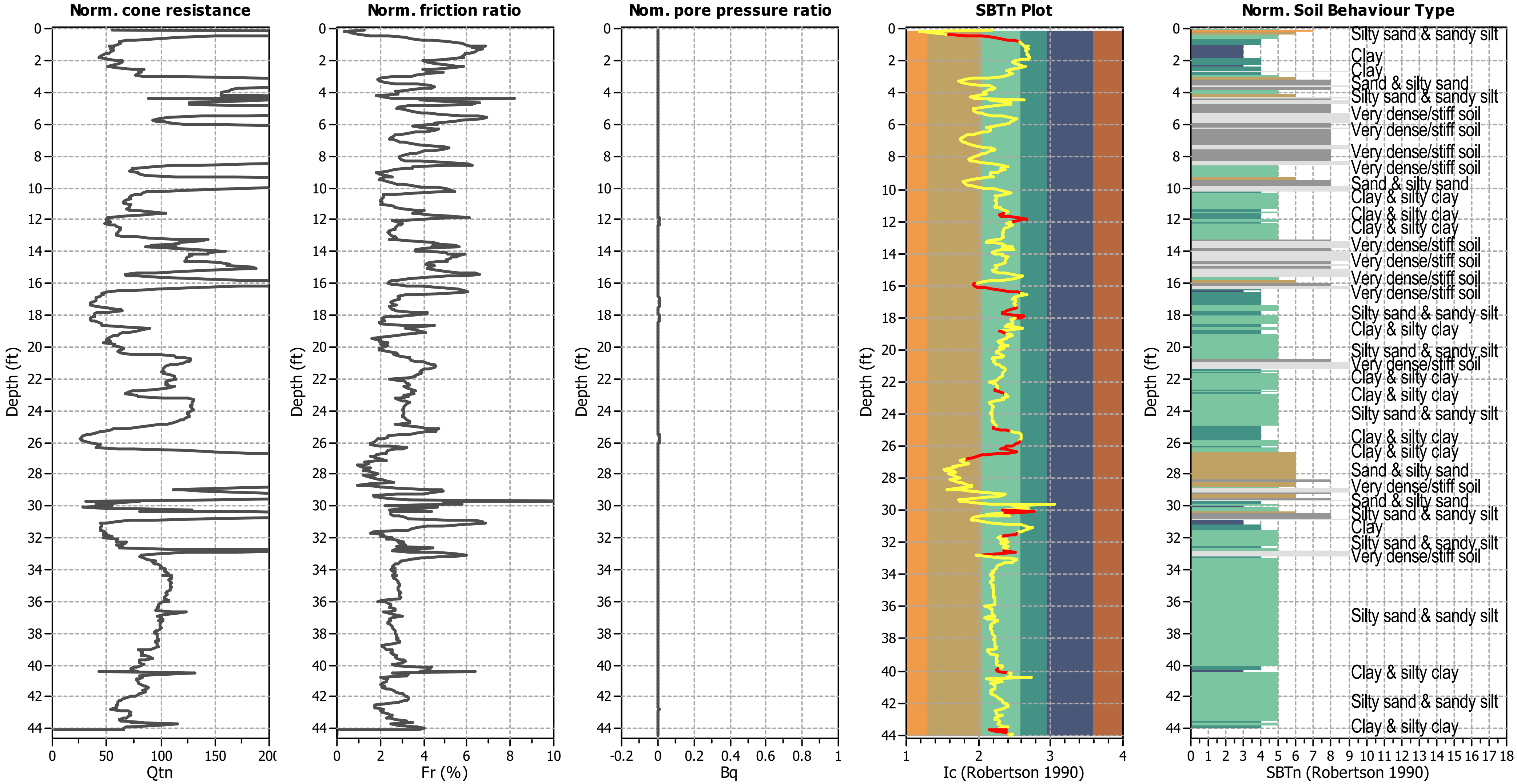
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.63	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.75	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots (normalized)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.63	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.75	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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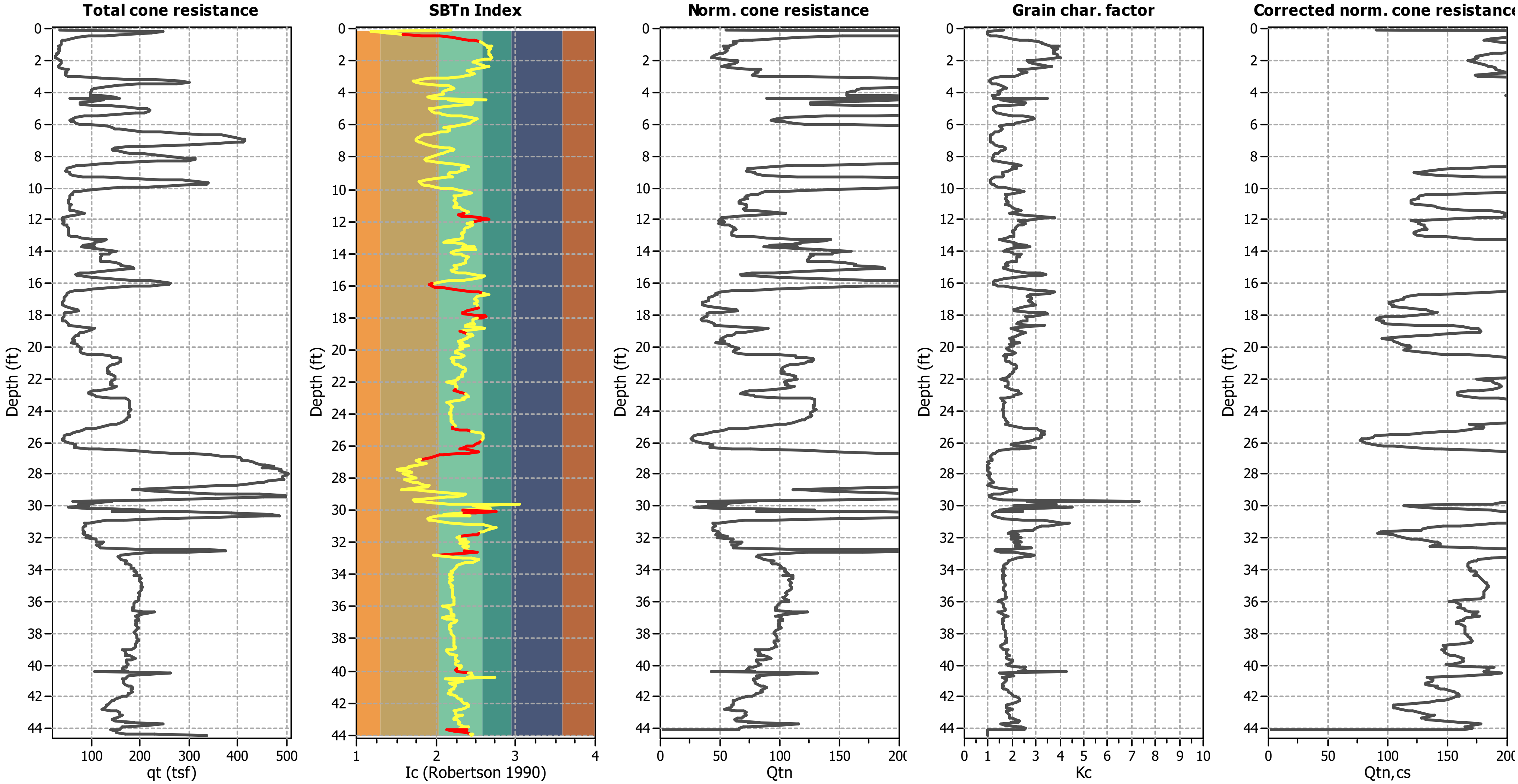
8. Very stiff sand to
9. Very stiff fine grained

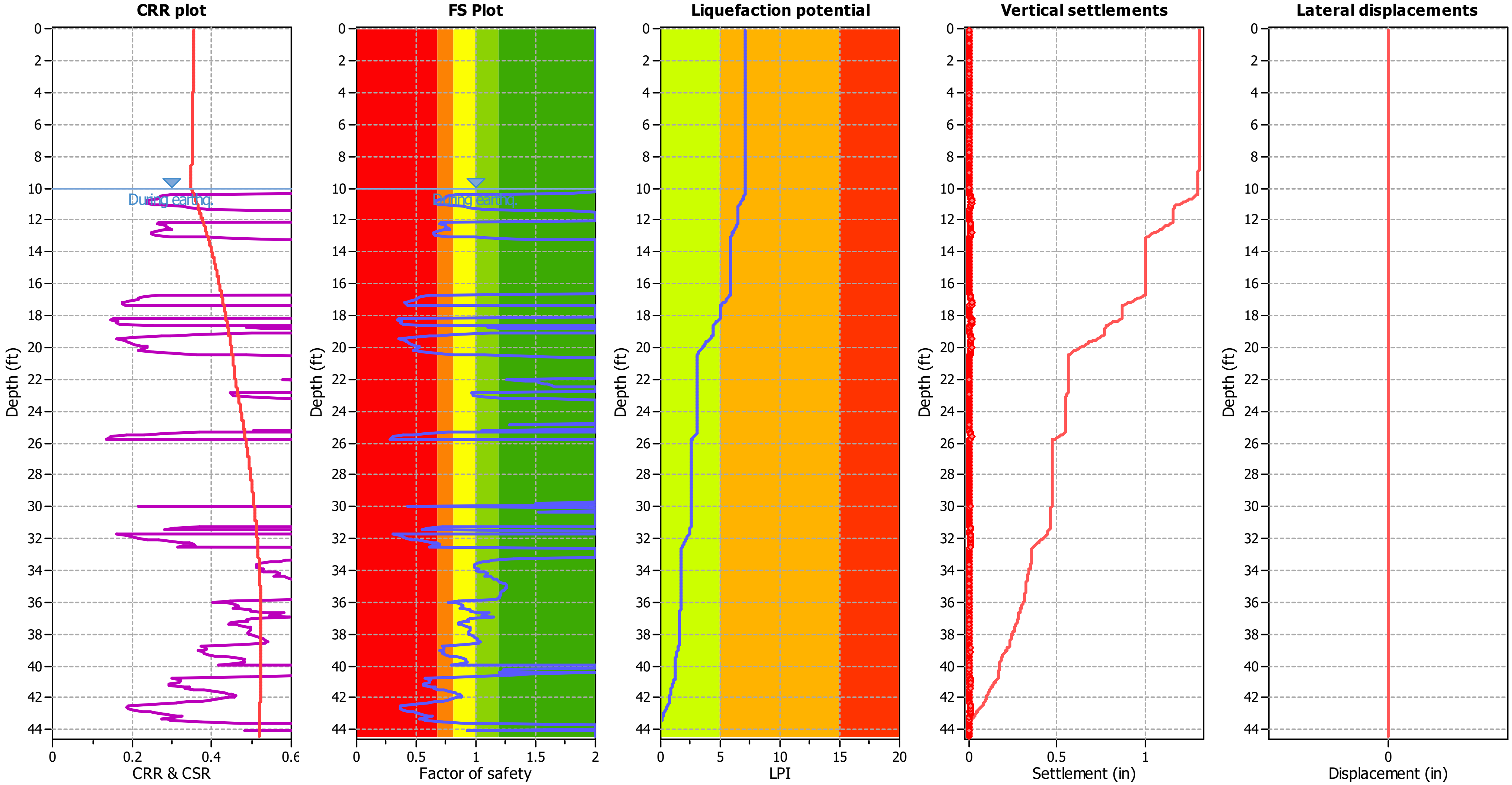
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.63	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.75	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	6.63	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.75	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

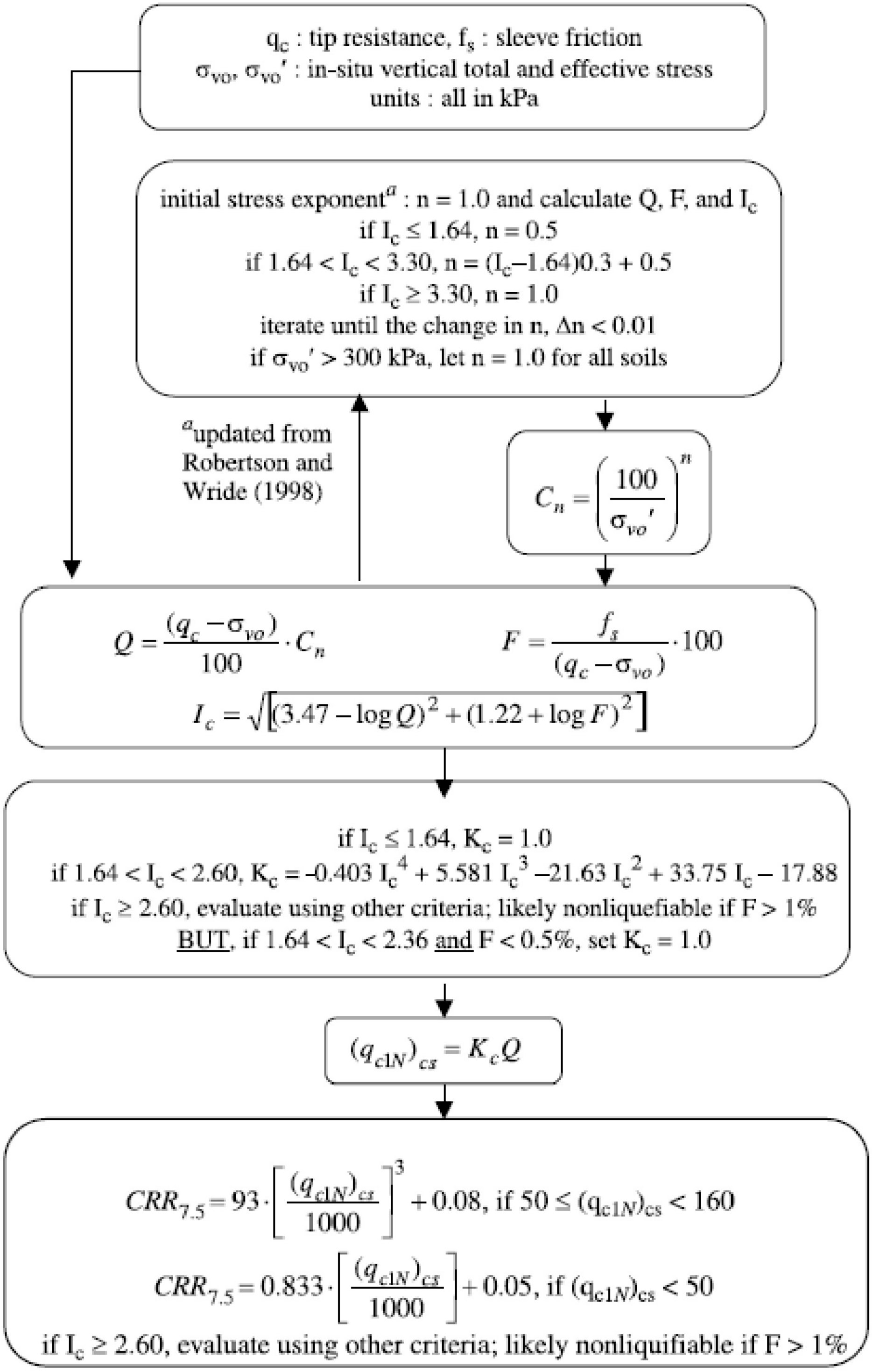
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

Procedure for the evaluation of soil liquefaction resistance, NCEER (1998)

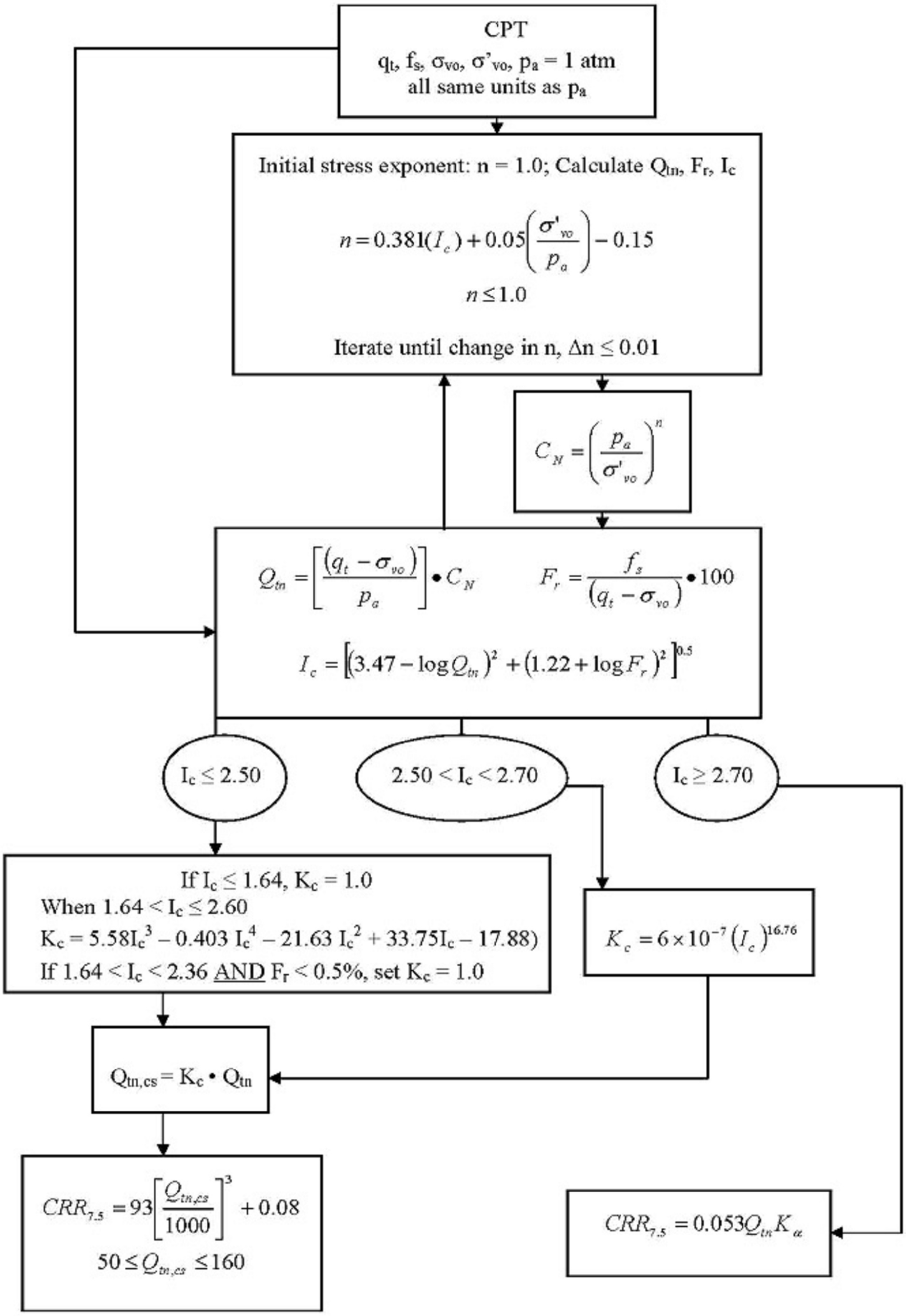
Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. The procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils). The revised procedure is presented below in the form of a flowchart[1]:



[1] "Estimating liquefaction-induced ground settlements from CPT for level ground", G. Zhang, P.K. Robertson, and R.W.I. Brachman

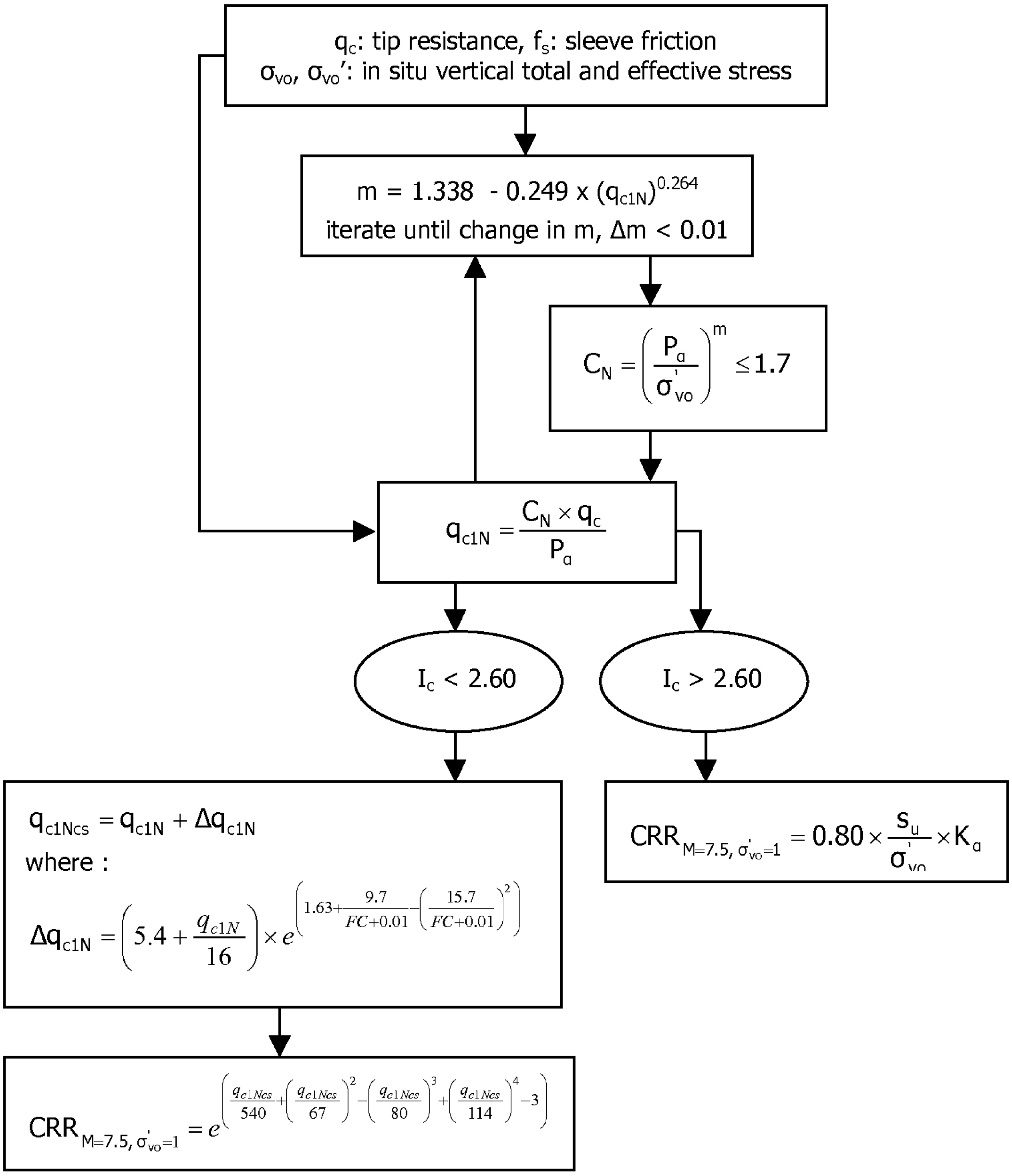
Procedure for the evaluation of soil liquefaction resistance (all soils), Robertson (2010)

Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. This procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils). The revised procedure is presented below in the form of a flowchart[1]:



[1] P.K. Robertson, 2009. "Performance based earthquake design using the CPT", Keynote Lecture, International Conference on Performance-based Design in Earthquake Geotechnical Engineering – from case history to practice, IS-Tokyo, June 2009

Procedure for the evaluation of soil liquefaction resistance, Idriss & Boulanger (2008)



Procedure for the evaluation of soil liquefaction resistance (sandy soils), Moss et al. (2006)

CPT

q_t, f_s, I_c

$I_c < I_c$ cut-off

Initial estimate using raw tip measurements, friction ratio. Calculate $q_{t,1}$. Repeat until an acceptable convergence tolerance is achieved.

$$c = f_1 \cdot \left(\frac{R_f}{f_3}\right)^{f_2}$$

$$C_q = \left(\frac{P_a}{\sigma'_v}\right)^{c}$$

$$q_{t,1} = C_q \cdot q_t$$

$$CRR = \exp\left[\frac{q_{t,1}^{1.045} + q_{t,1} \cdot (0.110 \cdot R_f) + (0.001 \cdot R_f) + c \cdot (1 + 0.850 \cdot R_f) - 0.848 \cdot \ln(M_w) - 0.002 \cdot \ln(\sigma'_v) - 20.923 + 1.632 \cdot \Phi^{-1}(P_L)}{7.177}\right]$$

Procedure for the evaluation of soil liquefaction resistance, Boulanger & Idriss(2014)

$$FS_{liq} = \frac{CRR_{M=7.5,\sigma_v'=1atm}}{CSR_{M=7.5,\sigma_v'=1atm}}$$

$$CSR_{M=7.5,\sigma_v'=1atm} = 0.65 \frac{\sigma_v}{\sigma_v'} \frac{a_{\max}}{g} r_d \frac{1}{MSF} \frac{1}{K_\sigma}$$

$$r_d = \exp\left[\alpha(z) + \beta(z) \cdot M\right]$$

$$\alpha(z) = -1.012 - 1.126 \sin\left(\frac{z}{11.73} + 5.133\right)$$

$$\beta(z) = 0.106 + 0.118 \sin\left(\frac{z}{11.28} + 5.142\right)$$

$$K_\sigma = 1 - C_\sigma \ln\left(\frac{\sigma_v'}{P_a}\right) \le 1.1$$

$$C_\sigma = \frac{1}{37.3 - 8.27\left(q_{c1Ncs}\right)^{0.264}} \le 0.3$$

$$MSF = 1 + \left(MSF_{max} - 1\right)\left(8.64 \exp\left(\frac{-M}{4}\right) - 1.325\right)$$

$$MSF_{max} = 1.09 + \left(\frac{q_{c1Ncs}}{180}\right)^3 \le 2.2$$

σ_v' *and* σ_v *at start of earthquake shaking*

$$CRR_{M=7.5,\sigma_v'=1atm} = \exp\left(\frac{q_{c1Ncs}}{113} + \left(\frac{q_{c1Ncs}}{1000}\right)^2 - \left(\frac{q_{c1Ncs}}{140}\right)^3 + \left(\frac{q_{c1Ncs}}{137}\right)^4 - 2.80\right)$$

$$q_{c1Ncs} = q_{c1N} + \Delta q_{c1N}$$

$$\Delta q_{c1N} = \left(11.9 + \frac{q_{c1N}}{14.6}\right) \exp\left(1.63 - \frac{9.7}{FC+2} - \left(\frac{15.7}{FC+2}\right)^2\right)$$

$$q_{c1N} = C_{N\sigma} \frac{q_c}{P_a}$$

$$C_{N\sigma} = \left(\frac{P_a}{\sigma_v'}\right)^m \le 1.7$$

$$m = 1.338 - 0.249\left(q_{c1Ncs}\right)^{0.264} \quad with \quad 0.264 \le m \le 0.782$$

$$FC = 80\left(I_C + C_{FC}\right) - 137 \quad with \quad 0\% \le FC \le 100\%$$

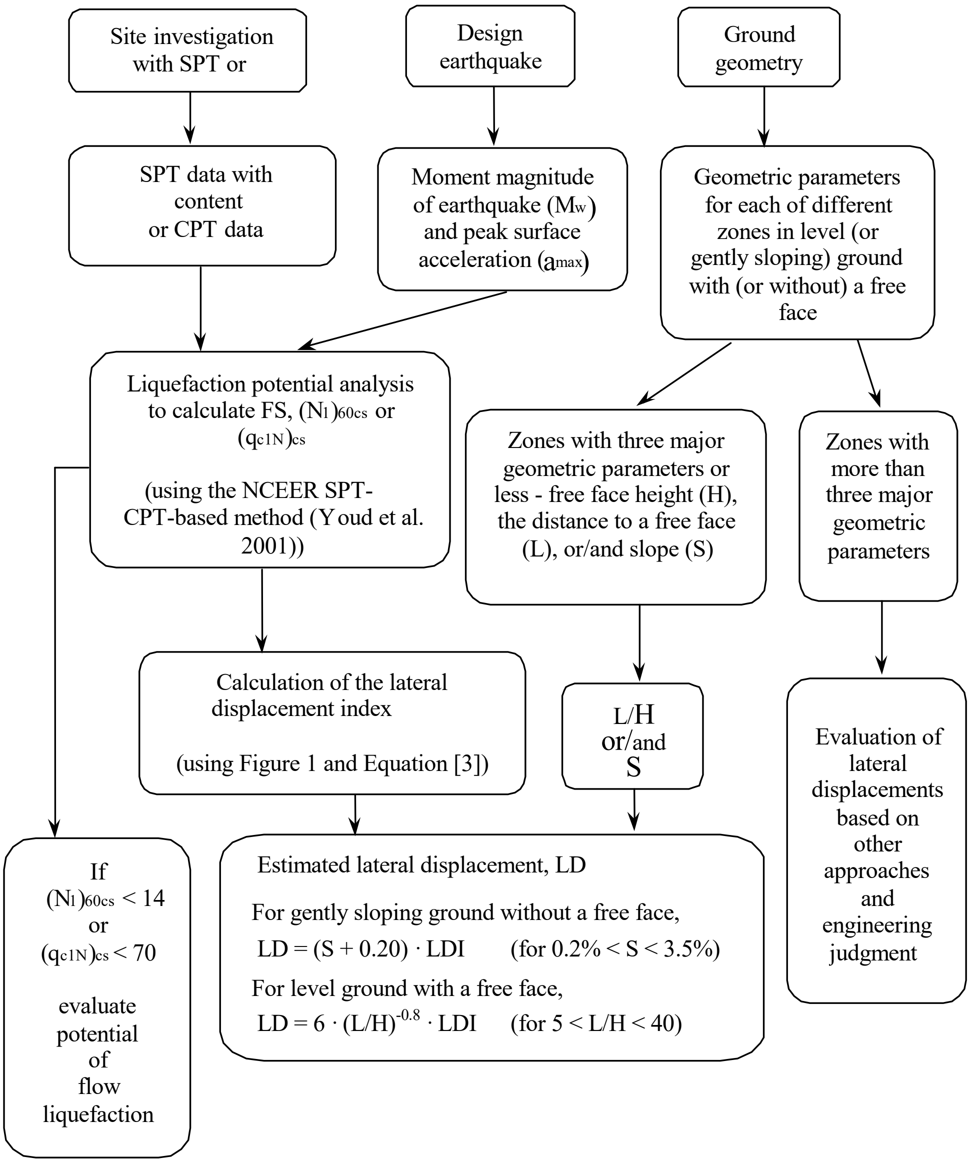
$$I_c = \left[\left(3.47 - \log(Q)\right)^2 + \left(1.22 + \log(F)\right)^2\right]^{0.5}$$

$$Q = \left(\frac{q_c - \sigma_{vc}}{P_a}\right)\left(\frac{P_a}{\sigma_{vc}'}\right)^n \; with\; 0.5 \le n \le 1.0 \; per\; Robertson\; \&\; Wride\; (1998)$$

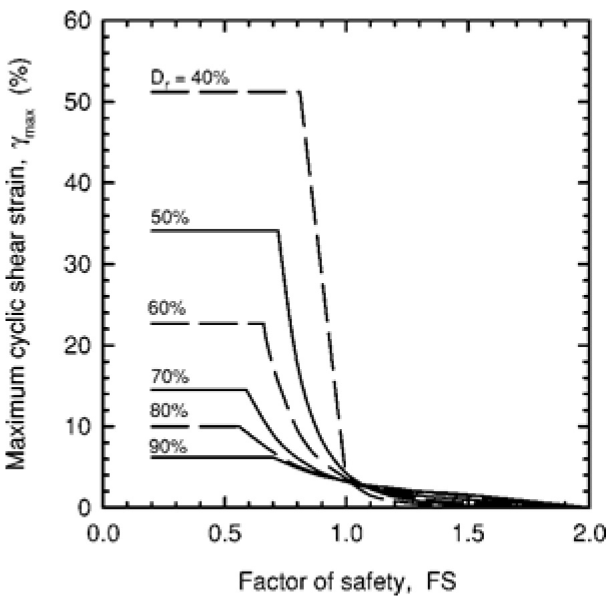
$$F = \left(\frac{f_s}{q_c - \sigma_{vc}}\right) \cdot 100\%$$

σ_v' *at time of CPT sounding*

Procedure for the evaluation of liquefaction-induced lateral spreading displacements



[1] Flow chart illustrating major steps in estimating liquefaction-induced lateral spreading displacements using the proposed approach



[1] **Figure 1**

$$LDI = \int_0^{Z_{max}} \gamma_{max} \, dz$$

[1] **Equation [3]**

[1] "Estimating liquefaction-induced ground settlements from CPT for level ground", G. Zhang, P.K. Robertson, and R.W.I. Brachman

Procedure for the estimation of seismic induced settlements in dry sands

Average shear stress, τ_{av}

$$\tau_{av} = CSR^{*} \cdot \sigma_{v0}' = 0.65 \cdot \frac{\alpha_{max}}{g} \cdot \sigma_{v0} \cdot \tau_a$$

Estimate small shear strain modulus, G_0

$$G_0 = 0.0188 \cdot \left[10^{(0.55 I_c - 1.68)}\right] \cdot (q_t - \sigma_v)$$

Estimate shear strain amplitude, γ (based on Pradel (1998))

$$\gamma = \left[\frac{1 + \alpha \cdot e^{bR}}{1 + \alpha}\right] \cdot R \cdot 100\ (\%)$$

$$R = \frac{\tau_{av}}{G_0} \text{ (Note } \tau_{av} \text{ and } G_0 \text{ same units)}$$

$$\alpha = 0.0389 \cdot \left(\frac{\sigma_v}{p_a}\right) + 0.124$$

$$b = 6400 \cdot \left(\frac{\sigma_v}{p_a}\right)$$

Estimate volumetric strain in 15 cycles

$$\varepsilon_{vol(15)} = \gamma \cdot \left[\frac{(N_1)_{60,cs}}{20}\right]^{-1.20}$$

$$(N_1)_{60,cs} = \frac{Q_{tn,cs}}{8.5 \cdot \left[1 - \frac{I_c}{4.6}\right]}$$

Volumetric strain in design earthquake

$$\Sigma_{vol} = \varepsilon_{vol(15)} \cdot \left[\frac{N_c}{15}\right]^{0.45}$$

$$N_c = (M - 4)^{2.17}$$

Seismic settlement, s

$$s = 2 \cdot \int_0^{GWT} \Sigma_{vol} \cdot dz$$

Robertson, P.K. and Lisheng, S., 2010, “Estimation of seismic compression in dry soils using the CPT” FIFTH INTERNATIONAL CONFERENCE ON RECENT ADVANCES IN GEOTECHNICAL EARTHQUAKE ENGINEERING AND SOIL DYNAMICS, Symposium in honor of professor I. M. Idriss, San Diego, CA

Liquefaction Potential Index (LPI) calculation procedure

Calculation of the Liquefaction Potential Index (LPI) is used to interpret the liquefaction assessment calculations in terms of severity over depth. The calculation procedure is based on the methology developed by Iwasaki (1982) and is adopted by AFPS.

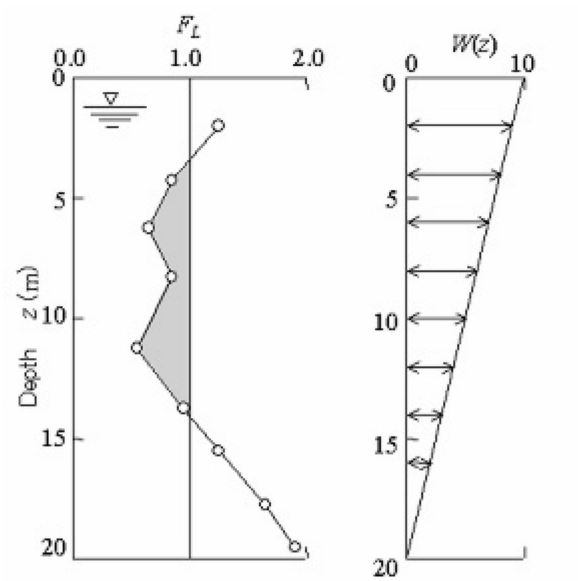
To estimate the severity of liquefaction extent at a given site, LPI is calculated based on the following equation:

$$\textbf{LPI} = \int_0^{20} (10 - 0{,}5_Z) \times F_L \times d_z$$

where:
F_L = 1 - F.S. when F.S. less than 1
F_L = 0 when F.S. greater than 1
z depth of measurment in meters

Values of LPI range between zero (0) when no test point is characterized as liquefiable and 100 when all points are characterized as susceptible to liquefaction. Iwasaki proposed four (4) discrete categories based on the numeric value of LPI:

- LPI = 0 : Liquefaction risk is very low
- 0 < LPI <= 5 : Liquefaction risk is low
- 5 < LPI <= 15 : Liquefaction risk is high
- LPI > 15 : Liquefaction risk is very high



Graphical presentation of the LPI calculation procedure

Shear-Induced Building Settlement (Ds) calculation procedure

The shear-induced building settlement (Ds) due to liquefaction below the building can be estimated using the relationship developed by Bray and Macedo (2017):

$$Ln(Ds) = c1 + c2 * LBS + 0.58 * Ln\left(Tanh\left(\frac{HL}{6}\right)\right) +$$
$$4.59 * Ln(Q) - 0.42 * Ln(Q)^2 - 0.02 * B +$$
$$0.84 * Ln(CAVdp) + 0.41 * Ln(Sa1) + \varepsilon$$

where Ds is in the units of mm, c1= -8.35 and c2= 0.072 for LBS ≤ 16, and c1= -7.48 and c2= 0.014 otherwise. Q is the building contact pressure in units of kPa, HL is the cumulative thickness of the liquefiable layers in the units of m, B is the building width in the units of m, CAVdp is a standardized version of the cumulative absolute velocity in the units of g-s, Sa1 is 5%-damped pseudo-acceleration response spectral value at a period of 1 s in the units of g, and ε is a normal random variable with zero mean and 0.50 standard deviation in Ln units. The liquefaction-induced building settlement index (LBS) is:

$$LBS = \sum W * \frac{\varepsilon_{shear}}{z} dz$$

where z (m) is the depth measured from the ground surface > 0, W is a foundation-weighting factor wherein W = 0.0 for z less than Df, which is the embedment depth of the foundation, and W = 1.0 otherwise. The shear strain parameter (ε_shear) is the liquefaction-induced free-field shear strain (in %) estimated using Zhang et al. (2004). It is calculated based on the estimated Dr of the liquefied soil layer and the calculated safety factor against liquefaction triggering (FSL).

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Leighton Consulting, Inc.
Geotechnical Engineering Consultants
Irvine, California
PN 13589.002

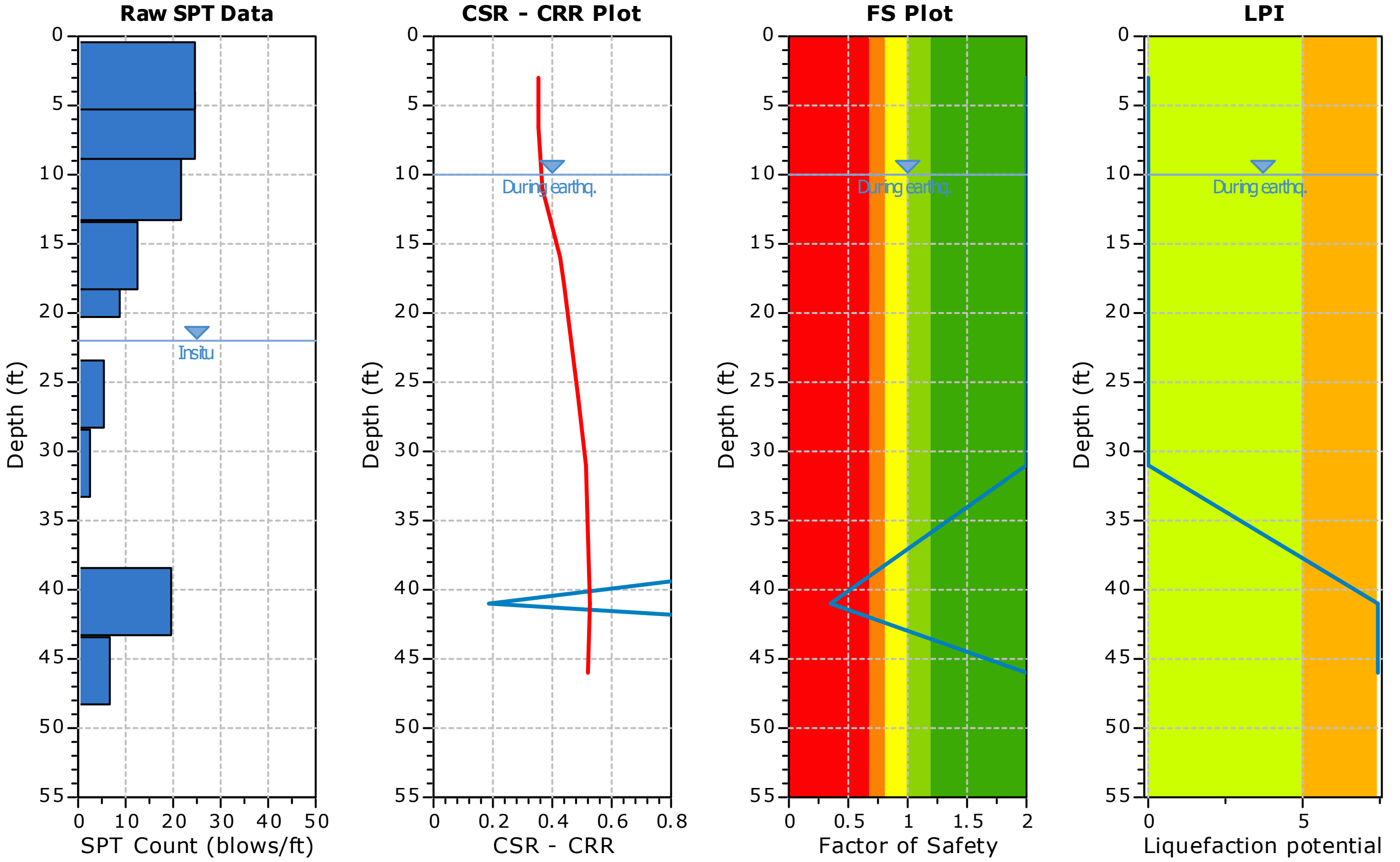
SPT BASED LIQUEFACTION ANALYSIS REPORT

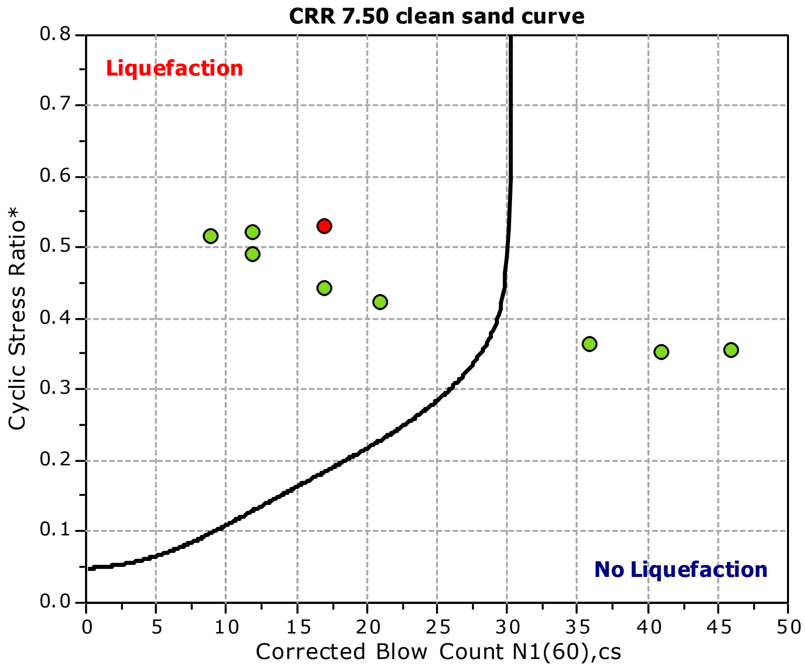
Project title : Proposed Self Storage Facility **SPT Name: LB-1**

Location : 21101 Ventura Blvd., Woodland Hills, CA

:: Input parameters and analysis properties ::

Analysis method:	NCEER 1998	G.W.T. (in-situ):	22.00 ft
Fines correction method:	NCEER 1998	G.W.T. (earthq.):	10.00 ft
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	6.63
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.75 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.15		

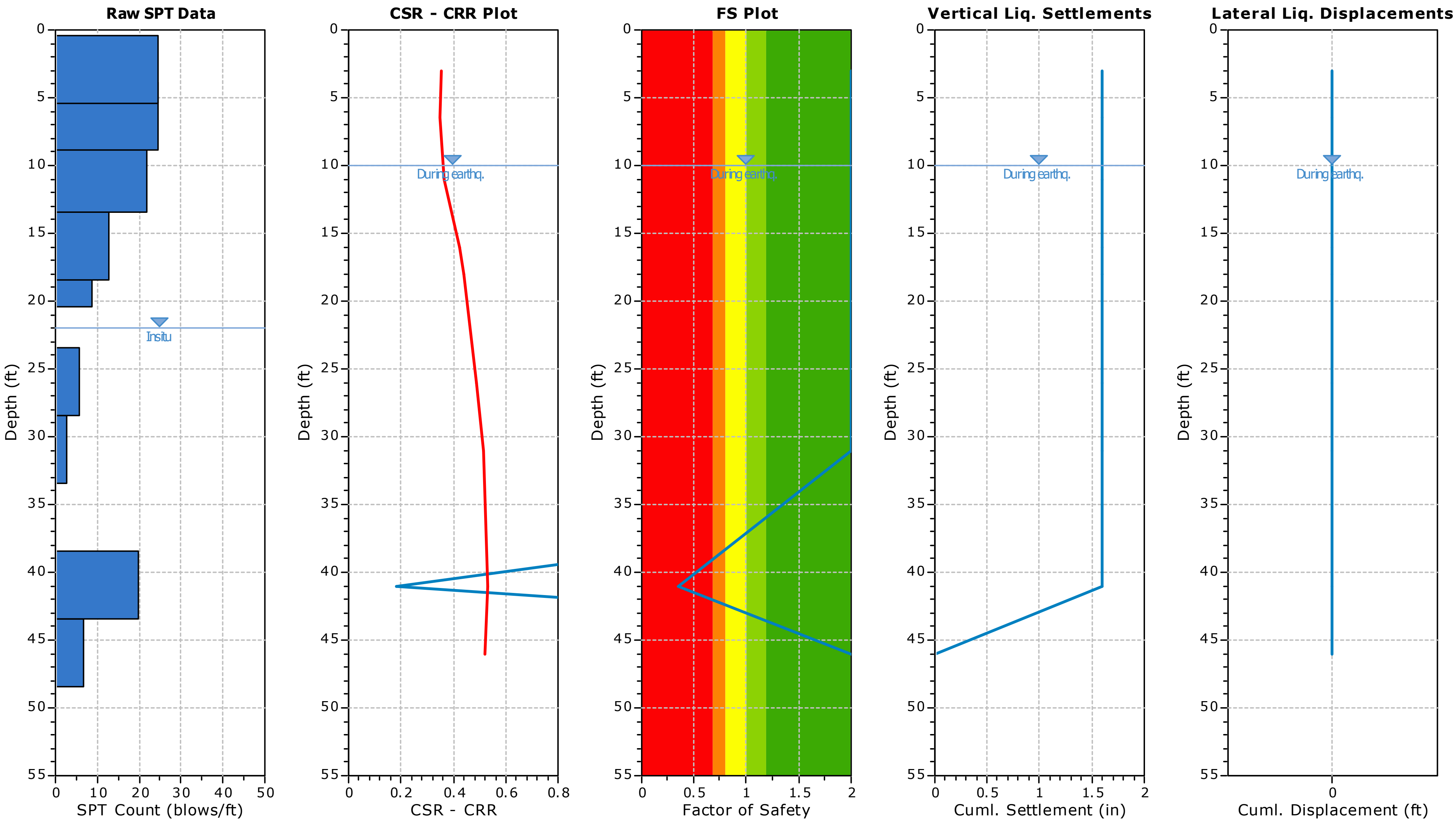




F.S. color scheme
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme
- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
3.00	25	60.00	125.00	5.00	No
6.50	25	60.00	125.00	3.00	No
11.00	22	70.00	126.00	5.00	No
16.00	13	70.00	120.00	3.00	No
18.00	9	70.00	120.00	5.00	No
26.00	6	70.00	117.00	10.00	No
31.00	3	70.00	117.00	10.00	No
41.00	20	5.00	115.00	5.00	Yes
46.00	7	70.00	115.00	5.00	No

Abbreviations

Depth: Depth at which test was performed (ft)
SPT Field Value: Number of blows per foot
Fines Content: Fines content at test depth (%)
Unit Weight: Unit weight at test depth (pcf)
Infl. Thickness: Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy: User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	Fines Content (%)	α	β	$(N_1)_{60cs}$	$CRR_{7.5}$
3.00	25	125.00	0.19	0.00	0.19	1.60	1.15	1.00	0.75	1.00	34	60.00	5.00	1.20	46	4.000
6.50	25	125.00	0.41	0.00	0.41	1.39	1.15	1.00	0.75	1.00	30	60.00	5.00	1.20	41	4.000
11.00	22	126.00	0.69	0.00	0.69	1.19	1.15	1.00	0.85	1.00	26	70.00	5.00	1.20	36	4.000
16.00	13	120.00	0.99	0.00	0.99	1.03	1.15	1.00	0.85	1.00	13	70.00	5.00	1.20	21	4.000
18.00	9	120.00	1.11	0.00	1.11	0.98	1.15	1.00	0.95	1.00	10	70.00	5.00	1.20	17	4.000
26.00	6	117.00	1.58	0.12	1.45	0.85	1.15	1.00	0.95	1.00	6	70.00	5.00	1.20	12	4.000
31.00	3	117.00	1.87	0.28	1.59	0.81	1.15	1.00	1.00	1.00	3	70.00	5.00	1.20	9	4.000
41.00	20	115.00	2.45	0.59	1.85	0.75	1.15	1.00	1.00	1.00	17	5.00	0.00	1.00	17	0.185
46.00	7	115.00	2.73	0.75	1.98	0.72	1.15	1.00	1.00	1.00	6	70.00	5.00	1.20	12	4.000

Abbreviations

σ_v: Total stress during SPT test (tsf)
u_o: Water pore pressure during SPT test (tsf)
σ'_{vo}: Effective overburden pressure during SPT test (tsf)
C_N: Overburden corretion factor
C_E: Energy correction factor
C_B: Borehole diameter correction factor
C_R: Rod length correction factor
C_S: Liner correction factor
$N_{1(60)}$: Corrected N_{SPT} to a 60% energy ratio
α, β: Clean sand equivalent clean sand formula coefficients
$N_{1(60)cs}$: Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$: Cydic resistance ratio for M=7.5

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS	
3.00	125.00	0.19	0.00	0.19	0.99	1.00	0.485	1.37	0.354	1.00	0.354	2.000	●
6.50	125.00	0.41	0.00	0.41	0.99	1.00	0.481	1.37	0.351	1.00	0.351	2.000	●
11.00	126.00	0.69	0.03	0.66	0.98	1.00	0.499	1.37	0.364	1.00	0.364	2.000	●
16.00	120.00	0.99	0.19	0.80	0.97	1.00	0.581	1.37	0.424	1.00	0.424	2.000	●

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS	
18.00	120.00	1.11	0.25	0.86	0.96	1.00	0.605	1.37	0.441	1.00	0.441	2.000	●
26.00	117.00	1.58	0.50	1.08	0.94	1.00	0.669	1.37	0.488	1.00	0.490	2.000	●
31.00	117.00	1.87	0.66	1.22	0.92	1.00	0.687	1.37	0.501	0.97	0.515	2.000	●
41.00	115.00	2.45	0.97	1.48	0.84	1.00	0.679	1.37	0.495	0.94	0.530	0.349	●
46.00	115.00	2.73	1.12	1.61	0.79	1.00	0.657	1.37	0.479	0.92	0.521	2.000	●

Abbreviations

$\sigma_{v,eq}$:	Total overburden pressure at test point, during earthquake (tsf)
$u_{o,eq}$:	Water pressure at test point, during earthquake (tsf)
$\sigma'_{vo,eq}$:	Effective overburden pressure, during earthquake (tsf)
r_d :	Nonlinear shear mass factor
α:	Improvement factor due to stone columns
CSR :	Cyclic Stress Ratio (adjusted for improvement)
MSF :	Magnitude Scaling Factor
$CSR_{eq,M=7.5}$:	CSR adjusted for M=7.5
K_{sigma}:	Effective overburden stress factor
CSR^*:	CSR fully adjusted (user FS applied)***
FS:	Calculated factor of safety against soil liquefaction

*** User FS: 1.00

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
3.00	2.000	0.00	9.54	3.50	0.00
6.50	2.000	0.00	9.01	3.50	0.00
11.00	2.000	0.00	8.32	4.50	0.00
16.00	2.000	0.00	7.56	5.00	0.00
18.00	2.000	0.00	7.26	2.00	0.00
26.00	2.000	0.00	6.04	8.00	0.00
31.00	2.000	0.00	5.28	5.00	0.00
41.00	0.349	0.65	3.75	10.00	7.45
46.00	2.000	0.00	2.99	5.00	0.00

Overall potential I_L : 7.45

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)
3.00	34	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	5.00	0.000
6.50	30	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	3.00	0.000

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)

Cumulative settlemetns: 0.000

Abbreviations

τ_{av}: Average cyclic shear stress
p: Average stress
G_{max}: Maximum shear modulus (tsf)
α, b: Shear strain formula variables
γ: Average shear strain
ε_{15}: Volumetric strain after 15 cycles
N_c: Number of cycles
ε_{Nc}: Volumetric strain for number of cycles N_c (%)
Δh: Thickness of soil layer (in)
ΔS: Settlement of soil layer (in)

:: Vertical settlements estimation for saturated sands ::

Depth (ft)	D_{50} (in)	q_c/N	e_v (%)	Δh (ft)	s (in)
11.00	0.00	5.00	0.00	5.00	0.000
16.00	0.00	5.00	0.00	3.00	0.000
18.00	0.00	5.00	0.00	5.00	0.000
26.00	0.00	5.00	0.00	10.00	0.000
31.00	0.00	5.00	0.00	10.00	0.000
41.00	0.00	5.00	2.67	5.00	1.602
46.00	0.00	5.00	0.00	5.00	0.000

Cumulative settlements: 1.602

Abbreviations

D_{50}: Median grain size (in)
q_c/N: Ratio of cone resistance to SPT
e_v: Post liquefaction volumetric strain (%)
Δh: Thickness of soil layer to be considered (ft)
s: Estimated settlement (in)

:: Lateral displacements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60}$	D_r (%)	γ_{max} (%)	d_z (ft)	LDI	LD (ft)
3.00	34	81.63	0.00	5.00	0.000	0.00
6.50	30	76.68	0.00	3.00	0.000	0.00
11.00	26	71.39	0.00	5.00	0.000	0.00
16.00	13	50.48	0.00	3.00	0.000	0.00
18.00	10	44.27	0.00	5.00	0.000	0.00
26.00	6	34.29	0.00	10.00	0.000	0.00
31.00	3	24.25	0.00	10.00	0.000	0.00
41.00	17	57.72	22.70	5.00	0.000	0.00
46.00	6	34.29	0.00	5.00	0.000	0.00

:: Lateral displacements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60}$	D_r (%)	γ_{max} (%)	d_z (ft)	LDI	LD (ft)

Cumulative lateral displacements: 0.00

Abbreviations

D_r: Relative density (%)
γ_{max}: Maximum amplitude of cyclic shear strain (%)
d_z: Soil layer thickness (ft)
LDI: Lateral displacement index (ft)
LD: Actual estimated displacement (ft)

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Leighton Consulting, Inc.
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PN 13589.002

SPT BASED LIQUEFACTION ANALYSIS REPORT

Project title : Proposed Self Storage Facility **SPT Name: LB-1**

Location : 21101 Ventura Blvd., Woodland Hills, CA

:: Input parameters and analysis properties ::

Analysis method:	NCEER 1998	G.W.T. (in-situ):	22.00 ft
Fines correction method:	NCEER 1998	G.W.T. (earthq.):	10.00 ft
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	6.57
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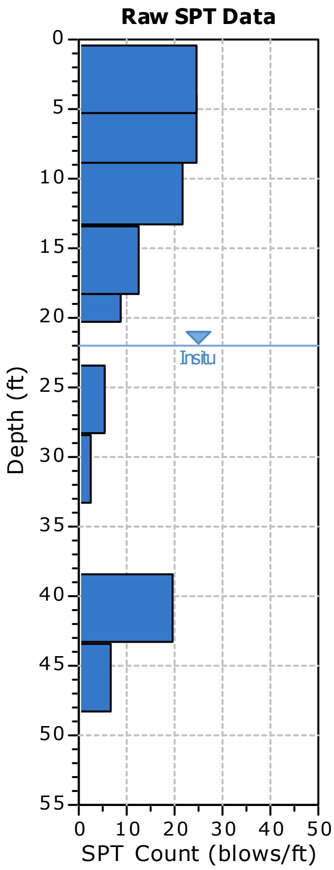
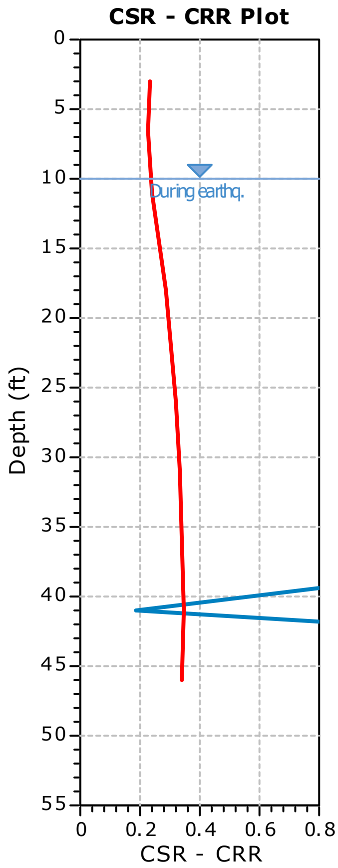
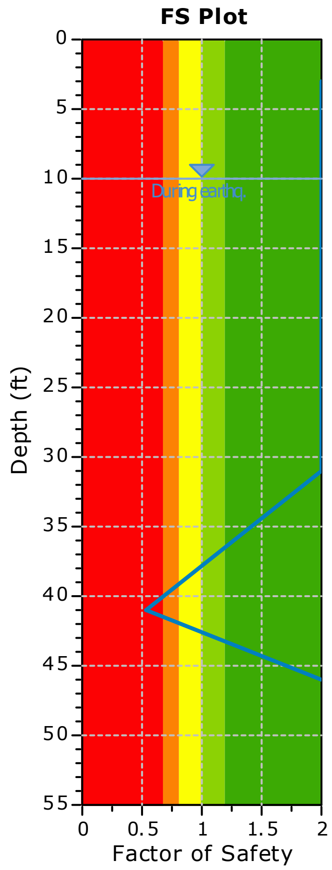
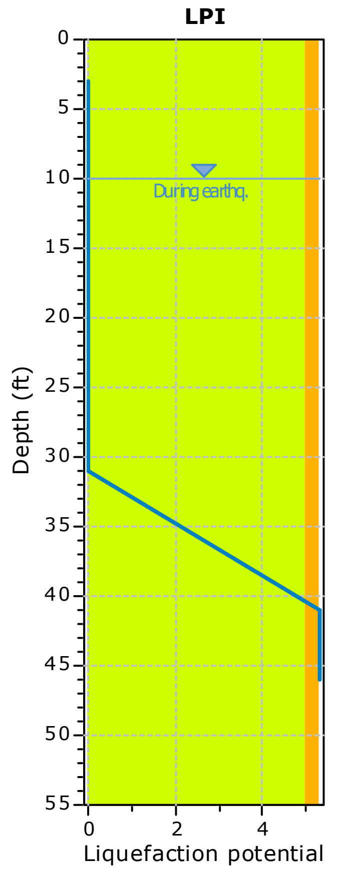
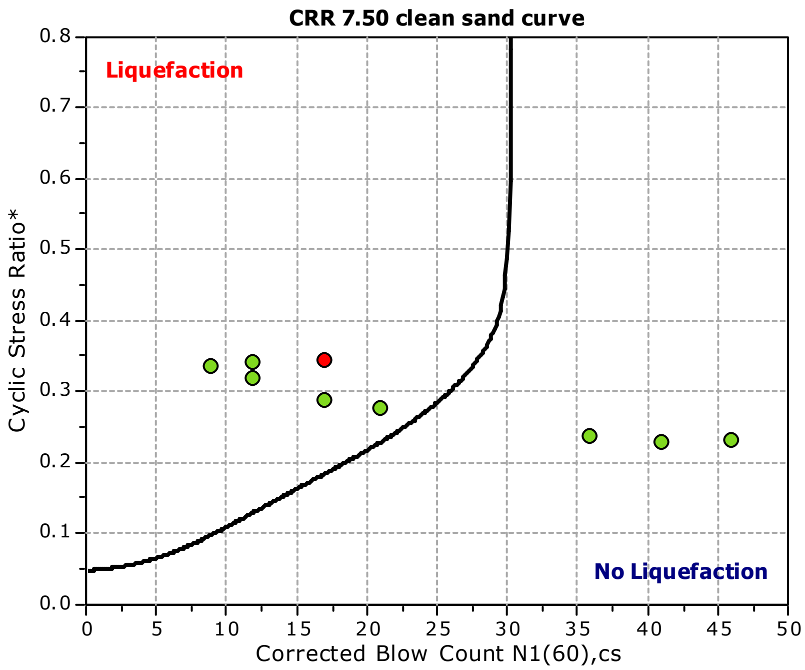
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.15		











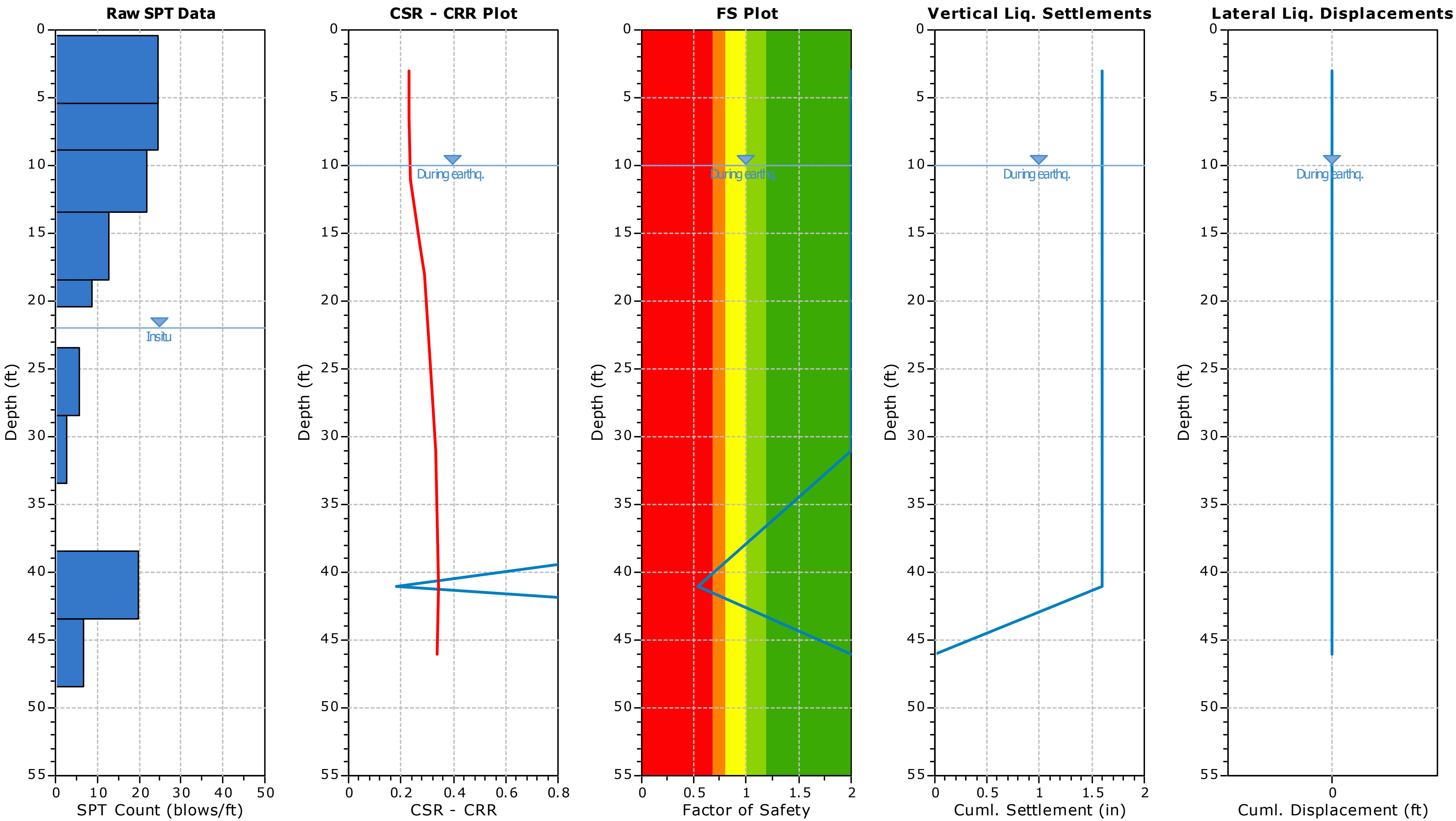
F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
3.00	25	60.00	125.00	5.00	No
6.50	25	60.00	125.00	3.00	No
11.00	22	70.00	126.00	5.00	No
16.00	13	70.00	120.00	3.00	No
18.00	9	70.00	120.00	5.00	No
26.00	6	70.00	117.00	10.00	No
31.00	3	70.00	117.00	10.00	No
41.00	20	5.00	115.00	5.00	Yes
46.00	7	70.00	115.00	5.00	No

Abbreviations

Depth: Depth at which test was performed (ft)
SPT Field Value: Number of blows per foot
Fines Content: Fines content at test depth (%)
Unit Weight: Unit weight at test depth (pcf)
Infl. Thickness: Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy: User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	Fines Content (%)	α	β	$(N_1)_{60cs}$	$CRR_{7.5}$
3.00	25	125.00	0.19	0.00	0.19	1.60	1.15	1.00	0.75	1.00	34	60.00	5.00	1.20	46	4.000
6.50	25	125.00	0.41	0.00	0.41	1.39	1.15	1.00	0.75	1.00	30	60.00	5.00	1.20	41	4.000
11.00	22	126.00	0.69	0.00	0.69	1.19	1.15	1.00	0.85	1.00	26	70.00	5.00	1.20	36	4.000
16.00	13	120.00	0.99	0.00	0.99	1.03	1.15	1.00	0.85	1.00	13	70.00	5.00	1.20	21	4.000
18.00	9	120.00	1.11	0.00	1.11	0.98	1.15	1.00	0.95	1.00	10	70.00	5.00	1.20	17	4.000
26.00	6	117.00	1.58	0.12	1.45	0.85	1.15	1.00	0.95	1.00	6	70.00	5.00	1.20	12	4.000
31.00	3	117.00	1.87	0.28	1.59	0.81	1.15	1.00	1.00	1.00	3	70.00	5.00	1.20	9	4.000
41.00	20	115.00	2.45	0.59	1.85	0.75	1.15	1.00	1.00	1.00	17	5.00	0.00	1.00	17	0.185
46.00	7	115.00	2.73	0.75	1.98	0.72	1.15	1.00	1.00	1.00	6	70.00	5.00	1.20	12	4.000

Abbreviations

σ_v: Total stress during SPT test (tsf)
u_o: Water pore pressure during SPT test (tsf)
σ'_{vo}: Effective overburden pressure during SPT test (tsf)
C_N: Overburden corretion factor
C_E: Energy correction factor
C_B: Borehole diameter correction factor
C_R: Rod length correction factor
C_S: Liner correction factor
$N_{1(60)}$: Corrected N_{SPT} to a 60% energy ratio
α, β: Clean sand equivalent clean sand formula coefficients
$N_{1(60)cs}$: Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$: Cydic resistance ratio for M=7.5

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS	
3.00	125.00	0.19	0.00	0.19	0.99	1.00	0.323	1.40	0.230	1.00	0.230	2.000	●
6.50	125.00	0.41	0.00	0.41	0.99	1.00	0.321	1.40	0.229	1.00	0.229	2.000	●
11.00	126.00	0.69	0.03	0.66	0.98	1.00	0.333	1.40	0.237	1.00	0.237	2.000	●
16.00	120.00	0.99	0.19	0.80	0.97	1.00	0.387	1.40	0.276	1.00	0.276	2.000	●

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS	
18.00	120.00	1.11	0.25	0.86	0.96	1.00	0.403	1.40	0.287	1.00	0.287	2.000	●
26.00	117.00	1.58	0.50	1.08	0.94	1.00	0.446	1.40	0.318	1.00	0.319	2.000	●
31.00	117.00	1.87	0.66	1.22	0.92	1.00	0.458	1.40	0.326	0.97	0.336	2.000	●
41.00	115.00	2.45	0.97	1.48	0.84	1.00	0.453	1.40	0.323	0.94	0.345	0.535	●
46.00	115.00	2.73	1.12	1.61	0.79	1.00	0.438	1.40	0.312	0.92	0.339	2.000	●

Abbreviations

$\sigma_{v,eq}$:	Total overburden pressure at test point, during earthquake (tsf)
$u_{o,eq}$:	Water pressure at test point, during earthquake (tsf)
$\sigma'_{vo,eq}$:	Effective overburden pressure, during earthquake (tsf)
r_d :	Nonlinear shear mass factor
α:	Improvement factor due to stone columns
CSR :	Cyclic Stress Ratio (adjusted for improvement)
MSF :	Magnitude Scaling Factor
$CSR_{eq,M=7.5}$:	CSR adjusted for M=7.5
K_{sigma}:	Effective overburden stress factor
CSR^*:	CSR fully adjusted (user FS applied)***
FS:	Calculated factor of safety against soil liquefaction

*** User FS: 1.00

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
3.00	2.000	0.00	9.54	3.50	0.00
6.50	2.000	0.00	9.01	3.50	0.00
11.00	2.000	0.00	8.32	4.50	0.00
16.00	2.000	0.00	7.56	5.00	0.00
18.00	2.000	0.00	7.26	2.00	0.00
26.00	2.000	0.00	6.04	8.00	0.00
31.00	2.000	0.00	5.28	5.00	0.00
41.00	0.535	0.46	3.75	10.00	5.31
46.00	2.000	0.00	2.99	5.00	0.00

Overall potential I_L : 5.31

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)
3.00	34	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	5.00	0.000
6.50	30	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	3.00	0.000

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)

Cumulative settlemetns: 0.000

Abbreviations

τ_{av}: Average cyclic shear stress
p: Average stress
G_{max}: Maximum shear modulus (tsf)
α, b: Shear strain formula variables
γ: Average shear strain
ε_{15}: Volumetric strain after 15 cycles
N_c: Number of cycles
ε_{Nc}: Volumetric strain for number of cycles N_c (%)
Δh: Thickness of soil layer (in)
ΔS: Settlement of soil layer (in)

:: Vertical settlements estimation for saturated sands ::

Depth (ft)	D_{50} (in)	q_c/N	e_v (%)	Δh (ft)	s (in)
11.00	0.00	5.00	0.00	5.00	0.000
16.00	0.00	5.00	0.00	3.00	0.000
18.00	0.00	5.00	0.00	5.00	0.000
26.00	0.00	5.00	0.00	10.00	0.000
31.00	0.00	5.00	0.00	10.00	0.000
41.00	0.00	5.00	2.67	5.00	1.602
46.00	0.00	5.00	0.00	5.00	0.000

Cumulative settlements: 1.602

Abbreviations

D_{50}: Median grain size (in)
q_c/N: Ratio of cone resistance to SPT
e_v: Post liquefaction volumetric strain (%)
Δh: Thickness of soil layer to be considered (ft)
s: Estimated settlement (in)

:: Lateral displacements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60}$	D_r (%)	γ_{max} (%)	d_z (ft)	LDI	LD (ft)
3.00	34	81.63	0.00	5.00	0.000	0.00
6.50	30	76.68	0.00	3.00	0.000	0.00
11.00	26	71.39	0.00	5.00	0.000	0.00
16.00	13	50.48	0.00	3.00	0.000	0.00
18.00	10	44.27	0.00	5.00	0.000	0.00
26.00	6	34.29	0.00	10.00	0.000	0.00
31.00	3	24.25	0.00	10.00	0.000	0.00
41.00	17	57.72	22.70	5.00	0.000	0.00
46.00	6	34.29	0.00	5.00	0.000	0.00

:: Lateral displacements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60}$	D_r (%)	γ_{max} (%)	d_z (ft)	LDI	LD (ft)

Cumulative lateral displacements: 0.00

Abbreviations

D_r: Relative density (%)
γ_{max}: Maximum amplitude of cyclic shear strain (%)
d_z: Soil layer thickness (ft)
LDI: Lateral displacement index (ft)
LD: Actual estimated displacement (ft)

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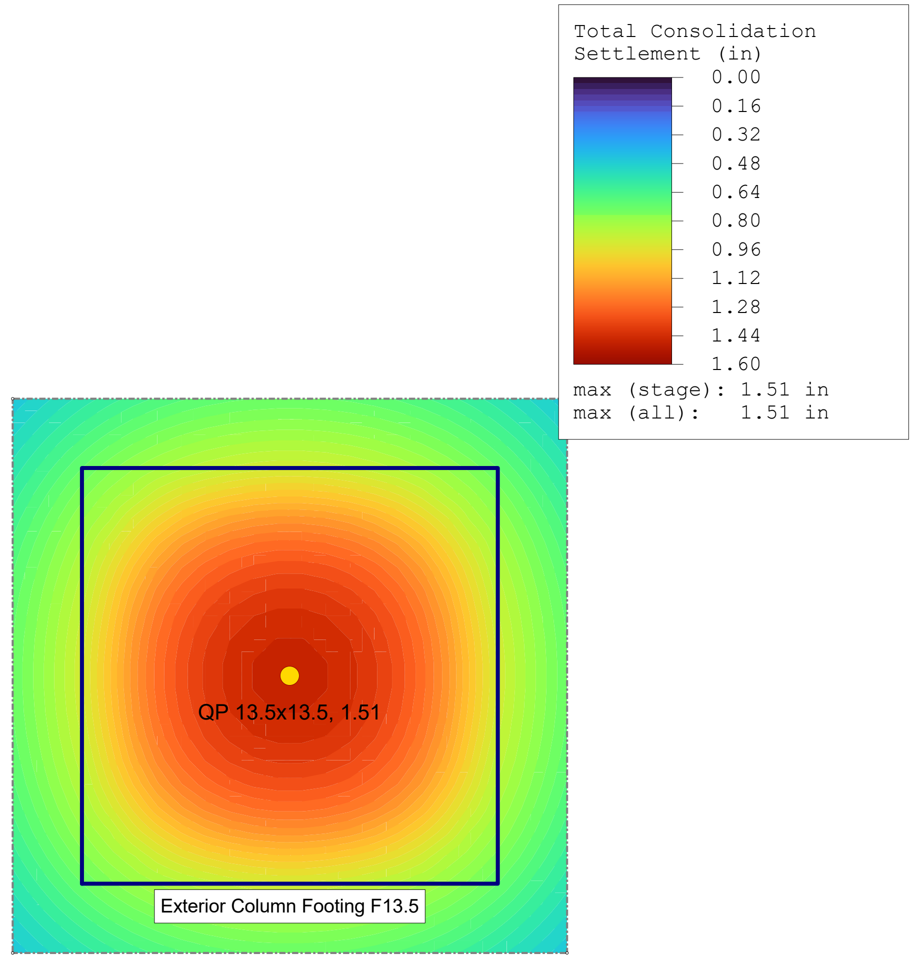
Spread Footing 13.5 x 13.5 feet

\\Ds-sc1\project\InFocus Projects\13501 - 14000\13589 JDA Woodland Hills\002 Geo\Analyses\S E T T L E 3\Spread Footing_135x135_Bous Vert_a1.s3z

Material Name	Color	Unit Weight (kips/ft3)	Es (ksf)	Eur (ksf)	Cc/Cce	Cr/Cre	OCR
Afc PSF		0.12	750	1000	-	-	-
Clay 1		0.12	-	-	0.07	0.014	3
Clay 2		0.12	-	-	0.125	0.022	1.25
Clay 3		0.12	-	-	0.059	0.0083	1.5
Clay 4		0.115	-	-	0.165	0.0205	1.58
Clay 5		0.125	-	-	0.1	0.01	2
Tush Formation		0.115	2200	3000	-	-	-

Compression parameters are Strain-Based



SETTLE3 5.016

Leighton

Project: Proposed Self Storage Building, 21101 Ventura Boulevard, Woodland Hills, Los Angeles, California				
Analyzed By: JEH	Units: feet	Stage No.: Stage 1	Project No.: 13589.002	File Name: Spread Footing_135x135_Bous Vert_a1.s3z
Date: August 10, 2022	Data Displayed: Total Consolidation Settlement			

Settle3 Analysis Information

Proposed Self Storage Building, 21101 Ventura Boulevard, Woodland Hills, Los Angeles, California

Project Settings

Document Name	Spread Footing_135x135_Bous Vert_a1
Project Title	Proposed Self Storage Building, 21101 Ventura Boulevard, Woodland Hills, Los Angeles, California
Analysis	Spread Footing 13.5 x 13.5 feet
Author	JEH
Company	Leighton Consulting, Inc.
Date Created	August 10, 2022
Last saved with Settle3 version	5.016
13589.002	
Stress Computation Method	Boussinesq
Stress Units	Imperial, stress as ksf
Settlement Units	inches

Advanced Settings

Start of secondary consolidation (% of primary)	95
Min. stress for secondary consolidation (% of initial)	1
Reset time when load changes for secondary consolidation	No
Minimum settlement ratio for subgrade modulus	0.9
Use average poisson's ratio to calculate layered stresses	
Update Cv in each time step (improves consolidation accuracy)	
Ignore negative effective stresses in settlement calculations	
Add field points to load edges	

Soil Profile

Layer Option	Horizontal Soil Layers
Vertical Axis	Elevation
Ground Elevation (ft)	899

Stage Settings

Stage #	Name
1	Stage 1

Results

Time taken to compute: 0.0899438 seconds

Stage: Stage 1

Data Type	Minimum	Maximum
Total Settlement [in]	0	1.76696
Total Consolidation Settlement [in]	0	1.51223
Virgin Consolidation Settlement [in]	0	0.418095
Recompression Consolidation Settlement [in]	0	1.10853
Immediate Settlement [in]	0	0.254732
Loading Stress ZZ [ksf]	0	3.5004
Loading Stress XX [ksf]	-0.0526427	2.75168
Loading Stress YY [ksf]	-0.0526427	2.75168
Effective Stress ZZ [ksf]	0	6.56486
Effective Stress XX [ksf]	0	6.53414
Effective Stress YY [ksf]	0	6.53414
Total Stress ZZ [ksf]	0	11.8065
Total Stress XX [ksf]	0	11.7757
Total Stress YY [ksf]	0	11.7757
Modulus of Subgrade Reaction (Total) [ksf/ft]	0	52.4655
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	0	495.155
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	0	58.6835
Total Strain	0	0.0187054
Pore Water Pressure [ksf]	0	5.2416
Degree of Consolidation [%]	0	100
Pre-consolidation Stress [ksf]	0.012	9.19196
Over-consolidation Ratio	1	2.57556
Void Ratio	0	1.09986
Hydroconsolidation Settlement [in]	0	0
Undrained Shear Strength	0	0.140806

Loads

1. Rectangular Load: "Exterior Column Footing F13.5"

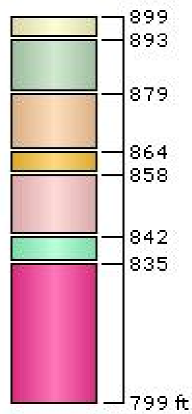
Length	13.5 ft
Width	13.5 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	182.25 ft2
Load	3.5 ksf
Elevation	897 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
8.156	3.432
21.656	3.432
21.656	16.932
8.156	16.932

Soil Layers

Layer #	Type	Thickness [ft]	Elevation [ft]
1	Afc PSF	6	899
2	Clay 1	14	893
3	Clay 2	15	879
4	Clay 3	6	864
5	Clay 4	16	858
6	Clay 5	7	842
7	Tush Formation	36	835



Soil Properties

Property	Afc PSF	Clay 1	Clay 2	Clay 3
Color				
Unit Weight [kips/ft3]	0.12	0.12	0.12	0.12
Saturated Unit Weight [kips/ft3]	0.12	0.12	0.12	0.12
K0	1	1	1	1
Immediate Settlement	Enabled	Disabled	Disabled	Disabled
Es [ksf]	750	-	-	-
Esur [ksf]	1000	-	-	-
Primary Consolidation	Disabled	Enabled	Enabled	Enabled
Material Type		Non-Linear	Non-Linear	Non-Linear
Cce	-	0.07	0.125	0.059
Cre	-	0.014	0.022	0.0083
e0	-	1.1	1.1	1.1
OCR	-	3	1.25	1.5
Undrained Su A [kips/ft2]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8
Piezo Line ID	1	1	1	1

Property	Clay 4	Clay 5	Tush Formation
Color			
Unit Weight [kips/ft3]	0.115	0.125	0.115
Saturated Unit Weight [kips/ft3]	0.115	0.125	0.115
K0	1	1	1
Immediate Settlement	Disabled	Disabled	Enabled
Es [ksf]	-	-	2200
Esur [ksf]	-	-	3000
Primary Consolidation	Enabled	Enabled	Disabled
Material Type	Non-Linear	Non-Linear	
Cce	0.165	0.1	-
Cre	0.0205	0.01	-
e0	1.1	1.1	-
OCR	1.58	2	-
Undrained Su A [kips/ft2]	0	0	0
Undrained Su S	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8
Piezo Line ID	1	1	1

Groundwater

Groundwater method	Piezometric Lines
Water Unit Weight	0.0624 kips/ft3

Piezometric Line Entities

ID	Elevation (ft)
1	883 ft

Query

Query Points

Point #	Query Point Name	(X,Y) Location	Number of Divisions
1	QP 13.5x13.5	14.906, 10.182	101

Field Point Grid

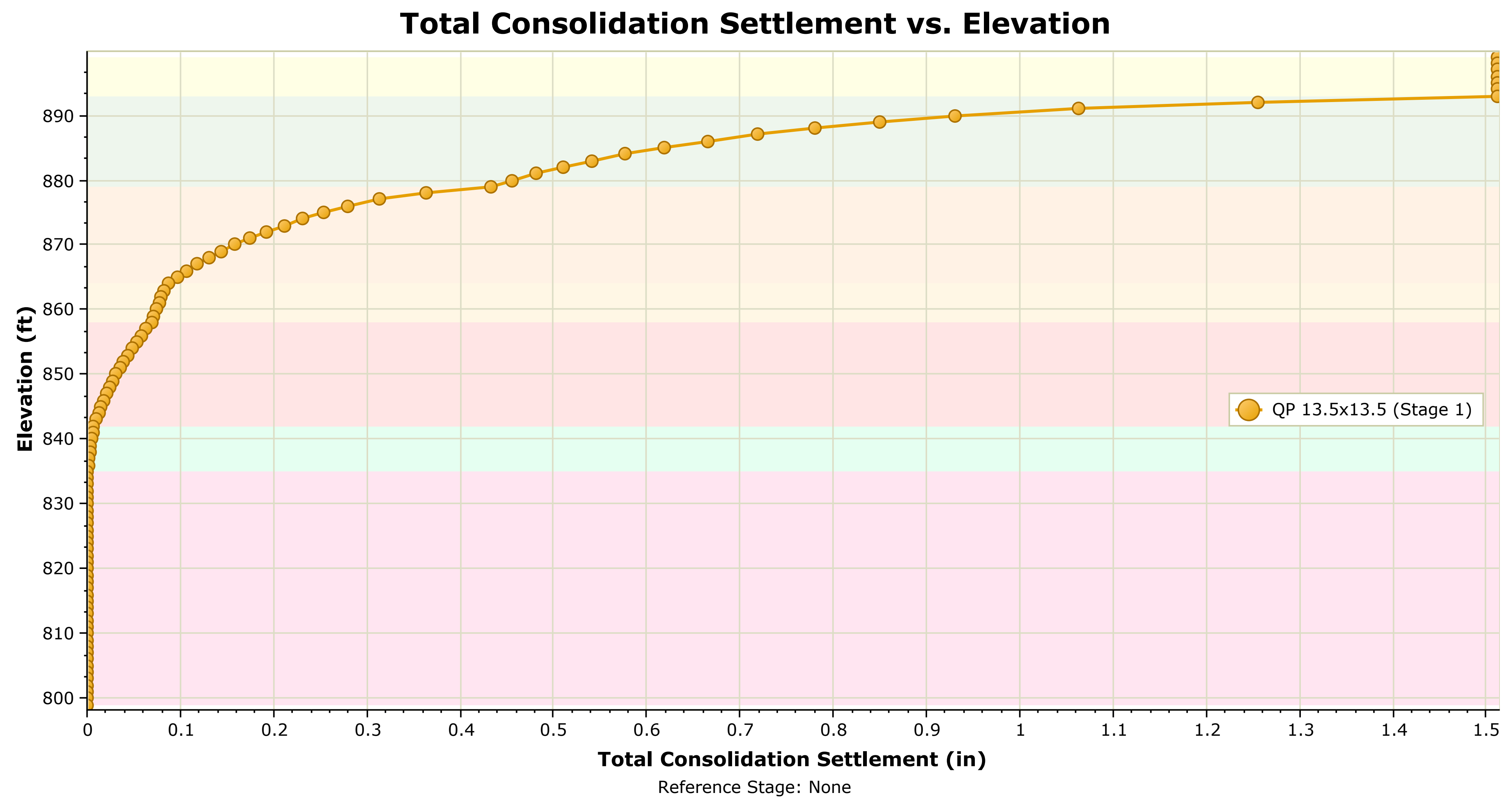
Number of points	652
Expansion Factor	1.5

Grid Coordinates

X [ft]	Y [ft]
28.406	23.682
28.406	-3.318
1.406	-3.318
1.406	23.682

Spread Footing 13.5 x 13.5 feet

\\Ds-sc1\project\InFocus Projects\13501 - 14000\13589 JDA Woodland Hills\002 Geo\Analyses\S E T T L E 3\Spread Footing_135x135_Bous Vert_a1.s3z



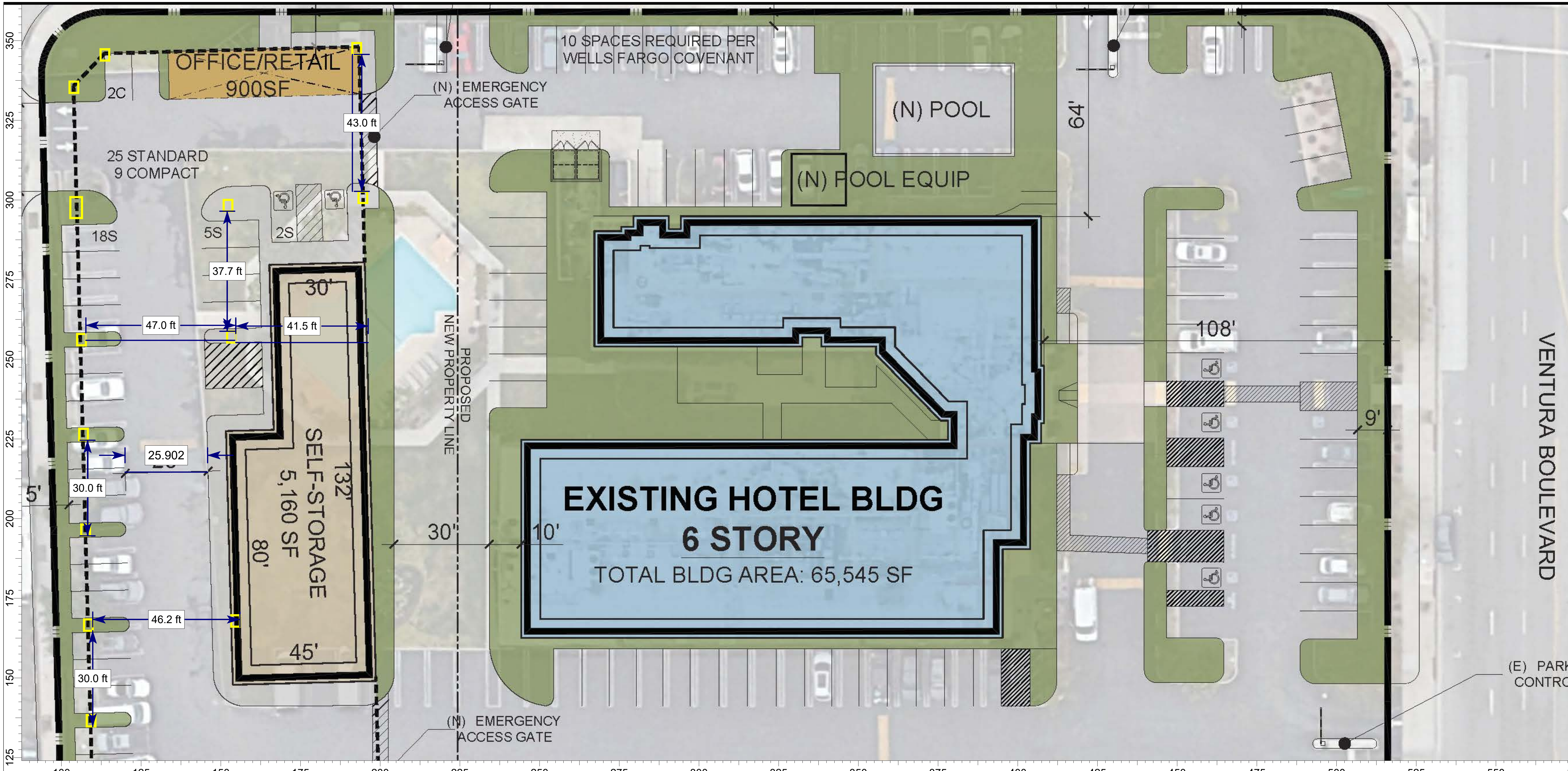
SETTLE3 5.016

Project: Proposed Self Storage Building, 21101 Ventura Boulevard, Woodland Hills, Los Angeles, California				
Analyzed By: JEH	Units: feet	Stage No.:	Project No.: 13589.002	File Name: Spread Footing_135x135_Bous Vert_a1.s3z
Date: August 10, 2022	Data Displayed:			

Architectural Site Plan

Estimated Foundation Layout and Loads

\\Ds-sc1\project\InFocus Projects\13501 - 14000\13589 JDA Woodland Hills\002 Geo\Analyses\SLIDE Site Plan\13589.002_Site Plan.slim



SLIDE 9.023

Leighton

Project: Proposed Self Storage Facility, 21101 Ventura Boulevard, Woodland Hills (Los Angeles) CA			
Analyzed By: JEH	Units: feet	Scale: 1:360	Project No.: 13589.002
Date: July 29, 2022	Condition: Sketch		File Name: 13589.002_Site Plan.slim

APPENDIX E

GENERAL EARTHWORK AND GRADING RECOMMENDATIONS

APPENDIX E

LEIGHTON CONSULTING, INC.
EARTHWORK AND GRADING GUIDE SPECIFICATIONS

TABLE OF CONTENTS

E-1.0 GENERAL

E-1.1 <u>Intent</u>

These Earthwork and Grading Guide Specifications are for grading and earthwork shown on the current, approved grading plan(s) and/or indicated in the Leighton Consulting, Inc. geotechnical report(s). These Guide Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the project-specific recommendations in the geotechnical report shall supersede these Guide Specifications. Leighton Consulting, Inc. shall provide geotechnical observation and testing during earthwork and grading. Based on these observations and tests, Leighton Consulting, Inc. may provide new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

E-1.2 <u>Role of Leighton Consulting, Inc.</u>

Prior to commencement of earthwork and grading, Leighton Consulting, Inc. shall meet with the earthwork contractor to review the earthwork contractor's work plan, to schedule sufficient personnel to perform the appropriate level of observation, mapping and compaction testing. During earthwork and grading, Leighton Consulting, Inc. shall observe, map, and document subsurface exposures to verify geotechnical design assumptions. If observed conditions are found to be significantly different than the interpreted assumptions during the design phase, Leighton Consulting, Inc. shall inform the owner, recommend appropriate changes in design to accommodate these observed conditions, and notify the review agency where required. Subsurface areas to be geotechnically observed, mapped, elevations recorded, and/or tested include (1) natural ground after clearing to receiving fill but before fill is placed, (2) bottoms of all "remedial removal" areas, (3) all key bottoms, and (4) benches made on sloping ground to receive fill.

Leighton Consulting, Inc. shall observe moisture-conditioning and processing of the subgrade and fill materials, and perform relative compaction testing of fill to determine the attained relative compaction. Leighton Consulting, Inc. shall provide *Daily Field Reports* to the owner and the Contractor on a routine and frequent basis.

E-1.3 <u>The Earthwork Contractor</u>

The earthwork contractor (Contractor) shall be qualified, experienced and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Guide

Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing grading and backfilling in accordance with the current, approved plans and specifications.

The Contractor shall inform the owner and Leighton Consulting, Inc. of changes in work schedules at least one working day in advance of such changes so that appropriate observations and tests can be planned and accomplished. The Contractor shall not assume that Leighton Consulting, Inc. is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish earthwork and grading in accordance with the applicable grading codes and agency ordinances, these Guide Specifications, and recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of Leighton Consulting, Inc., unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, adverse weather, etc., are resulting in a quality of work less than required in these specifications, Leighton Consulting, Inc. shall reject the work and may recommend to the owner that earthwork and grading be stopped until unsatisfactory condition(s) are rectified.

E-2.0 PREPARATION OF AREAS TO BE FILLED

E-2.1 <u>Clearing and Grubbing</u>

Vegetation, such as brush, grass, roots and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies and Leighton Consulting, Inc.. Care should be taken not to encroach upon or otherwise damage native and/or historic trees designated by the Owner or appropriate agencies to remain. Pavements, flatwork or other construction should not extend under the “drip line” of designated trees to remain.

Leighton Consulting, Inc. shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 3 percent of organic materials (by dry weight: ASTM D 2974). Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area. As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed.

E-2.2 Processing

Existing ground that has been declared satisfactory for support of fill, by Leighton Consulting, Inc., shall be scarified to a minimum depth of 6 inches (15 cm). Existing ground that is not satisfactory shall be over-excavated as specified in the following Section E-2.3. Scarification shall continue until soils are broken down and free of large clay lumps or clods and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

E-2.3 Overexcavation

In addition to removals and over-excavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be over-excavated to competent ground as evaluated by Leighton Consulting, Inc. during grading. All undocumented fill soils under proposed structure footprints should be excavated

E-2.4 Benching

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), (>20 percent grade) the ground shall be stepped or benched. The lowest bench or key shall be a minimum of 15 feet (4.5 m) wide and at least 2 feet (0.6 m) deep, into competent material as evaluated by Leighton Consulting, Inc.. Other benches shall be excavated a minimum height of 4 feet (1.2 m) into competent material or as otherwise recommended by Leighton Consulting, Inc.. Fill placed on ground sloping flatter than 5:1 (horizontal to vertical units), (<20 percent grade) shall also be benched or otherwise over-excavated to provide a flat subgrade for the fill.

E-2.5 Evaluation/Acceptance of Fill Areas

All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by Leighton Consulting, Inc. as suitable to receive fill. The Contractor shall obtain a written acceptance (*Daily Field Report*) from Leighton Consulting, Inc. prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys and benches.

E-3.0 FILL MATERIAL

E-3.1 Fill Quality

Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by Leighton Consulting, Inc. prior to placement. Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to Leighton Consulting, Inc. or mixed with other soils to achieve satisfactory fill material.

E-3.2 Oversize

Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 6 inches (15 cm), shall not be buried or placed in fill unless location, materials and placement methods are specifically accepted by Leighton Consulting, Inc.. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 feet (3 m) measured vertically from finish grade, or within 2 feet (0.61 m) of future utilities or underground construction.

E-3.3 Import

If importing of fill material is required for grading, proposed import material shall meet the requirements of Section E-3.1, and be free of hazardous materials ("contaminants") and rock larger than 3-inches (8 cm) in largest dimension. All import soils shall have an Expansion Index (EI) of 20 or less and a sulfate content no greater than (≤) 500 parts-per-million (ppm). A representative sample of a potential import source shall be given to Leighton Consulting, Inc. at least four full working days before importing begins, so that suitability of this import material can be determined and appropriate tests performed.

E-4.0 FILL PLACEMENT AND COMPACTION

E-4.1 Fill Layers

Approved fill material shall be placed in areas prepared to receive fill, as described in Section E-2.0, above, in near-horizontal layers not exceeding 8 inches (20 cm) in loose thickness. Leighton Consulting, Inc. may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers, and only if the building officials with the appropriate jurisdiction approve. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

E-4.2 Fill Moisture Conditioning

Fill soils shall be watered, dried back, blended and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM) Test Method D 1557.

E-4.3 Compaction of Fill

After each layer has been moisture-conditioned, mixed, and evenly spread, each layer shall be uniformly compacted to not-less-than (≥) 90 percent of the maximum dry density as determined by ASTM Test Method D 1557. In some cases, structural fill may be specified (see project-specific geotechnical report) to be uniformly compacted to at-least (≥) 95 percent of the ASTM D 1557 modified Proctor laboratory maximum dry density. For fills thicker than (>) 15 feet (4.5 m), the portion of fill deeper than 15 feet below proposed finish grade shall be compacted to 95 percent of the ASTM D 1557 laboratory maximum density. Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

E-4.4 Compaction of Fill Slopes

In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by back rolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet (1 to 1.2 m) in fill elevation, or by other methods producing satisfactory results acceptable to Leighton Consulting, Inc.. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of the ASTM D 1557 laboratory maximum density.

E-4.5 Compaction Testing

Field-tests for moisture content and relative compaction of the fill soils shall be performed by Leighton Consulting, Inc.. Location and frequency of tests shall be at our field representative(s) discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).

E-4.6 Compaction Test Locations

Leighton Consulting, Inc. shall document the approximate elevation and horizontal coordinates of each density test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that Leighton

Consulting, Inc. can determine the test locations with sufficient accuracy. Adequate grade stakes shall be provided.

E-5.0 EXCAVATION

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by Leighton Consulting, Inc. during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by Leighton Consulting, Inc. based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, then observed and reviewed by Leighton Consulting, Inc. prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by Leighton Consulting, Inc..

E-6.0 TRENCH BACKFILLS

E-6.1 Safety

The Contractor shall follow all OSHA and Cal/OSHA requirements for safety of trench excavations. Work should be performed in accordance with Article 6 of the *California Construction Safety Orders*, 2009 Edition or more current (see also: http://www.dir.ca.gov/title8/sb4a6.html).

E-6.2 Bedding and Backfill

All utility trench bedding and backfill shall be performed in accordance with applicable provisions of the 2018 Edition of the *Standard Specifications for Public Works Construction* (Green Book). Bedding material shall have a Sand Equivalent greater than 30 (SE>30). Bedding shall be placed to 1-foot (0.3 m) over the top of the conduit, and densified by jetting in areas of granular soils, if allowed by the permitting agency. Otherwise, the pipe-bedding zone should be backfilled with Controlled Low Strength Material (CLSM) consisting of at least one sack of Portland cement per cubic-yard of sand, and conforming to Section 201-6 of the 2018 Edition of the *Standard Specifications for Public Works Construction* (Green Book). Backfill over the bedding zone shall be placed and densified mechanically to a minimum of 90 percent of relative compaction (ASTM D 1557) from 1 foot (0.3 m) above the top of the conduit to the surface. Backfill above the pipe zone shall **not** be jetted. Jetting of the bedding around the conduits shall be observed by Leighton Consulting, Inc. and backfill above the pipe zone (bedding) shall be observed and tested by Leighton Consulting, Inc..

E-6.3 <u>Lift Thickness</u>

Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to Leighton Consulting, Inc. that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method, and only if the building officials with the appropriate jurisdiction approve.