GEOTECHNICAL EXPLORATION REPORT
MALIBU MIDDLE AND HIGH SCHOOL CAMPUS PLAN
PHASE I HIGH SCHOOL CORE PROJECT - BUILDING C
30237 MORNING VIEW DRIVE
MALIBU, LOS ANGELES COUNTY, CALIFORNIA

Prepared For **SANTA MONICA – MALIBU**
UNIFIED SCHOOL DISTRICT
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Santa Monica – Malibu Unified School District
c/o HMC Architects
633 West 5th Street, 3rd Floor
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Attention: Ms. Noemi Avila Zamudio,
Deputy Bond Program Manager

Subject: Geotechnical Exploration Report
Malibu Middle and High School Campus Plan
Phase I High School Core Project - Building C
30237 Morning View Drive
Malibu, Los Angeles County, California

In accordance with our proposal, dated June 17, 2020, authorized on July 16, 2020; Leighton Consulting, Inc. (Leighton) has prepared this geotechnical exploration report for the subject project. The purpose of our geotechnical exploration was to evaluate subsurface conditions at the project site and provide geotechnical recommendations to aid in design and construction.

We understand that the subject Phase I High School Core improvements include the demolition of the existing Juan Cabrillo Elementary School campus (11 buildings) and construction of a new Type II-B sprinklered 2-Story 80,000 square foot multi-purpose classrooms building (Building "C") at grade with no subterranean structures planned. Ancillary improvements include perimeter drive access, paved parking, utilities, hardscape and paver construction.

The Phase I High School Core Project - Building C, as currently proposed, is considered feasible from a geotechnical standpoint. Geotechnical recommendations with respect to earthwork and foundation design are presented in this report.

We appreciate the opportunity to provide our services for this project. We trust that the information contained herein meets the project requirements. 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 extension and e-mail address listed below.

Respectfully submitted,

LEIGHTON CONSULTING, INC.

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1.0 INTRODUCTION

1.1 Description and Proposed Development

Juan Cabrillo Elementary School (JCES) campus is located at 30237 Morning View Drive, immediately adjacent to and northeast of the Malibu Middle and High School (MMHS) campus in the city of Malibu, California. Currently, the JCES campus includes 11 classroom/administration buildings, playgrounds, playfields, asphalt paved parking lots, and various other ancillary improvements. The project site location (latitude N34.0251°, longitude -118.8286°) is shown on Figure 1, *Site Location Map*.

Our understanding of the project is based on review of the *Request for Qualifications/Proposals for Geotechnical Services, Malibu Middle & High School Campus Plan: Phase I New High School Core Project*, undated and our previous experience on site as geotechnical engineer of record. The MMHS Campus Plan, to be constructed in four phases (Phases 1, 2, 3, 4a and 4b) beginning in fall of 2022 will redevelop and modernize the existing MMHS campus and JCES to create three distinct areas: Middle School Core, High School Core and shared facilities with all Phases completed by 2030. Implementation of the campus plan will result in eventual demolition of all 11 buildings at JCES and 7 buildings on the MMHS campus.

The Phase I High School Core project (Proposed Building "C") will be located at the southwest corner of the campus, following demolition of the JCES Campus. We understand Building "C" is currently proposed as a two-story, Type II-B, 2-Story 80,000 square foot multi-purpose building with 25 classrooms, administration offices, supportive services, a library, 6 science labs, Art 3D sculpture and ceramics studio, lunch shelter, and a career center. The planned footprint of Building C improvements are is shown on Plate 1, *Geotechnical Map*.

1.2 Previous Geotechnical Explorations

Leighton Consulting, Inc. (Leighton) performed a series of previous geotechnical explorations at the MMHS and JCES campuses. Information collected during previous explorations supplemented explorations performed as part of the current study. The locations of previous explorations are presented on Plate 1.

2009: New Administration and Classroom Building: In early 2009, Leighton performed a geotechnical investigation in support of a proposed new administration and classroom building located in the southeastern portion of the campus, fronting Morning View Avenue, along with new bleachers and light poles for the existing football field. The field exploration program consisted of four (4) hollow-stem auger borings (B-1 through B-4) drilled to approximate depths ranging from 5 to 51½ feet below ground surface (bgs). Five (5) previous borings excavated by Associated Soils Engineering, Inc. (ASE, 1999) were incorporated into our analysis. The findings and recommendations were provided in a report dated January 16, 2009 (Leighton, 2009a).

2009-2010: Supplemental Explorations: In late 2009 through 2010, Leighton performed a series of supplemental geotechnical investigations throughout the MMHS campus in support of Parking Lot A, a stormwater collection and disposal system, and infiltration for the existing basketball courts. In total, four (4) additional hollow-stem augers and three (3) test pits were excavated on-site. The results of our findings were presented in a series of addenda to our geotechnical report (Leighton, 2009b through 2009d, and 2010a through 2010b).

2011: Stairway System: In support of a proposed stairway and ramp system located on the slope behind Building E, Leighton performed evaluation consisting of geologic mapping and slope stability analysis. The results of our findings and analysis are presented in a report dated October 10, 2011 (Leighton, 2011).

2016: Parking Lot E Access Road: A supplemental geotechnical exploration was performed for a proposed access road to Parking Lot E, consisting of one bucket auger (LBA-1) advanced to an approximate depth of 80 feet bgs. Subsurface stratigraphy was down-hole logged by our State of California Certified Engineering Geologist and geotechnical recommendations were provided in a report dated September 21, 2016 (Leighton, 2016a).

2016: Building E Replacement: In support of the proposed replacement of Building E, located on the southeast portion of the campus, Leighton advanced three (3) hollow-stem auger borings (LB-1 through LB-3) to depths ranging from 13½ feet to 36½ feet bgs. Findings and recommendations were presented in a report dated November 18, 2016 (Leighton, 2016b).

2017: Addendum 1 to Geotechnical Report, Response to Review Comments, Existing Building E Replacement: The response report was prepared to provide additional information to California Geological Survey (CGS) regarding slope stability for the 45-foot-high slope located east of Building E. Shear strength parameters and stability analyses were provided to CGS to aid in their review (Leighton, 2017a).

2017: Addendum 2 to Geotechnical Report, Response to City of Malibu Review Comments, Existing Building E Replacement: The response report was prepared to provide additional information to the City of Malibu regarding slope stability for a 45-foot-high slope located east of Building E. Shear strength parameters, seismic coefficients and stability analyses were provided to aid the City to aid in their review (Leighton, 2017b).

1.3 Purpose and Scope of Work

The purpose of our current geotechnical exploration was to evaluate subsurface conditions at the High School Core (JCES) Building C site and provide geotechnical recommendations to aid in design and construction of the project as currently proposed. The scope of this geotechnical exploration included the following tasks:

- **Background Review** – A background review was performed of our previous geotechnical reports and published geotechnical maps related to the project site and vicinity that are contained in Leighton's in-house library. References used in preparation of this report are listed in Section 7.0.

- **Pre-Field Exploration Activities** – Boring locations were marked during a pre-field site reconnaissance. Underground Service Alert (USA) was notified to locate and mark existing underground utilities prior to our subsurface exploration.

- **Field Exploration** – Our field exploration was performed on August 17, 2020, and consisted of drilling eight (8) hollow-stem auger borings (2020 LB-1 through 2020 LB-7 and 2020 LP-1) to depths ranging from 15 to 46½ feet bgs. In addition, six cone penetrometer (CPT) soundings (designated CPT-1 through CPT-6) were advanced to depths ranging from approximately 25 to 52 feet bgs. The approximate locations and depths of subsurface explorations (both current and previous) are shown on Plate 1.

Soil and bedrock encountered in the borings were logged in the field by a Leighton representative and described in accordance with the Unified Soil Classification System (ASTM D 2488). During drilling, bulk and relatively undisturbed drive samples were obtained from the borings for geotechnical laboratory testing and evaluation. The relatively undisturbed samples were collected utilizing a Modified California Ring sampler conducted in accordance with ASTM Test Method D 3550. Standard Penetration Tests (SPTs) were performed using a 24-inch-long, 1-3/8-inch I.D. and 2-inch O.D. split spoon sampler. The samplers were driven for a total penetration of 18 inches unless practical refusal, using a 140-pound automatic hammer falling freely for 30 inches in general accordance with ASTM Test Method D 1586. The number of blow counts per 6 inches of penetration was recorded on the boring logs. After completion of drilling the borings were backfilled with tamped soil cuttings. Logs of the hollow stem auger borings and CPTs, both past and present, are included in Appendix A, *Exploration Logs*.

- **Percolation Testing** – Boring LP-1 was converted to a temporary percolation test well upon completion of drilling and sampling. In-situ percolation testing was performed in general accordance with the County of Los Angeles Department of Public Works (LADPW) *Guidelines for Geotechnical Investigation and Reporting Low Impact Development Stormwater Infiltration* (LADPW, 2017). The results of the percolation testing are presented in Appendix B, *Percolation Test Results*. Refer to the discussion of infiltration rate presented in Section 2.3 *Infiltration*. Upon completion of the percolation testing, the well casing was removed and the boring backfilled with soil cuttings and patched with asphalt concrete (AC) to match existing site conditions.

- **Geotechnical Laboratory Testing** – Visual classification and laboratory testing were conducted on selected samples obtained during our field investigation. The laboratory testing program was performed to determine the geotechnical engineering properties of subsurface materials. The following laboratory tests were performed:

  - Moisture content and dry density (ASTM D2937),
  - Atterberg Limits (ASTM D4318),
  - Direct Shear (ASTM D3080),
  - Consolidation (ASTM D2435),
  - Expansion Index (ASTM D4829),

- Modified Proctor Compaction (ASTM D1557),
- R-Value (DOT CA Test 301), and
- Soluble sulfate, soluble chloride, pH and minimum resistivity (CTM 417 Part II, CTM 422, and CTM 532/643).

Results of moisture content and dry density testing are presented on the boring logs in Appendix A. Full laboratory test results are presented in Appendix C, *Laboratory Test Results*.

- **Engineering Analyses** – Geotechnical analysis was performed on the collected data to develop conclusions and recommendations for design and construction.
- **Report Preparation** – This geotechnical report presents our findings, conclusions, and recommendations.

2.0 GEOTECHNICAL FINDINGS

2.1 Regional Geologic Setting

The JCES campus is located within a narrow, terraced coastal strip separating the present-day beach from the high and steeper slopes of the Santa Monica Mountains. The Santa Monica Mountains are part of Transverse Ranges Geomorphic Province. The 11 geomorphic provinces in California represent topographic-geologic groupings of convenience based primarily on landforms, characteristics, lithologies, and late Cenozoic structural and geomorphic history. The Santa Monica Mountains thus have certain similarities with the Santa Ynez, San Gabriel, and San Bernardino mountain chains, as well as the offshore Channel Islands. The Transverse Ranges share pronounced topographic and structural east-west trends, in contrast to most major physiographic features of California which trend northwest-southeast or north-south.

Within the Cenozoic era, geologic evidence suggests the Santa Monica Mountains region has undergone two significant periods of crustal deformation, one in the middle Miocene time and a second period in Quaternary time up to the present. The former period was marked by crustal extension, volcanism, and the contemporaneous development of large detachment sheets (Campbell et al., 1996). The latter period encompasses the compressional regime active today. Compression of the western Santa Monica Mountains has resulted along several north-dipping thrust ramps such as the Malibu Coast Fault (e.g., Section A-A' of Campbell et al., 1996), and the Anacapa Fault of the Transverse Ranges province approximately 3 miles offshore from Point Dume. Because the Malibu Coast and Anacapa fault zones are active seismogenic structures close to the site, they are considered to be the most significant geologic structures influencing project design.

The Santa Monica Mountains expose a thick sequence of Cretaceous-age and younger sedimentary and extrusive igneous rocks. The sedimentary section includes sandstones, siltstones, and mudstones. The volcanic rocks include subaerial and subaqueous flows, breccias, and tuffs with reported aggregate stratigraphic thickness exceeding 28,000 feet for the eastern Santa Monica Mountains (Campbell et al., 1996). Close to the site, at least 4,500 to 5,000 feet of mostly marine sediments overlie crystalline metamorphic basement rock, according to well data (Campbell et al., 1996).

Important topographic and geomorphic features found in the Malibu coastal strip include Point Dume, and three principal marine terraces, at elevations from approximately 100 to 250 feet (Birkeland, 1972). The MMHS campus is situated on the middle of the three terraces. The terraces represent wave-cut platforms incised into bedrock or older surficial deposits. They are capped by marine shoreface deposits and non-marine stream terrace alluvium and debris flow deposits. Most of the terrace surfaces near the site have been dissected by erosion to a greater or lesser extent, with subsequent deposition of clastic gravel, sand, silt, and clay in the eroded channels.

2.2 Subsurface Conditions

Presented below is a brief description of the subsurface conditions encountered in the explorations completed at the site. More detailed descriptions of the geologic units encountered are presented on the logs in Appendix A. Subsurface 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.

Three geologic units were identified during field exploration, Artificial Fill (Afu), Quaternary Young Terrace Deposits (Qyd) equivalent to Quaternary Old Playa and Estuarine Deposits (Qol of Figure 2), and bedrock of the Tertiary age Monterey Formation (Tm). Campus site development has essentially masked or removed all surface exposures of natural geologic units and structure. Thus, it is not feasible to independently map the distribution of units over the project area. Local geology was developed from the map by the United States Geological Survey (Yerkes and Campbell, 2005). Figure 2, *Regional Geology Map*, illustrates the approximate distribution of geologic units at the site. The geologic profile interpreted from the boring data is presented on Plate 2, *Geologic Cross-Sections A-A', B-B'*.

The encountered geologic units are described below:

Undocumented Artificial Fill (Map Symbol: Afu): In general, artificial fill materials represent a relatively thin veneer across the project area and were encountered at thicknesses ranging from 1 to 7 feet. As encountered in our subsurface explorations, the site artificial fill materials consist predominantly of locally derived dark brown to black clay with minor amounts of olive brown silty sand. Because there is no readily available documentation regarding the origin or date of placement, it is considered uncertified and is recommended to be removed, reworked and replaced as engineered (certified) fill.

Quaternary Young Terrace Deposits (Map Symbol: Qyd): Beneath the veneer of artificial fill, Quaternary Young Terrace Deposits were encountered in each of our current subsurface explorations. In general, the terrace deposits consist of orange brown, medium stiff to hard clay and sandy clay grading to medium dense to very dense clayey silty sand and sand with varying proportions of siltstone and sandstone bedrock fragments.

Tertiary Monterey Formation (Map Symbol: Tm): Sandstone bedrock of the Tertiary Monterey Formation was encountered at a depth of 45 feet bgs in Boring LB-6 (Plate 1). Borings LB-3 and LB-5 encountered auger refusal at depths of approximately 21 feet and 27 feet bgs, respectively. Auger refusal likely occurred when the borings encountered the underlying bedrock of the Monterey Formation or basal gravel/cobbles at the wave cut interface. Based on previous explorations and regional geologic mapping, Monterey Formation bedrock is known to underlie the entire school campus at varying depths. Limited exposures of Monterey Formation bedrock were mapped during downhole logging and slope mapping as striking northwest to northeast and dipping at approximately 15 to 24 degrees southerly (Plate 1).

2.3 Infiltration

Percolation testing was performed within boring LP-1 to evaluate the infiltration characteristics of subsurface soils. The percolation test was conducted in general accordance with the County of Los Angeles Department of Public Works (LADPW) *Guidelines for Geotechnical Investigation and Reporting Low Impact Development Stormwater Infiltration* document (LADPW, 2017). Results of the percolation testing are presented in Appendix B, *Percolation Test Results.* The test location is shown on Plate 1.

A boring percolation test is useful for field measurements of the infiltration rate of soils, and is suited for testing when the design depth of the infiltration device is deeper than current existing grades, especially in areas where it is difficult to dig test pits, or where the depths of these test pits would be considerably deep. At the subject site, testing consisted of advancing the boring to the invert depth of typical infiltration devices.

The infiltration rate for our tests was calculated by dividing the rate of discharge by the infiltration surface area, or flow area. The volume of discharge was calculated by adding the total volume of water that dropped within the PVC pipe and within the annulus, and incorporating a porosity reduction factor to account for the porosity of the annulus material. The infiltration surface area was based on the average water height within the slotted pipe section of the test well.

Detailed results of the field testing data and measured infiltration rate for the test well are presented in Appendix B. The test result is summarized below:

Table 1 – Summary of Percolation Test Results

Boring	Zone Tested (feet bgs)	Measured Infiltration Rate (inch per hour)
LP-1	10 to 15	0.01

Based on the results of the percolation test performed and the low permeability clay soil that underlies the site, infiltration is not considered feasible according to County requirements (LADPW, 2017).

Dense sand interpreted in CPT's and sampled in several borings are cemented with iron oxide which decreases pore space between particles. As with other areas on this campus these sand zones have been determined to be limited in area representing channelization into the sediments as sea levels lowered over time. Infiltration into these channels may result in seepage downslope and offsite. The site is predominately underlain by expansive clay (lean and fat), infiltration of stormwater may mound due to shallow bedrock and laterally migrate along clay beds or along bedrock contact activating expansive clay. The school discharges wastewater to several septic systems on the campus. Details of the leach fields are unknown and infiltration of stormwater could impact the leach fields. Infiltration of stormwater is not recommended at this site.

2.4 Expansive Soil

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. Implementation of standard engineering and earthwork construction practices, such as proper foundation design and proper moisture conditioning of earthen fills will reduce the effects associated with expansive soils.

The near surface soils are considered to have a *high to very high expansion potential*. Expansion Index (EI) testing of two representative bulk samples from the upper 5 feet at the site indicate the site soils have an EI of 116 to 134. Upon completion of mass grading of the site, additional expansion testing should be performed to quantify EI values upon completion of grading.

2.5 Soil Corrosivity

Soil Resistivity: In general, soil resistivity, which is a measure of how easily electrical current flows through soils, is the most influential factor for ferrous corrosivity. Based on findings of studies presented in the American Society for Testing and Materials (ASTM) STP 1013 titled “Effects of Soil Characteristics on Corrosion” (February, 1989), an approximate relationship between soil resistivity and soil corrosiveness was developed as shown in Table 2 below.

Table 2 - 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

Sulfate Exposure: Sulfate ions in the soil can lower the soil resistivity and can be highly aggressive to Portland cement concrete by combining chemically with certain constituents of the concrete, principally tricalcium aluminate. This reaction is accompanied by expansion and eventual disruption of the concrete matrix. A potentially high sulfate content could also cause corrosion of reinforcing steel in concrete. Section 1904A of the 2016 California Building Code (CBC) defers to the American Concrete Institute’s (ACI’s) ACI 318-14 for concrete durability requirements. Table 19.3.1.1 of ACI 318-14 lists “*Exposure categories and classes*,” including sulfate exposure as follows:

Table 3 - Sulfate Concentration and Exposure

Soluble Sulfate in Water (parts-per-million)	Water-Soluble Sulfate (SO4) in soil (percentage by weight)	ACI 318-14 Sulfate Class
0-150	0.00 - 0.10	S0 (negligible)
150-1,500	0.10 - 0.20	S1 (moderate*)
1,500-10,000	0.20 - 2.00	S2 (severe)
>10,000	>2.00	S3 (very severe)

*or seawater

Two representative, near surface (0 to 5 feet bgs) bulk soil samples were tested to evaluate corrosion potential. The chemical analysis test results for the onsite soil from our geotechnical exploration are included in Appendix C, *Laboratory Test Results.* The results of chemical analysis are summarized below.

Table 4 - Corrosivity Test Results

Test Parameter	Test Results		General Classification of Hazard
	2020-LB-4 @ 0-5'	2020-LB-6 @0-5'	
Water-Soluble Sulfate-SO_4 in Soil (ppm)	148	235	Negligible to moderate sulfate exposure to buried concrete-S1 Exposure Class
Percent by Weight SO_4	0.148	0.235	
Water-Soluble Chloride in Soil (ppm)	80	240	Non-corrosive to buried concrete (per Caltrans Specifications)
Percent by Weight (Chl^-)	0.080	0.240	
pH	8.14	8.50	Mildly alkaline
Minimum Resistivity (saturated, ohm-cm)	800	945	Very severely corrosive to buried ferrous pipes

Based on geotechnical laboratory testing performed on selected soil samples collected from the site and review of previous laboratory test results, a synopsis of geotechnical properties of the site soils is provided in Table 5 below. Geotechnical laboratory testing results are presented in Appendix C, *Laboratory Test Results.*

Table 5 – Soil Geotechnical Properties Synopsis

Parameters	Soil Properties
In-situ Moisture:	Moist to very moist
In-situ Density:	stiff to hard for cohesive soils and medium dense to very dense for granular soils
Swell/Expansion Potential:	swell/expansion potential is high.
Collapse Potential:	Not susceptible to collapse when wetted
Strength:	Adequate to provide structural support
Corrosivity:	Moderate sulfate attack of concrete but severely corrosive to ferrous metals.

2.6 Groundwater

Groundwater was not encountered during our current investigation to the maximum depth drilled of approximately 46½ feet bgs. However, moist to very moist soils and bedrock were encountered within explored depths. Groundwater was encountered in our 2009 exploration (Leighton, 2009a) within Boring B-3 at a depth of 48½ feet (Elevation +46 feet mean sea level).

Groundwater depth was measured in an existing 4-inch diameter monitoring well (MW-2). The details of well construction are unknown, however the total depth measured was 77.4 feet bgs with a depth to static water level as 58.7 feet bgs corresponding to approximately Elevation (El.) +52 feet mean sea level (msl). The location of the monitoring well installed by others is shown on Plate 1.

Seasonal fluctuations should be expected during periods of intense localized rainfall. Groundwater is not expected to pose a constraint to construction.

3.0 GEOLOGIC-SEISMIC HAZARDS EVALUATION

Geologic and seismic hazards include surface faulting, seismic shaking, liquefaction and lateral spreading, seismically-induced settlement, landslides, flooding, and seiches and tsunamis. The following sections discuss these hazards and their potential impact at the project site.

3.1 Faulting

Based on our review of aerial photos, geologic and topographic maps including fault evaluation reports, no active faults or lineaments are known or mapped as crossing the project site (Hart and Bryant, 2007). The site is not situated on a pressure ridge, active geologic fault or active fault trace. As such the potential for fault rupture at the site is considered low during the life of the school, the student risk factor is therefore also considered low. The site is in close proximity (less than 5 miles) to two significant active faults that include the onshore Malibu Coast Fault and offshore Anacapa Fault. Both faults consist of slanted, north-dipping shear planes accommodating north-over-south crustal shortening in the present-day compressional stress field. Regional faults are shown on Figure 3, *Regional Fault and Historical Seismicity Map*.

Malibu Coast Fault: The closest active zoned fault to the site is the Malibu Coast Fault. The surface trace is located approximately 1.1 miles north of the site at its closest approach. The fault exhibits left-lateral oblique displacement, with a reported vertical slip rate component of about 0.4 millimeters per year (Lajoie et al., 1979) and a horizontal slip rate component of 0.3 millimeters per year (Petersen et al., 1996). The entire 23-mile-long fault zone is considered to be a potential source in the present statewide probabilistic seismic hazard model, and is considered capable of generating a maximum moment magnitude earthquake of 6.7 (Petersen et al., 1996).

Anacapa Fault: The offshore Anacapa Fault is located approximately 5 miles south of the site. Relatively little detailed knowledge exists regarding its average late Quaternary slip rate or total offset. It is apparent, however, from both oceanic sub-bottom reflection profiles (Greene and Kennedy, 1987), and historical seismicity that the fault is properly classified as active. Peterson et al. (1996) assigns a poorly constrained slip rate of 3.0 millimeters per year and a calculated maximum moment magnitude earthquake of 7.3 to this 46-mile-long fault.

The fault classification criteria adopted by the California Geological Survey (CGS) defines Earthquake Fault Zones along active or potentially active faults in accordance with the Alquist-Priolo Earthquake Fault Zoning Act of 1972. Thus, an active fault is one that has ruptured during Holocene time (the last 11,700 years). A fault that has ruptured during the last 1.8 million years (Pleistocene time) but has not been proven by direct evidence to have not moved within Holocene time is considered to be potentially active. A fault that has not moved during both Pleistocene and Holocene (Quaternary) time (that is no movement within the last 1.8 million years) is considered inactive. Review of the State of California Earthquake Fault Zones map for the 7.5 Minute Point Dume Quadrangle (CGS, 1994) indicate the site is not within an Alquist-Priolo Earthquake Fault Zone, consequently the State does not require a fault investigation.

Potentially Active Faults: The City of Malibu has employed a stricter, more conservative standard for fault surface rupture hazards. This standard was derived from the 1990 Seismic Safety Element for Los Angeles County, and was modified after CGS evaluated the Malibu Coast Fault Zone in 1994. As part of the City's requirements, any structures (habitable) within 500 feet of faults, regardless of activity, as mapped within CGS (1994) should be evaluated.

While not currently mapped as active zoned faults by the State of California, The Escondido Thrust Fault and Paradise Cove Fault are potentially active faults that are both mapped as traversing the campus of MMHS. The approximate locations of these features, inferred or interpreted by others as faults and/or depositional contacts between bedrock units, are shown on Plate 1.

Escondido Thrust Fault: As discussed in CGS (1994), the Escondido Thrust Fault is reportedly a poorly defined, low-angle, north-dipping thrust fault extending from Trancas Canyon easterly toward Escondido Beach. In general, the Escondido Thrust fault is mapped as separating the Tertiary Monterey Formation footwall from the older, overriding Trancas Formation and Zuma Volcanics thrust sheet. While there is evidence the eastern portions of the Escondido Fault displaces units no younger than 124,000 years old; western segments of the fault, coincident with previously mapped traces of the Malibu Coast Fault by Dibblee and Ehrensbeck (Diblee, 1993), exhibits displacements of Quaternary terrace deposits. However, the Escondido Thrust Fault shows **no** evidence of Holocene displacement and is thus considered not active per State of California definition. As mapped by Campbell and others (Campbell et. al, 1996), the Escondido Thrust Fault crosses the MMHS Campus approximately 600 feet northeast of the proposed footprint of

Building “C”. Dibblee maps the Escondido Thrust Fault within the MMHS and JCES campus as diverting to the southwest, and crossing the JCES campus approximately 100 feet north of the proposed footprint of Building “C” (Dibblee, 1993).

Paradise Cove Fault: The Paradise Cove Fault is a roughly east-west trending near vertical fault separating a tightly folded dolomitic unit from Monterey Formation Shale. The fault is not exposed at the surface but has been inferred based on stratigraphic contrast. As mapped by Campbell and others (Campbell, et al., 1996) and Dibblee (Dibblee, 1993), portions of the Paradise Canyon Fault west of Zuma Canyon appear to have been overridden by the Escondido Thrust Fault; implying the Escondido Thrust Fault is younger than the Paradise Cove Fault. As noted in CGS (1994), the Paradise Canyon Fault shows no evidence of Holocene displacement and is considered not active per State of California definition.

3.2 Historical Seismicity

An evaluation of historical seismicity from significant past earthquakes related to the site was performed. Peak ground accelerations (PGA) at the site resulting from significant past earthquakes between 1800 to 2020, with magnitudes M4.0 or greater, were estimated using the EQSEARCH computer program (Blake, 2000). This historical seismicity search was performed for a 100-kilometer (62-mile) radius from the project site, and is included in Appendix D, *Seismicity Data*. The largest earthquake magnitude found in the search was the M7.7 earthquake, known as the Arvin-Tehachapi quake that occurred on July 21, 1952 approximately 68.2 miles (109.7 kilometers) from the site producing an estimated site acceleration of approximately 0.052g. The largest estimated PGA found in the search was 0.330g from a M7.0 earthquake approximately 4.3 miles (6.9 kilometers) from the site occurring on September 24, 1827.

Review of additional data publicly available from the Center for Engineering Strong Motion Data (CESMD) website (http://strongmotioncenter.org/) was reviewed for stations in the vicinity of the project site. The data reviewed indicates that a site (CGS-CSMIP Station 24396) approximately 1.8 miles to the southeast of the project site experienced a peak ground acceleration of 0.130g from the M6.4 Northridge Earthquake that occurred on January 17, 1994.

3.3 Liquefaction and Lateral Spreading

Liquefaction is the loss of soil strength due to a buildup of excess pore-water pressure during strong and long-duration ground shaking. Liquefaction is associated primarily with loose (low density), saturated, relatively uniform fine- to medium-grained, clean cohesionless soils. As shaking action of an earthquake progresses, soil granules are rearranged and the soil densifies within a short period. This rapid densification of soil results in a buildup of pore-water pressure. When the pore-water pressure approaches the total overburden pressure, soil shear strength reduces abruptly and temporarily behaves similar to a fluid. For liquefaction to occur there must be:

(1) loose, clean granular soils,
(2) shallow groundwater, **and**
(3) strong, long-duration ground shaking

According to the State of California Seismic Hazard Zones Map (CGS, 2002), the site is not located within an area that has been identified as being potentially susceptible to liquefaction as shown on Figure 4, *Seismic Hazards Map*.

Moreover, due to the near-surface presence of stiff/hard, clay impacted terrace deposits and relatively shallow bedrock, the potential for liquefaction at this site is low.

Since the potential for liquefaction is considered low, the potential for lateral spreading to occur at the site is also considered low.

3.4 Seismically-Induced Settlement

Seismically-induced settlement consists of dry dynamic settlement (above groundwater) and liquefaction-induced settlement (below groundwater). These settlements occur primarily within loose to moderately dense sandy soil due to reduction in volume during and shortly after an earthquake event. Due to the predominantly stiff nature of underlying soils and shallow bedrock, seismically induced total and differential settlement under the building is expected to be negligible (Appendix E).

3.5 Landslides

The proposed project site is not located in an area mapped as potentially susceptible to seismically-induced landslides as shown on Figure 4, *Seismic Hazards Map*. No landslides are mapped or known to exist at the project site or vicinity. Previous grading and construction at the site has created stepped building pads and parking lots. The potential for seismically induced landslides at the site is considered low.

3.6 Flooding

The site is not located within a designated 100-500 year flood zone as defined by the Federal Emergency Management Agency (FEMA, 2008) as shown on Figure 5, *Flood Hazard Zone Map*, and therefore the potential for flooding is considered low.

Earthquake-induced flooding can be caused by failure of dams or other water-retaining structures as a result of an earthquake. Due to the absence of such structures near the site, the potential for earthquake-induced flooding at the site is considered low, see Figure 6, *Dam Inundation Map*.

3.7 Debris Flows

Leighton performed geologic reconnaissance to visually evaluate the areas impacted by debris flow and erosion that occurred after the Woolsey Fire during the November and December 2018 rain events at MMHS. During the rain event, a 48-inch-diameter storm drain at the cul-de-sac on Clover Heights Avenue was plugged with debris and debris flows overtopped the inlet structure, spilling onto the campus. See *Plate 1* for flow path.

It is generally accepted that debris flows most commonly occur on slopes with gradients ranging from 26 to 45 degrees (USGS,1975) The potential for debris flow depends on soil type, water content, and degree of vegetation in the source zone. Debris flows occurring in this area were the result of the Woolsey Fire where vegetation and structures were burned and stripped from the surrounding slopes. The loss of surficial support provided by vegetation and the accumulation of moisture from prolonged rain events in the loose and disturbed soil resulted in the debris flow.

Slopes with gradients between 11 and 26 degrees are recognized as transport zones across which debris flows generated from the upslope source areas are transported and where flow velocity remains relatively constant. In these areas, the loose soil and ash was stripped along the path of flow as observed in the narrow drainage incisions, thereby increasing flow volume. Areas where the slope gradient is 11 degrees or flatter, i.e. the MMHS campus, are depositional areas where mud is deposited as the flow velocity decelerates.

The source zone (north of the campus) emanates from the northwesterly trending, steep sided canyons north of Cuthbert Road which lack vegetation and mature trees within the canyon. The transport zone is generally defined as the mouth of the canyon from approximately Cuthbert Road where debris flows blocked the road, down gradient to south of the intersection of Harvester Road and Clover Heights Avenue, an area of approximately 2,400 linear feet in length. The depositional zone is identified as the MMHS campus ballfields, immediately south of the northern chain link fence boundary down gradient to the debris basin in the southwest region of the parking lot, or a linear distance of approximately 1,100 feet

Although there are relatively thick deposits of colluvium and ash on slopes above and surrounding the campus, the gradient of the flow pathway (depositional zone) as observed (see Plate 1), not considering minor slopes, is relatively flat, approximately 5 degrees.

Potential debris and mud flows could emanate from the main and tributary canyon upslope of the campus located approximately 2,400 feet north of the campus and transported down gradient. Although we don't have specific field data in the source area, considering the two rainfall events that occurred in November and early December 2018 which resulted in deposition from upslope debris flows, we do not anticipate a thick overburden of soil to remain on slopes in this area. Since the December 2018 debris flow the slopes above the campus have revegetated with light grasses and homes are being rebuilt and drainage pathways corrected.

Based on the relatively gentle slope inclination (±5 degrees) and long depositional zone (1,100 feet), which has a defined flow path, it is our opinion the occurrence of a debris flow emanating from the source area to cause significant structural damage to the MMHS campus is low.

3.8 Slope Softening and Slope Creep

As moisture infiltrates into the subsurface soil (such as from rain or irrigation), the wet density of the soil tends to increase, while the effective stress of the soil tends to decrease. In clay soils, slope softening can then occur, where the slope deforms in order to mobilize the shear stress needed to maintain stability. Slope softening can continue over time as the soils become more and more saturated. Slope creep is a related phenomenon whereby the soils on and adjacent to fill and natural soil slopes loosen with time due to weathering processes, incrementally moving downslope due to gravitational forces. A contributing factor at this site is the presence of expansive soil, which expands and shrinks during wetting and drying cycles. The expanding and shrinking of the soil could cause a ratcheting effect, where soil and relatively light surface improvements, such as concrete slabs, tend to move laterally toward the unconfined slope face during expansion and downward during periods of shrinkage. This would result in a gradual downward and lateral movement of the surficial soils (and surficial improvements). Principal factors that affect the severity of slope softening or slope creep include amount water infiltration into the subsurface soil, and the distance of improvements from the top of slope.

Building C, as currently designed, has flatwork (sidewalks and perimeter access road) planned near the top of the canyon along the west northwest side of the project (Plate 1). As such, measures will need to be implemented to limit the potential effects of slope softening/slope creep on the planned flatwork improvements. This is addressed in the recommendations Section 5.0 below.

3.9 Seiches and Tsunamis

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Tsunamis are predominately ocean waves generated by undersea large magnitude fault displacement or major ground movement. Based on review of the Tsunami Inundation Map for Emergency Planning, State of California, County of Los Angeles, Point Dume Quadrangle (CalEMA, 2009), the site is not located within a Tsunami Inundation area. However, the site is located within the tsunami runup area mapped by the City of Malibu General Plan (City of Malibu, 1995). The 100-year return period wave has a 5.1-foot height and the 500-year return period wave has an 8.7-foot height. The site is above Elevation +90 feet msl.

4.0 FINDINGS AND CONCLUSIONS

Presented below is a summary of findings and conclusions based on the results of our evaluation of the project site:

- This site is **not** located within a currently designated Alquist-Priolo Special Studies Zone for surface fault rupture. However, as is the case for most of Southern California, strong ground shaking has and will occur at this site. This site is also **not** within a currently designated liquefaction hazard zone. Due to primarily clay and dense sands, damaging liquefaction is unlikely to occur at this site. The site is not located in any State of California geologic or seismic hazard zone that could preclude the development of the proposed project.

- The City of Malibu has employed a stricter, more conservative standard for evaluation of fault surface rupture hazards. This standard was derived from the 1990 Seismic Safety Element for Los Angeles County, and was modified after the California Geological Survey evaluated the Malibu Coast Fault Zone in 1994. As part of the City's requirements, any structures (habitable) within 500 feet of faults, regardless of activity, should be evaluated.

- While not currently mapped as active zoned faults by the State of California, the Escondido Thrust Fault and Paradise Cove Fault are potentially active faults that are both mapped as traversing the campuses of MMHS and JCES. The approximate locations of these features, inferred by others as faults and/or depositional contacts between bedrock units, are shown on Plate 1.

- The site is underlain by undocumented artificial fill ranging in thickness from 1 to 7 feet overlying terrace deposits generally consisting of dense to very dense sands and stiff to hard lean to fat clay.

- Groundwater was not encountered during the current exploration. Groundwater is not expected to pose a constraint to construction. Current depth to static water level measured in a monitoring well is about 58 feet bgs corresponding to approximately El. +52 feet msl. The location of the monitoring well is shown on Plate 1.

- The potential for liquefaction and liquefaction-induced ground failure to occur at the site is considered low.

- The potential seismically-induced settlement at the site is estimated to be less than ¼ inch.

- Based on our observations and testing, the onsite soils that will be in contact with the planned structures are expected to have a high expansion potential. Additional testing is recommended during design stage or at completion of grading. For purposes of design, we recommend using an Expansion Index (EI) greater than 130.

- Concrete in contact with the onsite soil is expected to have negligible to moderate exposure to water-soluble sulfates and low exposure to chloride in the soil. The onsite soil, however, is considered severely corrosive to ferrous metal.

- The subsurface materials are anticipated to be readily excavated using conventional earthmoving equipment in good working condition.

- The proposed improvements may be supported on conventional shallow footing foundation systems established on engineered fill.

Based upon the results of our geotechnical evaluation of the site, the proposed improvements are considered feasible from a geotechnical standpoint.

5.0 RECOMMENDATIONS

The following recommendations have been developed based on the exhibited engineering properties of the onsite soils and their anticipated behavior both during and after construction. Recommendations are specifically provided for grading, design of foundations, seismic design considerations, floor slabs, and paving.

Proposed new replacement Building C may be supported on spread-type shallow foundation systems such as footings or post-tensioned concrete slabs with thickened edges established on engineered fill soils.

We recommend that all earthwork for the subject project be performed in accordance with the following recommendations and applicable governing agency grading requirements. Leighton should review the grading plan, foundation plans, and specifications when they are available to verify that the recommendations presented in this report have been properly interpreted and incorporated. Final loading and bearing pressure diagrams should be provided for our review once prepared to confirm recommendations and settlement estimates remain valid for the project as currently proposed.

5.1 Earthwork

Earthwork guide specifications are presented in Appendix F, *Earthwork and Grading Guide Specifications*. Earthwork for this new Building C constructed at grade is expected to include overexcavation and recompaction, backfill and buried utility rerouting. Project earthwork is expected to include complete demolition/removal of existing improvements in new foundation areas and complete overexcavation and recompaction of any remaining undocumented fill soils below new improvement footprints as described in the following subsections:

5.1.1 Site Preparation and Overexcavation

After the site is cleared, the soils should be carefully observed for the removal of all unsuitable deposits such as existing fill and highly expansive organic black fat clay. We recommend that after removal of pavements and hardscape, and complete demolition of existing structures within the proposed improvement footprint, all undocumented fill and unsuitable soils should be excavated from these proposed improvement footprints to expose native subgrade.

Overexcavation of Undocumented Fill: Undocumented fill was encountered as deep as 7 feet bgs in our borings. Deeper fill may be encountered between boring locations.

This overexcavation bottom should extend horizontally either the thickness of fill below spread footings or at least 5 feet horizontally beyond the outside edges of proposed footings, whichever is deeper. Any underground obstructions encountered should be removed. Those lines should be removed or rerouted where interfering with proposed construction.

Overexcavation of Bedrock: To mitigate the potential for differential settlement of the proposed new building caused by bearing stiffness variance between bedrock and fill or natural soil, we recommend that bedrock, although not anticipated, be overexcavated below footing elevations to establish at least 5 feet of engineered fill below footing bottoms. To facilitate earthwork logistics, we recommend that the entire footprint of the proposed new buildings and 5 feet beyond be overexcavated and replaced as engineered fill, where feasible.

***Overexcavation in the Creep Zone*:** Leighton understands concrete flatwork, perimeter access road and fences are planned atop and near the drainage channel on the western side of the property, within the "creep zone." In order to construct these improvements near the top of the slope within the anticipated creep zone and minimize or eliminate the effects of creep upon the wall and flatwork, recommendations provided in the following paragraph should be considered.

A minimum overexcavation of 5 feet and construction of a properly benched stability fill is recommended. Deeper overexcavation may be necessary depending on the depth of lateral cracks, the depth of which is best observed and evaluated during grading. A relatively inexpensive option to address creep related problems in top-of-slope walls and/or fences is to design the structure so that tilting or cracking will be less visually obvious, or such that it may be economically replaced or repaired. If, however, a better degree of creep mitigation is desired, the walls or fences may be provided with the deepened foundations or caissons and grade beams. In addition, the inclusion of frequent crack control joints (10-foot intervals or closer) within walls and flatwork should be considered.

Surface Drainage: Water should not be allowed to pond or accumulate anywhere except in detention basins. Drainage should be designed to collect and direct surface water away from structures to approved drainage facilities. Hardscape drains should be installed and drain to storm water disposal systems. Drainage patterns approved at the time of fine grading should be maintained throughout the life of proposed structures. Irrigation should not be allowed for at least 10-feet-horizontally around structures supported on shallow spread footings and/or with slabs-on-grade.

5.1.2 Subgrade Stabilization and Fill Placement

The subgrade is anticipated to consist of clayey soils, which are moisture sensitive and will require additional precautions especially during the rainy season. When grading is interrupted by heavy rains, fill operations should not be resumed until the moisture content and the dry density of the placed fill are satisfactory.

Clayey soils should be overexcavated as necessary to permit the placing of at least 2 feet of relatively non-expansive soils (Expansion Index, EI, less than 20) beneath concrete slabs and walks.

Moisture Content: After excavating as recommended, the moisture content of the clayey soils should be determined, and the clayey soils slowly and uniformly conditioned (moistened or dried) as necessary to bring the soils to a uniformly moist condition. Any surface soils that may have dried out too much should be slowly and uniformly moistened as necessary to bring these soils to uniformly moist condition. The moisture content of the clayey soils should be brought to approximately 3 to 5 percent over optimum moisture content to a depth of 12 inches below grade. The moisture content of the subgrade should be checked and approved prior to placing the required fill. When grading is interrupted by heavy rains, fill operations should not be resumed until the moisture content and the dry density of the placed fill are satisfactory.

Subgrade Evaluation: After moisture conditioning as required, the exposed soils should be rolled with heavy compaction equipment. At least the upper 12 inches of the exposed soils should be compacted to at least 90 percent relative compaction based on ASTM Test Method D 1557. If the subgrade soils are wet and soft, it may be necessary to place a layer of crushed rock or a geomembrane, or both, over the exposed soils to provide a base for the compaction of the required fill. In this case, the soft natural soils should be carefully excavated prior to placing the crushed rock layer.

Fill Placement and Compaction: The onsite soils, less any deleterious material or organic matter (fat organic black clay), can be used in required fills. Cobbles larger than 6 inches in largest diameter should not be used in the fill. Any required import material should consist of relatively non-expansive soils with an Expansion Index (EI) less than 30. The imported materials should contain sufficient fines (binder material) so as to be relatively impermeable and result in a stable subgrade when compacted. All proposed import materials should be approved by the geotechnical engineer of record prior to being placed at the site.

All fill soil should be placed in thin, loose lifts, moisture-conditioned, as necessary, to within 2 percent of optimum moisture content for granular soils and 3 to 5 % for clayey soils, and compacted to a minimum 90% relative compaction as determined by ASTM D 1557 standard test method (modified Proctor compaction curve) within building footprints. Aggregate base for pavement sections should be compacted to a minimum of 95% relative compaction. At least the upper 12 inches of the exposed soils in roadways and access drives, parking lots and (concrete –paver) flatwork areas, should be compacted to at least 95 percent relative compaction based on ASTM Test Method D 1557.

Wet and Pumping Soil: Subgrade stability issues may be encountered during subgrade preparation and site grading due to the moisture sensitivity of the clayey soils that are expected to comprise the subgrade and the relatively high in-situ moisture contents of the material based on recovered soil samples. Stability problems may be mitigated by either undercuting unstable soils or performing chemical or mechanical modification of the subgrade to allow grading activities to proceed. Chemical modification consists of the addition of either lime or Portland cement to a properly processed subgrade followed by recompaction. Chemical modification will require the geotechnical engineer's approval prior to implementing a modification program. Mechanical stabilization consists of the placement of a coarse (2- to 4-inch nominal particle diameter) crushed aggregate to serve as working mat. Depending upon the degree of instability, a geogrid may also be required in conjunction with the coarse aggregate.

5.1.3 Excavation Adjacent to Constructed Footings

It is essential that excavation not undermine foundations of structures that will remain in place along the boundaries of the project including leach fields and septic systems. As-built details of any structure(s) to remain should be provided to Leighton and the structural engineer prior to incorporation into the new design.

Excavations near existing footings that will remain, if any, should be performed with care so that the existing footings are not undermined and the subgrade supporting the footings is not disturbed. Excavation adjacent to existing foundations that extend below bearing elevation may require slot-cutting techniques or shoring to perform the excavation and protect the foundations.

5.1.4 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 State of California Occupational Safety and Health Administration (CalOSHA) requirements.

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

Temporary excavations should be treated in accordance with CalOSHA excavation regulations. The sides of excavations should be shored or sloped accordingly. CalOSHA allows the sides of unbraced excavations, up to a maximum height of 20 feet, to be cut to a ¾:1 (horizontal:vertical) slope for Type A soils, 1:1 for Type B soils, and 1½:1 for Type C soils.

The onsite soils are predominately clay within the proposed structural depths and generally conform to CalOSHA Type C soils. CalOSHA regulations are applicable in areas with no restriction of surrounding ground deformations. Shoring should be designed for areas with deformation restrictions. 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. Heavy construction loads, such as those resulting from stockpiles and heavy machinery, should be kept a minimum distance equivalent to the excavation height or 5 feet, whichever is greater, from the excavation unless the excavation is shored and these surcharges are considered in the design of the shoring system.

5.1.5 Pipe Bedding and Trench Backfill

Pipeline trenches should be backfilled with compacted fill in accordance with this report, and applicable *Standard Specifications for Public Works Construction* (Greenbook), current edition standards. Backfill in and above the pipe zone should be as follows:

Pipe Zone: 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, conforming to Section 201-6 of the 2018 Edition of the *Standard Specifications for Public Works Construction* (Greenbook). Due to expansive clays, sand bedding for conduits should not be allowed on this site below the building footprint to a lateral distance of 3 feet beyond the building footprint. CLSM bedding should be placed to 1 foot (0.3 m) over the top of the conduit and vibrated. CLSM should **not** be jetted.

Over Pipe Zone: Above the pipe zone, trenches can be backfilled with excavated on-site soils free of debris, organic and oversized material larger than (>) 3 inches in largest dimension. As an option, the whole trench can be backfilled with one-sack CLSM same as presented above for the pipe bedding zone. Oversized rock (cobbles and/or boulders) should either be removed from any backfill, or pulverized for use in backfill only above the pipe zone. Gravel larger than ¾ inch in diameter should be mixed with at least 80 percent soil by weight passing the No. 4 sieve. Native soil backfill over the pipe-bedding zone should be placed in thin lifts, moisture conditioned, as necessary, and mechanically compacted using a minimum standard of 90% relative compaction (relative to the laboratory modified Proctor maximum **dry** density), relative to the ASTM D 1557 laboratory maximum dry density within building footprints, or 85% under hardscape areas. Backfill above the pipe zone should **not** be jetted. In any case, backfill above the pipe zone (bedding) should be observed and tested by Leighton.

5.2 Corrosion Protection Measures

Water-soluble sulfates in soil can react adversely with concrete. As referenced in the 2019 California Building Code (CBC), Section 1904A, concrete subject to exposure to sulfates shall comply with requirements set forth in ACI 318. Based on laboratory testing results of the onsite soils from recent and prior investigations, concrete structures in contact with the onsite soil will likely have **"negligible" to "Moderate"** exposure to water-soluble sulfates in the soil. Therefore, common Type II Portland cement may be used for concrete construction in contact with site soils. Subgrade soil should be tested for water-soluble sulfate content prior to final design of the concrete structures once grading is complete. Import fill soil should be geotechnically tested for corrosivity and sulfate attack before import to the site. *Further testing of import soils should include analytical testing for chemicals of concern prior to import and acceptance.*

Based on corrosivity test results of the onsite soils from recent and prior investigations, the onsite soil is considered severely corrosive to ferrous metals. Therefore, based on these results, ferrous pipe 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. Ferrous pipe can also be protected by polyethylene bags, tap or coatings, di-electric fittings or other means to separate the pipe from on-site soils.

5.3 Foundations

Based on our preliminary investigation and our experience in the region, the proposed new buildings may be supported on spread-type shallow foundation systems such as footings or post-tensioned concrete slabs with thickened edges established on engineered fill or undisturbed natural soils.

5.3.1 Shallow Spread Footings

Footings for proposed structures should have a minimum embedment of 3 feet and have a minimum width of 2 feet. Footings for proposed temporary structures may be supported directly on grade.

Bearing Value: Footings established on engineered fill or undisturbed natural soils may be designed to impose an allowable bearing pressure of 4,000 pounds per square foot (psf).

The ultimate bearing capacity can be taken as 12,000 psf. This value does not incorporate a factor of safety and may only be used for an ultimate bearing capacity check with appropriate factored loads.

The recommended bearing value is a net value, and the weight of concrete in the footings can be taken as 50 pounds per cubic foot (pcf); the weight of soil backfill can be neglected when determining the downward loads.

Settlement: The above recommended allowable bearing capacities are generally based on a total post-construction settlement of about ½ inch for dead plus live column loads not exceeding 150 kips.

Differential settlement due to static loading is generally estimated at ¼ inch over a horizontal distance of 40 feet. Once developed by the structural engineer, we should review total dead and sustained live loads for each column including plan location and span distance, to evaluate if differential settlements between dissimilarly loaded columns will be tolerable. Excessive differential settlement can be mitigated with the use of reduced bearing pressures, deeper footing embedment, possibly changing overexcavation schemes and using imported base material under spread footings, or possibly other methods.

Lateral Resistance: Soil resistance available to withstand lateral loads on a shallow foundation is a function of the frictional resistance along the base of the footing and the passive resistance that may develop as the face of the structure tends to move into the soil. The frictional resistance between the base of the foundation and the subgrade soil may be computed using a coefficient of friction of 0.3. The passive resistance may be computed using an equivalent fluid pressure of 250 pounds-per-cubic-foot (pcf), assuming there is constant contact between the footing and undisturbed soil. The passive resistance can be increased by one-third when considering short-duration wind or seismic loads. The friction resistance and the passive resistance of the soils can be combined without reduction in determining the total lateral resistance.

Uplift Resistance: To evaluate uplift resistance provided by the dead weight of soils above the footing, the frustum of soil above the footing may be estimated by a 30 degree outward projection from vertical. A unit weight of 120 pcf may be used for the soil volume within the frustum.

To evaluate uplift resistance provided by the shear resistance soils above the footing, an allowable shear value of 75 psf may be used along vertical shear planes from the bottom of the footing to the ground surface along the perimeter the footings. A factor of safety of 3 was used to develop the allowable shear value

5.3.2 <u>Modulus of Subgrade Reaction</u>

For foundations established in undisturbed natural soil or engineered fill, an initial unit modulus of subgrade reaction (k_1) value of 100 pounds per cubic inch (pci) may be used.

The k_1 value presented herein, which corresponds to a 1-foot-square footing, should be reduced as shown below to incorporate foundation size effects:

$$k = k_1 \left(\frac{B+1}{2B}\right)^2$$

where B is the square footing width.

Leighton should review the resulting foundation deformation contours developed by the structural engineer for conformance with geotechnical settlement estimates.

5.3.3 <u>Flagpole-Type Foundations</u>

Canopy structures, light poles, and fencing may be supported on flagpole-type foundations. Flagpole-type foundations may be designed to impose an allowable vertical bearing pressure of 3,000 psf and an allowable lateral bearing pressure of 600 psf per foot below grade. The allowable vertical and lateral bearing pressures may be increased by one-third for short-duration loading such as wind or seismic loading. The recommended bearing value is a net value, and the weight of concrete in the flagpole footings can be taken as 50 pounds per cubic foot

5.4 <u>Seismic Design Parameters</u>

To accommodate effects of ground shaking produced by regional seismic events, seismic design can be performed by the project structural engineer in accordance with the 2019 CBC. The table below, *2019 CBC Mapped Seismic Parameters*, lists seismic design parameters based on the 2019 CBC, Section 1613A.3 (ASCE 7-16) methodology.

Table 6 - 2019 CBC Seismic Design Parameters (Mapped Values)

Categorization Coefficient	Design Value
Site Latitude	34.0251N
Site Longitude	-118.8286W
Site Class	D
Mapped spectral response acceleration parameter at short period, S_S	1.607
Mapped spectral response acceleration parameter at a period of 1 sec, S_1	0.557
Short Period (0.2 sec) Site Coefficient, F_a	1
Long Period (1.0 sec) Site Coefficient, F_v	null[5]
Adjusted spectral response acceleration parameter at short period, S_{MS}	1.607
Adjusted spectral response acceleration parameter at a period of 1 sec, S_{M1}	null[5]
Design spectral response acceleration parameter at short period, S_{DS}	1.071
Design spectral response acceleration parameter at a period of 1 sec, S_{D1}	null[5]
Site Modified Peak Ground Acceleration, PGA_M	0.764

1. All were derived from the SEA web page: https://seismicmaps.org/
2. All coefficients in units of g (spectral acceleration)
3. See Appendix C for details of the seismic evaluation.
4. *Requires C_s calculation, see below.
5. See Section 11.4.8 of ASCE 7-16

Based on the 2019 CBC Table 1613.2.3(2), the long period site coefficient should be determined in accordance with Section 11.4.8 of ASCE 7-16 since the mapped spectral response acceleration at 1 second is greater than 0.2g for Site Class D. In accordance with Section 11.4.8 of ASCE 7-16, a site-specific seismic analysis is required; however, the values provided herein 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 (Cs). The project structural engineer should review the seismic parameters.

The 2019 CBC site-specific seismic design parameters are summarized below. Details, including the site-specific response spectra are presented in Appendix D, *Seismicity Data*.

Table 7 - Site-Specific 2019 CBC Seismic Design Parameters

Categorization/Coefficients	Design Value
Adjusted Spectral Response Acceleration at 0.2s Period, S_{MS}	1.836g
Adjusted Spectral Response Acceleration at 1s Period, S_{M1}	1.140g
Design Spectral Response Acceleration at 0.2s Period, S_{DS}	1.224g
Design Spectral Response Acceleration at 1s Period, S_{D1}	0.760g

5.5 Slabs-on-Grade

More stringent requirements may be required by the structural engineer and/or architect; however, slabs-on-grade should have the following minimum recommended components:

- **Subgrade:** The near-surface soils are expansive and will shrink and swell with changes in the moisture content. Therefore, floor slabs-on-grade and adjacent concrete flatwork should be underlain by at least 2 feet of relatively non-expansive fill (EI < 20). Any concrete slabs-on-grade constructed directly on clayey subgrade should be designed by the structural engineer in accordance with 2019 CBC requirements for soils with a high expansion potential.

  Slab-on-grade subgrade soil should be moisture conditioned to or within 3% to 5% over optimum moisture content, to a minimum depth of 18 inches within building footprints, and compacted to 90% of the modified proctor (ASTM D 1557) laboratory maximum density prior to placing either a moisture barrier, steel and/or concrete.

- **Moisture Barrier:** A moisture barrier consisting of at least 15-mil-thick Stego-wrap vapor barriers (see: http://www.stegoindustries.com/products/stego_wrap_vapor_barrier.php), or equivalent, should then be placed below slabs where moisture-sensitive floor coverings or equipment will be placed.

- **Reinforced Concrete**: A conventionally reinforced concrete slab-on-grade with a thickness of at least 5 inches should within the building footprint and 6-inches for exterior SOG be placed in pedestrian areas without heavy loads. Reinforcing steel should be designed by the structural engineer, but as a minimum should be No. 3 rebar placed at 18 inches on-center, each direction (perpendicularly), mid-depth in the slab. A modulus of subgrade reaction (k) as a linear spring constant, of 75 pounds-per-square-inch per inch deflection (pci) can be used for design of heavily loaded slabs-on-grade, assuming a linear response up to deflections on the order of ¾ inch.

Minor cracking of concrete after curing due to expansion, drying and shrinkage is normal and will occur. 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. The use of low-slump concrete or low water/cement ratios can reduce the potential for shrinkage cracking.

5.6 Lateral Earth Pressures

Recommended lateral earth pressures are provided as equivalent fluid unit weights, in psf/ft. or pcf. These values do not contain an appreciable factor of safety, so the structural engineer should apply the applicable factors of safety and/or load factors during design.

On-site soils are likely **not** suitable to be used as retaining wall backfill due to its high expansion potential. However, site soils can be variable in composition and expansive characteristics. Should site soil be desired for reuse behind retaining walls the material should be tested to ensure Expansion Index is less than 20 (EI < 20). Recommended lateral earth pressures for retaining walls backfilled with sandy soils with drained conditions as shown on Figure 6, *Retaining Wall Backfill* are as follows:

Table 8 - Retaining Wall Design Earth Pressures

Retaining Wall Condition (Level Backfill)	Equivalent Fluid Pressure (pounds-per-cubic-foot)*
Active (cantilever)	40
At-Rest (braced)	60
Passive Resistance	250
Seismic Increment (add to active pressure)	25
Coefficient of Friction	0.3

*Only for level and drained properly compacted backfill

Walls that are free to rotate or deflect may be designed using active earth pressure. For walls that are fixed against rotation, the at-rest pressure should be used. For the seismic condition, the pressure should be added to the active earth pressure.

Retaining Wall Surcharges: In addition to the above lateral forces due to retained earth, surcharge due to above grade loads on the wall backfill, such as existing building foundations, should be considered in design of retaining walls.

Vertical surcharge loads behind a retaining wall on or in backfill within a 1:1 (horizontal:vertical) plane projection up and out from the retaining wall toe, should be considered as lateral and vertical surcharge. Unrestrained (cantilever) retaining walls should be designed to resist one-third of these surcharge loads applied as a uniform horizontal pressure on the wall. Braced walls should also be designed to resist an additional uniform horizontal-pressure equivalent to one-half of uniform vertical surcharge loads. Consideration should be given to underpinning existing structures to remain in this zone, to reduce surcharge loads on the wall and to reduce the potential for inducing damaging settlement within these existing buildings, due to soil movement within the wall influence zone.

In areas where autos and pickup trucks will drive we suggest assuming a uniform vertical surcharge of 300 psf, which would result in active and at-rest horizontal surcharges of 100 psf and 150 psf, respectively. This should be doubled in areas of heavy construction traffic (such as concrete trucks, heavy equipment delivery-trucks, etc.). If crane outrigger loads or other point load sources are applied as wall surcharge, this will require additional analyses based on load source and location relative to the wall.

Sliding and Overturning: Total depth of retained earth for design of walls and for uplift resistance, should be measured as the vertical height of the stem below the ground surface at the wall face for stem design, or measured at the heel of the footing for overturning and sliding. A soil unit weight of 120 pcf may be assumed for calculating the actual weight of the soil over the wall footing, if drained, or 60 pcf if submerged, for properly compacted backfill.

Drainage: Adequate drainage may be provided by a subdrain system positioned behind the walls. Typically, this system consists of a 4-inch minimum diameter perforated pipe placed near the base of the wall (perforations placed downward). The pipe should be bedded and backfilled with pervious backfill material described in Section 300-3.5.2 of the Standard Specifications for Public Works Construction

(Green Book), 2018 Edition. This pervious backfill should extend as shown on Figure 6 to within 2 feet of the outside finished grade. This pervious backfill and pipe should be wrapped in filter fabric, such as Mirafi 140N or equivalent, placed as described in Section 300-8.1 of the Standard Specifications for Public Works Construction (Green Book), 2018 Edition. The subdrain outlet should be connected to a free-draining outlet or sump.

Miradrain, Enkadrain, or similar drainage geocomposites may be used for wall drainage as an alternative to the Class 2 permeable material or drain rock backfill, particularly where horizontal space is limited adjacent to shoring (where walls are cast against shoring). These drainage panels should be connected to the perforated drainpipe at the base of the wall.

5.7 Pavement Design

To provide support for paving, the subgrade soils should be prepared as recommended in Section 5.1, Grading. Compaction of the subgrade, including trench backfills, to at least 90 to 95 percent as recommended relative compaction based on ASTM Test Method D 1557 and achieving a firm, hard and unyielding surface will be important for paving support. The upper 12 inches of vehicular pavement subgrade should be compacted to 95% relative compaction. The preparation of the paving area subgrade should be performed immediately prior to placement of the base course. Proper drainage of the paved areas should be provided since this will reduce moisture infiltration into the subgrade and increase the life of the paving.

Base Course: The base course for both asphalt concrete and Portland Cement Concrete paving should meet the specifications for Class 2 Aggregate Base as defined in Section 26 of the latest edition of the State of California, Department of Transportation, and Standard Specifications. Alternatively, the base course could meet the specifications for untreated base as defined in Section 200-2 of the latest edition of *Standard Specifications for Public Works Construction* (Greenbook). Crushed Miscellaneous Base (CMB) may be used for the base course provided the geotechnical consultant evaluates and tests it before delivery to the site.

Asphalt Concrete: The required asphalt paving and base thicknesses will depend on the expected wheel loads and volume of traffic (Traffic Index or TI). Assuming that the paving subgrade will consist of the onsite soils with an R-value of 5 (Appendix C) compacted to at least 90 percent relative compaction based on

ASTM Test Method D 1557 below 12 inches and 95% relative compaction in the upper 12 inches, the minimum recommended paving thicknesses are presented in the following table:

Area	Traffic Index	Asphalt Concrete (inches)	Base Course (inches)
Light Truck	5	3	10
Heavy Truck	6	4	12
Main Drives	7	4	16

The asphalt paving sections were determined using the Caltrans design method. We can determine the recommended paving and base course thicknesses for other Traffic Indices if required. Careful inspection is recommended to verify that the recommended thicknesses or greater are achieved, and that proper construction procedures are followed.

Portland Cement Concrete Paving: PCC paving and walks supported on clayey onsite soils should be underlain by at least 2 feet of engineered fill consisting of relatively non-expansive (EI < 20) soils.

PCC paving sections were determined in accordance with procedures developed by the Portland Cement Association. Concrete paving sections for a range of Traffic Indices are presented in the table below. We have assumed that the PCC will have a compressive strength $\left(f_c^{'}\right)$ of at least 3,000 pounds per square inch (psi).

Area	Traffic Index	Portland Cement Concrete (inches)	Base Course (inches)
Light Truck	5	6	4
Heavy Truck	6	6½	4
Main Drives	7	7	4

The paving should be provided with expansion joints at regular intervals no more than 15 feet in each direction. Load transfer devices, such as dowels or keys, are recommended at joints in the paving to reduce possible offsets. The paving sections in the above table have been developed based on the strength of unreinforced concrete. Steel reinforcing may be added to the paving to reduce cracking and to prolong the life of the paving.

Paver Construction: A Traffic Index of 5 or less was assumed in our analysis with additional consideration for weekly trash truck traffic and occasional fire engine traffic. The resulting interlocking paver section is presented below. The assumed traffic index should be reviewed by the project civil engineer to determine if the assumed value is suitable for the proposed project.

Interlocking Paver Section

Assumed Traffic Index	Interlocking Paver Minimum Thickness*	Bedding Sand	Class II Aggregate Base
5 or less	3 inches	1 to 1½ inches	16 inches

* Assumed paver thickness

Standard interlocking pavers suitable for traffic loading typically are 3 1/8 inches (80 mm) thick and should be underlain by approximately 1 to 1.5 inch of bedding sand conforming to ASTM C 33 gradation requirements per the manufacturer's recommendations. The sand should be washed, clean, non-plastic, and free from deleterious or foreign matter, and the sand layer should be underlain by an aggregate base layer of the minimum thickness, as indicated in the table above. The Class II aggregate base (Caltrans, R-value of 78, minimum) should be compacted to a minimum of 95 percent relative compaction determined in accordance with the modified Proctor compaction test (ASTM Test Method D1557). Prior to placement of base course, the underlying subgrade soil in the areas of vehicular paver construction should be overexcavated to a minimum depth of 12 inches, moisture-conditioned to 3 to 5 percent above optimum moisture content, and recompacted to a minimum of 95 percent relative compaction in accordance with ASTM Test Method D 1557.

In order to minimize differential movement of the concrete pavers, we also recommend that the pavers be placed after the majority of the heavy duty construction traffic (such as concrete trucks) has occurred. If this is not feasible, to prevent distortion, construction matting can be considered to protect the pavers to the degree possible during the remaining construction at the site.

Should crushed miscellaneous base (CMB) be considered for placement in paver areas, the CMB material should be submitted to the geotechnical engineer for evaluation and approval prior to implementing into site construction as the geotechnical engineering properties of the CMB may affect the thickness of recommended base course. Field observations and periodic testing, as needed during placement of the base course materials, should be undertaken by the geotechnical engineer of record to ensure that the requirements of the standard specifications are fulfilled

5.8 Additional Geotechnical Services

Our geotechnical recommendations are contingent upon Leighton Consulting, Inc., providing geotechnical observation and testing services during earthwork and foundation construction. There is a potential for encountering deeper undocumented fill, underground obstructions (septic and leach field facilities) or otherwise unacceptable existing soils between or beyond our boring locations. We are unaware of any existing fill placement documentation for this site. Therefore, inconsistent existing fill materials may be encountered during construction, possibly requiring revised geotechnical recommendations.

Our geotechnical recommendations provided in this report are based on information available at the time the report was prepared and may change as plans are developed. Additional geotechnical exploration, testing and/or analysis may be required based on final plans. Leighton Consulting, Inc. should review site grading and foundation plans when available, to comment further on geotechnical aspects of this project and check to see general conformance of final project plans to recommendations presented in this report.

Leighton Consulting, Inc. should be retained to provide geotechnical observation and testing during excavation and all phases of earthwork. Our conclusions and recommendations should be reviewed and verified by us during construction and revised accordingly if geotechnical conditions encountered vary from our findings and interpretations. Geotechnical observation and testing should be provided:

- During all excavation,
- During compaction of all fill materials,
- After excavation of all footings and prior to placement of concrete,
- During deep foundation installation if required,
- During utility trench backfilling and compaction,
- During pavement subgrade and base preparation, and/or
- If and when any unusual geotechnical conditions are encountered.

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, expressed 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 obtained, as warranted.

This report is issued with the understanding that it is the responsibility of the owner or a duly authorized agent acting on behalf of the owner, 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 findings of this report are considered valid as of the present date. However, changes in the condition of a property can occur with the passage of time, whether due to natural processes or the work of man on the subject or adjacent properties. In addition, changes in standards of practice may occur from legislation or the broadening of knowledge. Accordingly, the findings of this report may at some future time be invalidated wholly or partially by changes outside Leighton's control.

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 should be considered preliminary if unanticipated conditions are encountered and additional explorations, testing, and analyses may be necessary to develop alternative recommendations.

This report has been prepared for the expressed use of Santa Monica – Malibu Unified School District and its design consultants, and only as related expressly to the assessment of the geotechnical constraints of developing the subject site and for construction purposes. This report may not be used by others or for other projects without the expressed written consent of Santa Monica – Malibu School District and our firm.

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 investigations as they deem necessary to satisfy themselves as to the surface and/or subsurface conditions to be encountered and the procedures to be used in the performance of work on the subject site.

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Project: 11382.016	Eng/Geol: CCK/JAR	Figure 1
Scale:1 " = 2,000 '	Date: October 2020	Leighton
Base Map: ESRI ArcGIS Online 2020 Thematic Information: Leighton Author: Leighton Geomatics (btran)		

SITE LOCATION MAP

Phase I New High School Core Project
30215 Morning View Drive
Malibu, Los Angeles County, California

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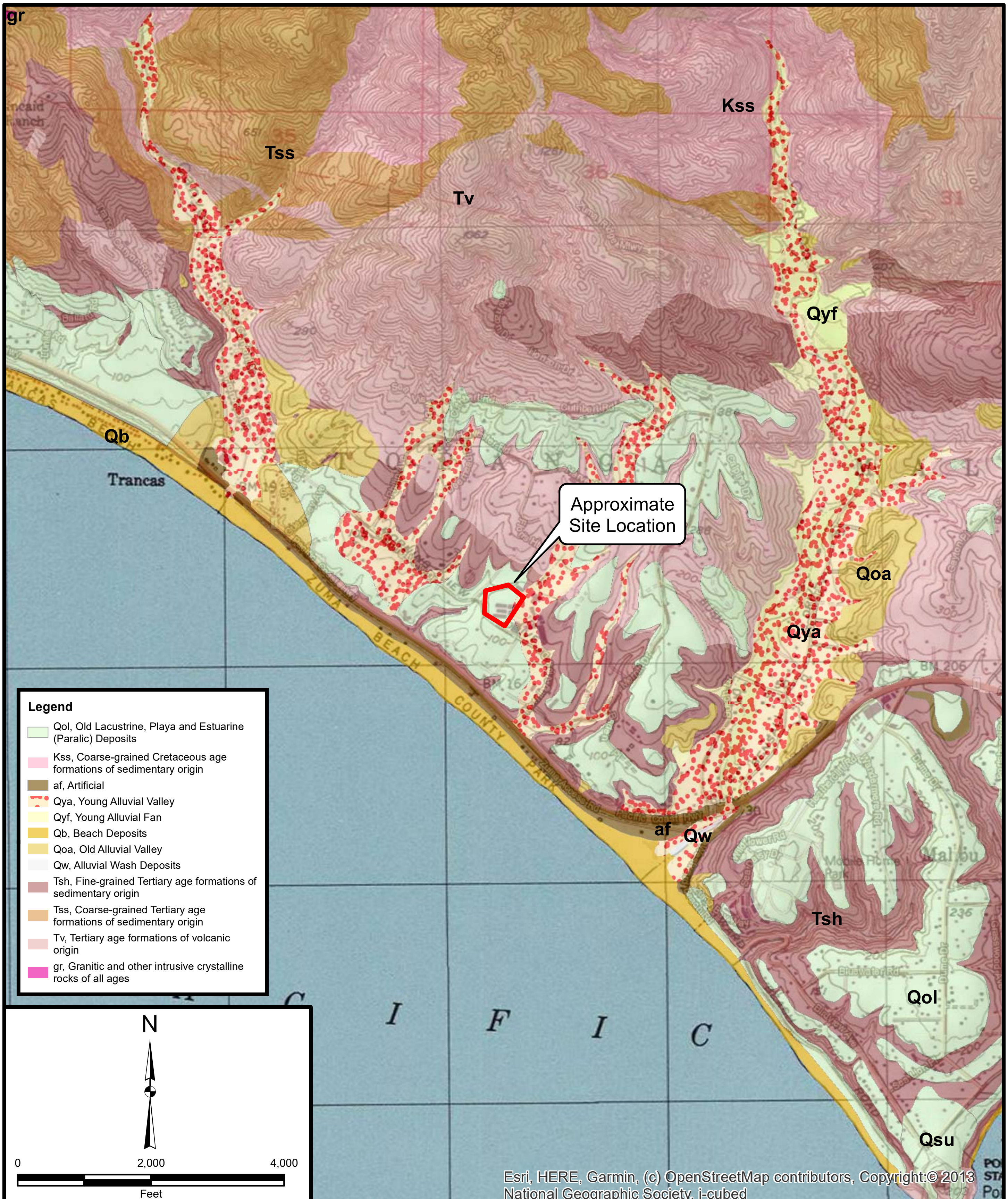

Project: 11382.016	Eng/Geol: CCK/JAR
Scale:1 " = 2,000 '	Date: October 2020

Base Map: ESRI ArcGIS Online 2020
Thematic Information: Leighton, USGS
Author: Leighton Geomatics (btran)

REGIONAL GEOLOGY MAP

Phase I New High School Core Project
30215 Morning View Drive
Malibu, Los Angeles County, California

Figure 2

Leighton

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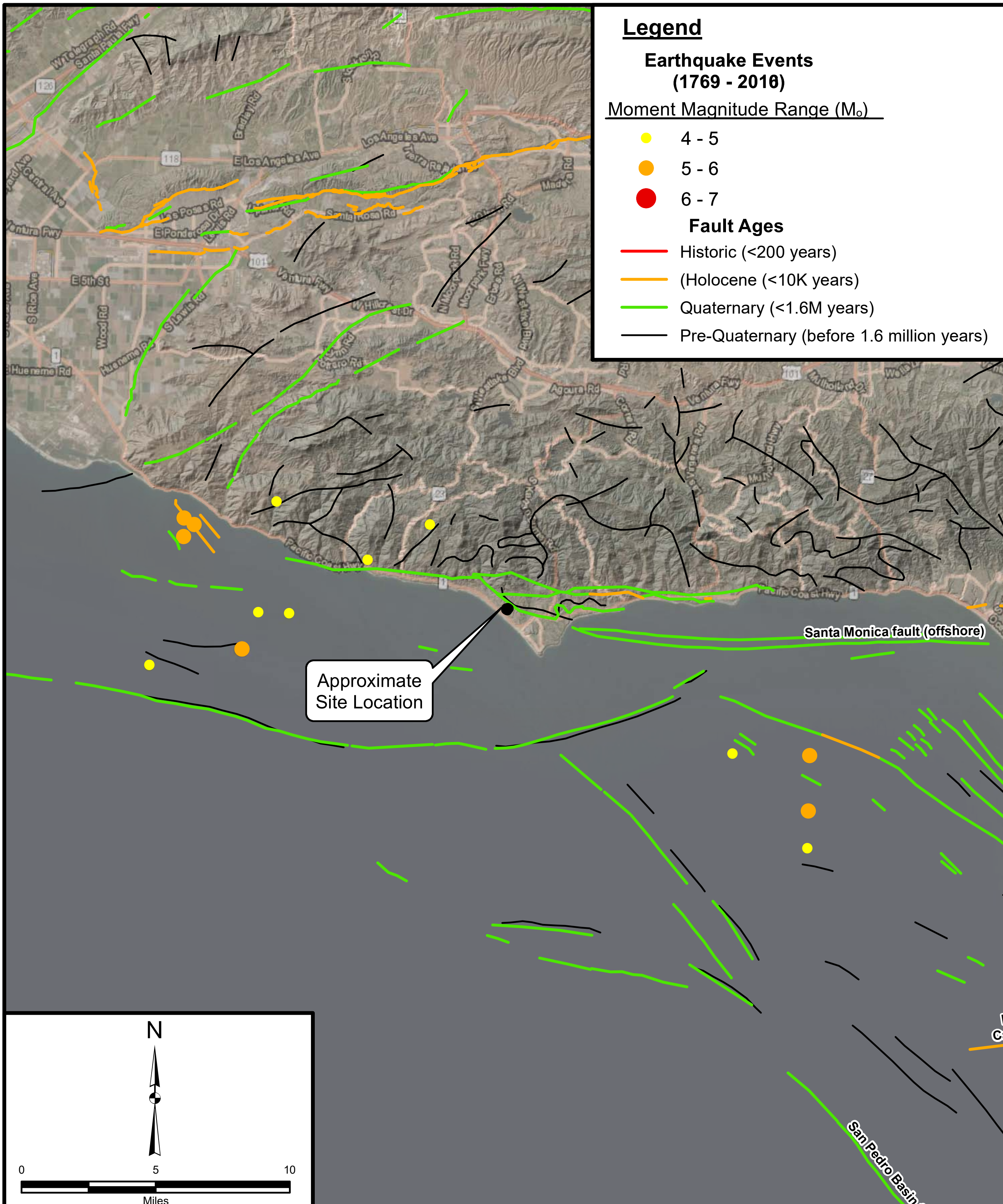

Project: 11382.016	Eng/Geol: CCK/JAR	REGIONAL FAULT AND HISTORICAL SEISMICITY MAP	Figure 3
Scale:1 " = 5 miles	Date: October 2020	Phase I New High School Core Project 30215 Morning View Drive Malibu, Los Angeles County, California	Leighton

Base Map: ESRI ArcGIS Online 2020
Thematic Information: Leighton, Bryant, W. A. (compiler), 2005, Digital Database of Quaternary and Younger Faults from the Fault Activity Map of California, version 2.0: CGS, USGS, SCEC.
Author: Leighton Geomatics (btran)

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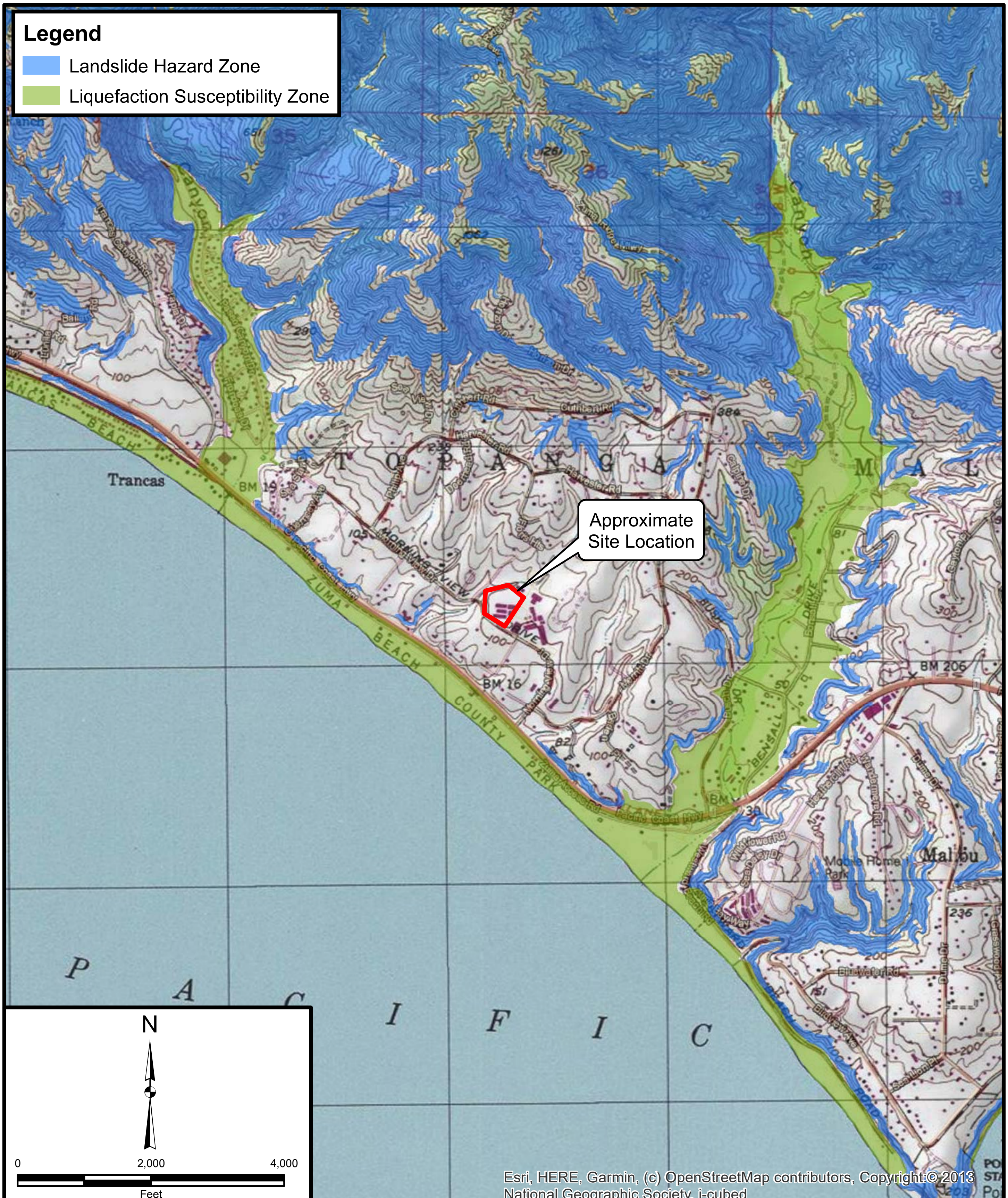

Project: 11382.016	Eng/Geol: CCK/JAR
Scale:1 " = 2,000 '	Date: October 2020

Base Map: ESRI ArcGIS Online 2020
Thematic Information: Leighton, CGS
Author: Leighton Geomatics (btran)

SEISMIC HAZARD MAP

Phase I New High School Core Project
30215 Morning View Drive
Malibu, Los Angeles County, California

Figure 4

Leighton

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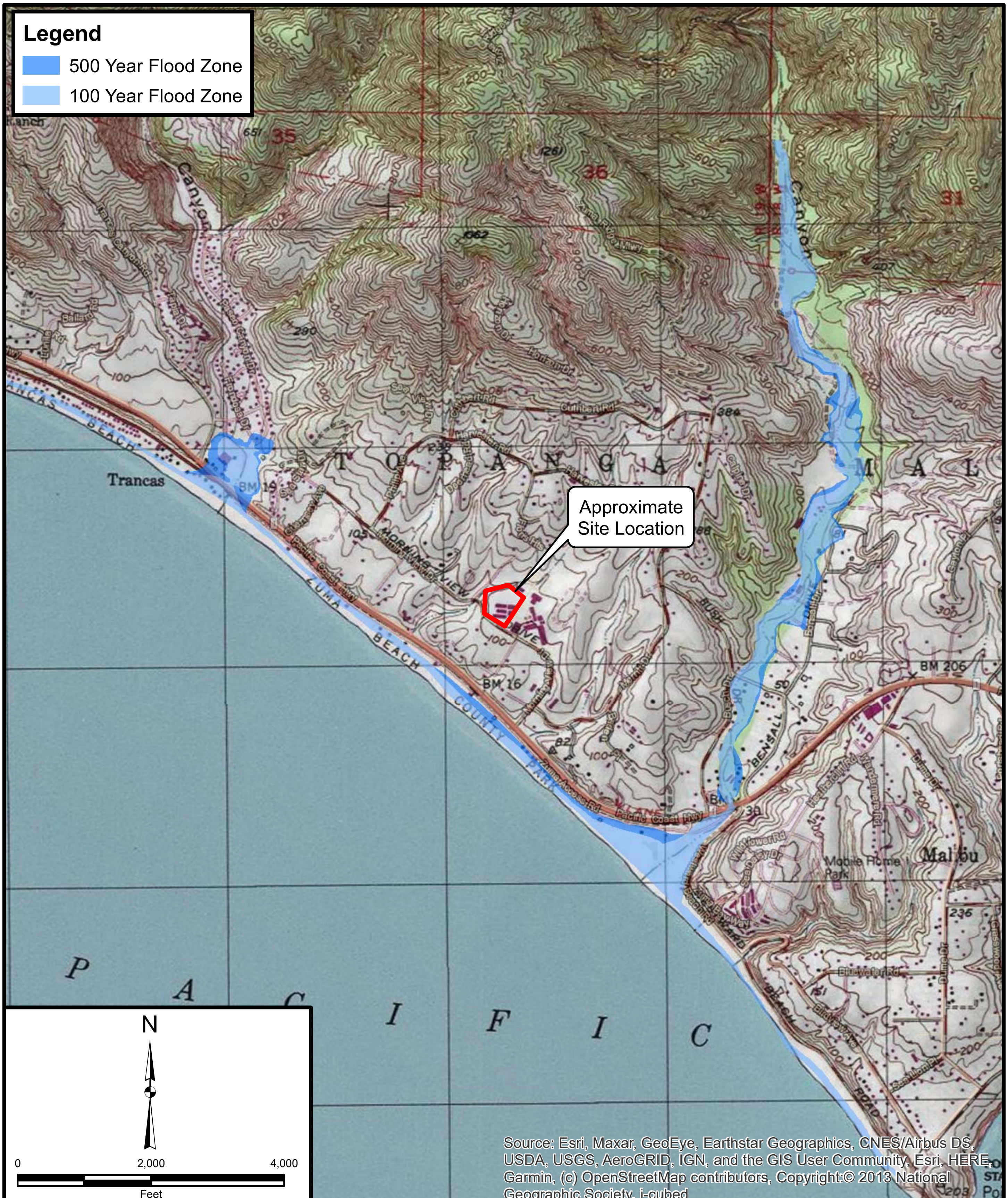

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Scale:1 " = 2,000 '	Date: October 2020

Base Map: ESRI ArcGIS Online 2020
Thematic Information: Leighton, CA DWR, FEMA
Author: Leighton Geomatics (btran)

FLOOD HAZARD ZONE MAP

Phase I New High School Core Project
30215 Morning View Drive
Malibu, Los Angeles County, California

Figure 5

Leighton

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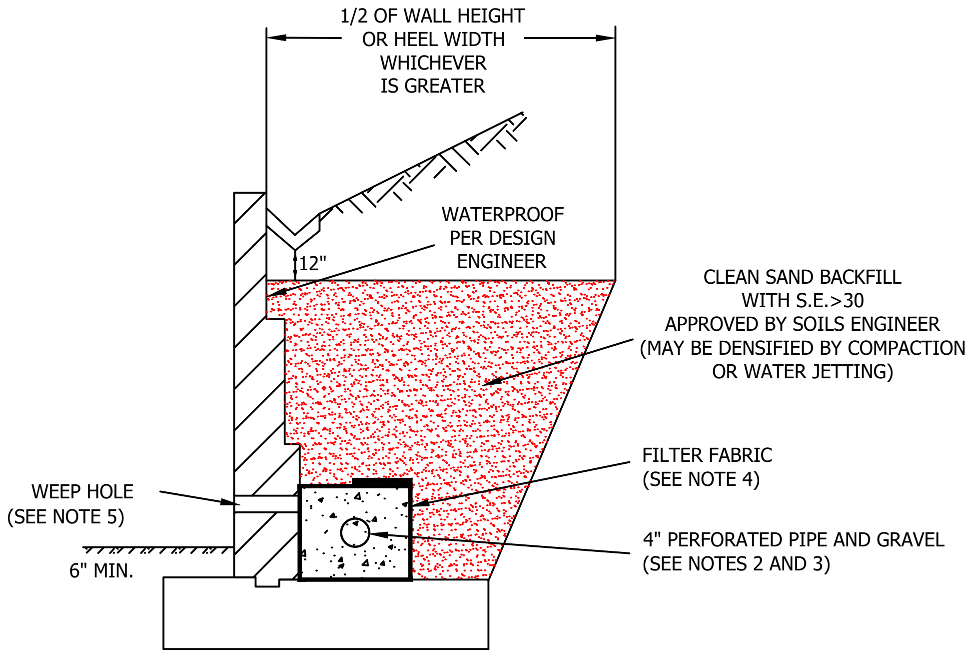
SUBDRAIN OPTIONS AND BACKFILL WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF >50



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

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FIGURE 6

APPENDIX A

EXPLORATION LOGS

GEOTECHNICAL BORING LOG 2020-LB-1

Project No.	11382.016	**Date Drilled**	8-17-20
Project	SMMUSD MMHS Phase 1	**Logged By**	MM
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	113'
Location	See Plate 1- Geotechnical Map	**Sampled By**	MM

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.

Elevation Feet	Depth Feet	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
113	0							@0': 3 inches of asphalt concrete over 6 inches of base course.	
			B-1				CL	**Artificial Fill, undocumented (Afu):** @0.75': CLAY; dark brown to black; moist; trace fine sand; medium to high plasticity.	
110									
	5		R-1	6 9 12	85	28		**Non-Marine Terrace Deposits (Qyd):** @5': CLAY; stiff; dark brown; moist; trace fine sand and gravel.	
			R-2	7 14 21	93	28		@7': CLAY; very stiff; dark brown; moist; little fine sand; fine gravel; medium to high plasticity; trace carbonate precipitate.	
105									
	10		R-3	10 16 26	94	28	CL	@10': CLAY; hard; dark brown; moist; little fine sand; carbonate veins; MnO points.	
100									
	15		S-4	3 5 7		25		@15': CLAY; very stiff; dark brown; moist; little fine sand; fine gravel; moderate plasticity; trace carbonate nodules and MnO points.	
95									
	20		R-5	8 11 19	103	16		@20': CLAY; very stiff; brown; moist; fine sand and gravel; moderate plasticity; trace carbonates.	
90									
	25		S-6	3 3 5		25	SC	@25': Clayey SAND; medium dense; brown to orange brown; moist; fine to coarse sand; mostly fine sand; no to low plasticity; carbonate veins.	
85									
	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

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

GEOTECHNICAL BORING LOG 2020-LB-1

Project No.	11382.016	**Date Drilled**	8-17-20
Project	SMMUSD MMHS Phase 1	**Logged By**	MM
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	113'
Location	See Plate 1- Geotechnical Map	**Sampled By**	MM

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Bulk / Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	30			R-7		12 17 22	102	22	CL	@30': Sandy CLAY; hard; orange brown; moist; mostly fine sand; fine gravel; moderate plasticity.	
80										**Total Depth: 31.5 feet** **No groundwater encountered.** **Backfilled with soil cuttings, tamped, and patched with asphalt.**	
	35										
75											
	40										
70											
	45										
65											
	50										
60											
	55										
55											
	60										

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

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

GEOTECHNICAL BORING LOG 2020-LB-2

Project No.	11382.016	**Date Drilled**	8-17-20
Project	SMMUSD MMHS Phase 1	**Logged By**	MM
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	115'
Location	See Plate 1- Geotechnical Map	**Sampled By**	MM

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No. (Bulk / Driven)	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
115	0								@0': 3 inches of asphalt concrete over 5 inches of base course.	
				B-1				CL	**Artificial Fill, undocumented (Afu):** @0.66': CLAY; dark brown to black; moist; trace fine sand; medium to high plasticity.	
110	5			S-1	2 4 9		21		@5': CLAY; very stiff; dark brown to black; moist; trace fine sand and gravel; medium to high plasticity; hydrocarbon odor.	
									Non-Marine Terrace Deposits (Qyd): @6.5': CLAY; dark brown to orange brown; moist.	
105	10			S-2	4 7 10		27	CL	@10': CLAY; very stiff; brown; moist; little fine sand and gravel; carbonate veins.	
100	15			S-3	2 6 10		25		@15': CLAY; very stiff; brown; moist; little fine sand and gravel; carbonate veins.	
95	20			S-4	5 5 10		6	SC	@20': Clayey SAND; medium dense; brown; moist; fine to coarse sand; mostly fine sand; fine gravel.	
									Total Depth: 21.5 feet **No groundwater encountered.** **Backfilled with soil cuttings, tammped, and patched with asphalt.**	
90	25									
85	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

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GEOTECHNICAL BORING LOG 2020-LB-3

Project No.	11382.016	**Date Drilled**	8-17-20
Project	SMMUSD MMHS Phase 1	**Logged By**	MM
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	119'
Location	See Plate 1- Geotechnical Map	**Sampled By**	MM

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.

Elevation Feet	Depth Feet	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0							@0': 2.5 inches of asphalt concrete over 4 inches of base course.	
			B-1				CL	**Artificial Fill, undocumented (Afu):** @0.54': CLAY; dark brown to black; moist; few to little fine sand; few fine gravel.	
115	5		R-1	5 7 14	102	23	CL	**Non-Marine Terrace Deposits (Qyd):** @5': CLAY; very stiff; dark brown; moist; little fine sand; few fine gravel.	
			R-2A	17 50/4"	102	23		@7': Hard, no sample recovered.	
110			R-2B	4 10 21				Rig encountered difficult drilling, auger grinding on rock. Auger refusal at 7'10". Stepped over 5 feet and redrilled down to 7'. @7': CLAY; hard; dark brown; moist; few fine sand; little fine gravel; medium plasticity.	
	10		R-3	11 12 15	102	18		@10': Sandy CLAY; very stiff; orange brown; moist; fine sand and gravel, low to moderate plasticity.	
105	15		S-4	15 25 50/4"		15	SC	@15': Clayey SAND with gravel; hard; yellowish brown; moist; fine to coarse sand; fine to coarse gravel; some mechanically broken rock fragments.	
100	20		R-5	50/4"	109	15		@20': Clayey SAND; very dense; yellowish brown; moist; fine to coarse sand; mostly fine sand; fine to coarse gravel; FeO spots and stains along pedogenic surfaces.	
								Total Depth: 21.33 feet **Auger refusal at 21.33'.** **No groundwater encountered.** **Backfilled with soil cuttings, tamped, and patched with asphalt.**	
95	25								
90	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

GEOTECHNICAL BORING LOG 2020-LB-4

Project No.	11382.016	**Date Drilled**	8-17-20
Project	SMMUSD MMHS Phase 1	**Logged By**	MM
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	118'
Location	See Plate 1- Geotechnical Map	**Sampled By**	MM

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.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Bulk / Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0			B-1	Bulk		97	14	CL	@0': 2 inches of asphalt concrete over 4 inches of base course. **Artificial Fill, undocumented (Afu):** @0.5': CLAY; dark brown; moist; medium to high plasticity.	MD, DS, EI, CN RV, CR
115									CL	**Non-Marine Terrace Deposits (Qyd):** @2': CLAY with sand; yellowish brown; moist; little fine sand and gravel.	
	5			R-1	Driven	10 12 16	110	18		@5': CLAY with sand; very stiff; yellowish brown; moist; mostly fine sand; fine gravel.	AL, DS, CN
110				R-2	Driven	7 12 21	105	21		@7': CLAY; very stiff; yellowish brown; moist; fine sand and gravel; low to moderate plasticity; carbonate veins throughout.	AL
	10			R-3	Driven	5 10 16	105	17		@10': CLAY; very stiff; orange brown; moist; fine sand and gravel; low to moderate plasticity; carbonate veins throughout; caliche.	AL, DS, CN
105											
	15			R-4	Driven	8 19 32	107	20		@15': CLAY; hard; orange brown; moist; fine sand and gravel; low to moderate plasticity; carbonate veins.	AL
100											
	20			S-5	Driven	5 16 14		22	CH	@20': CLAY, fat; hard; orange brown; moist; fine to coarse sand; fine gravel; low to medium plasticity; MnO spotting.	AL
95											
	25			R-6	Driven	15 20 50/5"	104	21		@25': CLAY, fat; hard; orange brown; moist; fine sand; fine gravel.	AL
90											
	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

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GEOTECHNICAL BORING LOG 2020-LB-4

Project No.	11382.016	**Date Drilled**	8-17-20
Project	SMMUSD MMHS Phase 1	**Logged By**	MM
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	118'
Location	See Plate 1- Geotechnical Map	**Sampled By**	MM

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Bulk / Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	30			S-7		5 8 14		26	SC	@30': Clayey SAND; medium dense; orange brown; moist; fine to coarse sand; mostly fine sand; fine gravel; no to low plasticity.	
85										**Total Depth: 31.5 feet** **No groundwater encountered.** **Backfilled with soil cuttings, tamped, and patched with asphalt.**	
	35										
80											
	40										
75											
	45										
70											
	50										
65											
	55										
60											
	60										

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

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GEOTECHNICAL BORING LOG 2020-LB-5

Project No.	11382.016	**Date Drilled**	8-17-20
Project	SMMUSD MMHS Phase 1	**Logged By**	MM
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	113'
Location	See Plate 1- Geotechnical Map	**Sampled By**	MM

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.

Elevation Feet	Depth Feet	Graphic Log (N / S)	Attitudes	Sample No.	Bulk / Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			B-1						@0': 4 inches of asphalt concrete over 1 inches of base course.	
									CL	**Artificial Fill, undocumented (Afu):** @0.41': Silty SAND; grayish olive brown; moist; mostly fine sand.	
110										**Non-Marine Terrace Deposits (Qyd):** @1': Sandy CLAY; orange brown; moist; little fine sand.	
	5			R-1		3 4 5	96	24		@5': CLAY with Sand; medium stiff; orange brown; moist; fine sand; few fine gravel; moderately plastic; laminated with dark brown to black layers of clay.	
105				R-2		3 5 9	97	22		@7': CLAY with Sand; stiff; orange brown and dark brown; moist; fine sand; fine gravel. Transitions to dark brown CLAY at 8'.	
	10			R-3		3 8 12	95	26		@10': CLAY; very stiff; dark brown to black; moist; trace fine sand; moderate plasticity. Transitions to orange brown CLAY; moist; few fine sand; fine to coarse gravel.	
100											
	15			S-4		3 3 6		24		@15': Sandy CLAY; stiff; orange brown; moist; fine sand; fine gravel; carbonate layers and blebs.	
95											
	20			R-5		11 19 43	106	21		@20': Sandy CLAY; hard; orange brown; moist; fine sand; little fine gravel; low plasticity; carbonate veins.	
90										Rig encountered difficult drilling, rattling, possible cobbles.	
	25			S-6		6 17 17		27	SC	@25': Clayey SAND; very dense; orange brown; moist; mostly fine sand; fine gravel; no to low plasticity.	
				R-7		50/2"					
85										**Total Depth: 27.16 feet** **Auger refusal at 27.16'.** **No groundwater encountered.** **Backfilled with soil cuttings, tamped, and patched with asphalt.**	
	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	DS	DIRECT SHEAR	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	EI	EXPANSION INDEX	SE	SAND EQUIVALENT
CN	CONSOLIDATION	H	HYDROMETER	SG	SPECIFIC GRAVITY
CO	COLLAPSE	MD	MAXIMUM DENSITY	UC	UNCONFINED COMPRESSIVE STRENGTH
CR	CORROSION	PP	POCKET PENETROMETER		
CU	UNDRAINED TRIAXIAL	RV	R VALUE		

GEOTECHNICAL BORING LOG 2020-LB-6

Project No.	11382.016	**Date Drilled**	8-17-20
Project	SMMUSD MMHS Phase 1	**Logged By**	MM
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	112'
Location	See Plate 1- Geotechnical Map	**Sampled By**	MM

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.

Elevation Feet	Depth Feet	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0						@0': 6 inches of asphalt concrete over 6 inches of base course.	
		B-1		99	12		**Artificial Fill, undocumented (Afu):** @1': Silty SAND; olive brown; moist; mostly fine sand over CLAY; dark brown; moist; moderate plasticity.	MD, DS, EI, CN RV, CR
110						CL	**Non-Marine Terrace Deposits (Qyd):** @2': CLAY with sand; orange brown; moist; fine sand; moderate plasticity.	
	5	R-1	7 7 14	99	23	CH	@5': CLAY, fat; stiff; brown; moist; mostly fine sand; low plasticity; carbonate layers.	AL
105		R-2	7 14 19	100	25		@7': CLAY, fat; very stiff; dark brown; moist; medium plasticity; carbonate veins; caliche.	AL, DS, CN
	10	R-3	19 24 21	105	16		@10': CLAY, fat; hard; orange brown; moist; fine sand; fine to coarse gravel; low to medium plasticity	AL
100								
	15	R-4	12 16 48	113	16	CL	@15': CLAY; hard; orange brown; moist; fine sand; fine gravel; medium plasticity; trace carbonate veins; caliche.	AL, DS, CN
95								
	20	S-5	7 8 16		20	CH	@20': CLAY, fat; hard; orange brown; moist; little fine sand and gravel; MnO veins throughout.	AL
90								
	25	R-6	9 13 30	110	19	CL	@25': CLAY; very stiff; orange brown; moist; little fine sand and gravel; trace MnO staining.	AL
85								
	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

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GEOTECHNICAL BORING LOG 2020-LB-6

Project No.	11382.016	**Date Drilled**	8-17-20
Project	SMMUSD MMHS Phase 1	**Logged By**	MM
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	112'
Location	See Plate 1- Geotechnical Map	**Sampled By**	MM

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Bulk / Driven	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			S-7		8 24 29		5	SP	@30': SAND, poorly-graded; very dense; yellowish orange brown; moist; fine to coarse sand; mostly fine sand; trace fine gravel.	
80											
										@33': SAND, poorly-graded, with gravel; fine and rounded gravel, oxidation ring around gravel.	
	35			R-8		42 50/5"	103	9		@35': SAND, poorly-graded; very dense; yellowish brown; moist; mostly fine sand; fine rounded gravel.	
75											
	40			S-8		10 11 36		33		@40': SAND, poorly-graded with CLAY beds; dense SAND; hard CLAY; gray to orange brown; moist; fine to coarse sand; mostly fine sand; dark FeO stained sand layer; laminated clay layers; FeO staining along clay laminations and pedogenic surfaces.	
70											
	45			R-9		50/4"	62	57		**(Tm):** @45': SANDSTONE; olive brown; moist; fine sand; trace fine rounded gravel.	
65										**Total Depth: 46.5 feet** **No groundwater encountered.** **Backfilled with soil cuttings, tamped, and patched with asphalt.**	
	50										
60											
	55										
55											
	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	DS	DIRECT SHEAR	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	EI	EXPANSION INDEX	SE	SAND EQUIVALENT
CN	CONSOLIDATION	H	HYDROMETER	SG	SPECIFIC GRAVITY
CO	COLLAPSE	MD	MAXIMUM DENSITY	UC	UNCONFINED COMPRESSIVE STRENGTH
CR	CORROSION	PP	POCKET PENETROMETER		
CU	UNDRAINED TRIAXIAL	RV	R VALUE		

GEOTECHNICAL BORING LOG 2020-LB-7

Project No.	11382.016	**Date Drilled**	8-17-20
Project	SMMUSD MMHS Phase 1	**Logged By**	MM
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	113'
Location	See Plate 1- Geotechnical Map	**Sampled By**	MM

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.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No. (Bulk / Driven)	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0								@0': 6 inches of asphalt concrete over 6 inches of base course.	
				B-1					**Artificial Fill, undocumented (Afu):** @1': Silty SAND; olive brown; moist; mostly fine sand.	
110								CL	**Non-Marine Terrace Deposits (Qyd):** @1.5': Sandy CLAY; orange brown; moist; fine sand and gravel.	
	5			R-1	6 11 20	105	16		@5': Sandy CLAY; very stiff; orange brown; moist; fine sand; fine gravel; carbonate veins.	
105				R-2	10 16 29	109	18		@7': Sandy CLAY; hard; orange brown; moist; fine sand and gravel; few carbonates; spotty MnO at 8'.	
	10			R-3	11 20 50/5"	109	18		@10': Sandy CLAY; hard; orange brown; moist; fine sand and gravel; carbonate veins and nodules; spotty MnO.	
100										
	15			S-4	7 10 17		22		@15': Sandy CLAY; hard; brown to orange brown; moist; fine sand; fine gravel, granitic fragments; 1/4-inch-thick carbonate veins	
95										
	20			R-5	12 20 50/5"	114	18	SC	@20': Clayey SAND; very dense; orange brown; moist; fine to coarse sand; mostly fine sand.	
90										
	25			S-6	10 20 37		4	SP	@25': SAND, poorly-graded; dense; yellowish orange brown; moist; fine to coarse sand; mostly fine sand.	
85										
	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

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GEOTECHNICAL BORING LOG 2020-LB-7

Project No.	11382.016	**Date Drilled**	8-17-20
Project	SMMUSD MMHS Phase 1	**Logged By**	MM
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	113'
Location	See Plate 1- Geotechnical Map	**Sampled By**	MM

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Bulk / Driven	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-7	Driven	12 12 10	102	5	SP	@30': SAND, poorly-graded; medium dense; yellowish brown to grayish brown; moist; fine to coarse sand; round, fine gravel at the top of sample; bedded and laminated Silty SAND at 31.5'.	
80										**Total Depth: 31.5 feet** **No groundwater encountered.** **Backfilled with soil cuttings, tamped, and patched with asphalt.**	
	35										
75											
	40										
70											
	45										
65											
	50										
60											
	55										
55											
	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

GEOTECHNICAL BORING LOG 2020-LP-1

Project No.	11382.016	**Date Drilled**	8-17-20
Project	SMMUSD MMHS Phase 1	**Logged By**	MM
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	112'
Location	See Plate 1- Geotechnical Map	**Sampled By**	MM

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Bulk / Driven	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0									@0': 3 inches of asphalt concrete over 3 inches of base course.	
				B-1	Bulk					**Artificial Fill, undocumented (Afu):** @0.5': Silty SAND; olive brown; moist; fine sand; fine gravel.	
110									CL	**Non-Marine Terrace Deposits (Qyd):** @1': Sandy CLAY; orange brown; moist.	
	5			R-1	Driven	5, 5, 7	99	21		@5': Sandy CLAY; stiff; orange brown; moist; fine sand; fine to coarse gravel; moderately plastic; dark brown to black clay intermixed.	
105				R-2	Driven	5, 10, 17	96	26		@7': CLAY; very stiff; dark brown to black; moist; trace fine sand; medium to high plasticity.	
	10			R-3	Driven	9, 12, 18	106	21		@10': CLAY with SAND; very stiff; brown; moist; little fine sand; carbonate veins throughout.	
100											
				S-4	Driven	7, 5, 9		19	SC	@13.5': Clayey SAND; medium dense; orange brown; moist; mostly fine sand; fine gravel; no to low plasticity.	
	15									**Total Depth: 15 feet** **No groundwater encountered.** **Converted boring to temporary percolation well on 08/17/2020.** **Installed solid 2-inch-diameter pipe from 0-10 feet.** **Installed slotted pipe from 10-15 feet.** **Added Monterey #3 SAND around pipe from 15-6.75 feet.** **Backfilled with soil cuttings and patched with asphalt upon completion of testing on 08/18/2020.**	
95											
	20										
90											
	25										
85											
	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
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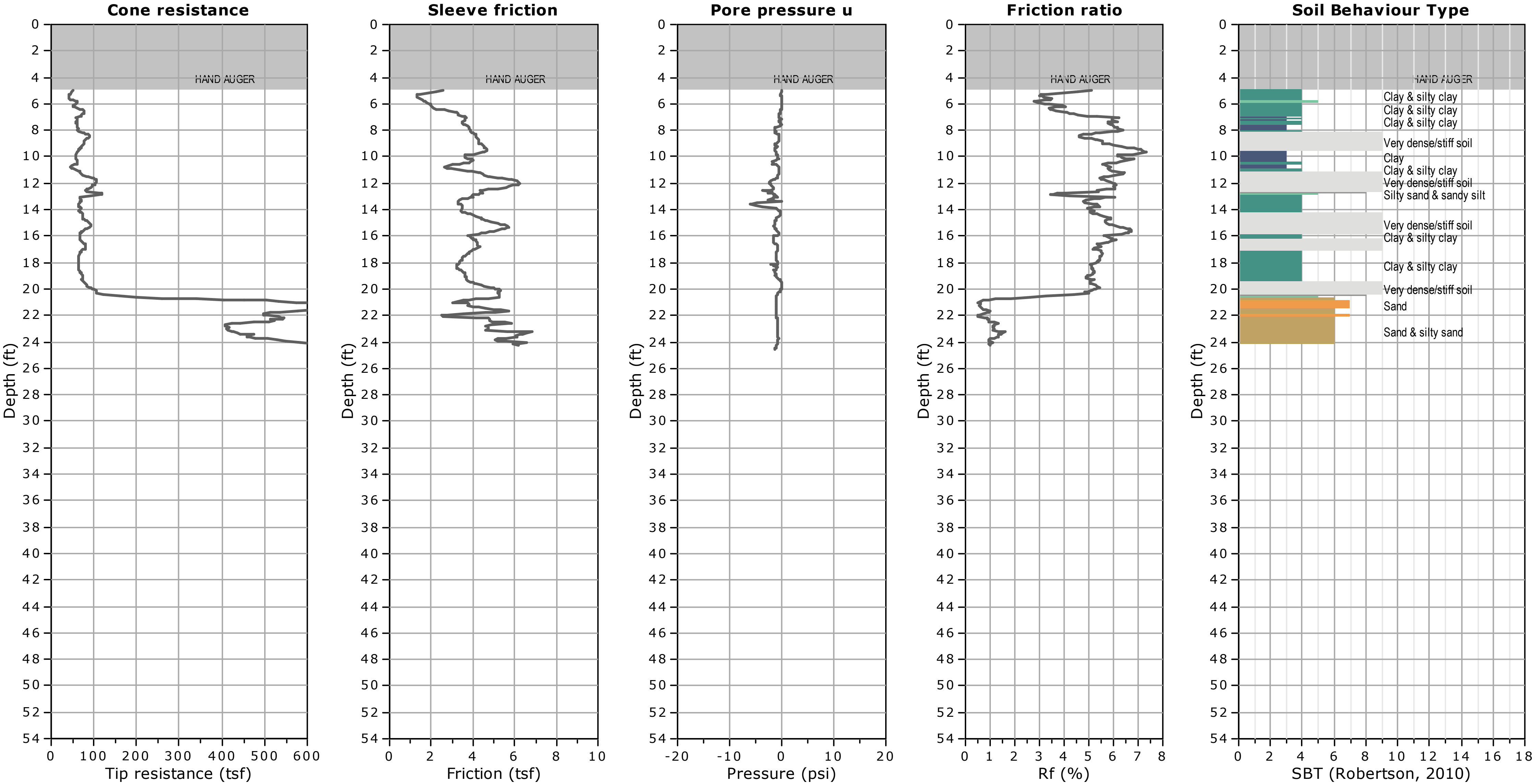
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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

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

Project: Leighton Consulting / Malibu High School
Location: Malibu, CA

CPT-1
Total depth: 24.61 ft, Date: 8/17/2020



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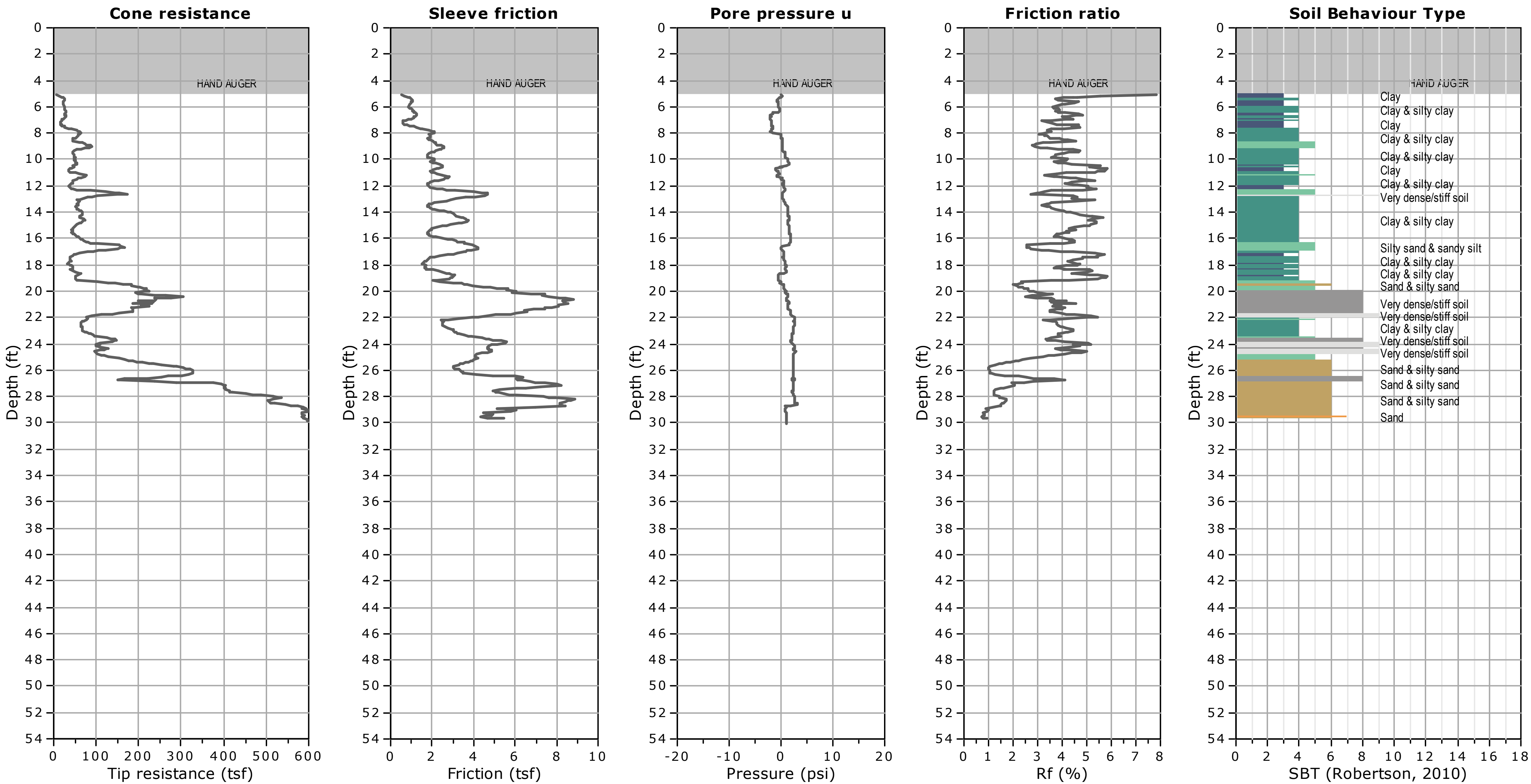
Project: Leighton Consulting / Malibu High School
Location: Malibu, CA

CPT-2
Total depth: 30.07 ft, Date: 8/17/2020



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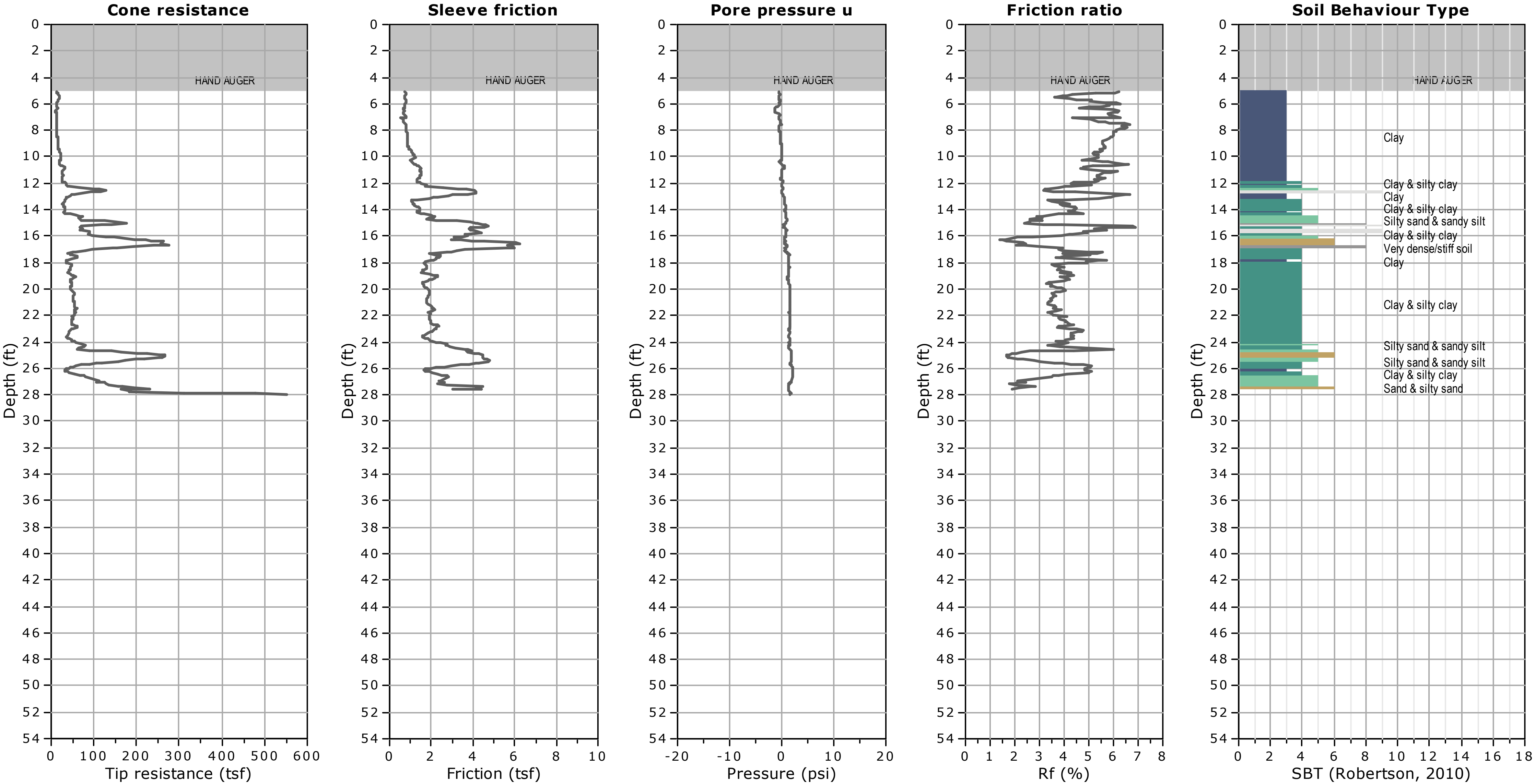
www.kehoetesting.com

Project: Leighton Consulting / Malibu High School

Location: Malibu, CA

CPT-3

Total depth: 27.95 ft, Date: 8/17/2020



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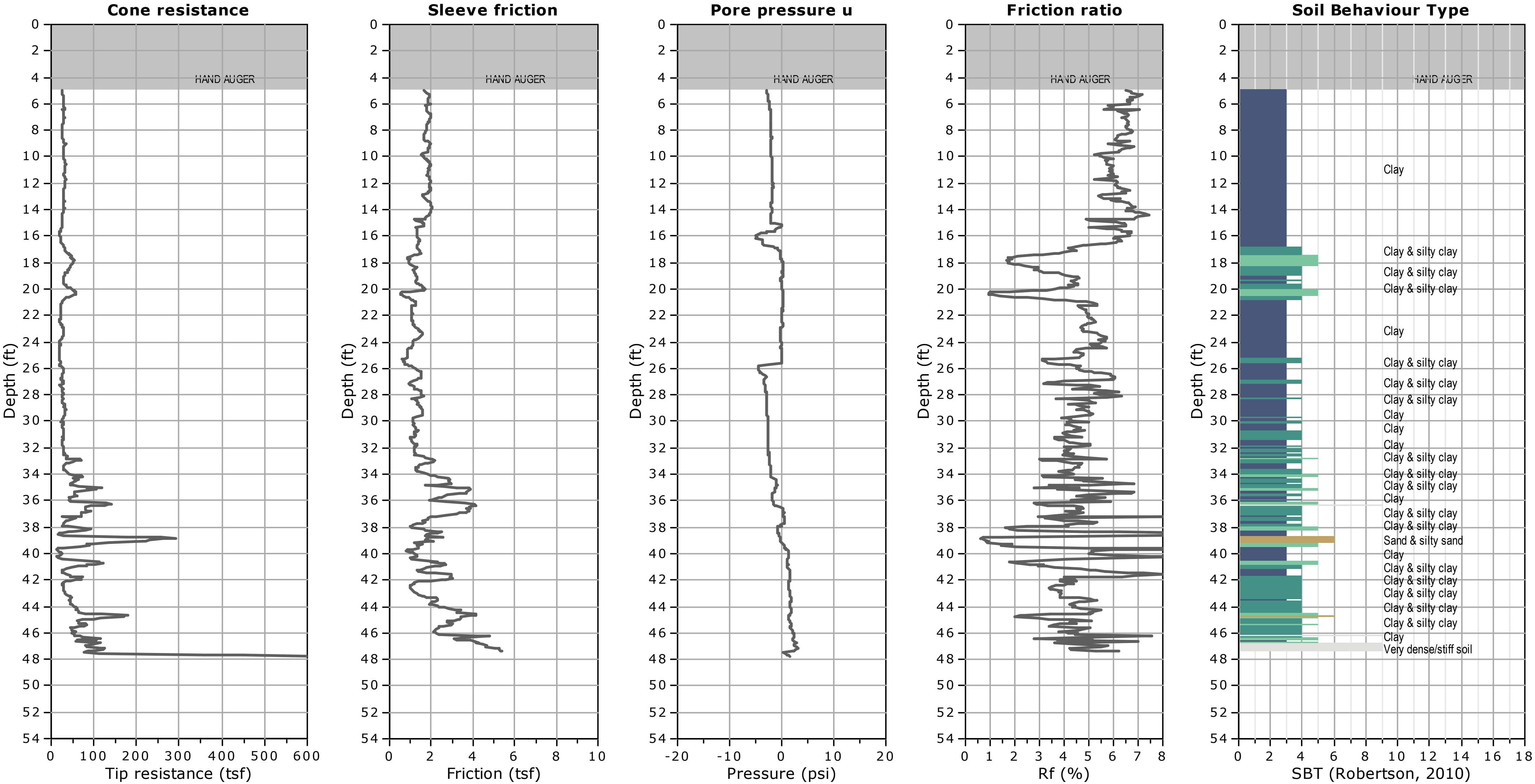
www.kehoetesting.com

Project: **Leighton Consulting / Malibu High School**
Location: **Malibu, CA**

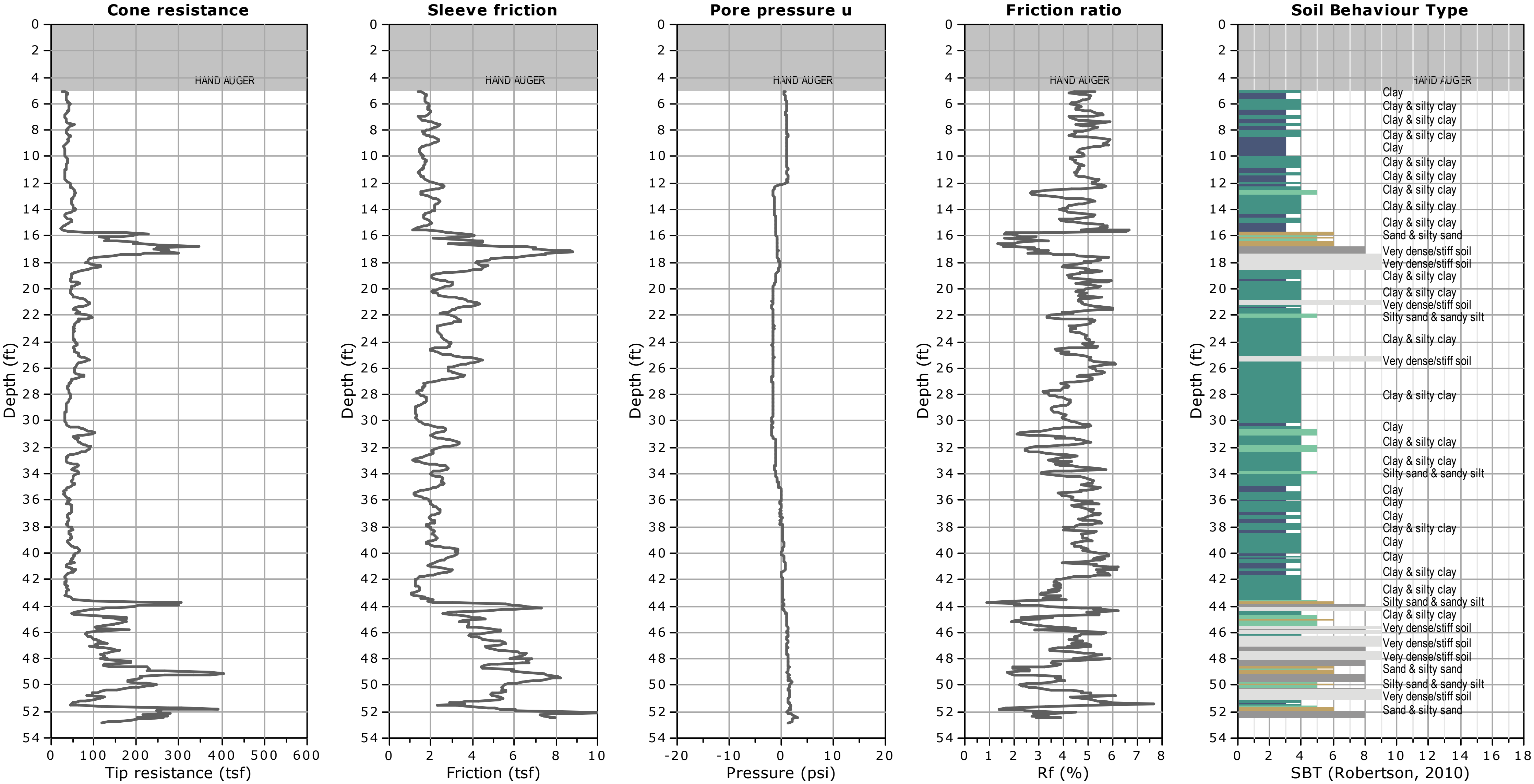
CPT-4
Total depth: 47.74 ft, Date: 8/17/2020



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

Project: **Leighton Consulting / Malibu High School** **CPT-5**

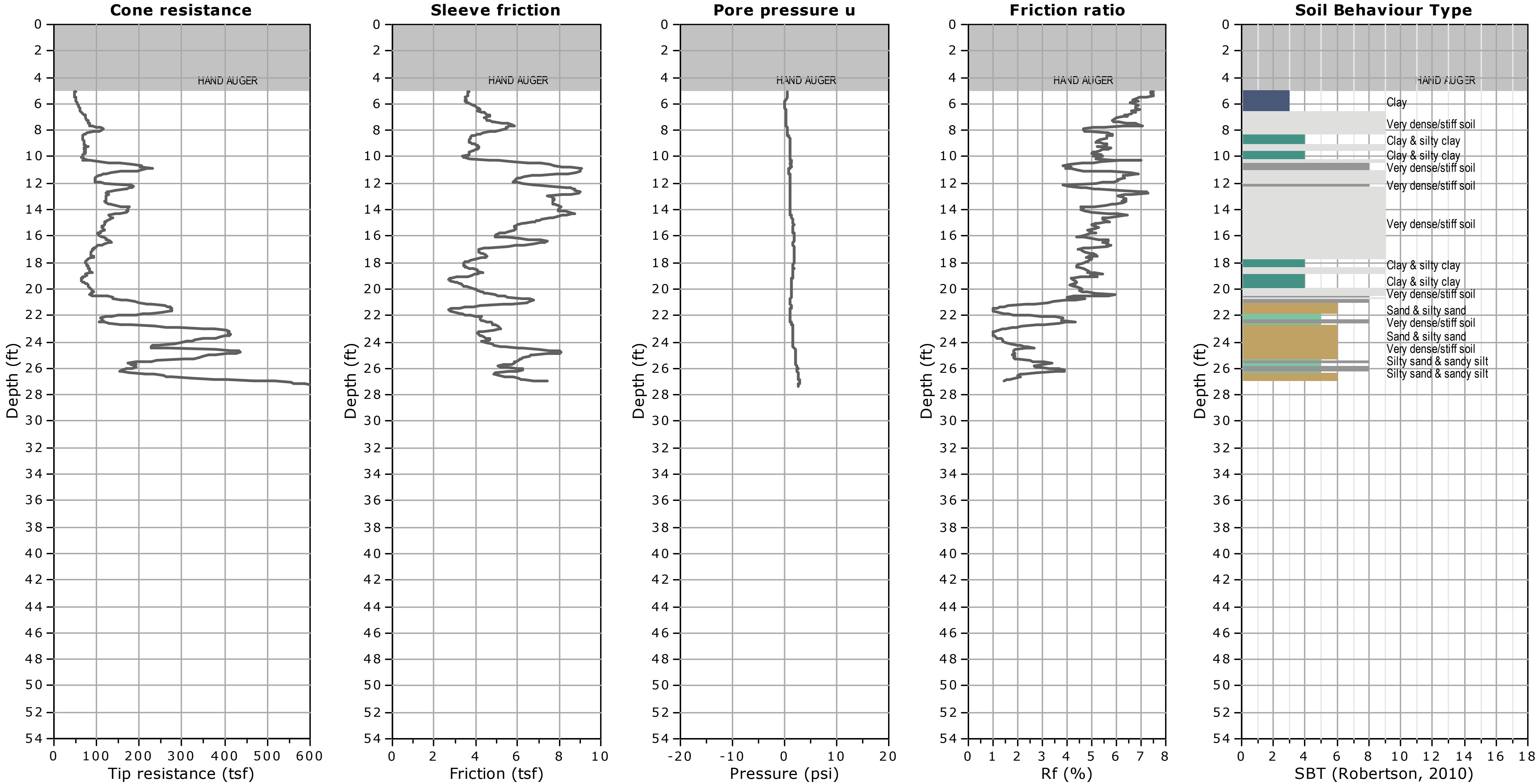
Location: **Malibu, CA** Total depth: 52.84 ft, Date: 8/17/2020



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

Project: Leighton Consulting / Malibu High School
Location: Malibu, CA

CPT-6
Total depth: 27.38 ft, Date: 8/17/2020



Leighton Consulting
Malibu High School
Malibu, CA

CPT Shear Wave Measurements

Location	Tip Depth (ft)	Geophone Depth (ft)	Travel Distance (ft)	S-Wave Arrival (msec)	S-Wave Velocity from Surface (ft/sec)	Interval S-Wave Velocity (ft/sec)
CPT-4	5.02	4.02	4.49	8.56	525	
	10.07	9.07	9.29	15.52	598	689
	15.12	14.12	14.26	22.34	638	729
	20.44	19.44	19.54	29.54	662	734
	25.00	24.00	24.08	35.00	688	832
	30.05	29.05	29.12	40.14	725	980
	35.07	34.07	34.13	44.52	767	1144
	40.06	39.06	39.11	47.86	817	1492
	45.05	44.05	44.10	52.68	837	1034
	47.70	46.70	46.74	54.28	861	1655

Shear Wave Source Offset - 2 ft

S-Wave Velocity from Surface = Travel Distance/S-Wave Arrival
Interval S-Wave Velocity = (Travel Dist2-Travel Dist1)/(Time2-Time1)

APPENDIX B

PERCOLATION TEST RESULTS

Boring Percolation Test Data Sheet

Project Number:	11382.016	**Test Hole Number:**	LP-1
Project Name:	SMMUSD MMHS PH 1	**Date Excavated:**	8/17/2020
Earth Description:	Older Alluvium	**Date Tested:**	8/18/2020
Liquid Description:	Tap water	**Depth of boring (ft):**	15
Tested By:	MM	**Radius of boring (in):**	4
Time Interval Standard		**Radius of casing (in):**	1
Start Time for Pre-Soak:	8/18/20 8:21 AM	**Length of slotted of casing (ft):**	5
Start Time for Standard:	8/18/20 9:21 AM	**Depth to Initial Water Depth (ft):**	7.75
Standard Time Interval Between Readings, mins:	30	**Porosity of Annulus Material, *n* :**	0.35
		Bentonite Plug at Bottom:	no

Percolation Data

Reading	Time	Time Interval, Δt (min.)	Initial/Final Depth to Water (ft.)	Initial/Final Water Height, H_0/H_f (in.)	Total Water Drop, Δd (in.)	Infiltration Rate (in./hr.)
P1	8:21	30	7.75	87.0	0.2	0.00
	8:51		7.77	86.8		
P2	8:51	30	7.77	86.8	0.5	0.01
	9:21		7.81	86.3		
1	9:21	30	7.81	86.3	0.2	0.00
	9:51		7.83	86.0		
2	9:51	30	7.83	86.0	0.2	0.00
	10:21		7.85	85.8		
3	10:21	30	7.85	85.8	0.4	0.01
	10:51		7.88	85.4		
4	10:51	30	7.88	85.4	0.2	0.00
	11:21		7.90	85.2		
5	11:21	30	7.90	85.2	0.2	0.00
	11:51		7.92	85.0		
6	11:51	30	7.92	85.0	0.4	0.01
	12:21		7.95	84.6		
7	12:21	30	7.95	84.6	0.2	0.00
	12:51		7.97	84.4		
8	12:51	30	7.97	84.4	0.2	0.00
	13:21		7.99	84.1		

Infiltration Rate (I) = Discharge Volume/Surface Area of Test Section/Time Interval

Measured Infiltration Rate, I (Average of Last 3 ReadingsLast Readings) = 0.01 in./hr.

APPENDIX C

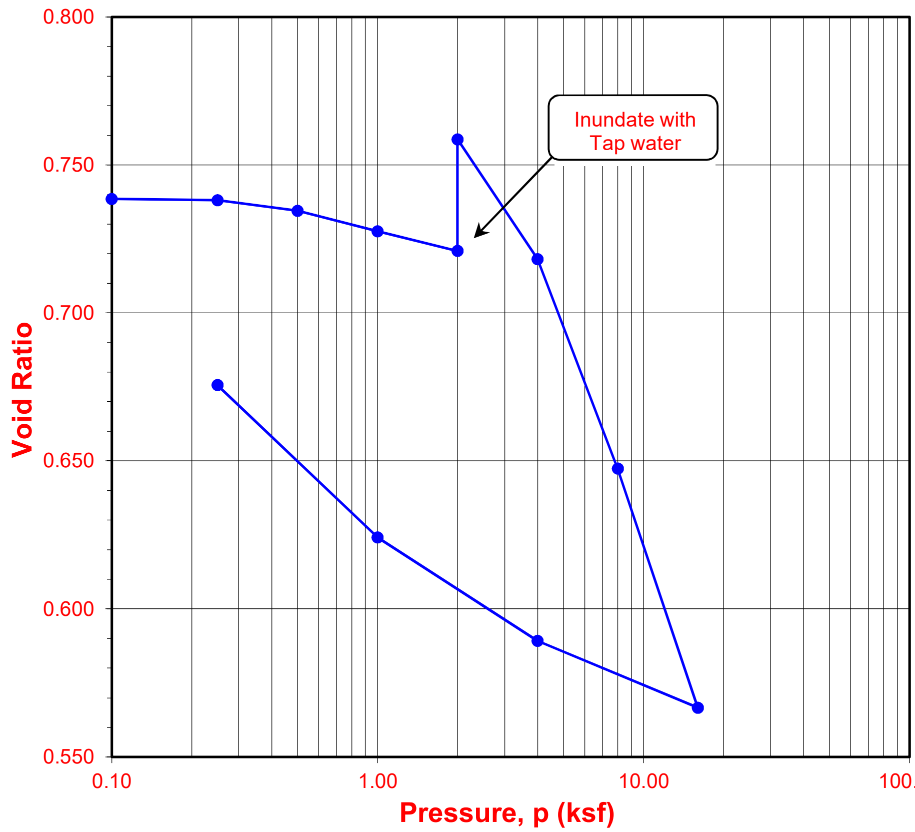
LABORATORY TEST RESULTS

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Leighton

Project Name:	SMMUSD MMHS Phase 1	Tested By:	GB/YN	Date:	08/26/20
Project No.:	11382.016	Checked By:	J. Ward	Date:	09/15/20
Boring No.:	2020-LB-4	Depth (ft.):	0-5		
Sample No.:	B-1	Sample Type:	90% Remold		
Soil Identification:	Dark yellowish brown lean clay with sand (CL)s				

Sample Diameter (in.)	2.415
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	175.82
Weight of Ring (g)	43.23
Height after consol. (in.)	0.9643
Before Test	
Wt.Wet Sample+Cont. (g)	189.90
Wt.of Dry Sample+Cont. (g)	176.18
Weight of Container (g)	75.84
Initial Moisture Content (%)	13.7
Initial Dry Density (pcf)	97.0
Initial Saturation (%)	50
Initial Vertical Reading (in.)	0.2835
After Test	
Wt.of Wet Sample+Cont. (g)	248.25
Wt. of Dry Sample+Cont. (g)	218.66
Weight of Container (g)	59.58
Final Moisture Content (%)	25.54
Final Dry Density (pcf)	99.9
Final Saturation (%)	100
Final Vertical Reading (in.)	0.2443
Specific Gravity (assumed)	2.70
Water Density (pcf)	62.43

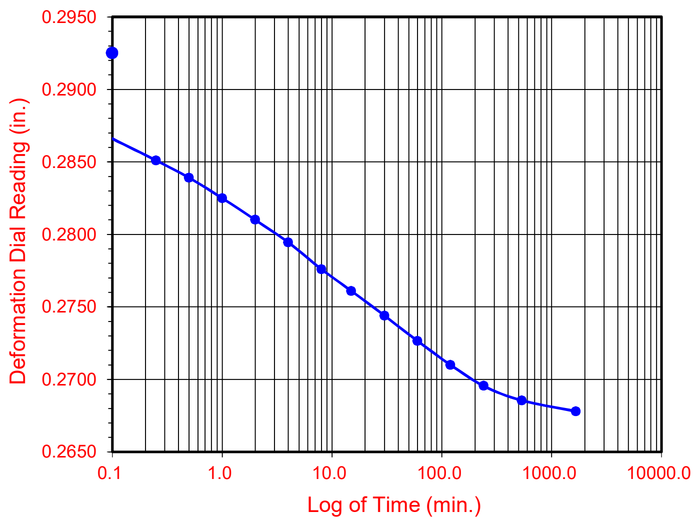


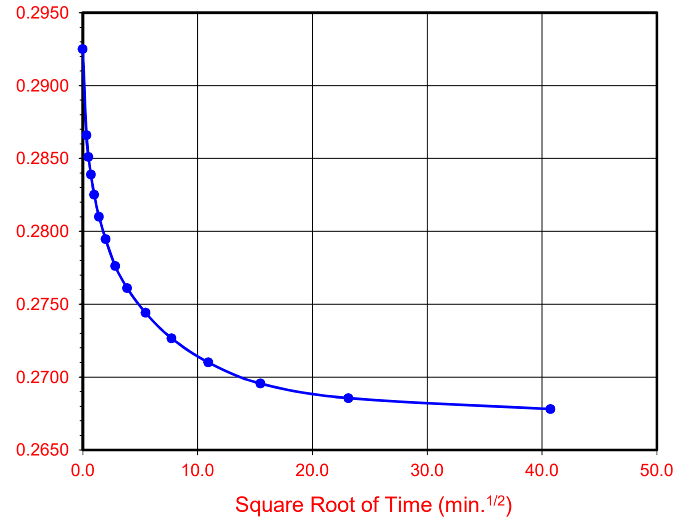
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.2840	1.0005	0.00	-0.05	0.739	-0.05
0.25	0.2833	0.9998	0.05	0.02	0.738	-0.03
0.50	0.2806	0.9971	0.11	0.29	0.735	0.18
1.00	0.2757	0.9922	0.20	0.78	0.728	0.58
2.00	0.2708	0.9873	0.31	1.27	0.721	0.96
2.00	0.2925	1.0090	0.31	-0.90	0.759	-1.21
4.00	0.2678	0.9843	0.45	1.57	0.718	1.12
8.00	0.2255	0.9420	0.61	5.80	0.647	5.19
16.00	0.1770	0.8935	0.81	10.65	0.567	9.84
4.00	0.1914	0.9079	0.67	9.21	0.589	8.54
1.00	0.2133	0.9298	0.49	7.02	0.624	6.53
0.25	0.2443	0.9608	0.35	3.92	0.676	3.57

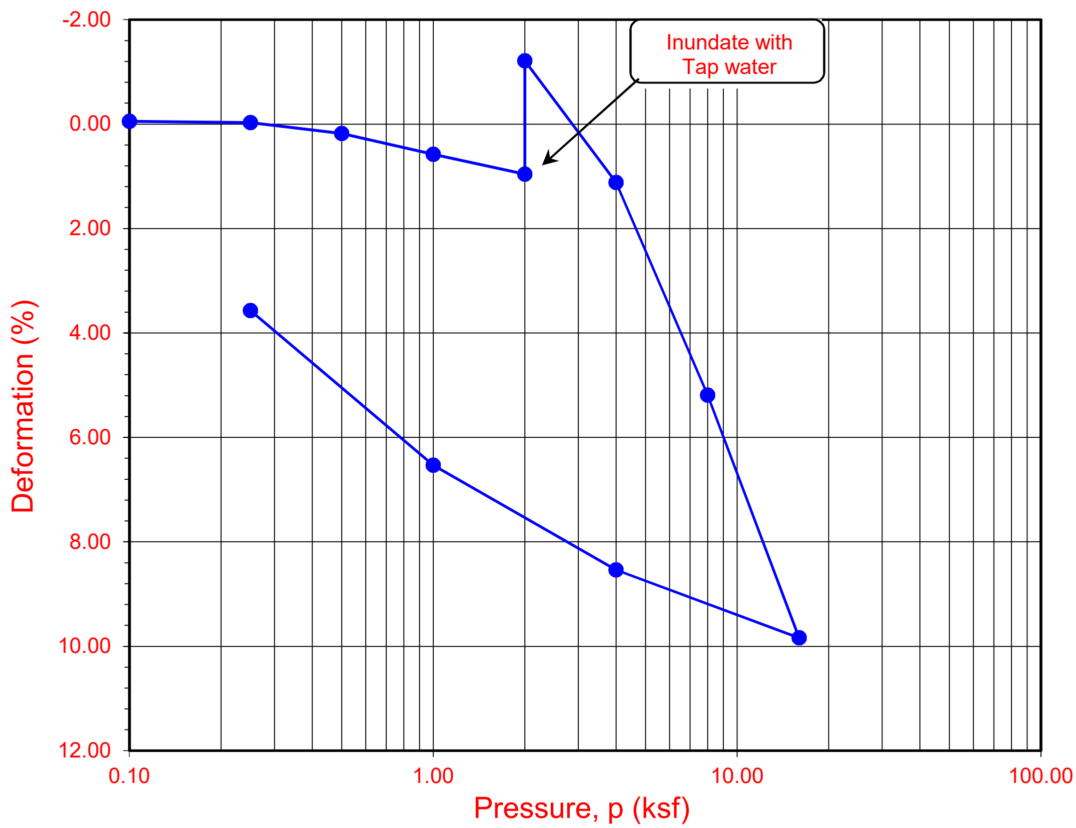
Time Readings @ 4 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
8/30/20	5:20:00	0.0	0.0	0.2925
8/30/20	5:20:06	0.1	0.3	0.2866
8/30/20	5:20:15	0.2	0.5	0.2851
8/30/20	5:20:30	0.5	0.7	0.2839
8/30/20	5:21:00	1.0	1.0	0.2825
8/30/20	5:22:00	2.0	1.4	0.2810
8/30/20	5:24:00	4.0	2.0	0.2795
8/30/20	5:28:00	8.0	2.8	0.2776
8/30/20	5:35:00	15.0	3.9	0.2761
8/30/20	5:50:00	30.0	5.5	0.2744
8/30/20	6:20:00	60.0	7.7	0.2727
8/30/20	7:20:00	120.0	11.0	0.2710
8/30/20	9:20:00	240.0	15.5	0.2696
8/30/20	14:15:00	535.0	23.1	0.2686
8/31/20	9:01:00	1661.0	40.8	0.2678

Time Readings @ 4 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
2020-LB-4	**B-1**	**0-5**	**13.7**	**25.5**	**97.0**	**99.9**	**0.738**	**0.676**	**50**	**100**

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

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 11382.016

SMMUSD MMHS Phase 1

09-20

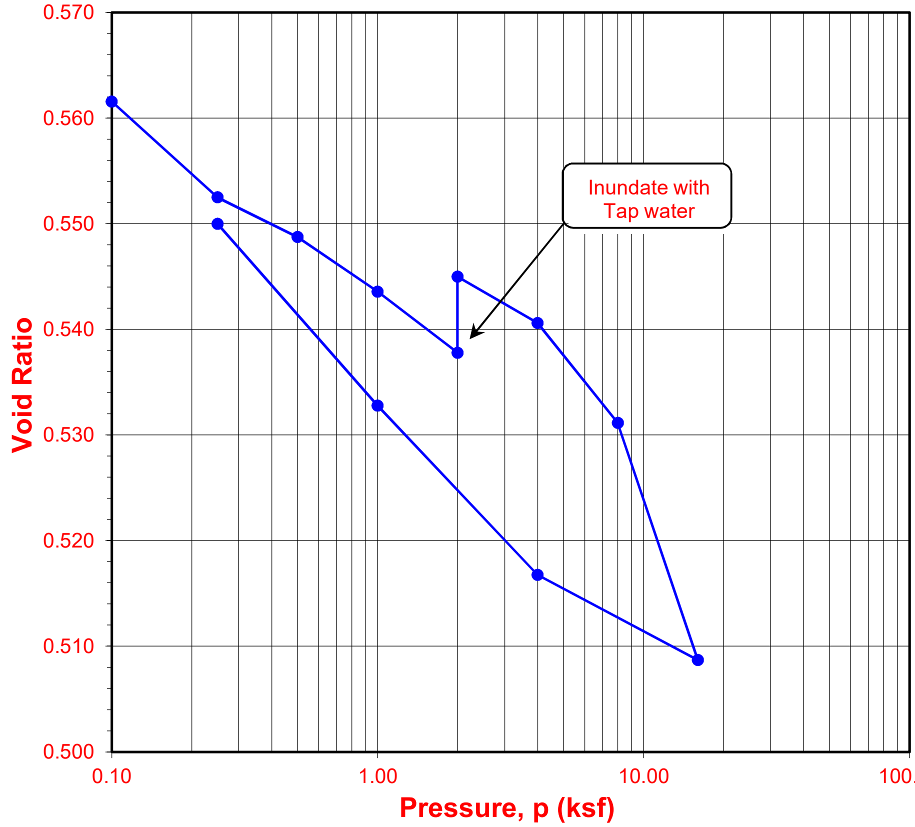
ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Leighton

Project Name:	SMMUSD MMHS Phase 1	Tested By:	GB/YN	Date: 08/22/20
Project No.:	11382.016	Checked By:	J. Ward	Date: 09/14/20
Boring No.:	2020-LB-4	Depth (ft.):	5.0	
Sample No.:	R-1	Sample Type:	Ring	
Soil Identification:	Dark yellowish brown lean clay with sand (CL)s			

Sample Diameter (in.)	2.415
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	200.00
Weight of Ring (g)	44.67
Height after consol. (in.)	0.9914
Before Test	
Wt.Wet Sample+Cont. (g)	181.04
Wt.of Dry Sample+Cont. (g)	162.59
Weight of Container (g)	58.00
Initial Moisture Content (%)	17.6
Initial Dry Density (pcf)	109.8
Initial Saturation (%)	86
Initial Vertical Reading (in.)	0.2600
After Test	
Wt.of Wet Sample+Cont. (g)	262.52
Wt. of Dry Sample+Cont. (g)	234.57
Weight of Container (g)	61.85
Final Moisture Content (%)	21.83
Final Dry Density (pcf)	107.4
Final Saturation (%)	100
Final Vertical Reading (in.)	0.2482
Specific Gravity (assumed)	2.75
Water Density (pcf)	62.43

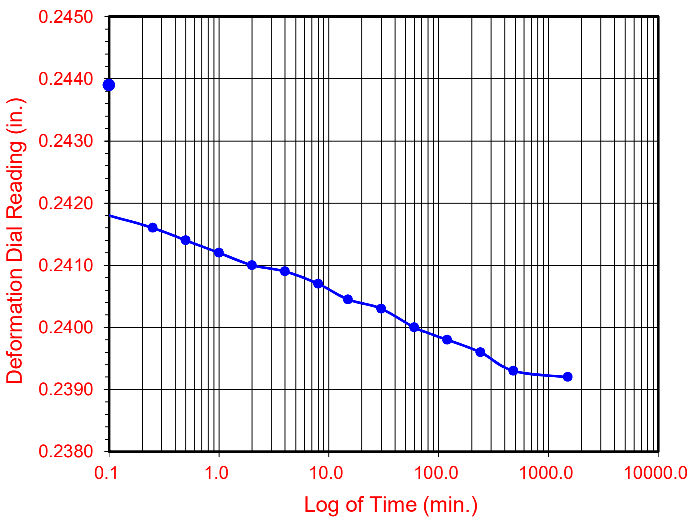


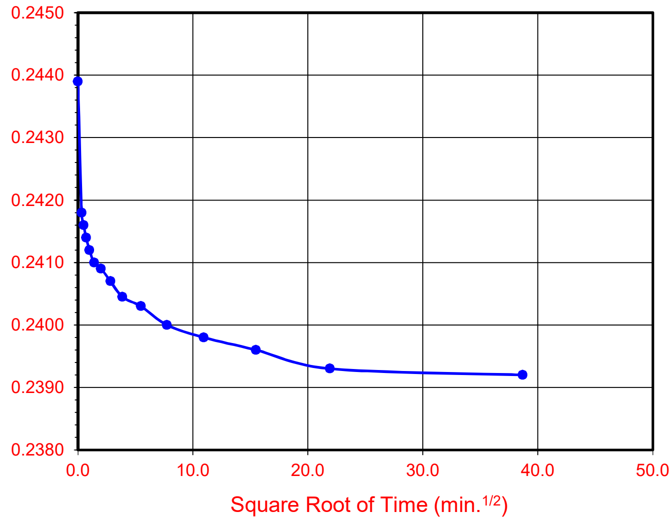
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.2588	0.9988	0.00	0.12	0.562	0.12
0.25	0.2521	0.9921	0.09	0.79	0.552	0.70
0.50	0.2488	0.9888	0.18	1.12	0.549	0.94
1.00	0.2444	0.9844	0.29	1.56	0.544	1.27
2.00	0.2393	0.9793	0.43	2.07	0.538	1.64
2.00	0.2439	0.9839	0.43	1.61	0.545	1.18
4.00	0.2392	0.9792	0.62	2.08	0.541	1.46
8.00	0.2310	0.9710	0.84	2.91	0.531	2.07
16.00	0.2143	0.9543	1.07	4.57	0.509	3.50
4.00	0.2219	0.9619	0.83	3.82	0.517	2.99
1.00	0.2350	0.9750	0.54	2.50	0.533	1.96
0.25	0.2482	0.9882	0.32	1.18	0.550	0.86

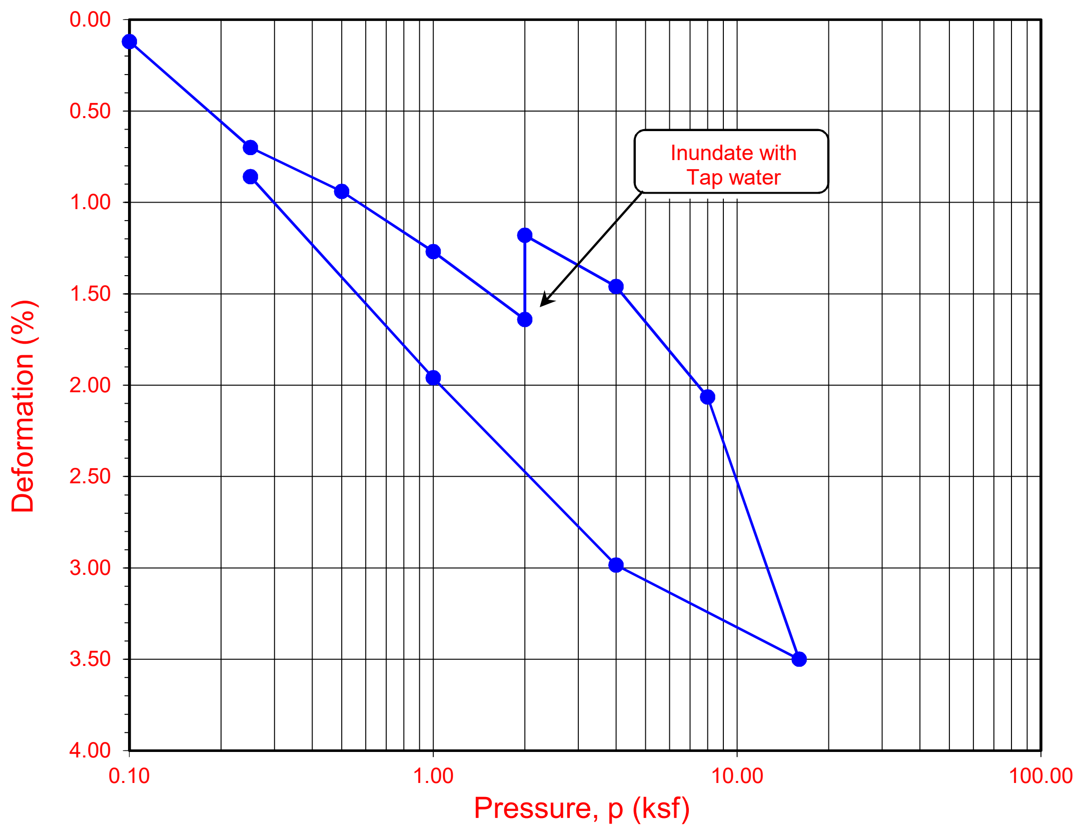
Time Readings @ 4 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
8/25/20	7:25:00	0.0	0.0	0.2439
8/25/20	7:25:06	0.1	0.3	0.2418
8/25/20	7:25:15	0.2	0.5	0.2416
8/25/20	7:25:30	0.5	0.7	0.2414
8/25/20	7:26:00	1.0	1.0	0.2412
8/25/20	7:27:00	2.0	1.4	0.2410
8/25/20	7:29:00	4.0	2.0	0.2409
8/25/20	7:33:00	8.0	2.8	0.2407
8/25/20	7:40:00	15.0	3.9	0.2405
8/25/20	7:55:00	30.0	5.5	0.2403
8/25/20	8:25:00	60.0	7.7	0.2400
8/25/20	9:25:00	120.0	11.0	0.2398
8/25/20	11:25:00	240.0	15.5	0.2396
8/25/20	15:25:00	480.0	21.9	0.2393
8/26/20	8:22:00	1497.0	38.7	0.2392

Time Readings @ 4 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
2020-LB-4	**R-1**	**5.0**	**17.6**	**21.8**	**109.8**	**107.4**	**0.563**	**0.550**	**86**	**100**

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

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 11382.016

SMMUSD MMHS Phase 1

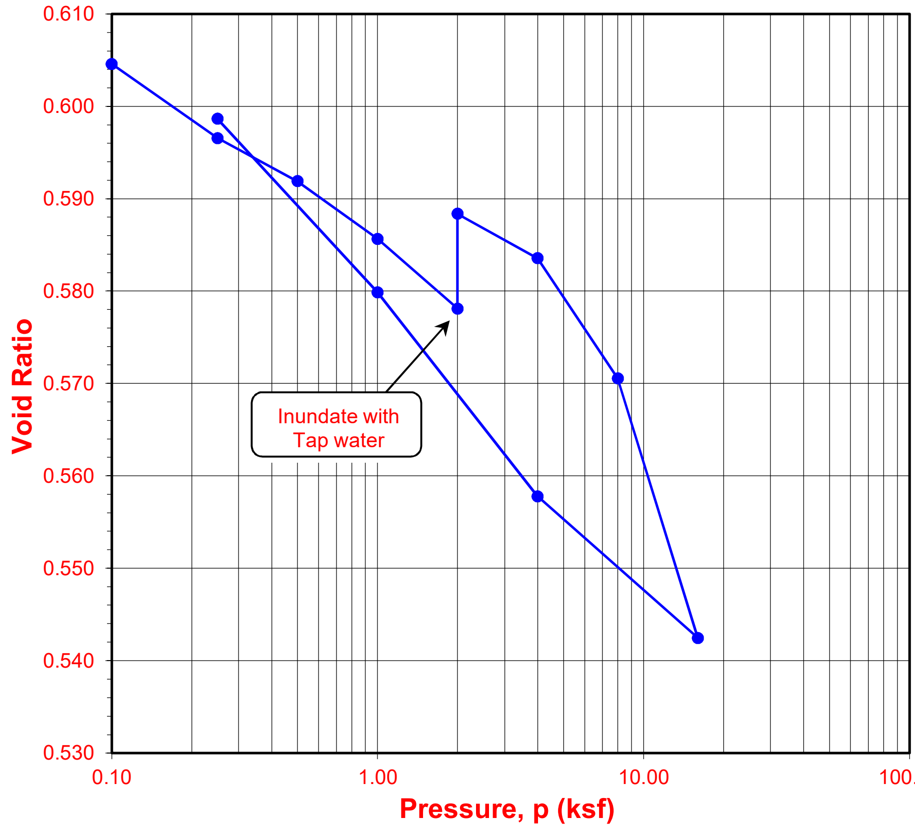
09-20

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Leighton

Project Name:	SMMUSD MMHS Phase 1	Tested By: GB/YN	Date: 08/22/20
Project No.:	11382.016	Checked By: J. Ward	Date: 09/14/20
Boring No.:	2020-LB-4	Depth (ft.): 10.0	
Sample No.:	R-3	Sample Type:	Ring
Soil Identification:	Yellowish brown lean clay (CL), caliche noted		

Sample Diameter (in.)	2.415
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	187.46
Weight of Ring (g)	39.55
Height after consol. (in.)	0.9956
Before Test	
Wt.Wet Sample+Cont. (g)	176.00
Wt.of Dry Sample+Cont. (g)	155.43
Weight of Container (g)	35.70
Initial Moisture Content (%)	17.2
Initial Dry Density (pcf)	105.0
Initial Saturation (%)	77
Initial Vertical Reading (in.)	0.2774
After Test	
Wt.of Wet Sample+Cont. (g)	247.00
Wt. of Dry Sample+Cont. (g)	216.42
Weight of Container (g)	58.00
Final Moisture Content (%)	25.73
Final Dry Density (pcf)	99.3
Final Saturation (%)	100
Final Vertical Reading (in.)	0.2713
Specific Gravity (assumed)	2.70
Water Density (pcf)	62.43

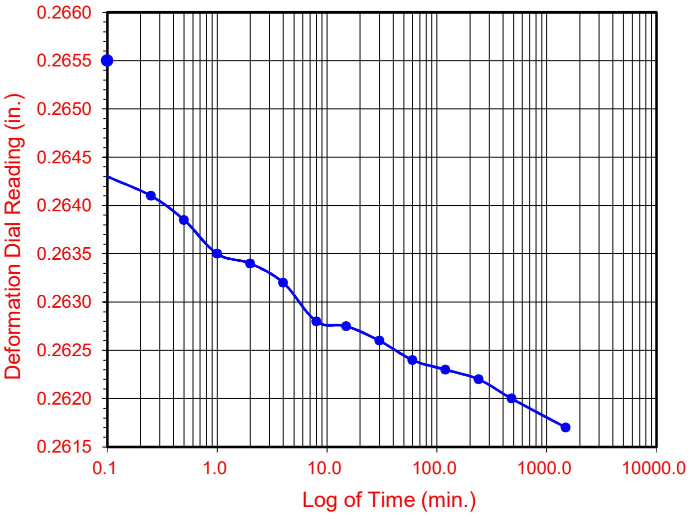


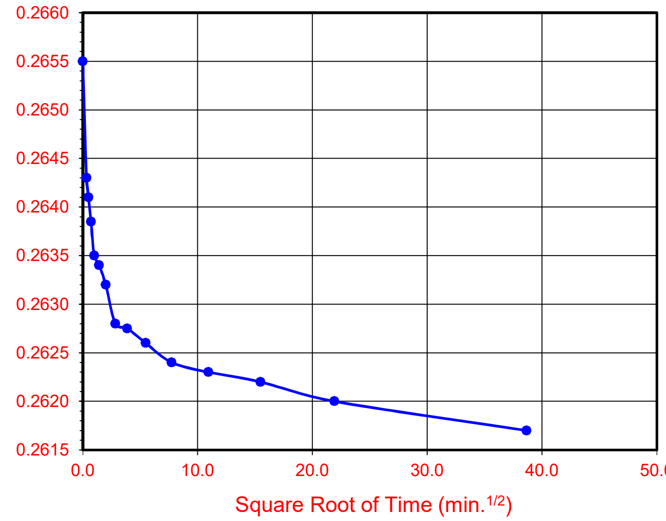
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.2767	0.9993	0.00	0.07	0.605	0.07
0.25	0.2716	0.9942	0.01	0.58	0.597	0.57
0.50	0.2685	0.9911	0.03	0.89	0.592	0.86
1.00	0.2643	0.9869	0.06	1.31	0.586	1.25
2.00	0.2591	0.9817	0.11	1.83	0.578	1.72
2.00	0.2655	0.9881	0.11	1.19	0.588	1.08
4.00	0.2617	0.9843	0.19	1.57	0.584	1.38
8.00	0.2522	0.9748	0.33	2.52	0.571	2.19
16.00	0.2328	0.9554	0.52	4.46	0.542	3.94
4.00	0.2440	0.9666	0.36	3.35	0.558	2.99
1.00	0.2590	0.9816	0.23	1.84	0.580	1.61
0.25	0.2713	0.9939	0.17	0.61	0.599	0.44

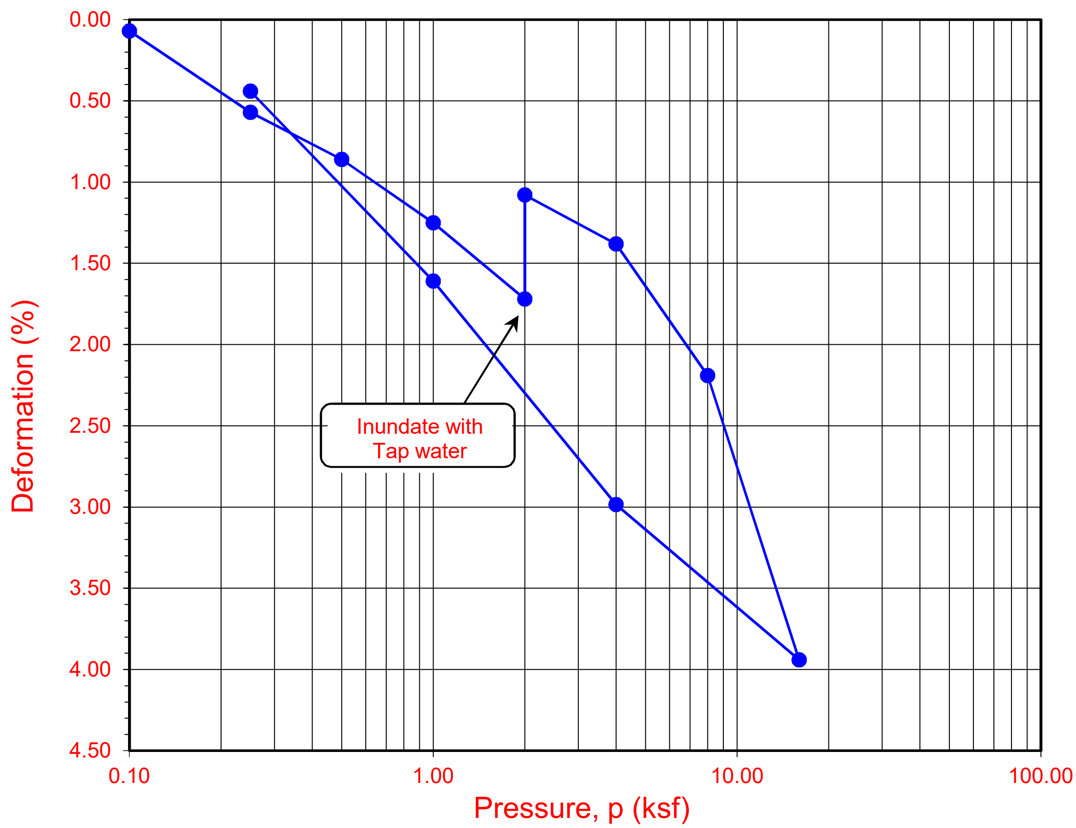
Time Readings @ 4 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
8/25/20	7:30:00	0.0	0.0	0.2655
8/25/20	7:30:06	0.1	0.3	0.2643
8/25/20	7:30:15	0.2	0.5	0.2641
8/25/20	7:30:30	0.5	0.7	0.2639
8/25/20	7:31:00	1.0	1.0	0.2635
8/25/20	7:32:00	2.0	1.4	0.2634
8/25/20	7:34:00	4.0	2.0	0.2632
8/25/20	7:38:00	8.0	2.8	0.2628
8/25/20	7:45:00	15.0	3.9	0.2628
8/25/20	8:00:00	30.0	5.5	0.2626
8/25/20	8:30:00	60.0	7.7	0.2624
8/25/20	9:30:00	120.0	11.0	0.2623
8/25/20	11:30:00	240.0	15.5	0.2622
8/25/20	15:30:00	480.0	21.9	0.2620
8/26/20	8:24:00	1494.0	38.7	0.2617

Time Readings @ 4 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
2020-LB-4	**R-3**	**10.0**	**17.2**	**25.7**	**105.0**	**99.3**	**0.606**	**0.599**	**77**	**100**

Soil Identification: Yellowish brown lean clay (CL), caliche noted

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 11382.016

SMMUSD MMHS Phase 1

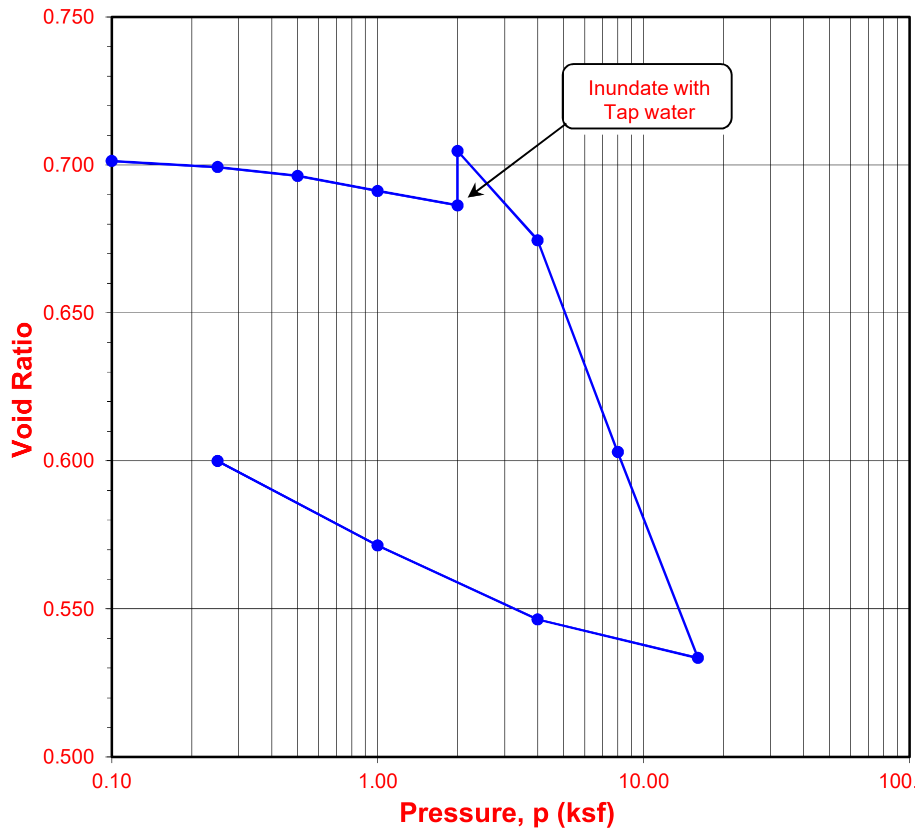
09-20

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Leighton

Project Name:	SMMUSD MMHS Phase 1	Tested By: GB/YN	Date:	08/26/20
Project No.:	11382.016	Checked By: J. Ward	Date:	09/15/20
Boring No.:	2020-LB-6	Depth (ft.): 0-5		
Sample No.:	B-1	Sample Type:	90% Remold	
Soil Identification:	Dark yellowish brown lean clay with sand (CL)s			

Sample Diameter (in.)	2.415
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	173.83
Weight of Ring (g)	40.69
Height after consol. (in.)	0.9402
Before Test	
Wt.Wet Sample+Cont. (g)	214.82
Wt.of Dry Sample+Cont. (g)	200.37
Weight of Container (g)	77.79
Initial Moisture Content (%)	11.8
Initial Dry Density (pcf)	99.1
Initial Saturation (%)	45
Initial Vertical Reading (in.)	0.3294
After Test	
Wt.of Wet Sample+Cont. (g)	255.91
Wt. of Dry Sample+Cont. (g)	230.20
Weight of Container (g)	70.38
Final Moisture Content (%)	21.58
Final Dry Density (pcf)	105.4
Final Saturation (%)	97
Final Vertical Reading (in.)	0.2663
Specific Gravity (assumed)	2.70
Water Density (pcf)	62.43

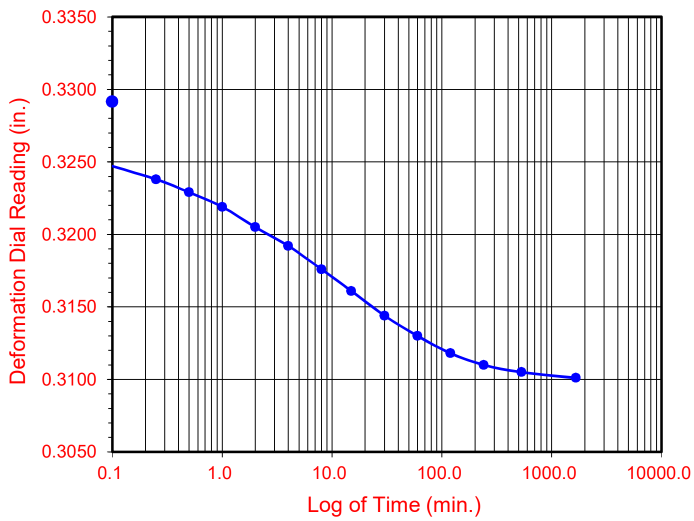


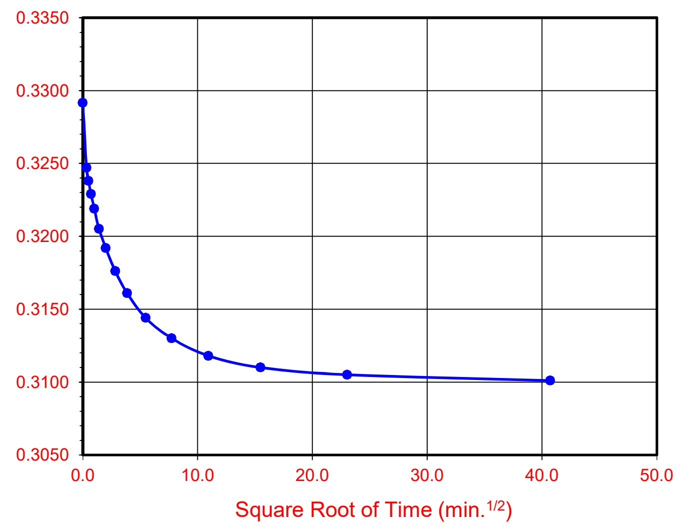
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.3292	0.9998	0.00	0.03	0.701	0.03
0.25	0.3277	0.9983	0.03	0.18	0.699	0.15
0.50	0.3256	0.9962	0.06	0.38	0.696	0.32
1.00	0.3221	0.9927	0.11	0.73	0.691	0.62
2.00	0.3184	0.9890	0.20	1.11	0.686	0.91
2.00	0.3292	0.9998	0.20	0.03	0.705	-0.17
4.00	0.3101	0.9807	0.33	1.93	0.675	1.60
8.00	0.2666	0.9372	0.48	6.28	0.603	5.80
16.00	0.2238	0.8944	0.67	10.56	0.533	9.89
4.00	0.2332	0.9038	0.50	9.63	0.546	9.13
1.00	0.2489	0.9195	0.39	8.05	0.571	7.66
0.25	0.2663	0.9369	0.33	6.31	0.600	5.98

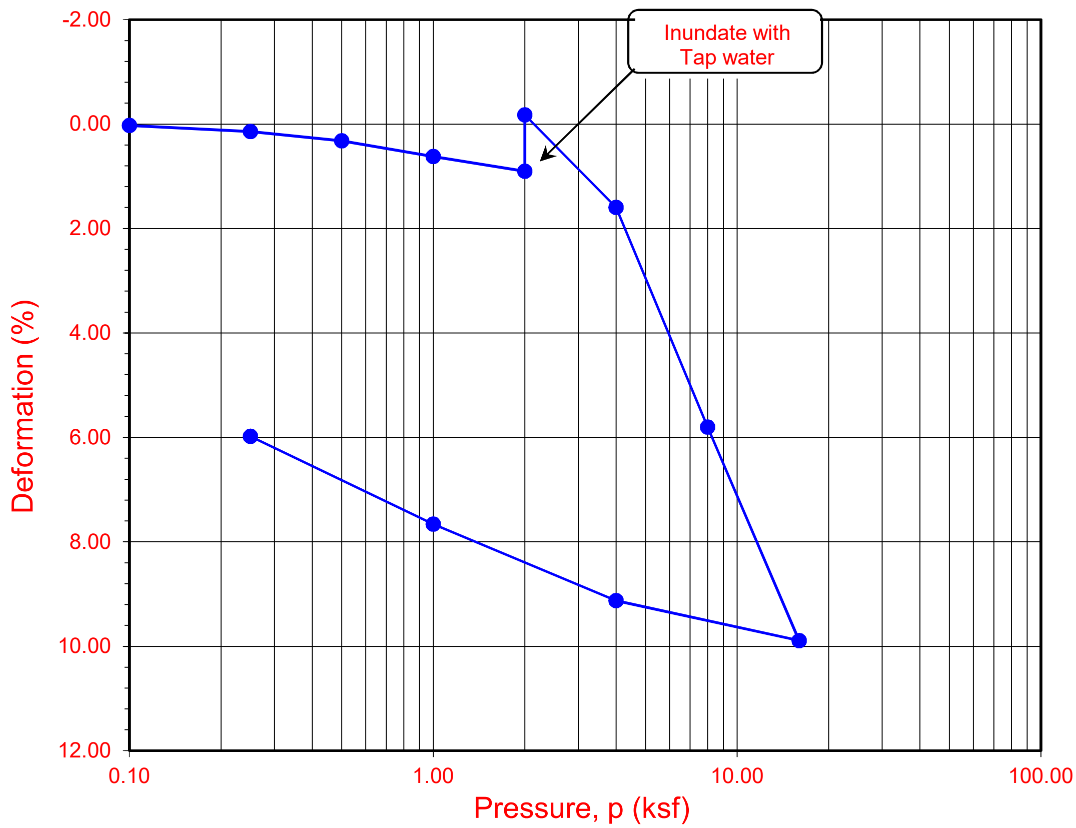
Time Readings @ 4 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
8/30/20	5:25:00	0.0	0.0	0.3292
8/30/20	5:25:06	0.1	0.3	0.3247
8/30/20	5:25:15	0.2	0.5	0.3238
8/30/20	5:25:30	0.5	0.7	0.3229
8/30/20	5:26:00	1.0	1.0	0.3219
8/30/20	5:27:00	2.0	1.4	0.3205
8/30/20	5:29:00	4.0	2.0	0.3192
8/30/20	5:33:00	8.0	2.8	0.3176
8/30/20	5:40:00	15.0	3.9	0.3161
8/30/20	5:55:00	30.0	5.5	0.3144
8/30/20	6:25:00	60.0	7.7	0.3130
8/30/20	7:25:00	120.0	11.0	0.3118
8/30/20	9:25:00	240.0	15.5	0.3110
8/30/20	14:15:00	530.0	23.0	0.3105
8/31/20	9:03:00	1658.0	40.7	0.3101

Time Readings @ 4 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
2020-LB-6	**B-1**	**0-5**	**11.8**	**21.6**	**99.1**	**105.4**	**0.702**	**0.600**	**45**	**97**

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

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 11382.016

SMMUSD MMHS Phase 1

09-20

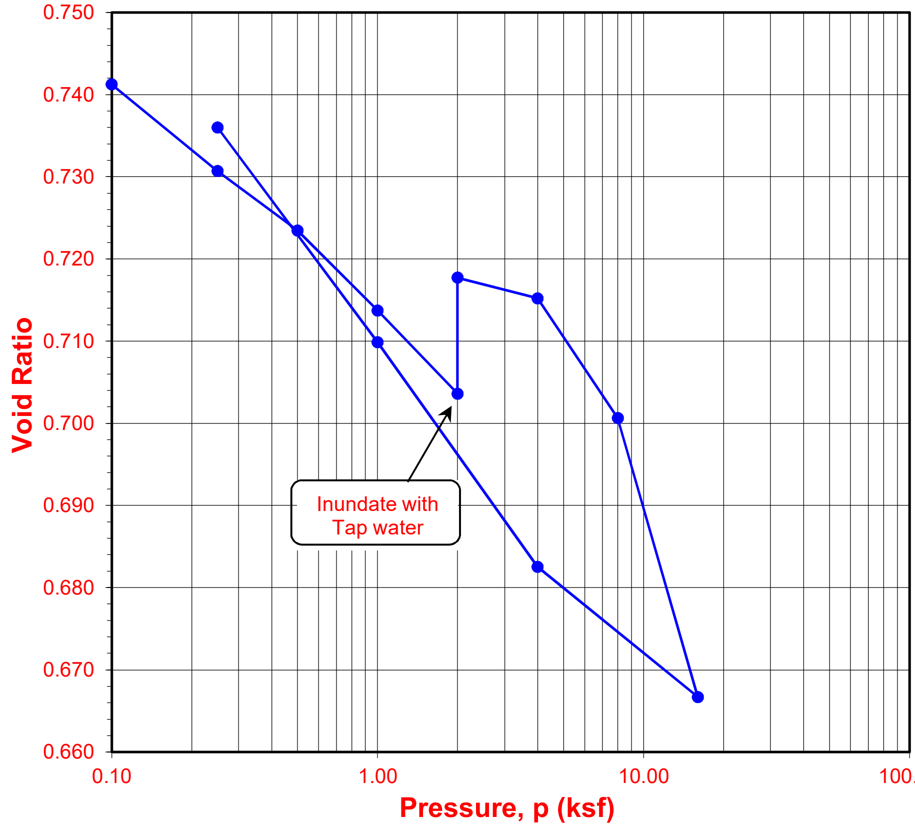
ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Leighton

Project Name:	SMMUSD MMHS Phase 1	Tested By:	GB/YN	Date:	08/22/20
Project No.:	11382.016	Checked By:	J. Ward	Date:	09/14/20
Boring No.:	2020-LB-6	Depth (ft.):	7.0		
Sample No.:	R-2	Sample Type:	Ring		
Soil Identification:	Dark yellowish brown fat clay (CH), caliche noted				

Sample Diameter (in.)	2.415
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	192.94
Weight of Ring (g)	42.48
Height after consol. (in.)	0.9967
Before Test	
Wt.Wet Sample+Cont. (g)	203.31
Wt.of Dry Sample+Cont. (g)	174.40
Weight of Container (g)	57.27
Initial Moisture Content (%)	24.7
Initial Dry Density (pcf)	100.4
Initial Saturation (%)	93
Initial Vertical Reading (in.)	0.2929
After Test	
Wt.of Wet Sample+Cont. (g)	229.56
Wt. of Dry Sample+Cont. (g)	197.21
Weight of Container (g)	35.71
Final Moisture Content (%)	27.18
Final Dry Density (pcf)	99.3
Final Saturation (%)	100
Final Vertical Reading (in.)	0.2869
Specific Gravity (assumed)	2.80
Water Density (pcf)	62.43

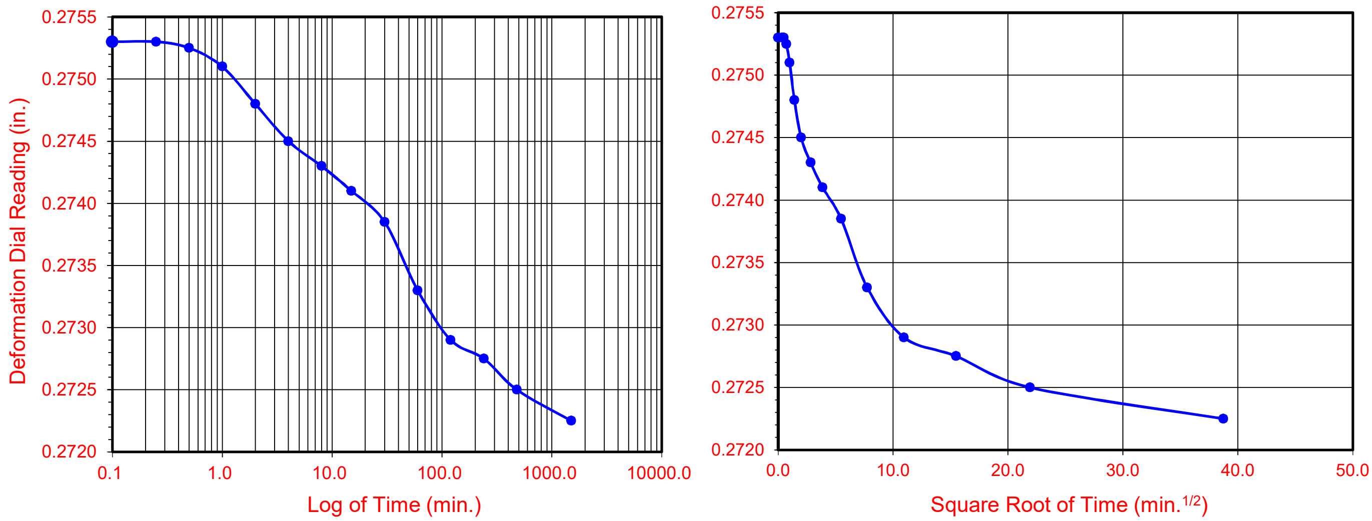


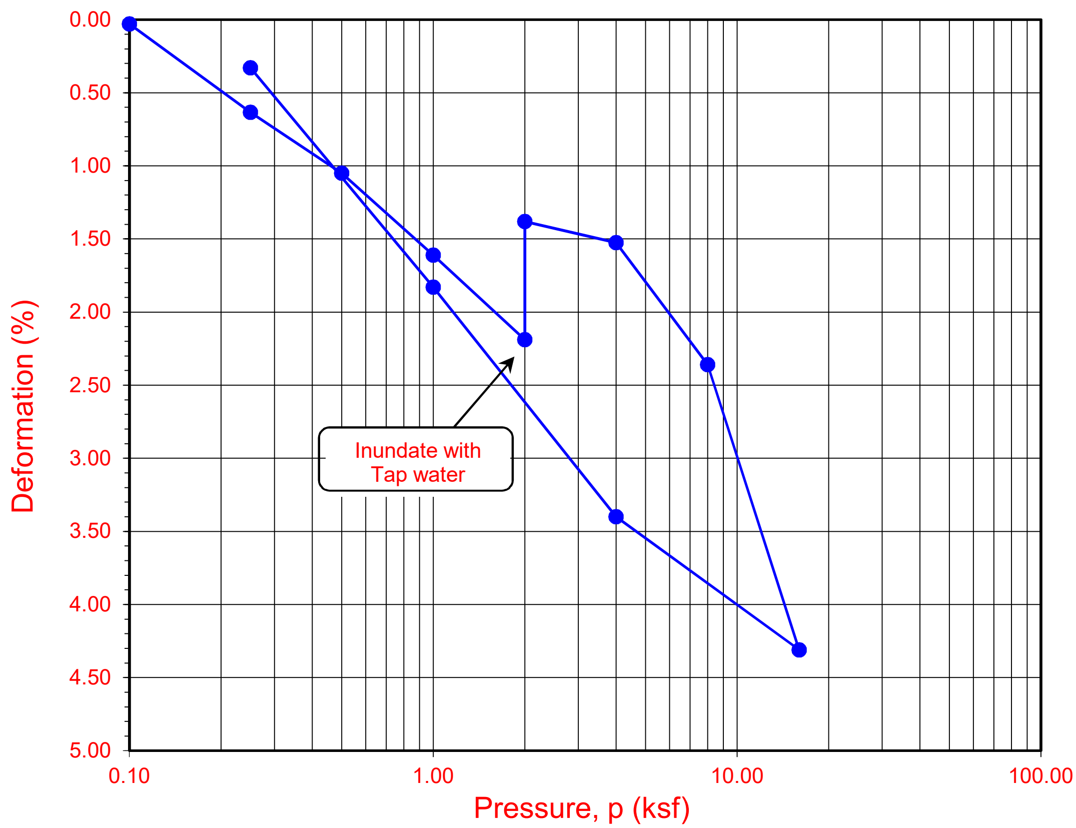
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.2926	0.9997	0.00	0.03	0.741	0.03
0.25	0.2861	0.9932	0.05	0.69	0.731	0.64
0.50	0.2811	0.9882	0.13	1.18	0.723	1.05
1.00	0.2745	0.9816	0.23	1.84	0.714	1.61
2.00	0.2672	0.9743	0.38	2.57	0.704	2.19
2.00	0.2753	0.9824	0.38	1.76	0.718	1.38
4.00	0.2723	0.9794	0.54	2.07	0.715	1.53
8.00	0.2621	0.9692	0.72	3.08	0.701	2.36
16.00	0.2404	0.9475	0.94	5.25	0.667	4.31
4.00	0.2517	0.9588	0.72	4.12	0.683	3.40
1.00	0.2698	0.9769	0.48	2.31	0.710	1.83
0.25	0.2869	0.9940	0.27	0.60	0.736	0.33

Time Readings @ 4 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
8/25/20	7:20:00	0.0	0.0	0.2753
8/25/20	7:20:06	0.1	0.3	0.2753
8/25/20	7:20:15	0.2	0.5	0.2753
8/25/20	7:20:30	0.5	0.7	0.2753
8/25/20	7:21:00	1.0	1.0	0.2751
8/25/20	7:22:00	2.0	1.4	0.2748
8/25/20	7:24:00	4.0	2.0	0.2745
8/25/20	7:28:00	8.0	2.8	0.2743
8/25/20	7:35:00	15.0	3.9	0.2741
8/25/20	7:50:00	30.0	5.5	0.2739
8/25/20	8:20:00	60.0	7.7	0.2733
8/25/20	9:20:00	120.0	11.0	0.2729
8/25/20	11:20:00	240.0	15.5	0.2728
8/25/20	15:20:00	480.0	21.9	0.2725
8/26/20	8:20:00	1500.0	38.7	0.2723

Time Readings @ 4 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
2020-LB-6	**R-2**	**7.0**	**24.7**	**27.2**	**100.4**	**99.3**	**0.742**	**0.736**	**93**	**100**

Soil Identification: Dark yellowish brown fat clay (CH), caliche noted

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 11382.016

SMMUSD MMHS Phase 1

09-20

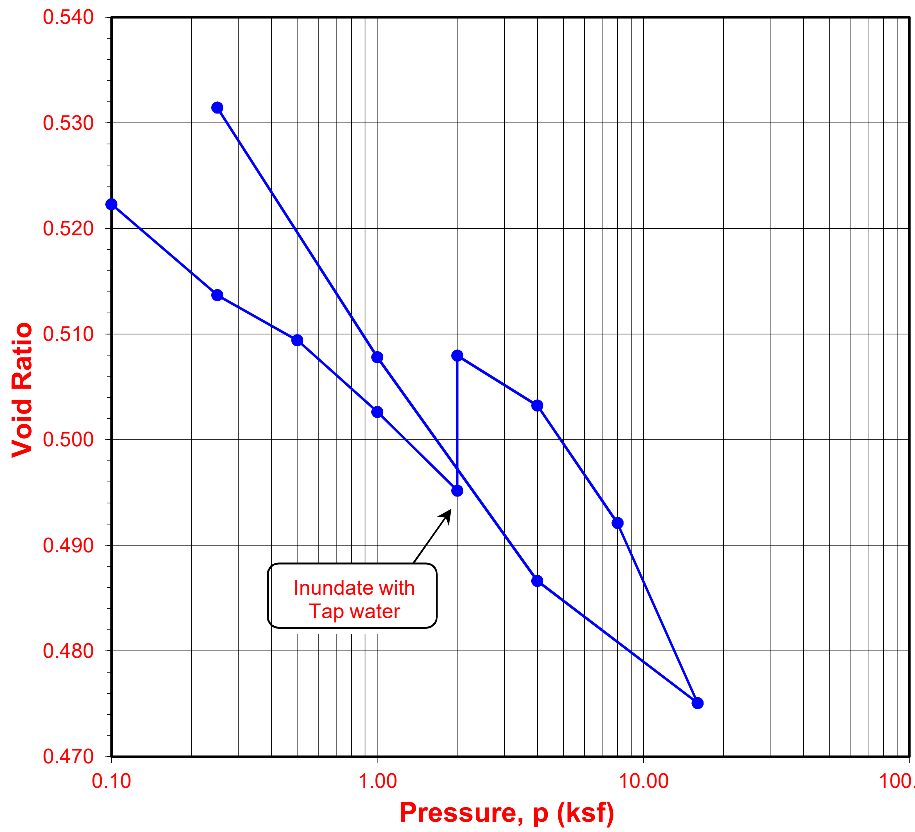
ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Leighton

Project Name:	SMMUSD MMHS Phase 1	Tested By:	GB/YN	Date: 08/22/20
Project No.:	11382.016	Checked By:	J. Ward	Date: 09/14/20
Boring No.:	2020-LB-6	Depth (ft.):	15.0	
Sample No.:	R-4	Sample Type:	Ring	
Soil Identification:	Dark yellowish brown lean clay (CL), caliche noted			

Sample Diameter (in.)	2.415
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	199.63
Weight of Ring (g)	41.73
Height after consol. (in.)	1.0055
Before Test	
Wt.Wet Sample+Cont. (g)	187.85
Wt.of Dry Sample+Cont. (g)	170.40
Weight of Container (g)	61.85
Initial Moisture Content (%)	16.1
Initial Dry Density (pcf)	113.1
Initial Saturation (%)	85
Initial Vertical Reading (in.)	0.2891
After Test	
Wt.of Wet Sample+Cont. (g)	269.89
Wt. of Dry Sample+Cont. (g)	242.33
Weight of Container (g)	68.28
Final Moisture Content (%)	20.83
Final Dry Density (pcf)	109.4
Final Saturation (%)	100
Final Vertical Reading (in.)	0.2934
Specific Gravity (assumed)	2.76
Water Density (pcf)	62.43

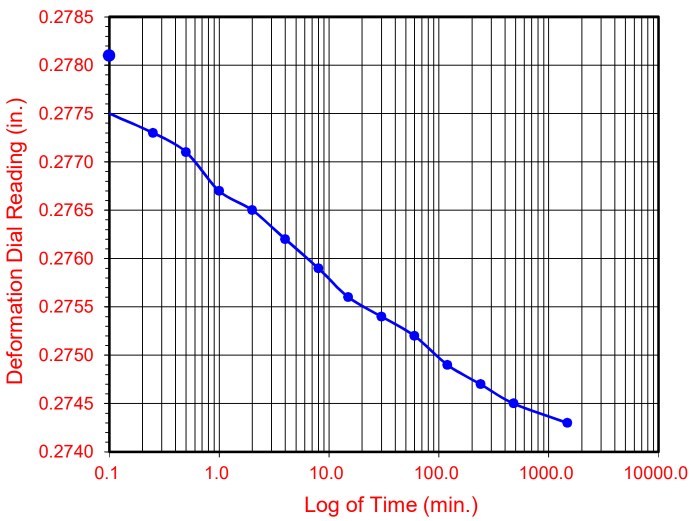


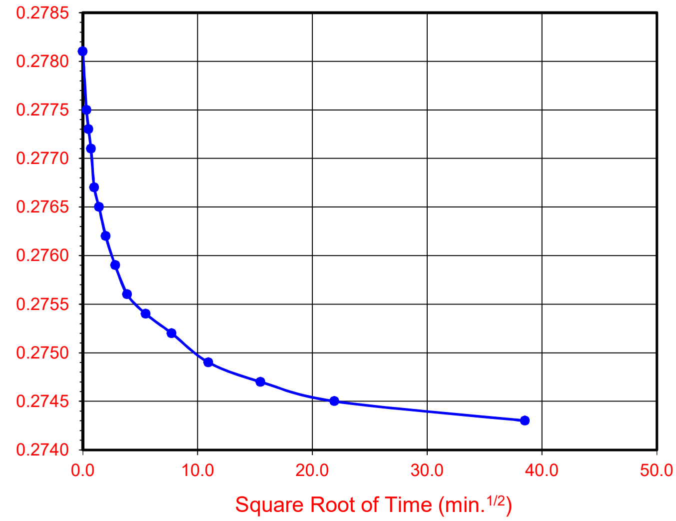
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.2886	0.9995	0.00	0.05	0.522	0.05
0.25	0.2828	0.9937	0.02	0.64	0.514	0.62
0.50	0.2798	0.9907	0.04	0.93	0.509	0.89
1.00	0.2750	0.9859	0.07	1.41	0.503	1.34
2.00	0.2697	0.9806	0.11	1.94	0.495	1.83
2.00	0.2781	0.9890	0.11	1.10	0.508	0.99
4.00	0.2743	0.9852	0.18	1.48	0.503	1.30
8.00	0.2660	0.9769	0.28	2.31	0.492	2.03
16.00	0.2535	0.9644	0.41	3.56	0.475	3.15
4.00	0.2622	0.9731	0.30	2.69	0.487	2.39
1.00	0.2772	0.9881	0.19	1.19	0.508	1.00
0.25	0.2934	1.0043	0.12	-0.43	0.531	-0.55

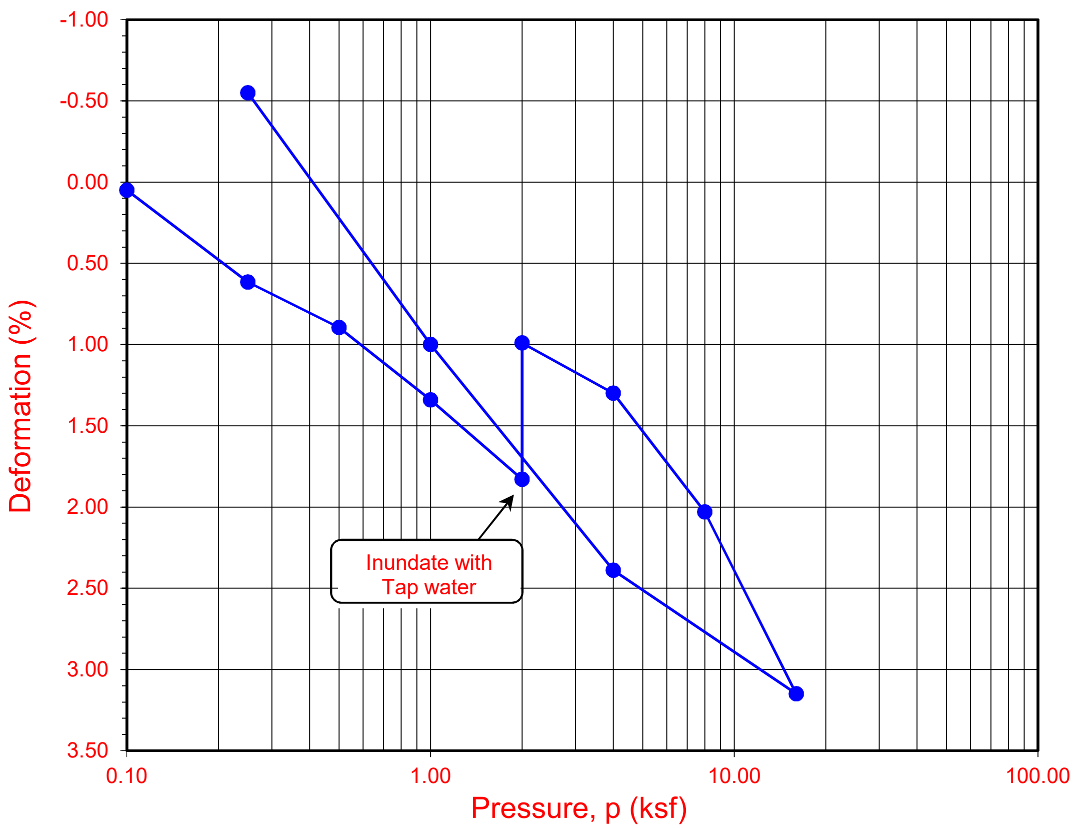
Time Readings @ 4 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
8/25/20	7:42:00	0.0	0.0	0.2781
8/25/20	7:42:06	0.1	0.3	0.2775
8/25/20	7:42:15	0.2	0.5	0.2773
8/25/20	7:42:30	0.5	0.7	0.2771
8/25/20	7:43:00	1.0	1.0	0.2767
8/25/20	7:44:00	2.0	1.4	0.2765
8/25/20	7:46:00	4.0	2.0	0.2762
8/25/20	7:50:00	8.0	2.8	0.2759
8/25/20	7:57:00	15.0	3.9	0.2756
8/25/20	8:12:00	30.0	5.5	0.2754
8/25/20	8:42:00	60.0	7.7	0.2752
8/25/20	9:42:00	120.0	11.0	0.2749
8/25/20	11:42:00	240.0	15.5	0.2747
8/25/20	15:42:00	480.0	21.9	0.2745
8/26/20	8:26:00	1484.0	38.5	0.2743

Time Readings @ 4 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
2020-LB-6	**R-4**	**15.0**	**16.1**	**20.8**	**113.1**	**109.4**	**0.523**	**0.531**	**85**	**100**

Soil Identification: Dark yellowish brown lean clay (CL), caliche noted

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 11382.016

SMMUSD MMHS Phase 1

09-20

Leighton

TESTS for SULFATE CONTENT CHLORIDE CONTENT and pH of SOILS

Project Name: SMMUSD MMHS Phase 1 Tested By : G. Berdy Date: 08/25/20

Project No. : 11382.016 Checked By: J. Ward Date: 09/15/20

Boring No.	2020-LB-4	2020-LB-6		
Sample No.	B-1	B-1		
Sample Depth (ft)	0-5	0-5		
Soil Identification:	Dark yellowish brown (CL)s	Dark yellowish brown (CL)s		
Wet Weight of Soil + Container (g)	0.00	0.00		
Dry Weight of Soil + Container (g)	0.00	0.00		
Weight of Container (g)	1.00	1.00		
Moisture Content (%)	0.00	0.00		
Weight of Soaked Soil (g)	100.23	100.27		

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	17	152		
Crucible No.	21	12		
Furnace Temperature (°C)	860	860		
Time In / Time Out	8:15/9:00	8:15/9:00		
Duration of Combustion (min)	45	45		
Wt. of Crucible + Residue (g)	22.1668	20.7479		
Wt. of Crucible (g)	22.1632	20.7422		
Wt. of Residue (g) (A)	0.0036	0.0057		
PPM of Sulfate (A) x 41150	148.14	234.56		
PPM of Sulfate, Dry Weight Basis	**148**	**235**		

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	15	15		
ml of AgNO3 Soln. Used in Titration (C)	0.6	1.4		
PPM of Chloride (C -0.2) * 100 * 30 / B	80	240		
PPM of Chloride, Dry Wt. Basis	**80**	**240**		

pH TEST, DOT California Test 643

pH Value	**8.14**	**8.50**		
Temperature °C	20.1	20.1		

SOIL RESISTIVITY TEST

DOT CA TEST 643

Leighton

Project Name:	SMMUSD MMHS Phase 1	Tested By :	G. Berdy	Date:	08/28/20
Project No. :	11382.016	Checked By:	J. Ward	Date:	09/15/20
Boring No.:	2020-LB-4	Depth (ft.) :	0-5		
Sample No. :	B-1				

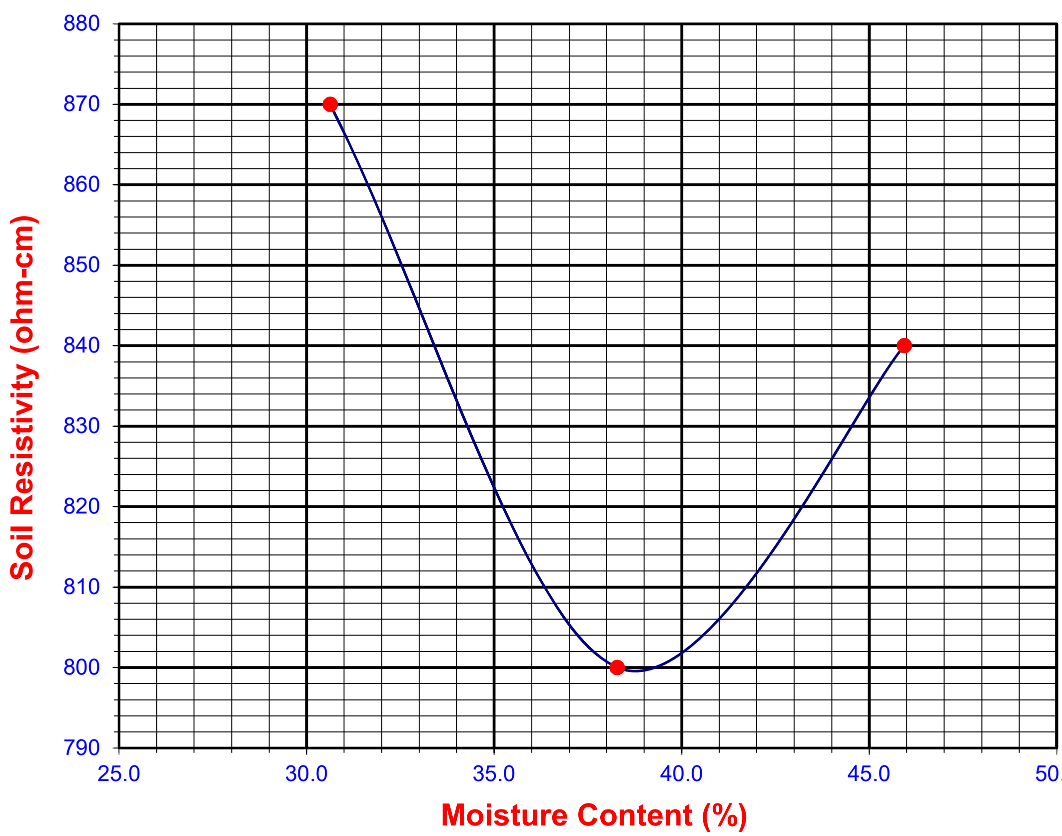
Soil Identification:* Dark yellowish brown (CL)s

*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	40	30.63	870	870
2	50	38.28	800	800
3	60	45.94	840	840
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)	130.60
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	
800	**38.8**	**148**	**80**	**8.14**	**20.1**



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SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name:	SMMUSD MMHS Phase 1	Tested By :	G. Berdy Date: 08/28/20
Project No. :	11382.016	Checked By:	J. Ward Date: 09/15/20
Boring No.:	2020-LB-6	Depth (ft.) :	0-5
Sample No. :	B-1		

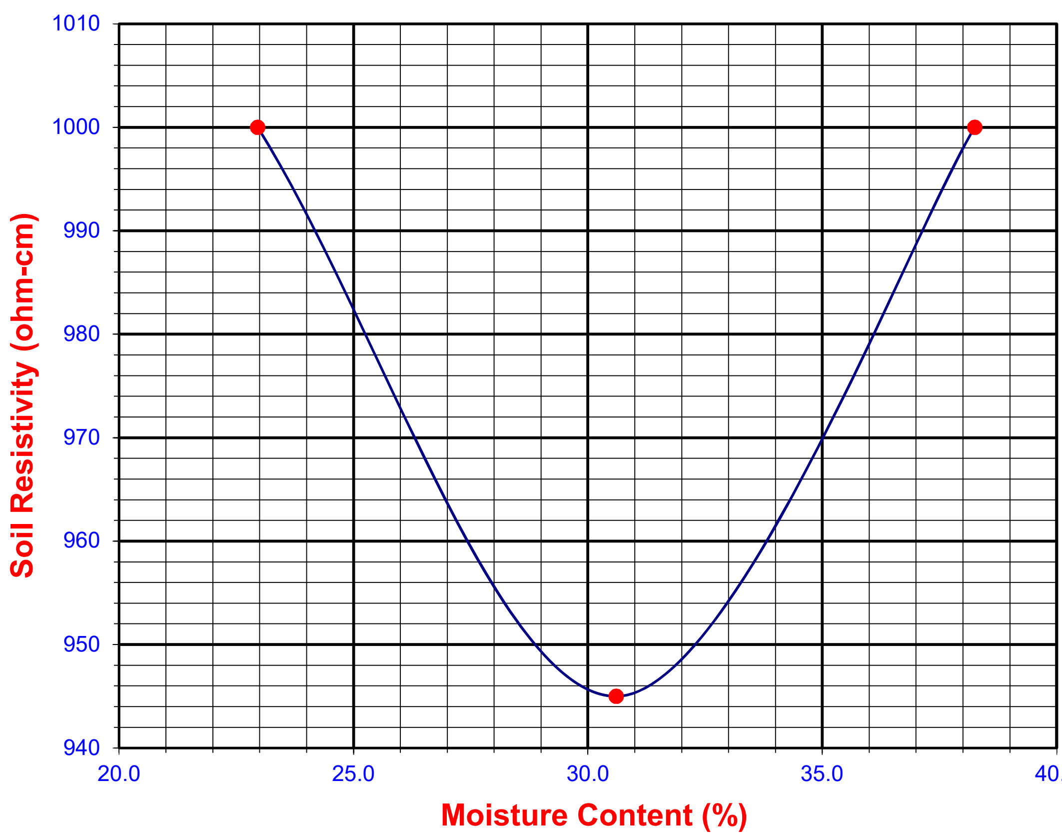
Soil Identification:* Dark yellowish brown (CL)s

*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	22.95	1000	1000
2	40	30.60	945	945
3	50	38.26	1000	1000
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)	130.70
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	
945	**30.6**	**235**	**240**	**8.50**	**20.1**



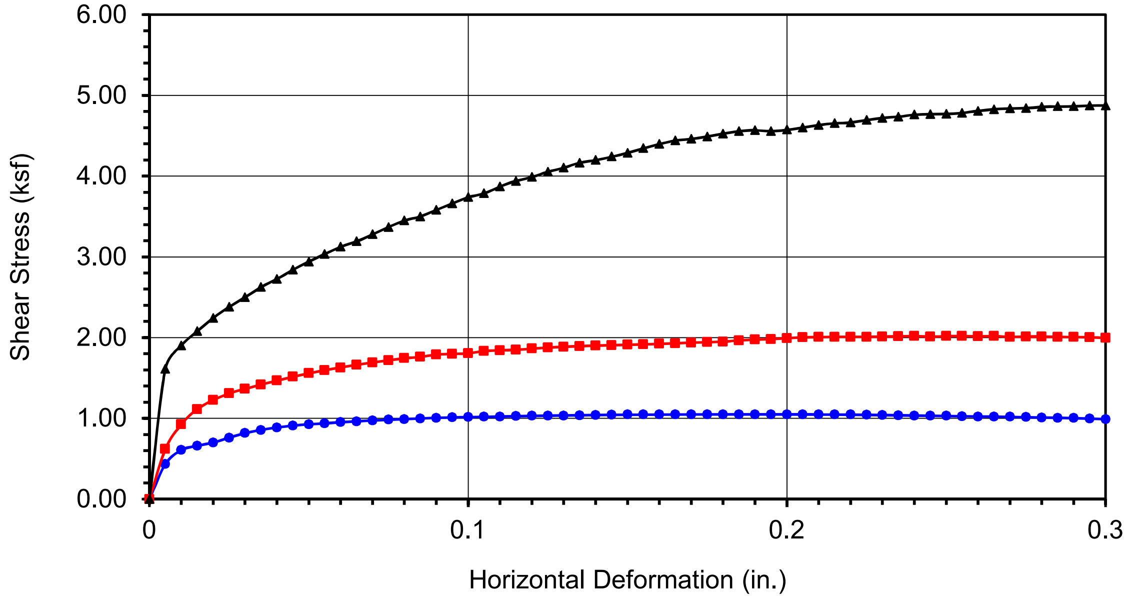
DIRECT SHEAR TEST

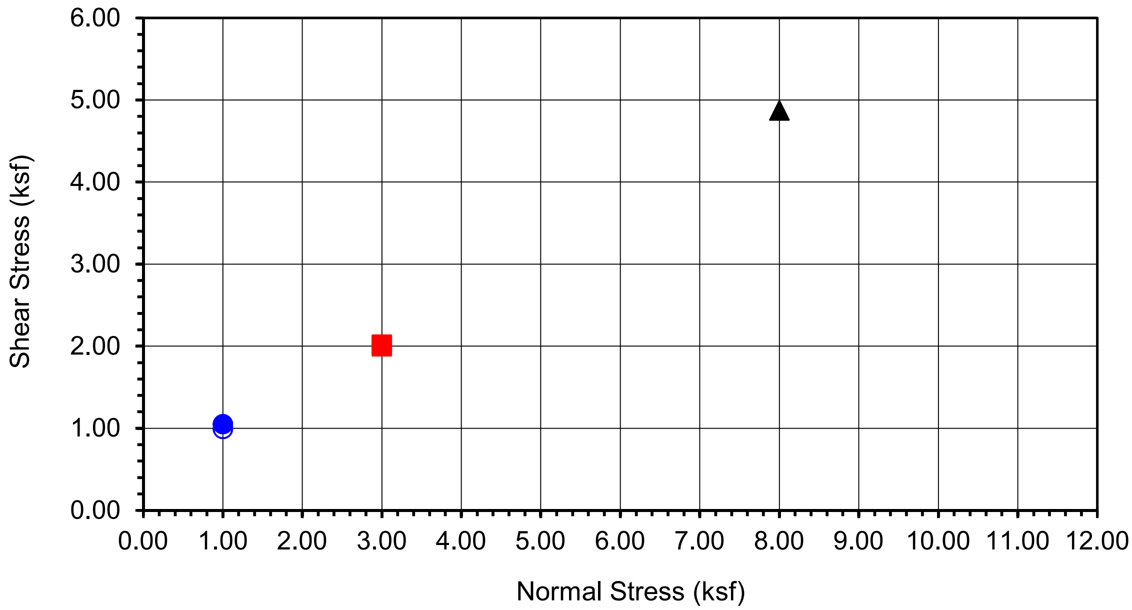
Consolidated Drained - ASTM D 3080

Leighton

Project Name:	SMMUSD MMHS Phase 1	Tested By:	G. Bathala	Date:	08/27/20
Project No.:	11382.016	Checked By:	J. Ward	Date:	09/15/20
Boring No.:	LB-4	Sample Type:	90% Remold		
Sample No.:	B-1	Depth (ft.):	0-5		
Soil Identification:	Dark yellowish brown lean clay with sand (CL)s				

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	178.05	178.28	178.27
Weight of Ring(gm):	45.82	45.78	45.72
Before Shearing			
Weight of Wet Sample+Cont.(gm):	189.90	189.90	189.90
Weight of Dry Sample+Cont.(gm):	176.18	176.18	176.18
Weight of Container(gm):	75.84	75.84	75.84
Vertical Rdg.(in): Initial	0.0000	0.2376	0.0000
Vertical Rdg.(in): Final	0.0216	0.2411	-0.0397
After Shearing			
Weight of Wet Sample+Cont.(gm):	221.00	200.89	196.21
Weight of Dry Sample+Cont.(gm):	187.23	172.91	171.01
Weight of Container(gm):	73.42	59.53	57.19
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-4**
Sample No.	**B-1**
Depth (ft)	**0-5**

Sample Type:

90% Remold

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

Normal Stress (kip/ft²)	1.000	3.000	8.000
Peak Shear Stress (kip/ft²)	● 1.050	■ 2.018	▲ 4.873
Shear Stress @ End of Test (ksf)	○ 0.990	□ 1.996	△ 4.873
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 (%)	13.67	13.67	13.67
Dry Density (pcf)	96.7	96.9	97.0
Saturation (%)	49.7	50.0	50.0
Soil Height Before Shearing (in.)	1.0216	0.9965	0.9603
Final Moisture Content (%)	29.7	24.7	22.1

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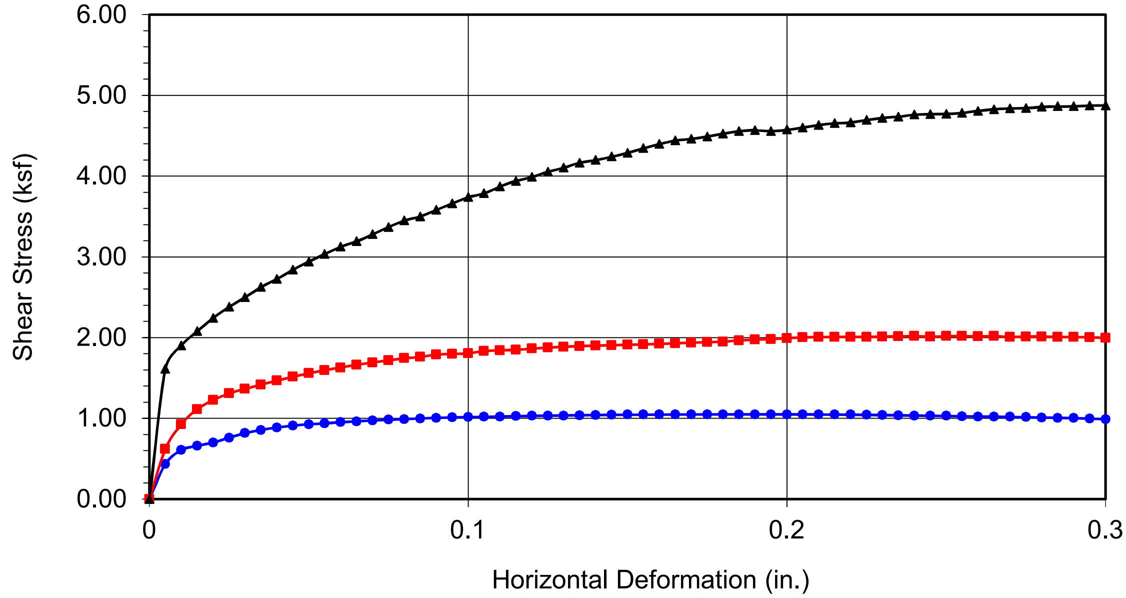
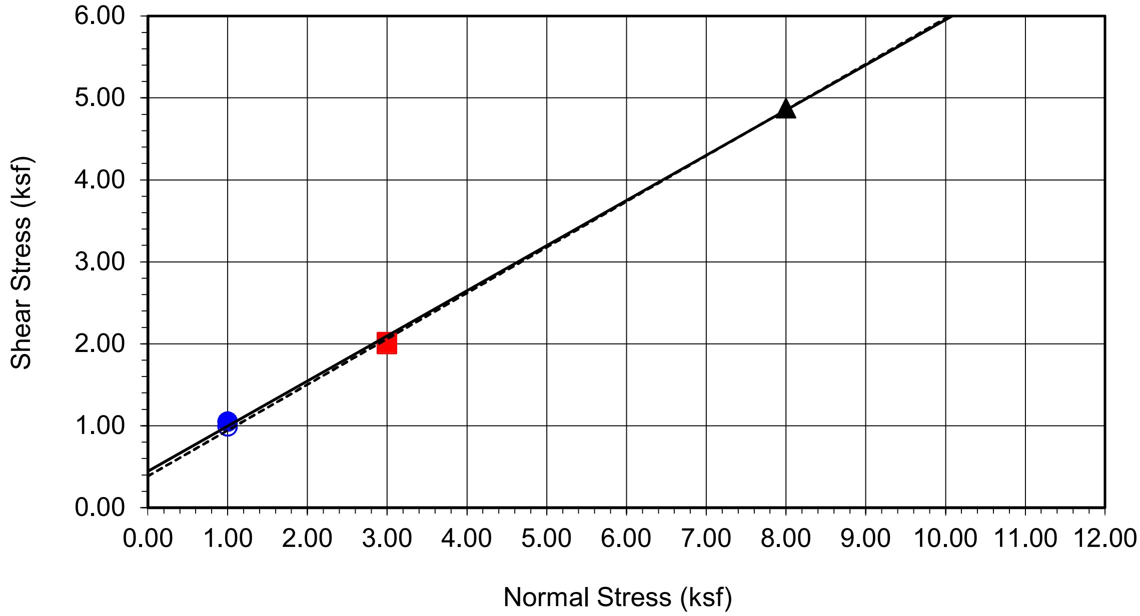
DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 11382.016

SMMUSD MMHS Phase 1

08-20





Boring No.	**LB-4**
Sample No.	**B-1**
Depth (ft)	**0-5**
Sample Type:	90% Remold

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

Strength Parameters

	C (psf)	ϕ (°)
Peak	443	29
Ultimate	385	29

Normal Stress (kip/ft²)	1.000	3.000	8.000
Peak Shear Stress (kip/ft²)	● 1.050	■ 2.018	▲ 4.873
Shear Stress @ End of Test (ksf)	○ 0.990	□ 1.996	△ 4.873
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 (%)	13.67	13.67	13.67
Dry Density (pcf)	96.7	96.9	97.0
Saturation (%)	49.7	50.0	50.0
Soil Height Before Shearing (in.)	1.0216	0.9965	0.9603
Final Moisture Content (%)	29.7	24.7	22.1

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DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 11382.016

SMMUSD MMHS Phase 1

08-20

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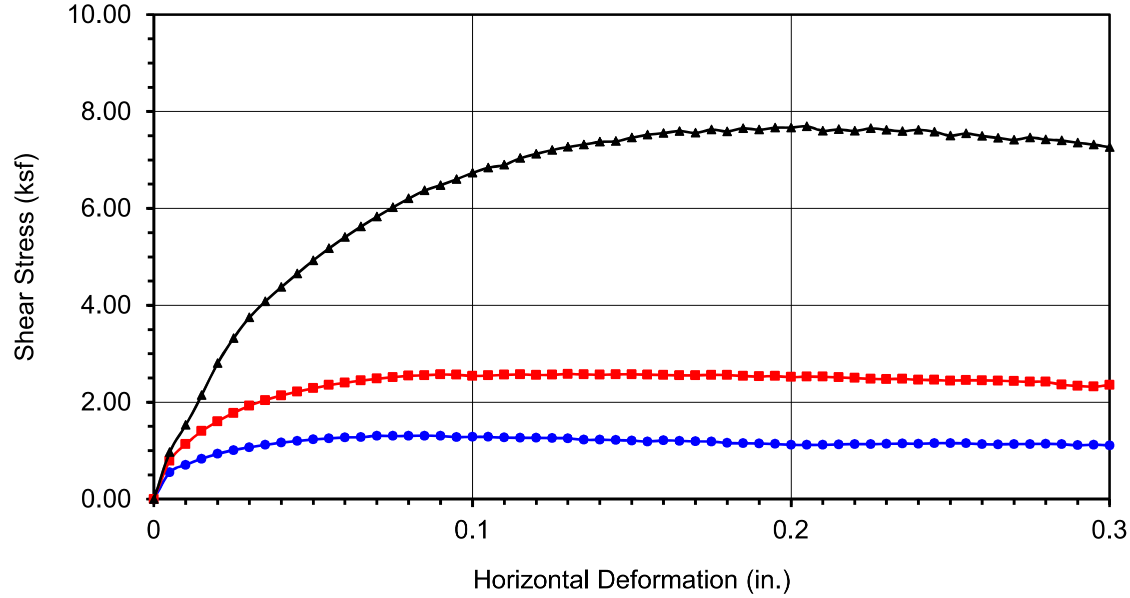
DIRECT SHEAR TEST

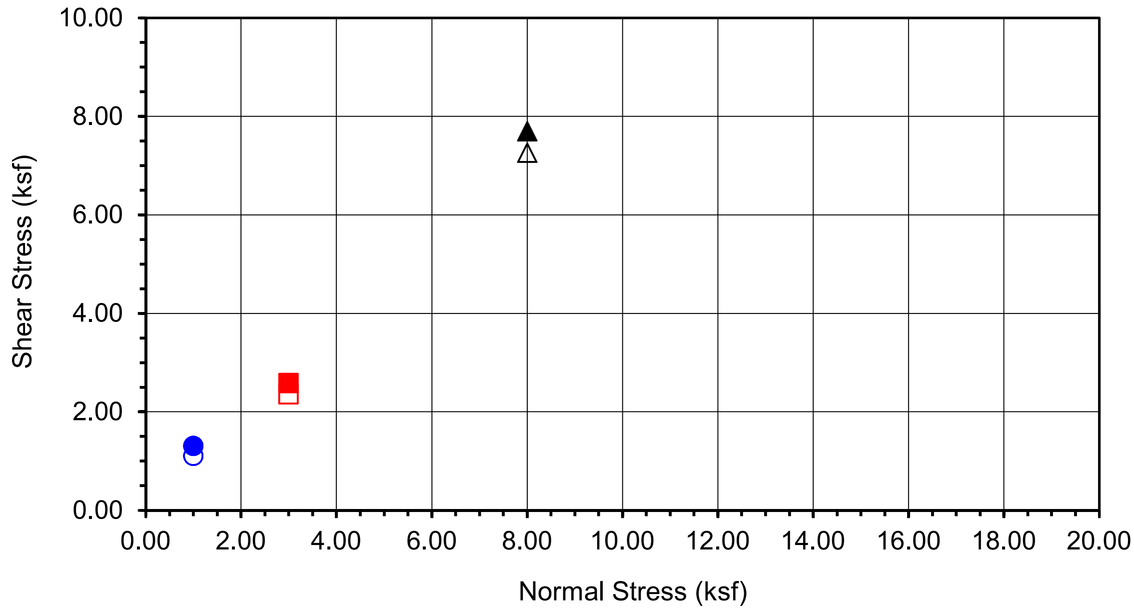
Consolidated Drained - ASTM D 3080

Project Name:	SMMUSD MMHS Phase 1	Tested By:	G. Bathala	Date:	08/24/20
Project No.:	11382.016	Checked By:	J. Ward	Date:	09/14/20
Boring No.:	LB-4	Sample Type:	Ring		
Sample No.:	R-1	Depth (ft.):	5.0		

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

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	193.26	198.28	197.95
Weight of Ring(gm):	42.89	45.84	45.43
Before Shearing			
Weight of Wet Sample+Cont.(gm):	181.04	181.04	181.04
Weight of Dry Sample+Cont.(gm):	162.59	162.59	162.59
Weight of Container(gm):	58.00	58.00	58.00
Vertical Rdg.(in): Initial	0.2272	0.0000	0.0000
Vertical Rdg.(in): Final	0.2329	-0.0168	-0.0285
After Shearing			
Weight of Wet Sample+Cont.(gm):	230.23	210.16	207.93
Weight of Dry Sample+Cont.(gm):	203.33	184.48	184.45
Weight of Container(gm):	77.79	58.64	58.53
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-4**
Sample No.	**R-1**
Depth (ft)	**5**
Sample Type: Ring	
Soil Identification: Dark yellowish brown lean clay with sand (CL)s	

Normal Stress (kip/ft²)	1.000	3.000	8.000
Peak Shear Stress (kip/ft²)	● 1.308	■ 2.581	▲ 7.693
Shear Stress @ End of Test (ksf)	○ 1.107	□ 2.355	△ 7.262
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 (%)	17.64	17.64	17.64
Dry Density (pcf)	106.3	107.8	107.8
Saturation (%)	81.3	84.4	84.6
Soil Height Before Shearing (in.)	0.9943	0.9832	0.9715
Final Moisture Content (%)	21.4	20.4	18.6

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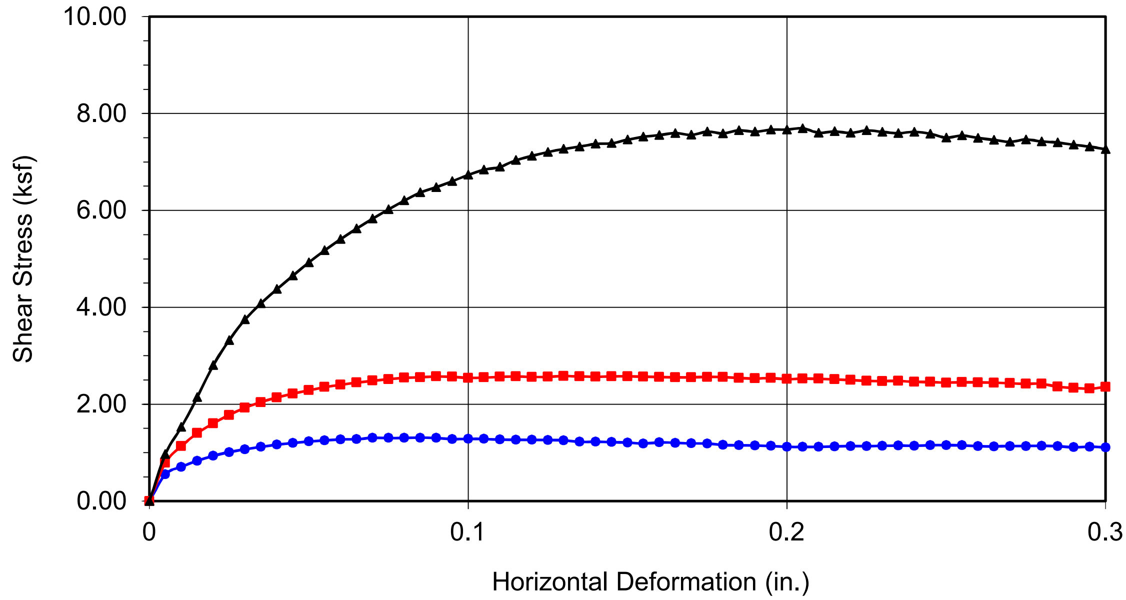
DIRECT SHEAR TEST RESULTS

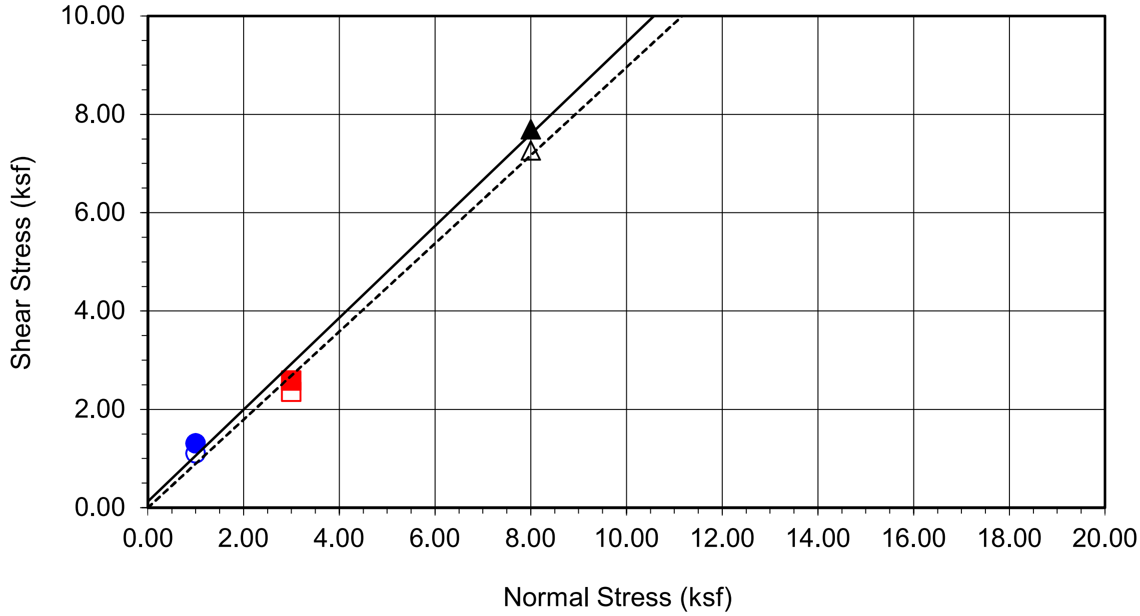
Consolidated Drained - ASTM D 3080

Project No.: 11382.016

SMMUSD MMHS Phase 1

08-20





Boring No.	**LB-4**
Sample No.	**R-1**
Depth (ft)	**5**
Sample Type:	Ring

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

Strength Parameters

	C (psf)	ϕ (°)
Peak	127	43
Ultimate	0	42

Normal Stress (kip/ft²)	1.000	3.000	8.000
Peak Shear Stress (kip/ft²)	● 1.308	■ 2.581	▲ 7.693
Shear Stress @ End of Test (ksf)	○ 1.107	□ 2.355	△ 7.262
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 (%)	17.64	17.64	17.64
Dry Density (pcf)	106.3	107.8	107.8
Saturation (%)	81.3	84.4	84.6
Soil Height Before Shearing (in.)	0.9943	0.9832	0.9715
Final Moisture Content (%)	21.4	20.4	18.6

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DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 11382.016

SMMUSD MMHS Phase 1

08-20

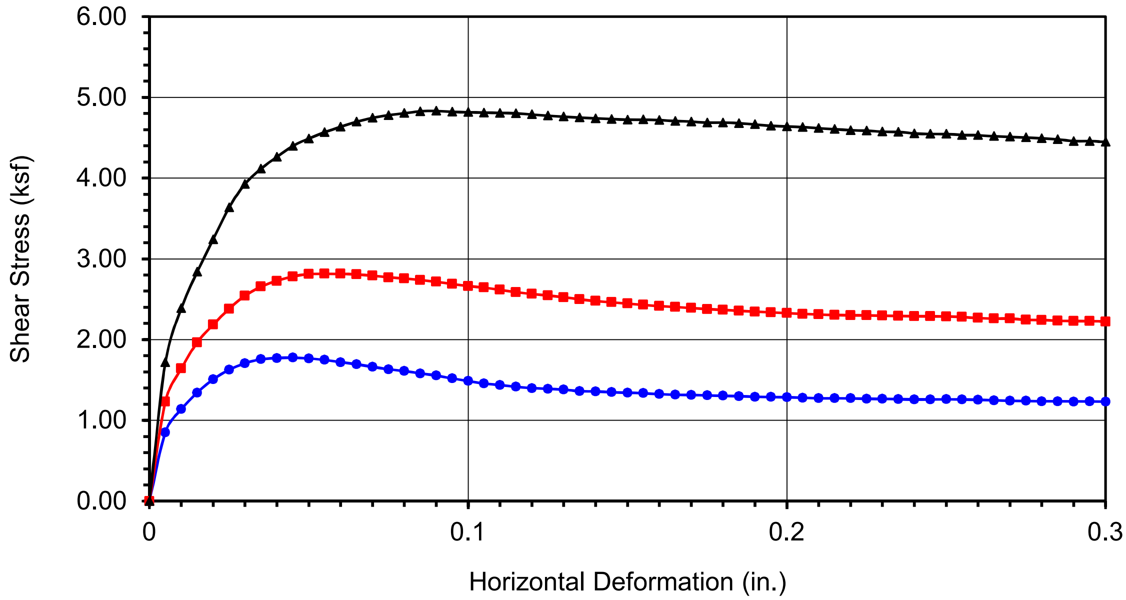
DIRECT SHEAR TEST

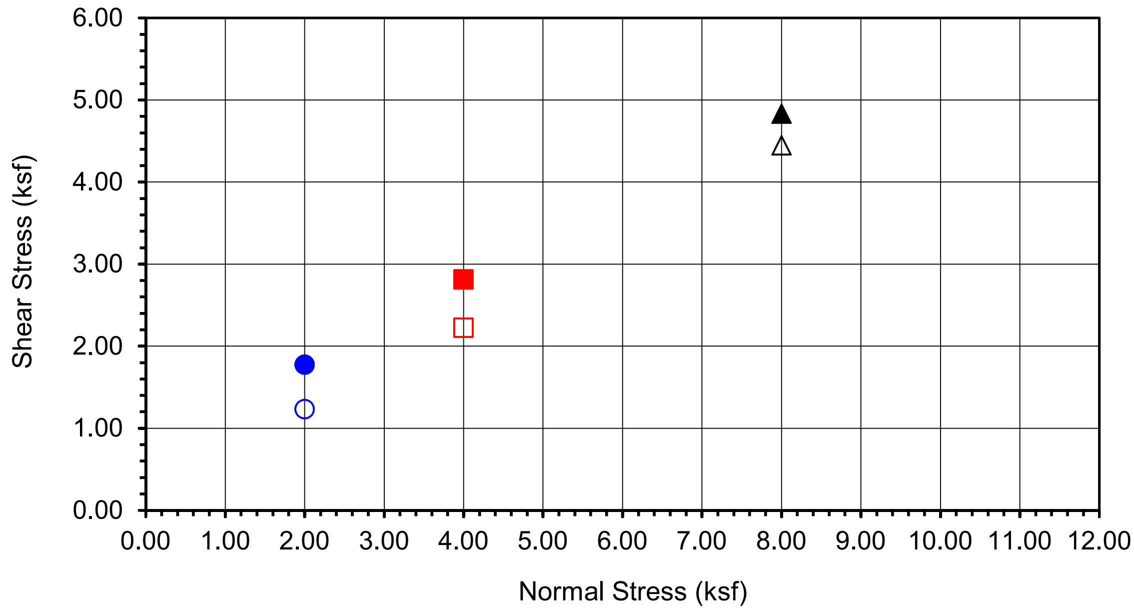
Consolidated Drained - ASTM D 3080

Leighton

Project Name:	SMMUSD MMHS Phase 1	Tested By:	G. Bathala	Date:	08/24/20
Project No.:	11382.016	Checked By:	J. Ward	Date:	09/14/20
Boring No.:	LB-4	Sample Type:	Ring		
Sample No.:	R-3	Depth (ft.):	10.0		
Soil Identification:	Yellowish brown lean clay (CL), caliche noted				

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	187.76	191.18	187.73
Weight of Ring(gm):	41.29	42.12	38.39
Before Shearing			
Weight of Wet Sample+Cont.(gm):	176.00	176.00	176.00
Weight of Dry Sample+Cont.(gm):	155.43	155.43	155.43
Weight of Container(gm):	35.70	35.70	35.70
Vertical Rdg.(in): Initial	0.0000	0.2460	0.2252
Vertical Rdg.(in): Final	-0.0104	0.2672	0.2435
After Shearing			
Weight of Wet Sample+Cont.(gm):	207.38	206.57	205.36
Weight of Dry Sample+Cont.(gm):	178.77	177.98	177.60
Weight of Container(gm):	59.07	57.41	57.19
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-4**
Sample No.	**R-3**
Depth (ft)	**10**
Sample Type: Ring	
Soil Identification: Yellowish brown lean clay (CL), caliche noted	

Normal Stress (kip/ft²)	2.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 1.776	■ 2.814	▲ 4.832
Shear Stress @ End of Test (ksf)	○ 1.232	□ 2.223	△ 4.448
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 (%)	17.18	17.18	17.18
Dry Density (pcf)	104.0	105.8	106.0
Saturation (%)	74.6	78.2	78.6
Soil Height Before Shearing (in.)	0.9896	0.9788	0.9817
Final Moisture Content (%)	23.9	23.7	23.1

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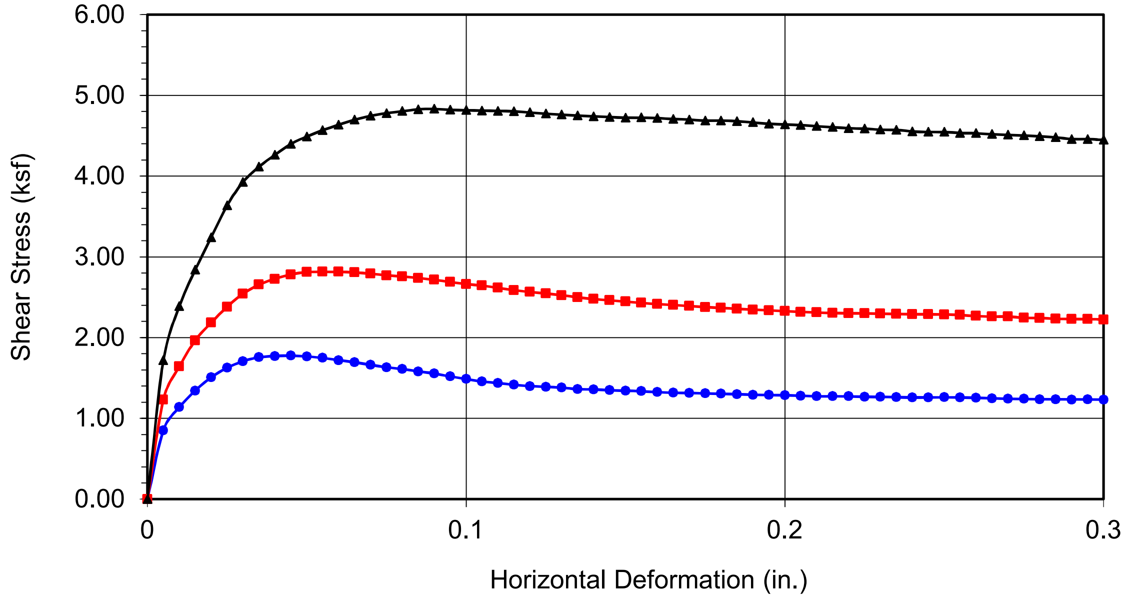
DIRECT SHEAR TEST RESULTS

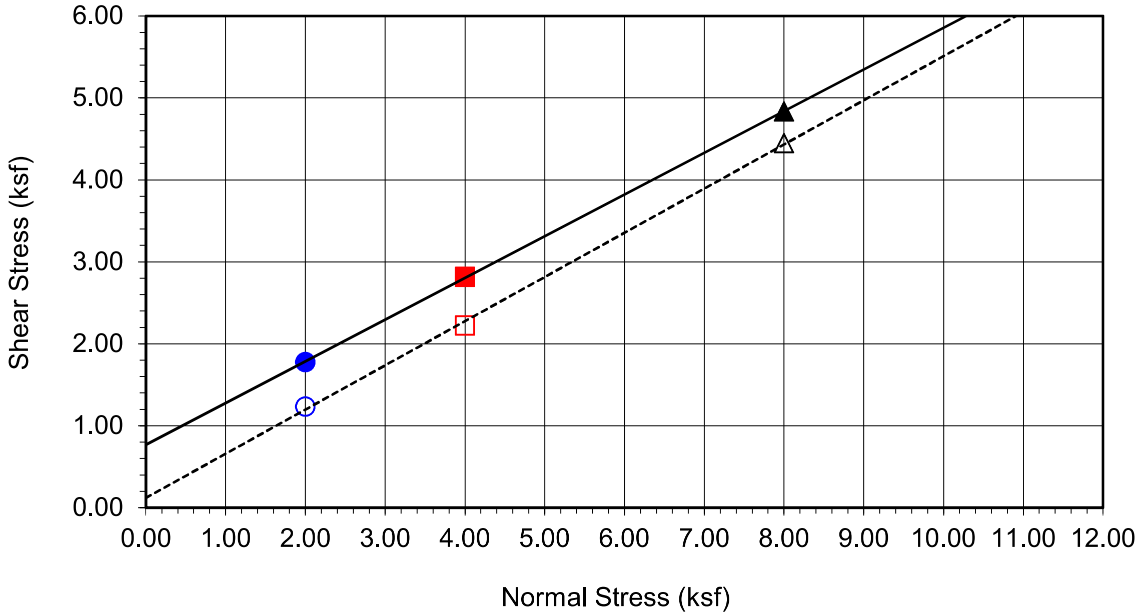
Consolidated Drained - ASTM D 3080

Project No.: 11382.016

SMMUSD MMHS Phase 1

08-20





Boring No.	**LB-4**
Sample No.	**R-3**
Depth (ft)	**10**
Sample Type:	Ring

Soil Identification:
Yellowish brown lean clay (CL), caliche noted

Strength Parameters

	C (psf)	ϕ (°)
Peak	767	27
Ultimate	120	28

Normal Stress (kip/ft²)	2.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 1.776	■ 2.814	▲ 4.832
Shear Stress @ End of Test (ksf)	○ 1.232	□ 2.223	△ 4.448
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 (%)	17.18	17.18	17.18
Dry Density (pcf)	104.0	105.8	106.0
Saturation (%)	74.6	78.2	78.6
Soil Height Before Shearing (in.)	0.9896	0.9788	0.9817
Final Moisture Content (%)	23.9	23.7	23.1

Leighton

DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 11382.016

SMMUSD MMHS Phase 1

08-20

DIRECT SHEAR TEST

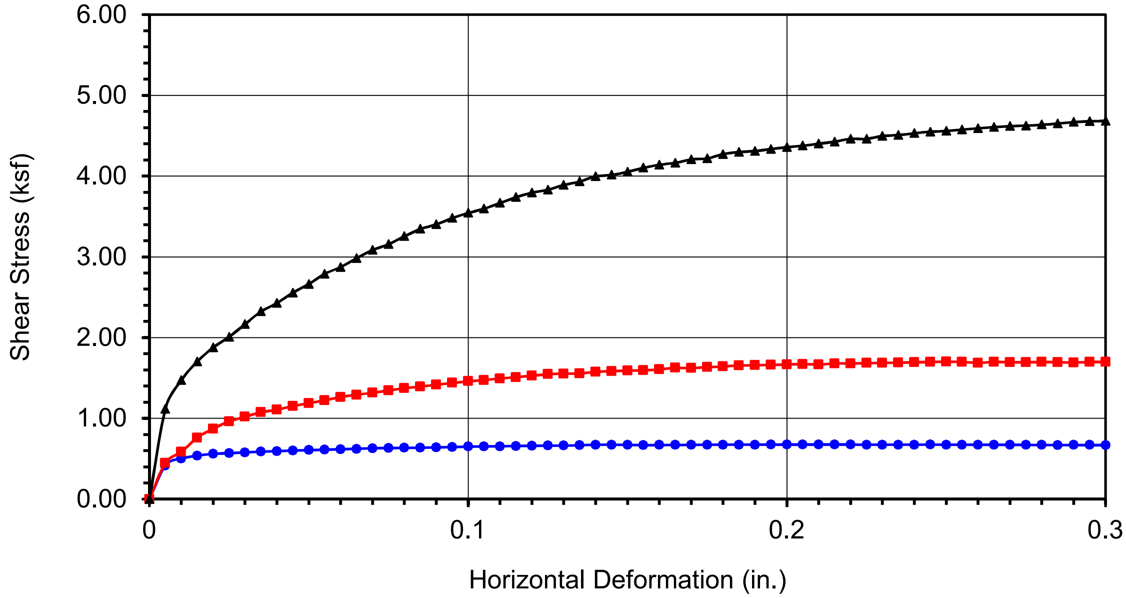
Consolidated Drained - ASTM D 3080

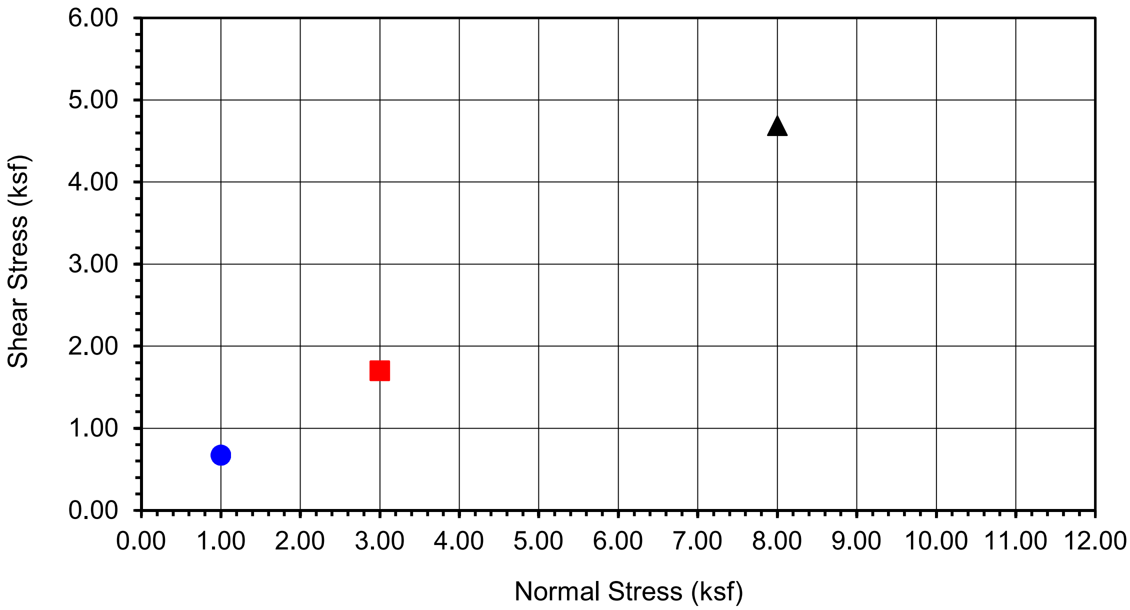
Leighton

Project Name:	SMMUSD MMHS Phase 1	Tested By:	G. Bathala	Date:	08/26/20
Project No.:	11382.016	Checked By:	J. Ward	Date:	09/15/20
Boring No.:	LB-6	Sample Type:	90% Remold		
Sample No.:	B-1	Depth (ft.):	0-5		

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

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	178.21	178.92	179.13
Weight of Ring(gm):	45.72	45.77	45.83
Before Shearing			
Weight of Wet Sample+Cont.(gm):	214.82	214.82	214.82
Weight of Dry Sample+Cont.(gm):	200.37	200.37	200.37
Weight of Container(gm):	77.79	77.79	77.79
Vertical Rdg.(in): Initial	0.2253	0.0000	0.0000
Vertical Rdg.(in): Final	0.2081	-0.0173	-0.0634
After Shearing			
Weight of Wet Sample+Cont.(gm):	184.45	211.10	211.04
Weight of Dry Sample+Cont.(gm):	154.07	184.12	188.23
Weight of Container(gm):	36.76	65.78	70.37
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-6**
Sample No.	**B-1**
Depth (ft)	**0-5**

Sample Type:

90% Remold

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

Normal Stress (kip/ft²)	1.000	3.000	8.000
Peak Shear Stress (kip/ft²)	● 0.676	■ 1.701	▲ 4.684
Shear Stress @ End of Test (ksf)	○ 0.666	□ 1.698	△ 4.684
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 (%)	11.79	11.79	11.79
Dry Density (pcf)	98.6	99.1	99.2
Saturation (%)	44.8	45.4	45.5
Soil Height Before Shearing (in.)	1.0172	0.9827	0.9366
Final Moisture Content (%)	25.9	22.8	19.4

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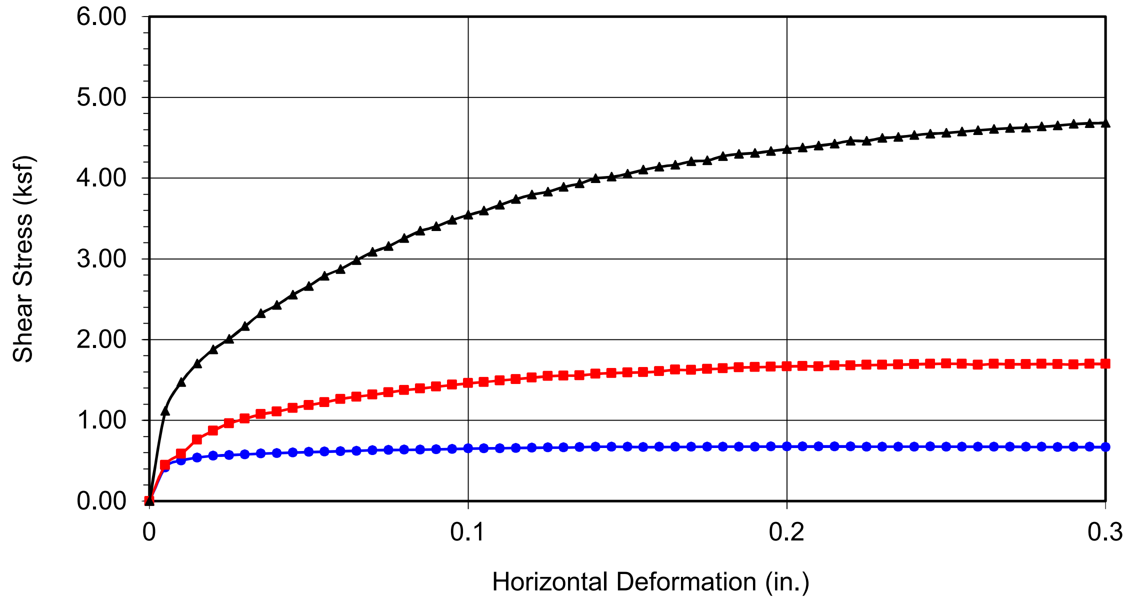
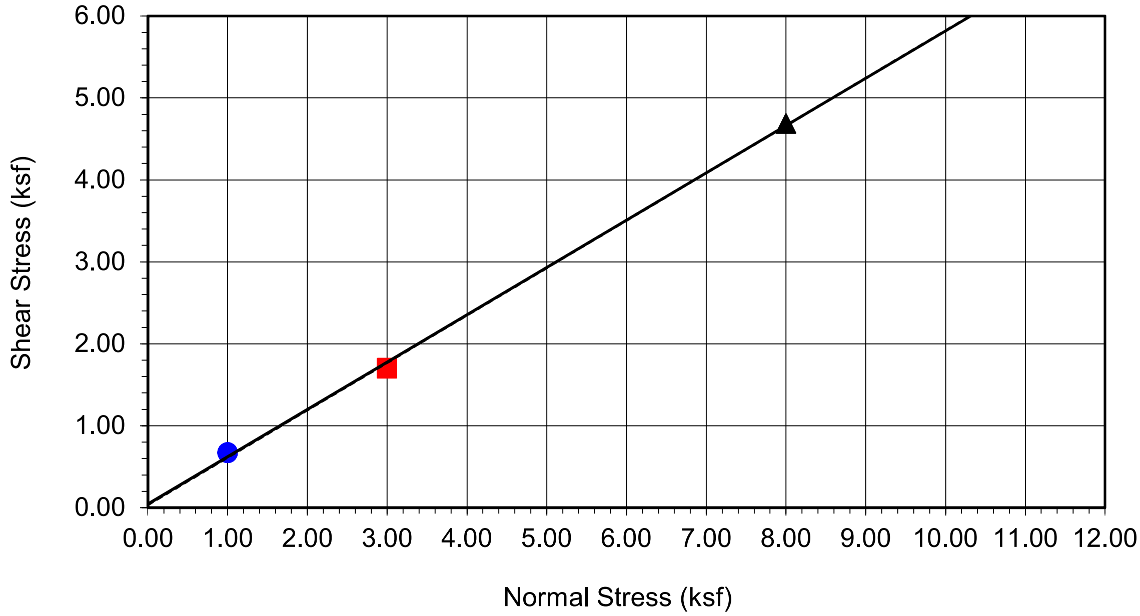
DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 11382.016

SMMUSD MMHS Phase 1

08-20





Boring No.	**LB-6**
Sample No.	**B-1**
Depth (ft)	**0-5**
Sample Type:	90% Remold

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

Strength Parameters

	C (psf)	ϕ (°)
Peak	45	30
Ultimate	35	30

Normal Stress (kip/ft²)	1.000	3.000	8.000
Peak Shear Stress (kip/ft²)	● 0.676	■ 1.701	▲ 4.684
Shear Stress @ End of Test (ksf)	○ 0.666	□ 1.698	△ 4.684
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 (%)	11.79	11.79	11.79
Dry Density (pcf)	98.6	99.1	99.2
Saturation (%)	44.8	45.4	45.5
Soil Height Before Shearing (in.)	1.0172	0.9827	0.9366
Final Moisture Content (%)	25.9	22.8	19.4

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DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 11382.016
SMMUSD MMHS Phase 1
08-20

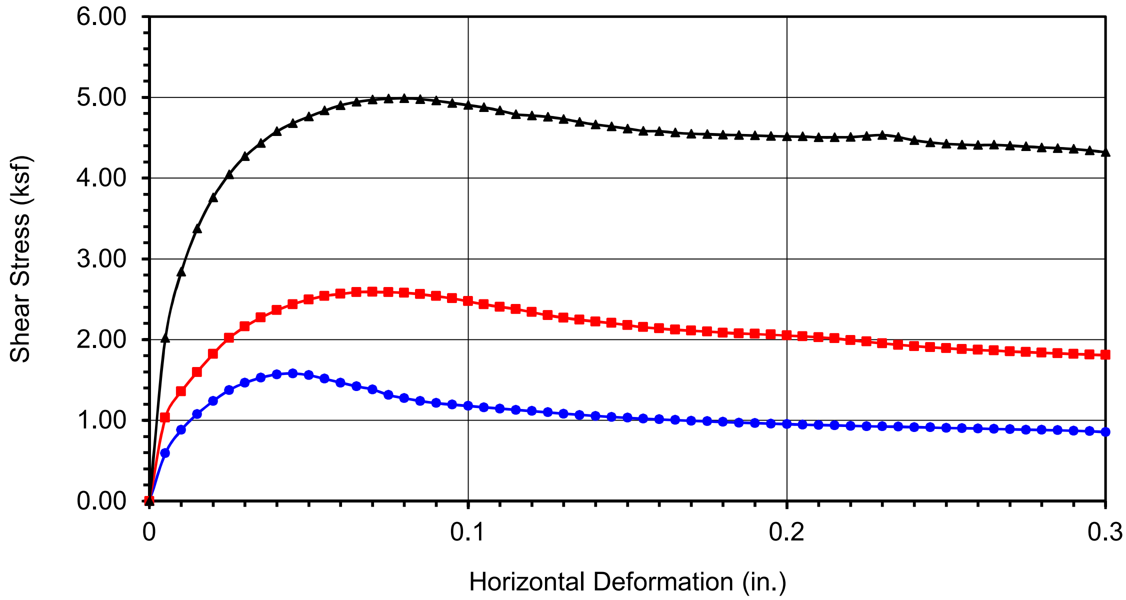
DIRECT SHEAR TEST

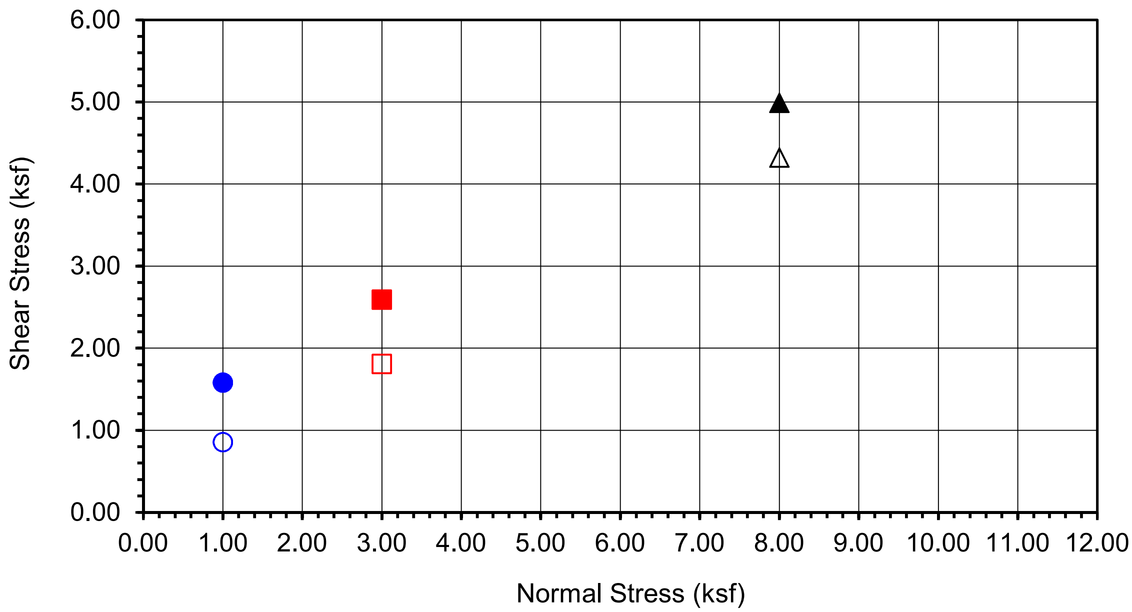
Consolidated Drained - ASTM D 3080

Project Name:	SMMUSD MMHS Phase 1	Tested By:	G. Bathala	Date:	08/25/20
Project No.:	11382.016	Checked By:	J. Ward	Date:	09/14/20
Boring No.:	LB-6	Sample Type:	Ring		
Sample No.:	R-2	Depth (ft.):	7.0		

Soil Identification: Dark yellowish brown fat clay (CH), caliche noted

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	192.16	192.73	192.97
Weight of Ring(gm):	43.31	43.31	41.87
Before Shearing			
Weight of Wet Sample+Cont.(gm):	203.31	203.31	203.31
Weight of Dry Sample+Cont.(gm):	174.40	174.40	174.40
Weight of Container(gm):	57.27	57.27	57.27
Vertical Rdg.(in): Initial	0.0000	0.2626	0.2340
Vertical Rdg.(in): Final	-0.0009	0.2786	0.2631
After Shearing			
Weight of Wet Sample+Cont.(gm):	207.81	207.95	207.32
Weight of Dry Sample+Cont.(gm):	175.21	177.21	177.87
Weight of Container(gm):	57.42	58.53	57.18
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-6**
Sample No.	**R-2**
Depth (ft)	**7**

Sample Type:

Ring

Soil Identification:
Dark yellowish brown fat clay (CH), caliche noted

Normal Stress (kip/ft²)	1.000	3.000	8.000
Peak Shear Stress (kip/ft²)	● 1.581	■ 2.590	▲ 4.989
Shear Stress @ End of Test (ksf)	○ 0.855	□ 1.808	△ 4.320
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 (%)	24.68	24.68	24.68
Dry Density (pcf)	99.3	99.7	100.8
Saturation (%)	95.5	96.4	99.1
Soil Height Before Shearing (in.)	0.9991	0.9840	0.9709
Final Moisture Content (%)	27.7	25.9	24.4

Leighton

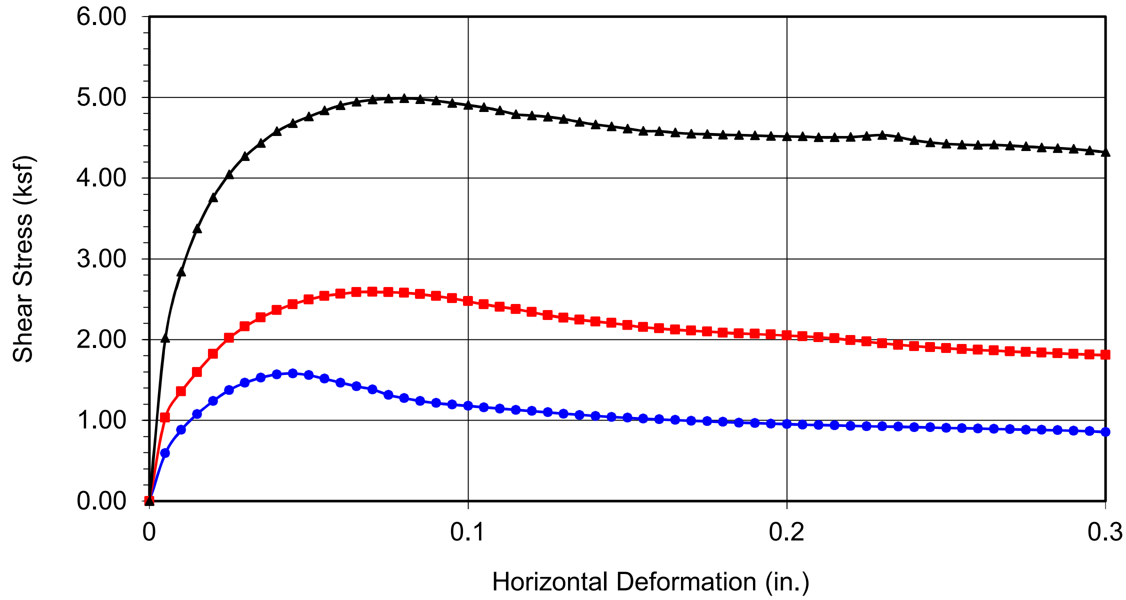
DIRECT SHEAR TEST RESULTS

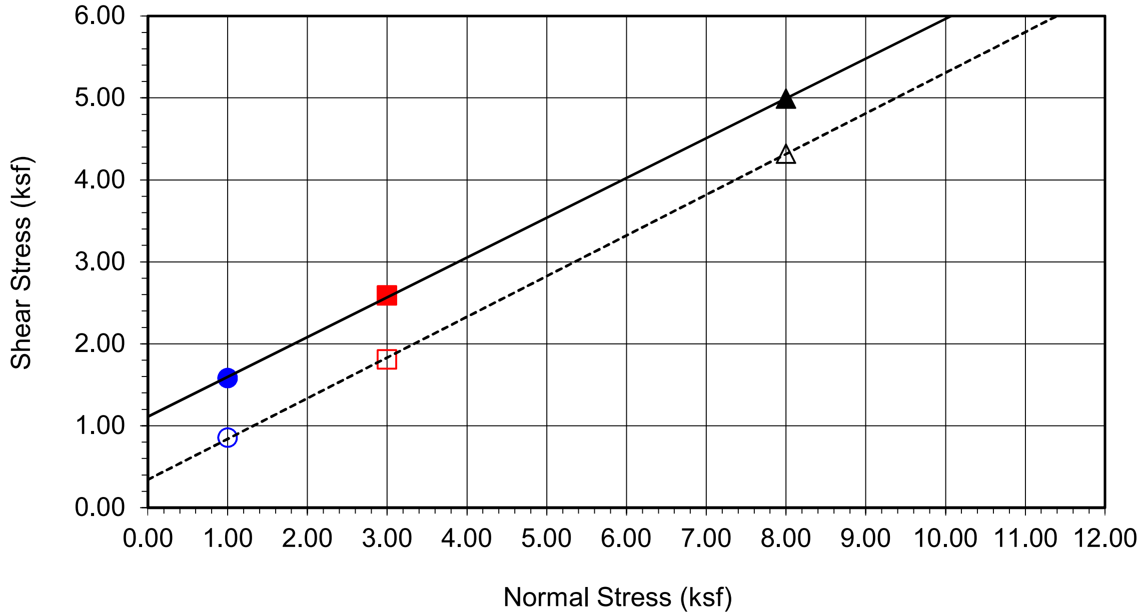
Consolidated Drained - ASTM D 3080

Project No.: 11382.016

SMMUSD MMHS Phase 1

08-20





Boring No.	**LB-6**
Sample No.	**R-2**
Depth (ft)	**7**
Sample Type:	Ring

Soil Identification:
Dark yellowish brown fat clay (CH), caliche noted

Strength Parameters

	C (psf)	ϕ (°)
Peak	1111	26
Ultimate	342	26

Normal Stress (kip/ft²)	1.000	3.000	8.000
Peak Shear Stress (kip/ft²)	● 1.581	■ 2.590	▲ 4.989
Shear Stress @ End of Test (ksf)	○ 0.855	□ 1.808	△ 4.320
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 (%)	24.68	24.68	24.68
Dry Density (pcf)	99.3	99.7	100.8
Saturation (%)	95.5	96.4	99.1
Soil Height Before Shearing (in.)	0.9991	0.9840	0.9709
Final Moisture Content (%)	27.7	25.9	24.4

Leighton

DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 11382.016

SMMUSD MMHS Phase 1

08-20

Leighton

DIRECT SHEAR TEST

Consolidated Drained - ASTM D 3080

Project Name: SMMUSD MMHS Phase 1
Tested By: G. Bathala
Date: 08/25/20

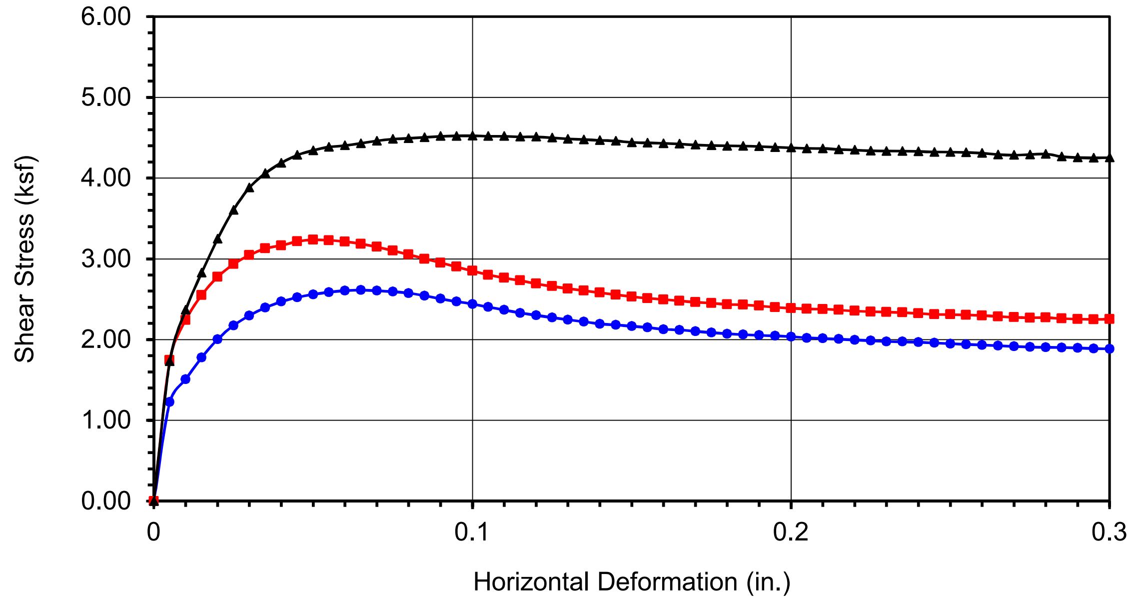
Project No.: 11382.016
Checked By: J. Ward
Date: 09/14/20

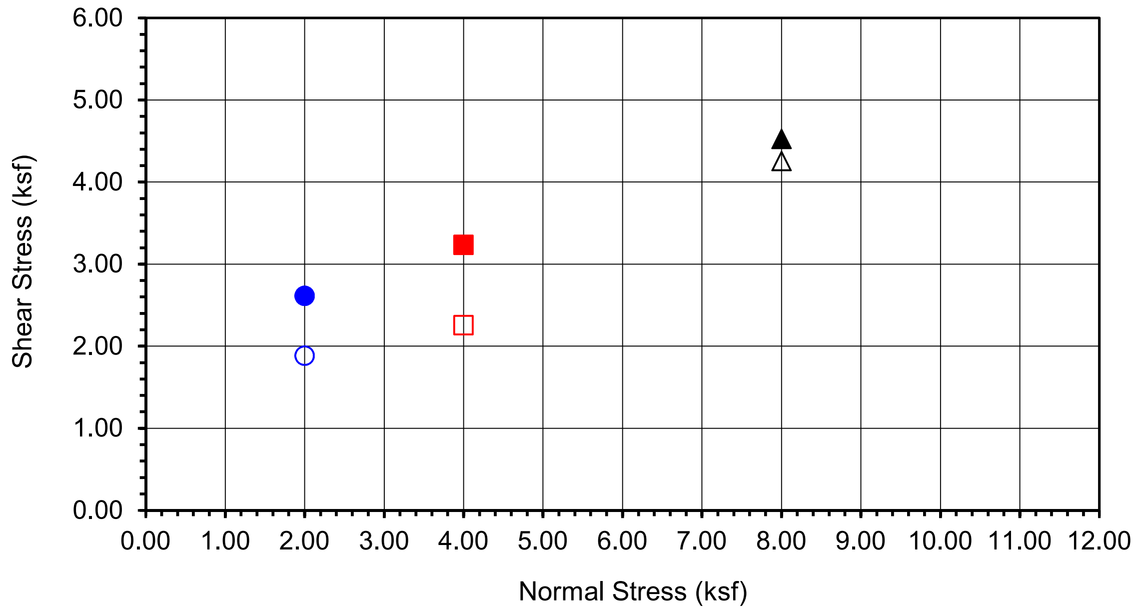
Boring No.: LB-6
Sample Type: Ring

Sample No.: R-4
Depth (ft.): 15.0

Soil Identification: Dark yellowish brown lean clay (CL), caliche noted

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	194.02	201.18	203.29
Weight of Ring(gm):	42.70	45.59	45.96
Before Shearing			
Weight of Wet Sample+Cont.(gm):	187.85	187.85	187.85
Weight of Dry Sample+Cont.(gm):	170.40	170.40	170.40
Weight of Container(gm):	61.85	61.85	61.85
Vertical Rdg.(in): Initial	0.0000	0.2576	0.0000
Vertical Rdg.(in): Final	-0.0138	0.2763	-0.0356
After Shearing			
Weight of Wet Sample+Cont.(gm):	215.84	214.15	215.39
Weight of Dry Sample+Cont.(gm):	186.71	187.37	189.48
Weight of Container(gm):	63.36	57.87	59.53
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-6**
Sample No.	**R-4**
Depth (ft)	**15**

Sample Type:

Ring

Soil Identification:
Dark yellowish brown lean clay (CL), caliche noted

Normal Stress (kip/ft²)	2.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 2.612	■ 3.235	▲ 4.524
Shear Stress @ End of Test (ksf)	○ 1.886	□ 2.254	△ 4.254
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 (%)	16.08	16.08	16.08
Dry Density (pcf)	108.4	111.5	112.7
Saturation (%)	78.2	84.8	87.6
Soil Height Before Shearing (in.)	0.9862	0.9813	0.9644
Final Moisture Content (%)	23.6	20.7	19.9

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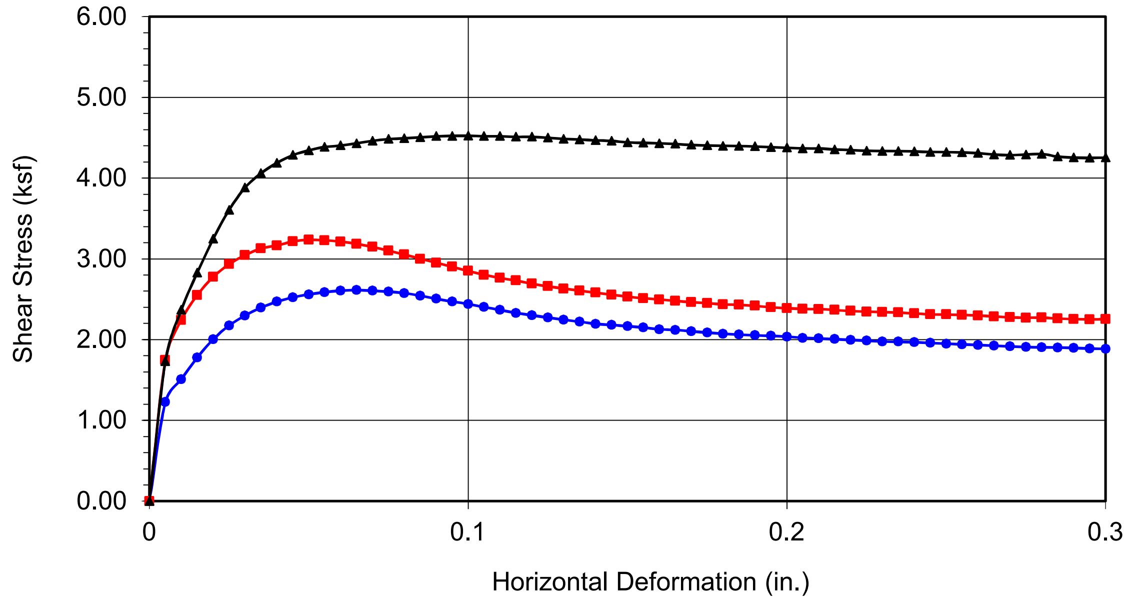
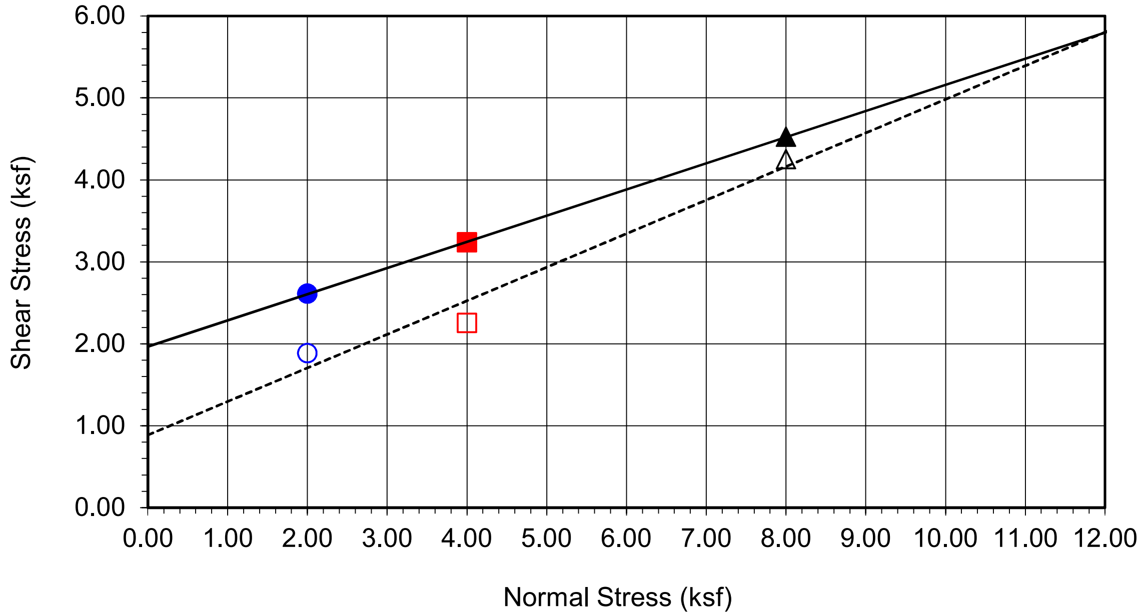
DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 11382.016

SMMUSD MMHS Phase 1

08-20





Boring No.	**LB-6**
Sample No.	**R-4**
Depth (ft)	**15**
Sample Type:	Ring

Soil Identification:
Dark yellowish brown lean clay (CL), caliche noted

Strength Parameters

	C (psf)	ϕ (°)
Peak	1968	18
Ultimate	886	22

Normal Stress (kip/ft²)	2.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 2.612	■ 3.235	▲ 4.524
Shear Stress @ End of Test (ksf)	○ 1.886	□ 2.254	△ 4.254
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 (%)	16.08	16.08	16.08
Dry Density (pcf)	108.4	111.5	112.7
Saturation (%)	78.2	84.8	87.6
Soil Height Before Shearing (in.)	0.9862	0.9813	0.9644
Final Moisture Content (%)	23.6	20.7	19.9

Leighton

DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 11382.016

SMMUSD MMHS Phase 1

08-20

EXPANSION INDEX of SOILS

ASTM D 4829

Leighton

Project Name:	SMMUSD MMHS Phase 1	Tested By:	SF/JG	Date:	09/01/20
Project No.:	11382.016	Checked By:	J. Ward	Date:	09/15/20
Boring No.:	2020-LB-4	Depth (ft.):	0-5		
Sample No.:	B-1				
Soil Identification:	Dark yellowish brown lean clay with sand (CL)s				

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	1.1335
Wt. Comp. Soil + Mold	(g)	534.10	430.80
Wt. of Mold	(g)	166.20	0.00
Specific Gravity (Assumed)		2.70	2.70
Container No.		O	O
Wet Wt. of Soil + Cont.	(g)	725.40	597.00
Dry Wt. of Soil + Cont.	(g)	636.60	488.92
Wt. of Container	(g)	0.00	166.20
Moisture Content	(%)	13.95	33.49
Wet Density	(pcf)	111.0	114.6
Dry Density	(pcf)	97.4	85.9
Void Ratio		0.731	0.963
Total Porosity		0.422	0.491
Pore Volume	(cc)	87.4	115.1
Degree of Saturation (%) [S_{meas}]		**51.5**	93.9

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.)
09/01/20	8:20	1.0	0	0.6375
09/01/20	8:30	1.0	10	0.6375
Add Distilled Water to the Specimen				
09/01/20	10:38	1.0	128	0.7050
09/02/20	7:15	1.0	1365	0.7710
09/02/20	8:34	1.0	1444	0.7710

Expansion Index (EI_{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	**134**

EXPANSION INDEX of SOILS

ASTM D 4829

Leighton

Project Name:	SMMUSD MMHS Phase 1	Tested By:	SF/JG	Date:	09/01/20
Project No.:	11382.016	Checked By:	J. Ward	Date:	09/15/20
Boring No.:	2020-LB-6	Depth (ft.):	0-5		
Sample No.:	B-1				
Soil Identification:	Dark yellowish brown lean clay with sand (CL)s				

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	1.1160
Wt. Comp. Soil + Mold (g)	537.50	432.60
Wt. of Mold (g)	163.50	0.00
Specific Gravity (Assumed)	2.70	2.70
Container No.	O	O
Wet Wt. of Soil + Cont. (g)	759.30	596.10
Dry Wt. of Soil + Cont. (g)	678.00	497.43
Wt. of Container (g)	0.00	163.50
Moisture Content (%)	11.99	29.55
Wet Density (pcf)	112.8	116.9
Dry Density (pcf)	100.7	90.3
Void Ratio	0.674	0.868
Total Porosity	0.402	0.465
Pore Volume (cc)	83.3	107.3
Degree of Saturation (%) [S meas]	**48.1**	91.9

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.)
09/01/20	9:17	1.0	0	0.5915
09/01/20	9:27	1.0	10	0.5915
Add Distilled Water to the Specimen				
09/01/20	10:35	1.0	68	0.6425
09/02/20	7:15	1.0	1308	0.7075
09/02/20	8:30	1.0	1383	0.7075

Expansion Index (EI meas) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	**116**

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Leighton

Project Name:	SMMUSD MMHS Phase 1	Tested By:	J. Gonzalez	Date:	08/25/20
Project No.:	11382.016	Checked By:	A. Santos	Date:	08/26/20
Boring No.:	2020-LB-4	Depth (ft.):	0-5		
Sample No.:	B-1				
Soil Identification:	Dark yellowish brown lean clay with sand (CL)s				

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)	3530	3678	3710	3670		
Weight of Mold (g)	1868	1868	1868	1868		
Net Weight of Soil (g)	1662	1810	1842	1802		
Wet Weight of Soil + Cont. (g)	315.3	381.5	354.3	450.0		
Dry Weight of Soil + Cont. (g)	289.8	343.4	313.8	390.2		
Weight of Container (g)	41.3	39.3	39.3	40.1		
Moisture Content (%)	10.26	12.53	14.75	17.08		
Wet Density (pcf)	110.0	119.8	121.9	119.3		
Dry Density (pcf)	99.8	106.5	106.3	101.9		

Maximum Dry Density (pcf) **107.1** **Optimum Moisture Content (%)** **13.5**

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

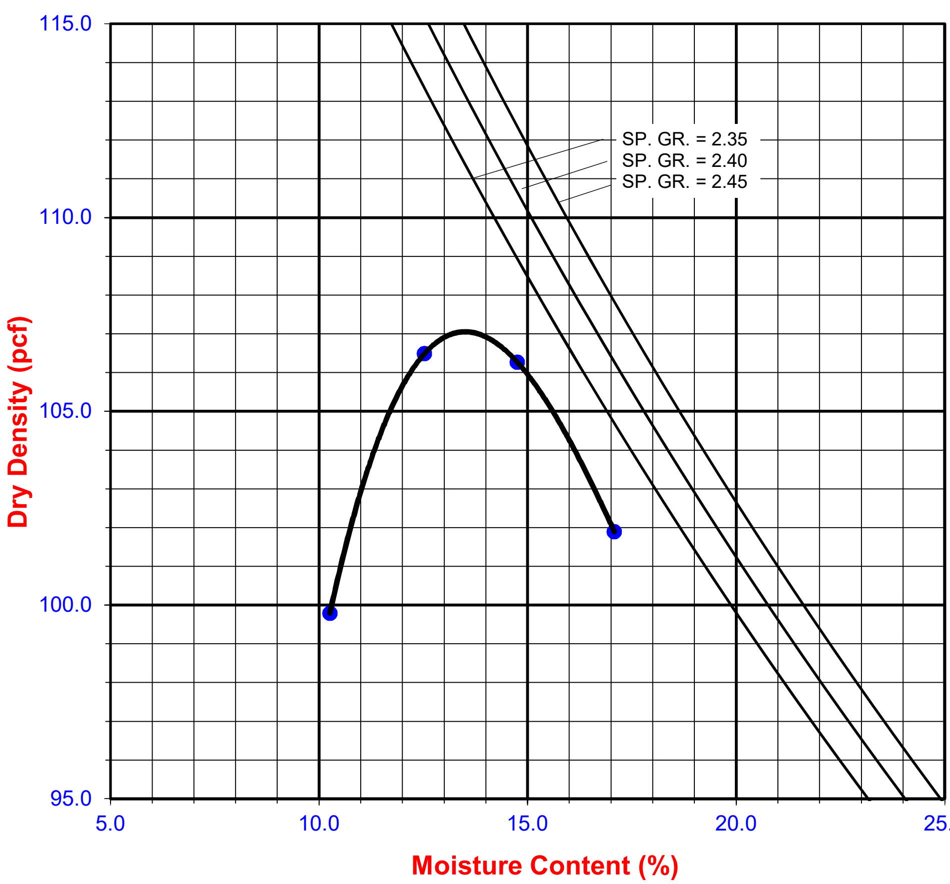
[ ] **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Leighton

Project Name:	SMMUSD MMHS Phase 1	Tested By:	J. Gonzalez	Date:	08/24/20
Project No.:	11382.016	Checked By:	A. Santos	Date:	08/25/20
Boring No.:	2020-LB-6	Depth (ft.):	0-5		
Sample No.:	B-1				
Soil Identification:	Dark yellowish brown lean clay with sand (CL)s				

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)	3598	3691	3709			
Weight of Mold (g)	1868	1868	1868			
Net Weight of Soil (g)	1730	1823	1841			
Wet Weight of Soil + Cont. (g)	362.9	346.6	322.4			
Dry Weight of Soil + Cont. (g)	339.0	317.2	284.7			
Weight of Container (g)	39.7	39.9	38.9			
Moisture Content (%)	7.99	10.60	15.34			
Wet Density (pcf)	114.5	120.7	121.9			
Dry Density (pcf)	106.1	109.1	105.7			

Maximum Dry Density (pcf) 109.4 **Optimum Moisture Content (%)** 11.6

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

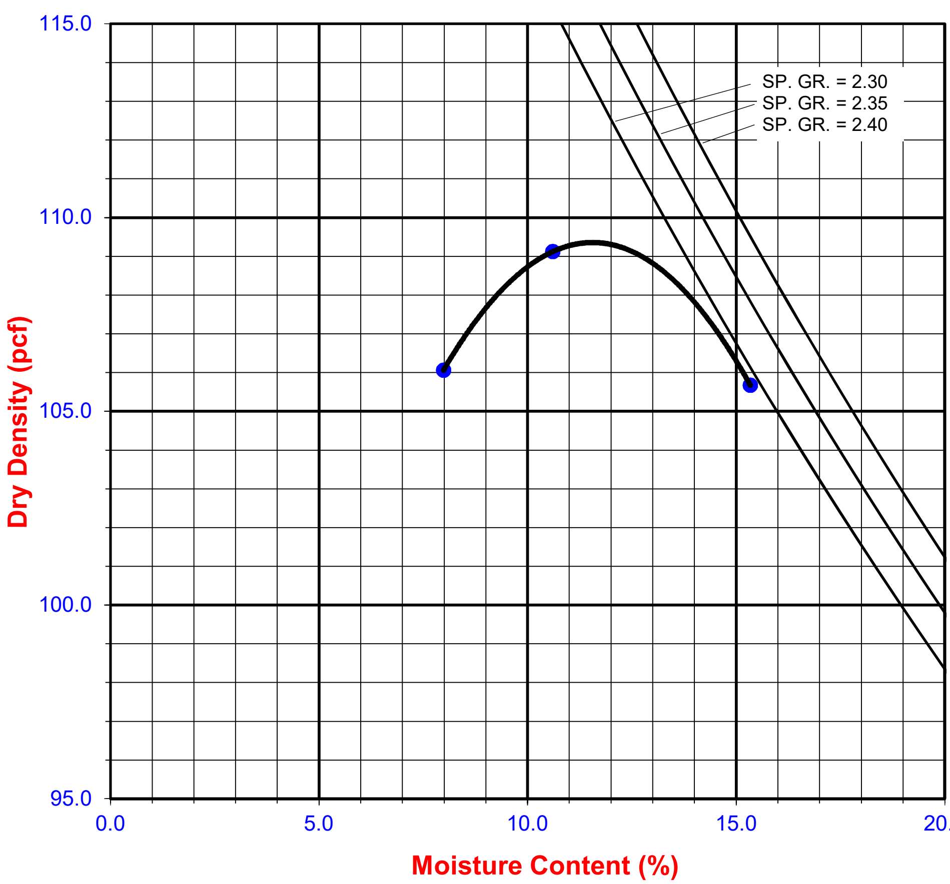
[ ] **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



ATTERBERG LIMITS

ASTM D 4318

Leighton

Project Name:	SMMUSD MMHS Phase 1	Tested By:	Y. Nguyen	Date:	08/28/20
Project No. :	11382.016	Input By:	G. Bathala	Date:	08/30/20
Boring No.:	2020-LB-4	Checked By:	J. Ward		
Sample No.:	R-1	Depth (ft.)	5.0		
Soil Identification:	Dark yellowish brown lean clay with sand (CL)s				

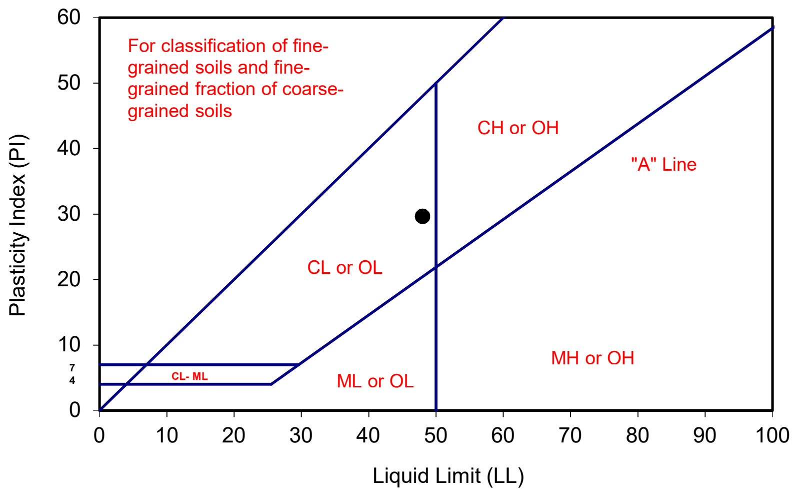
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			31	23	16	
Wet Wt. of Soil + Cont. (g)	8.91	8.82	20.23	20.50	21.09	
Dry Wt. of Soil + Cont. (g)	7.70	7.60	14.08	14.17	14.45	
Wt. of Container (g)	1.01	1.05	1.01	1.05	1.05	
Moisture Content (%) [Wn]	18.09	18.63	47.05	48.25	49.55	

Liquid Limit	**48**
Plastic Limit	**18**
Plasticity Index	**30**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) | 20.44

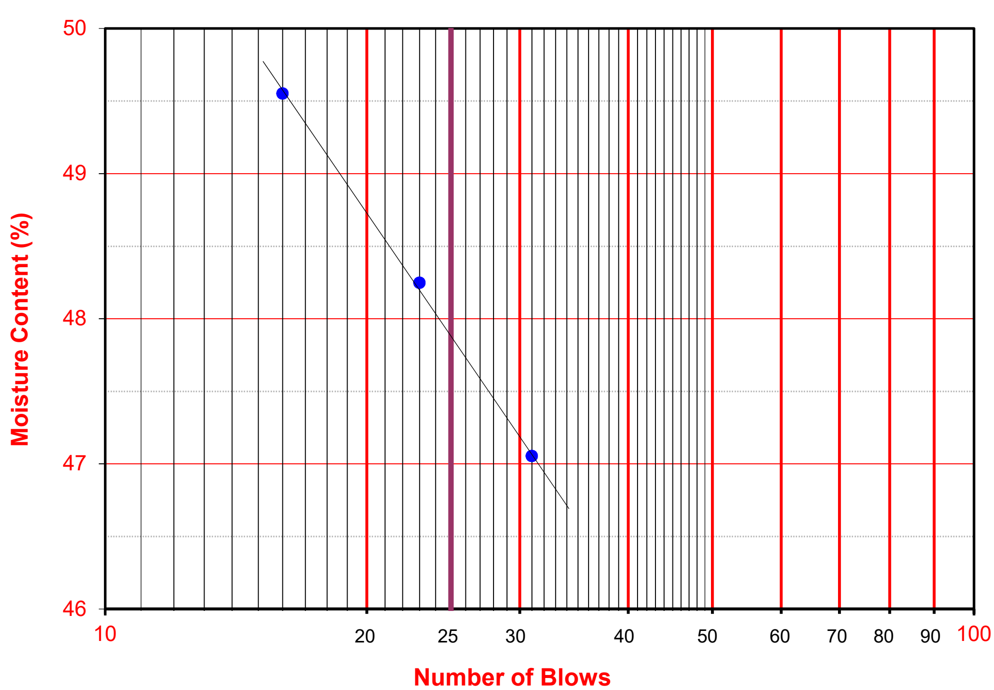
One - Point Liquid Limit Calculation

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



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

Leighton

Project Name: SMMUSD MMHS Phase 1
Tested By: S. Felter
Date: 08/31/20
Project No. : 11382.016
Input By: G. Bathala
Date: 09/02/20
Boring No.: 2020-LB-4
Checked By: J. Ward
Sample No.: R-2B
Depth (ft.) 7.0
Soil Identification: Brown lean clay (CL)

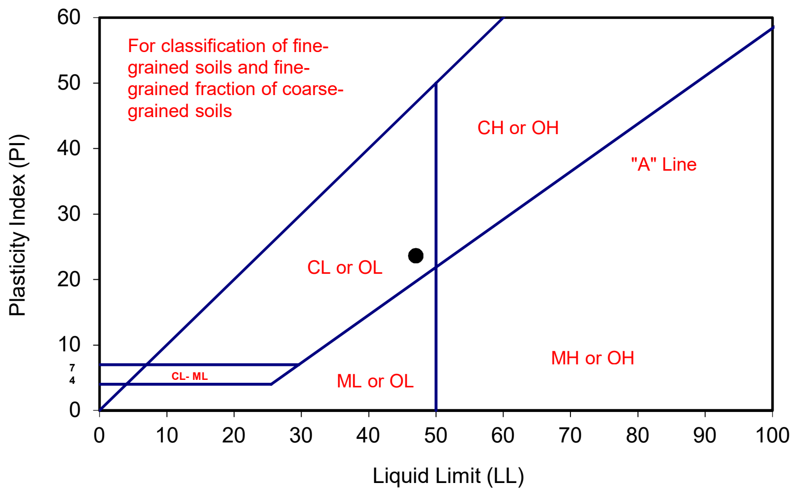
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			35	27	19	
Wet Wt. of Soil + Cont. (g)	8.62	8.61	20.78	20.78	20.10	
Dry Wt. of Soil + Cont. (g)	7.21	7.17	14.78	14.43	13.86	
Wt. of Container (g)	1.10	1.08	1.01	1.05	1.02	
Moisture Content (%) [Wn]	23.08	23.65	43.57	47.46	48.60	

Liquid Limit	**47**
Plastic Limit	**23**
Plasticity Index	**24**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) 19.71

One - Point Liquid Limit Calculation

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



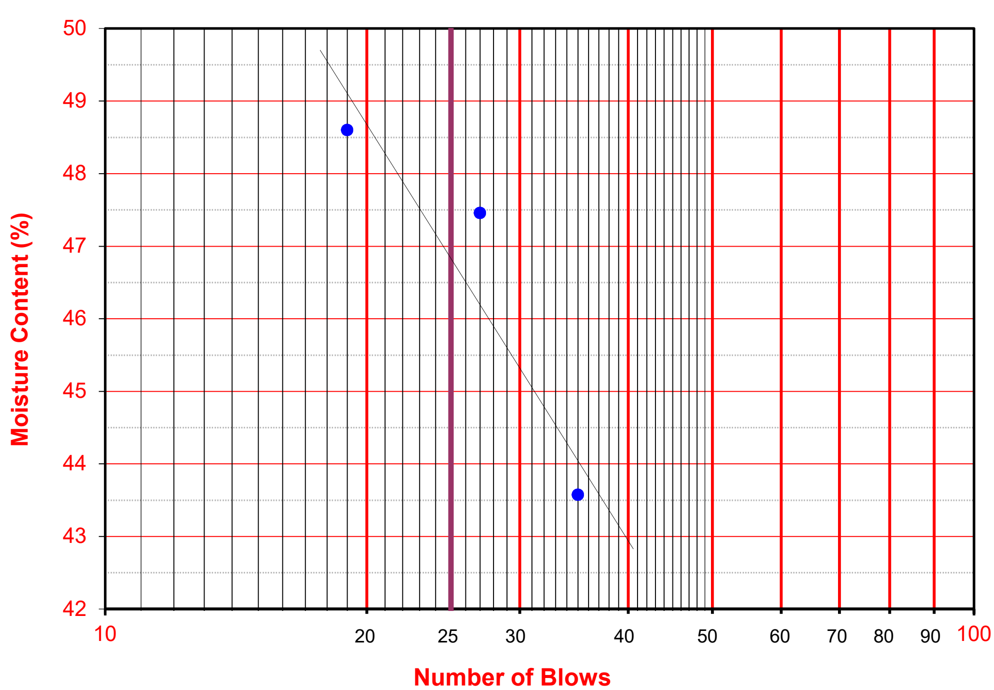
PROCEDURES USED

☐ Wet Preparation
Multipoint - Wet

☒ Dry Preparation
Multipoint - Dry

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test



ATTERBERG LIMITS
ASTM D 4318

Leighton

Project Name:	SMMUSD MMHS Phase 1	Tested By:	Y. Nguyen	Date:	08/27/20
Project No. :	11382.016	Input By:	G. Bathala	Date:	08/30/20
Boring No.:	2020-LB-4	Checked By:	J. Ward		
Sample No.:	R-3	Depth (ft.)	10.0		
Soil Identification:	Yellowish brown lean clay (CL), caliche noted				

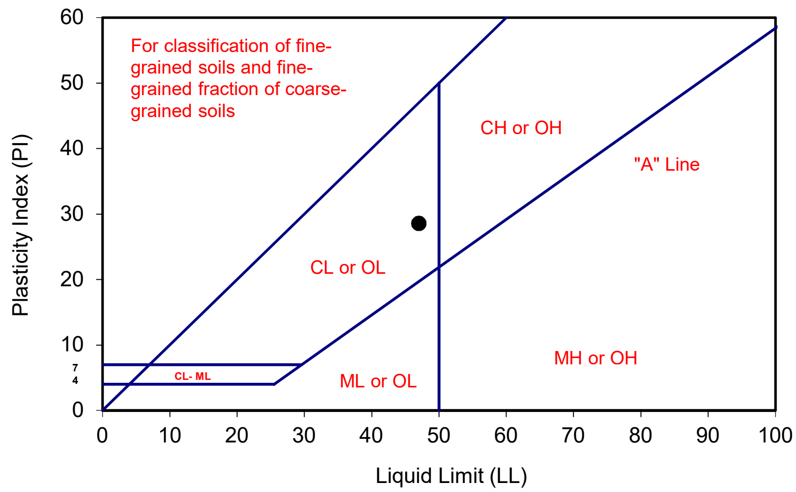
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			28	23	18	
Wet Wt. of Soil + Cont. (g)	9.36	9.27	19.90	20.70	23.39	
Dry Wt. of Soil + Cont. (g)	8.06	7.99	13.88	14.32	16.06	
Wt. of Container (g)	1.02	1.02	1.04	1.00	1.02	
Moisture Content (%) [Wn]	18.47	18.36	46.88	47.90	48.74	

Liquid Limit	**47**
Plastic Limit	**18**
Plasticity Index	**29**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) | 19.71

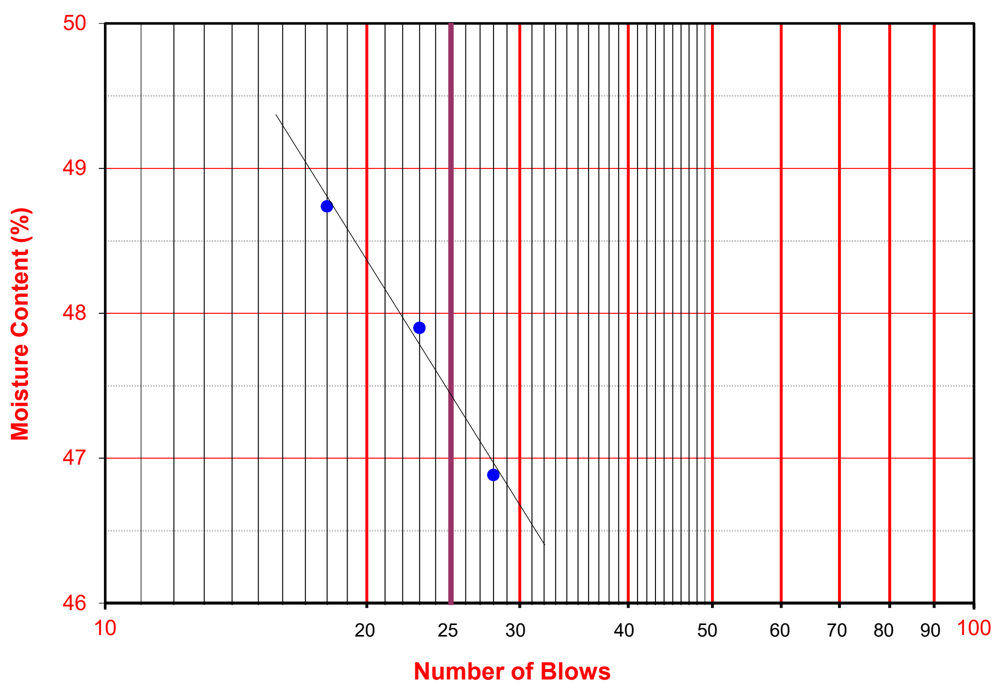
One - Point Liquid Limit Calculation

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



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

Leighton

Project Name:	SMMUSD MMHS Phase 1	Tested By:	Y. Nguyen	Date:	09/02/20
Project No. :	11382.016	Input By:	G. Bathala	Date:	09/03/20
Boring No.:	2020-LB-4	Checked By:	J. Ward		
Sample No.:	R-4	Depth (ft.)	15.0		
Soil Identification:	Yellowish brown lean clay (CL)				

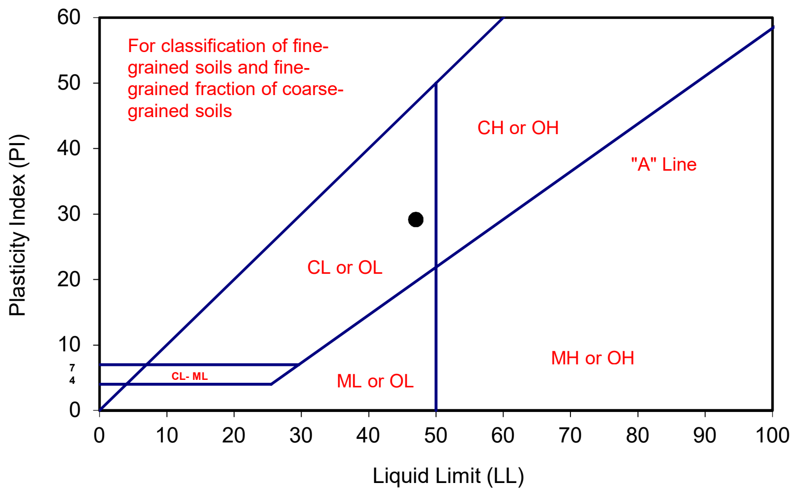
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			31	26	21	
Wet Wt. of Soil + Cont. (g)	9.49	9.56	20.55	20.60	20.79	
Dry Wt. of Soil + Cont. (g)	8.22	8.27	14.52	14.45	14.38	
Wt. of Container (g)	1.07	1.08	1.06	1.10	1.07	
Moisture Content (%) [Wn]	17.76	17.94	44.80	46.07	48.16	

Liquid Limit	**47**
Plastic Limit	**18**
Plasticity Index	**29**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) 19.71

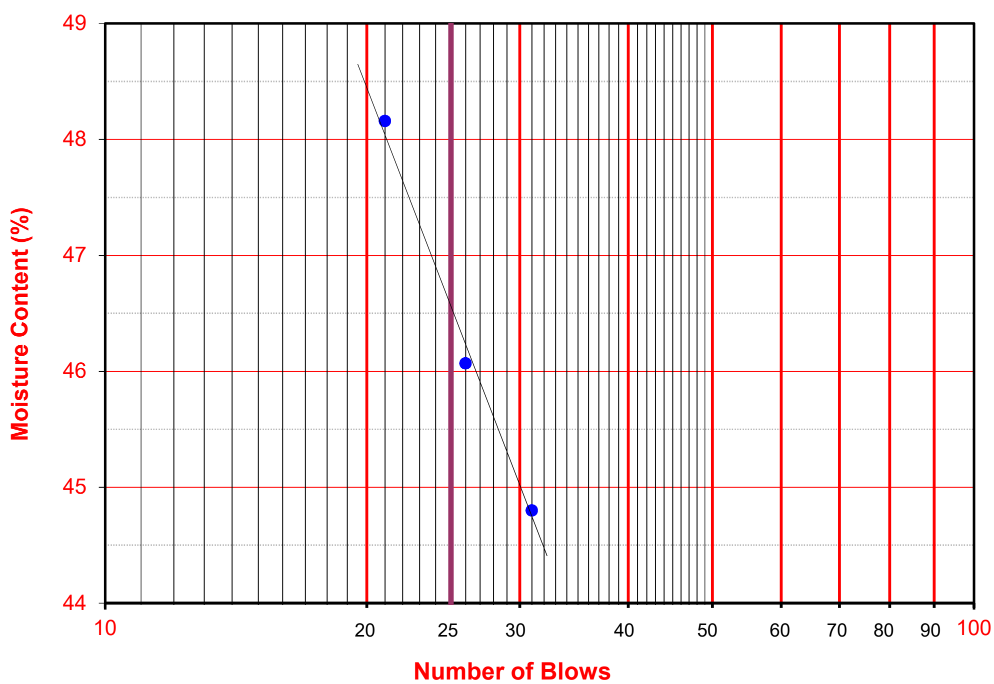
One - Point Liquid Limit Calculation

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



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

Leighton

Project Name:	SMMUSD MMHS Phase 1	Tested By:	Y. Nguyen	Date:	09/03/20
Project No. :	11382.016	Input By:	G. Bathala	Date:	09/04/20
Boring No.:	2020-LB-4	Checked By:	J. Ward		
Sample No.:	R-6	Depth (ft.)	25.0		
Soil Identification:	Yellowish brown fat clay (CH)				

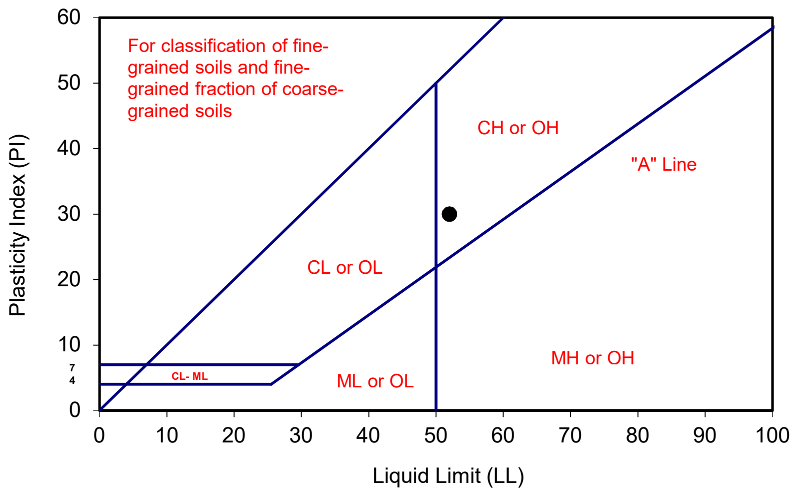
TEST NO.	PLASTIC LIMIT 1	PLASTIC LIMIT 2	LIQUID LIMIT 1	LIQUID LIMIT 2	LIQUID LIMIT 3	LIQUID LIMIT 4
Number of Blows [N]			29	22	15	
Wet Wt. of Soil + Cont. (g)	10.20	10.27	20.67	20.83	20.40	
Dry Wt. of Soil + Cont. (g)	8.54	8.61	14.08	13.99	13.51	
Wt. of Container (g)	1.05	1.02	1.10	1.05	1.06	
Moisture Content (%) [Wn]	22.16	21.87	50.77	52.86	55.34	

Liquid Limit	**52**
Plastic Limit	**22**
Plasticity Index	**30**
Classification	**CH**

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

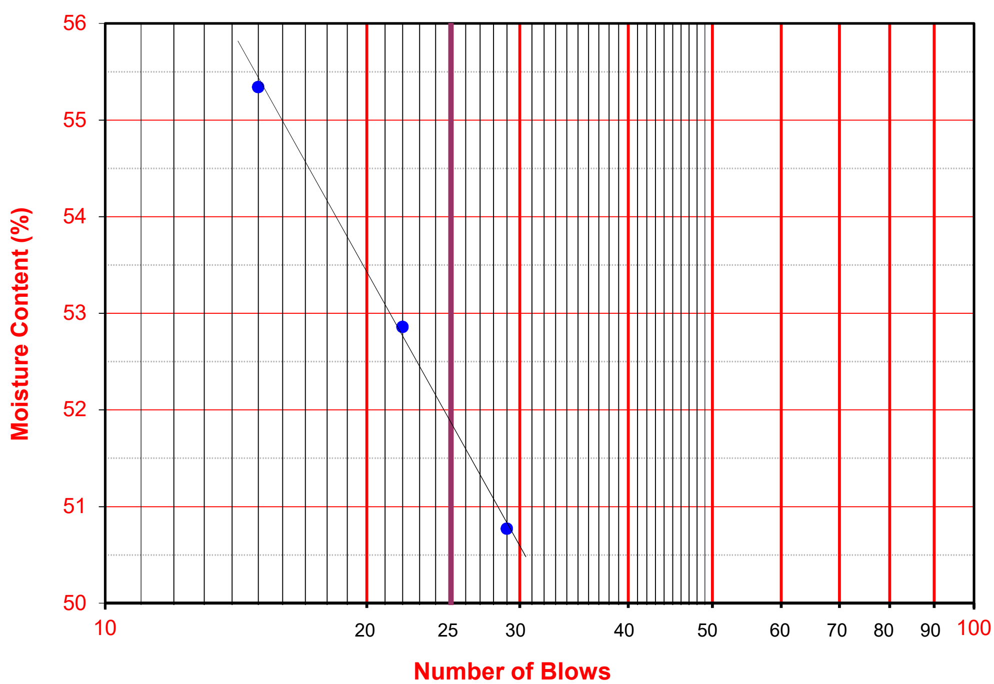
One - Point Liquid Limit Calculation

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



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

Leighton

Project Name:	SMMUSD MMHS Phase 1	Tested By:	Y. Nguyen	Date:	09/04/20
Project No. :	11382.016	Input By:	G. Bathala	Date:	09/08/20
Boring No.:	2020-LB-4	Checked By:	J. Ward		
Sample No.:	S-5	Depth (ft.)	20.0		
Soil Identification:	Brown fat clay (CH)				

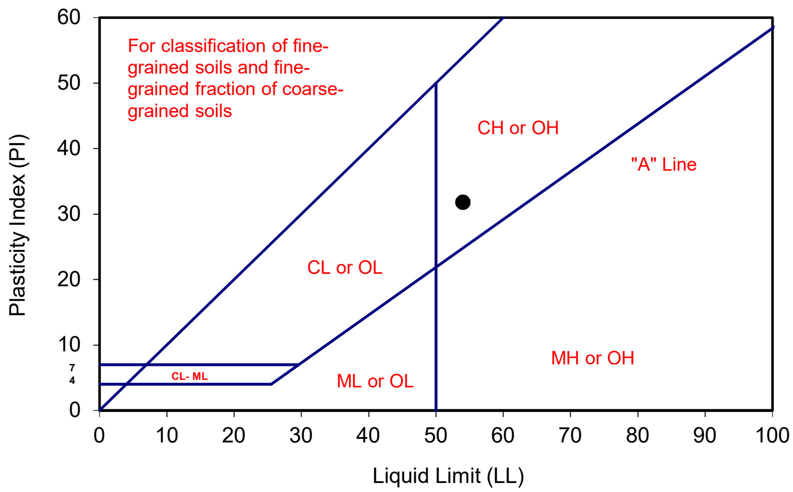
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			29	23	16	
Wet Wt. of Soil + Cont. (g)	10.15	10.22	21.59	21.29	20.70	
Dry Wt. of Soil + Cont. (g)	8.50	8.54	14.50	14.12	13.58	
Wt. of Container (g)	1.01	1.03	1.03	1.01	1.03	
Moisture Content (%) [Wn]	22.03	22.37	52.64	54.69	56.73	

Liquid Limit	**54**
Plastic Limit	**22**
Plasticity Index	**32**
Classification	**CH**

PI at "A" - Line = 0.73(LL-20) 24.82

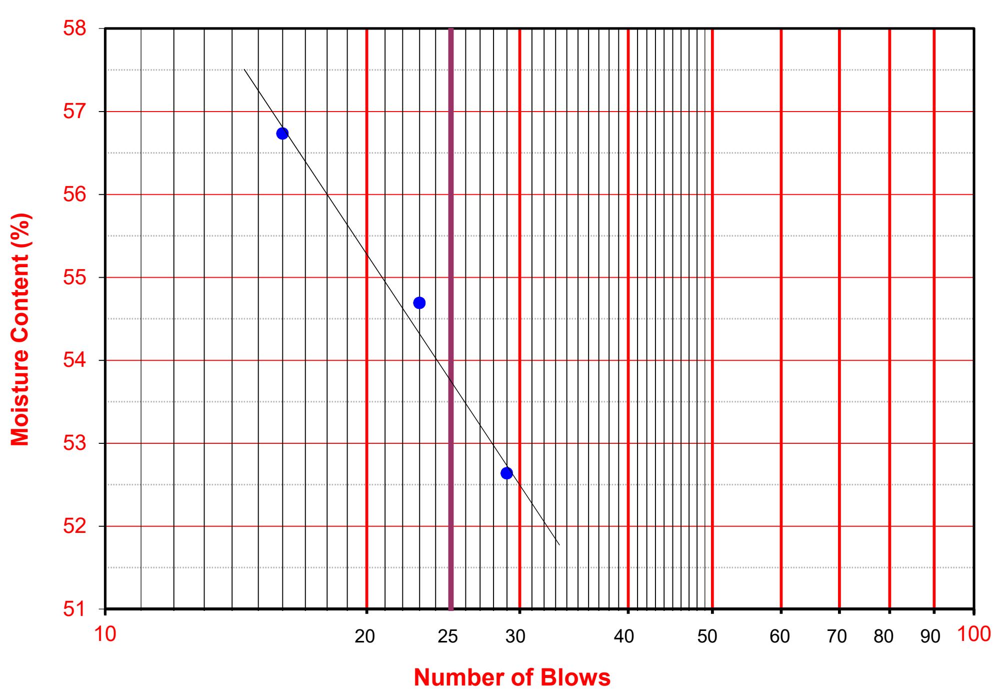
One - Point Liquid Limit Calculation

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



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

Leighton

Project Name:	SMMUSD MMHS Phase 1	Tested By:	Y. Nguyen	Date:	09/03/20
Project No. :	11382.016	Input By:	G. Bathala	Date:	09/08/20
Boring No.:	2020-LB-6	Checked By:	J. Ward		
Sample No.:	R-1	Depth (ft.)	5.0		
Soil Identification:	Yellowish brown fat clay (CH)				

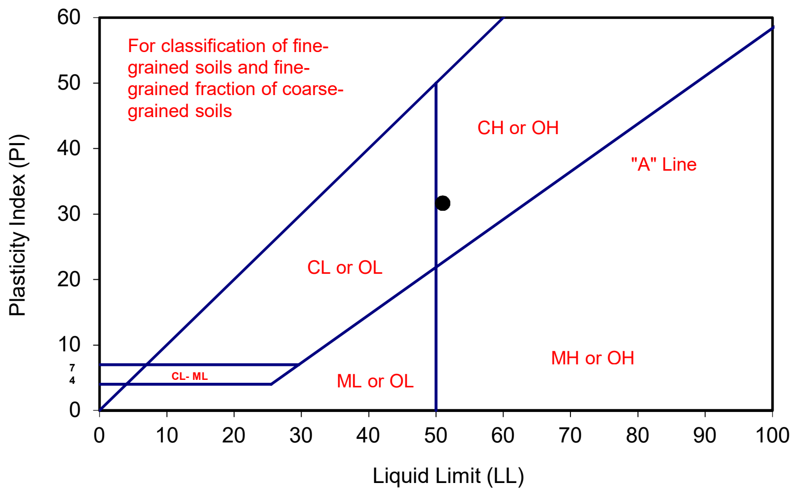
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			30	22	15	
Wet Wt. of Soil + Cont. (g)	10.11	10.17	21.40	20.68	20.32	
Dry Wt. of Soil + Cont. (g)	8.63	8.70	14.59	14.03	13.69	
Wt. of Container (g)	1.01	1.06	1.00	1.03	1.08	
Moisture Content (%) [Wn]	19.42	19.24	50.11	51.15	52.58	

Liquid Limit	**51**
Plasticity Index	**32**
Plastic Limit	**19**
Classification	**CH**

PI at "A" - Line = 0.73(LL-20) | 22.63

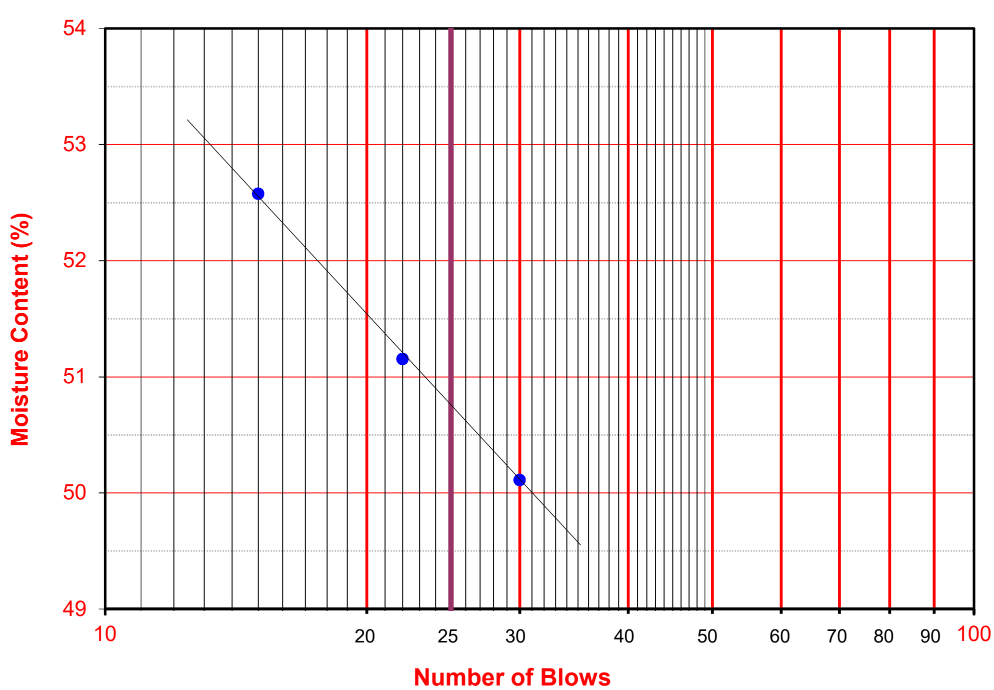
One - Point Liquid Limit Calculation

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



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:	SMMUSD MMHS Phase 1	Tested By:	Y. Nguyen	Date:	08/29/20
Project No. :	11382.016	Input By:	G. Bathala	Date:	09/03/20
Boring No.:	2020-LB-6	Checked By:	J. Ward		
Sample No.:	R-2	Depth (ft.)	7.0		
Soil Identification:	Dark yellowish brown fat clay (CH), caliche noted				

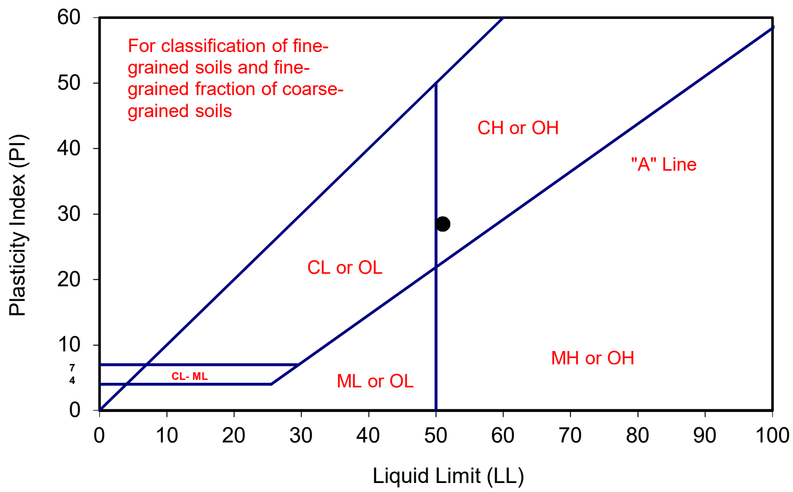
TEST NO.	PLASTIC LIMIT 1	PLASTIC LIMIT 2	LIQUID LIMIT 1	LIQUID LIMIT 2	LIQUID LIMIT 3	LIQUID LIMIT 4
Number of Blows [N]			28	22	15	
Wet Wt. of Soil + Cont. (g)	10.25	10.29	19.68	22.05	20.69	
Dry Wt. of Soil + Cont. (g)	8.58	8.58	13.47	14.92	13.84	
Wt. of Container (g)	1.08	1.08	1.04	1.00	1.05	
Moisture Content (%) [Wn]	22.27	22.80	49.96	51.22	53.56	

Liquid Limit	**51**
Plastic Limit	**23**
Plasticity Index	**28**
Classification	**CH**

PI at "A" - Line = 0.73(LL-20) 22.63

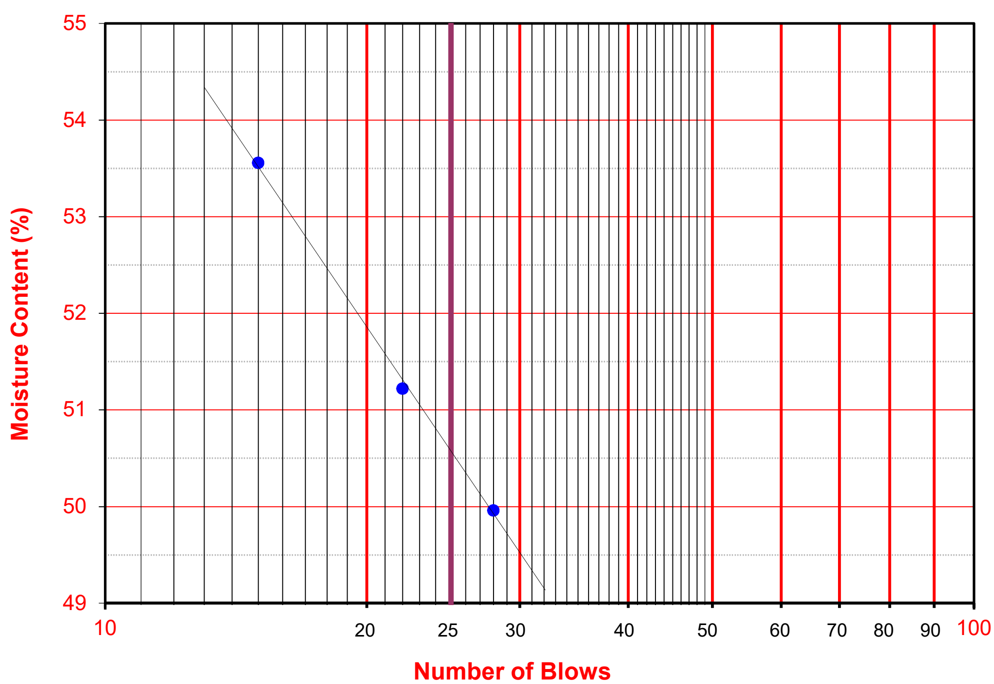
One - Point Liquid Limit Calculation

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



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

Leighton

Project Name:	SMMUSD MMHS Phase 1	Tested By:	Y. Nguyen	Date:	09/02/20
Project No. :	11382.016	Input By:	G. Bathala	Date:	09/03/20
Boring No.:	2020-LB-6	Checked By:	J. Ward		
Sample No.:	R-3	Depth (ft.)	10.0		
Soil Identification:	Yellowish brown fat clay (CH)				

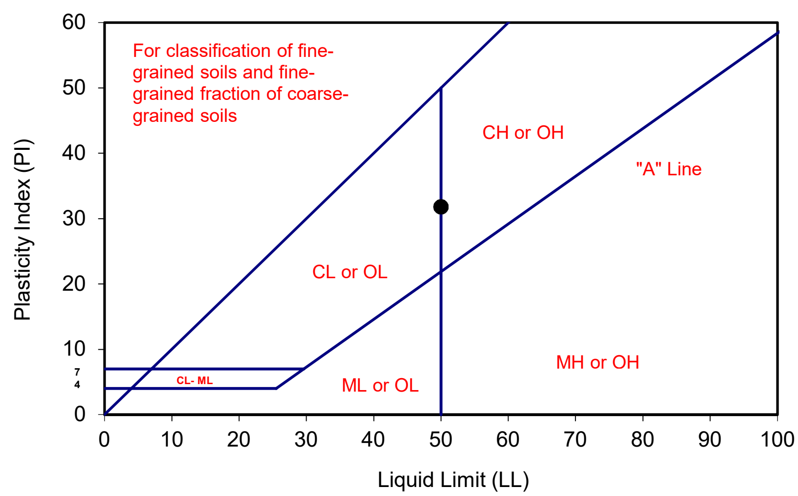
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			29	24	19	
Wet Wt. of Soil + Cont. (g)	9.92	10.08	20.20	19.60	19.14	
Dry Wt. of Soil + Cont. (g)	8.57	8.68	13.87	13.36	12.95	
Wt. of Container (g)	1.06	1.07	1.07	0.99	1.08	
Moisture Content (%) [Wn]	17.98	18.40	49.45	50.44	52.15	

Liquid Limit	**50**
Plastic Limit	**18**
Plasticity Index	**32**
Classification	**CH**

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

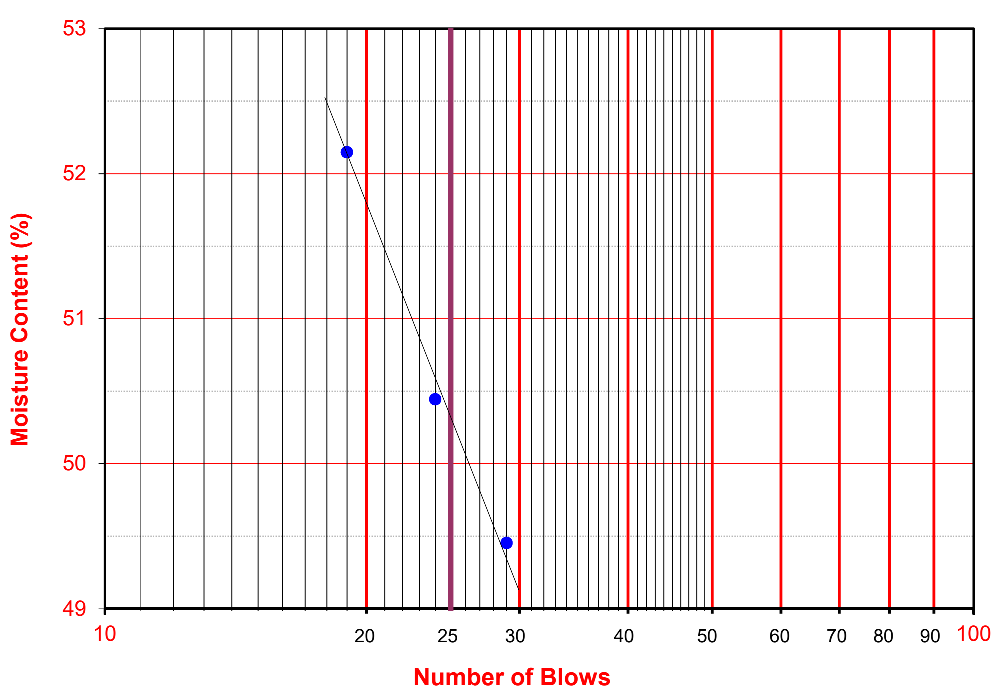
One - Point Liquid Limit Calculation

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



PROCEDURES USED

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



ATTERBERG LIMITS

ASTM D 4318

Leighton

Project Name:	SMMUSD MMHS Phase 1	Tested By:	Y. Nguyen	Date:	09/02/20
Project No. :	11382.016	Input By:	G. Bathala	Date:	09/03/20
Boring No.:	2020-LB-6	Checked By:	J. Ward		
Sample No.:	R-4	Depth (ft.)	15.0		
Soil Identification:	Dark yellowish brown lean clay (CL), caliche noted				

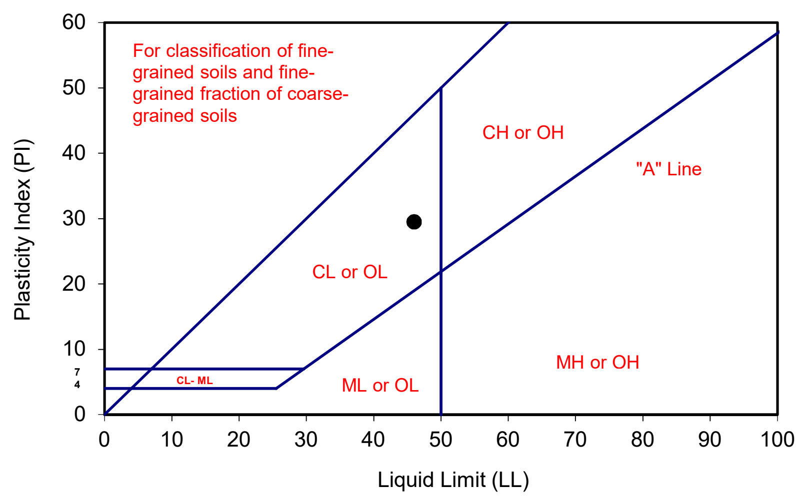
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			32	26	19	
Wet Wt. of Soil + Cont. (g)	8.87	8.94	21.90	20.20	20.90	
Dry Wt. of Soil + Cont. (g)	7.77	7.81	15.45	14.18	14.55	
Wt. of Container (g)	1.05	1.00	1.08	1.04	1.10	
Moisture Content (%) [Wn]	16.37	16.59	44.89	45.81	47.21	

Liquid Limit	**46**
Plastic Limit	**16**
Plasticity Index	**30**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) | 18.98

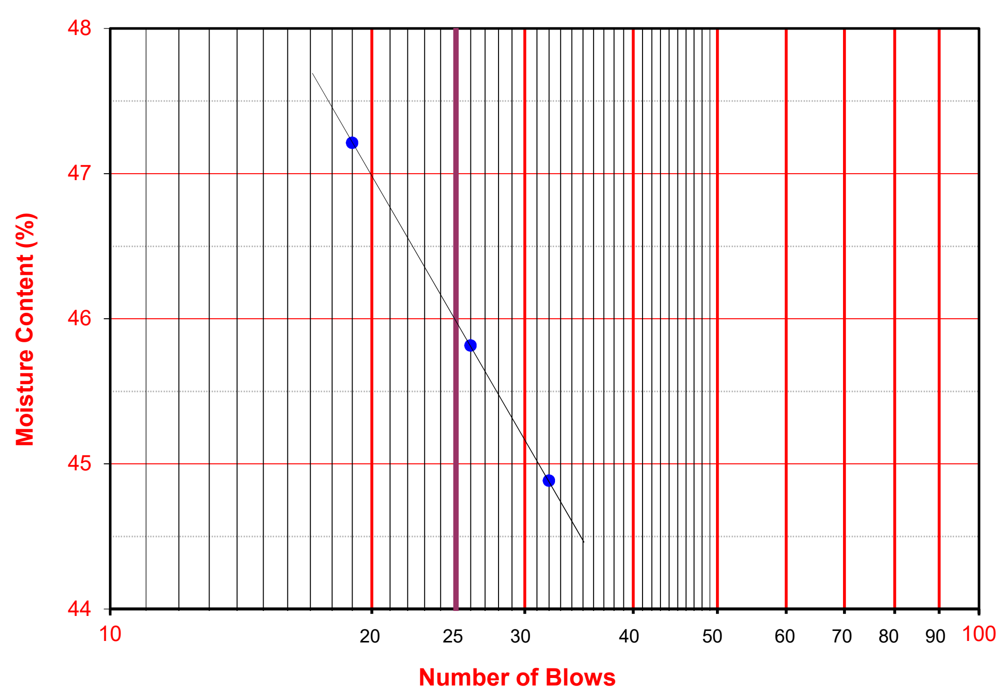
One - Point Liquid Limit Calculation

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



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

Leighton

Project Name:	SMMUSD MMHS Phase 1	Tested By:	Y. Nguyen	Date:	09/02/20
Project No. :	11382.016	Input By:	G. Bathala	Date:	09/04/20
Boring No.:	2020-LB-6	Checked By:	J. Ward		
Sample No.:	R-6	Depth (ft.)	25.0		
Soil Identification:	Brown lean clay (CL)				

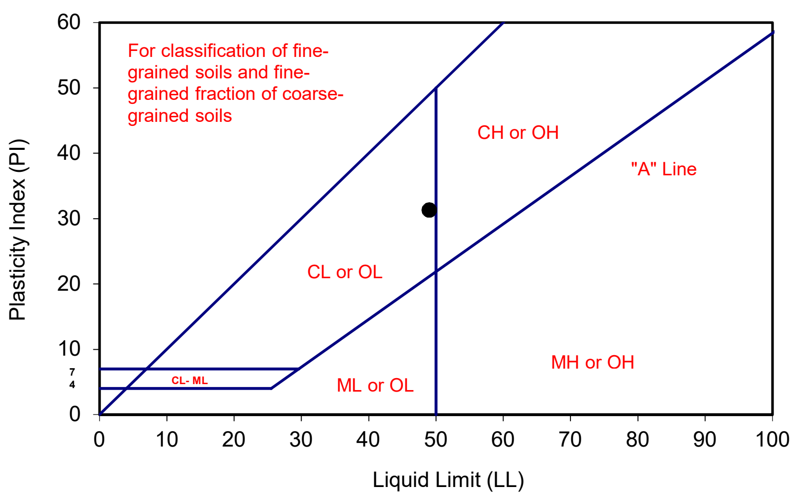
TEST NO.	PLASTIC LIMIT 1	PLASTIC LIMIT 2	LIQUID LIMIT 1	LIQUID LIMIT 2	LIQUID LIMIT 3	LIQUID LIMIT 4
Number of Blows [N]			32	25	17	
Wet Wt. of Soil + Cont. (g)	21.46	21.13	33.05	33.93	33.56	
Dry Wt. of Soil + Cont. (g)	19.91	19.62	26.72	27.17	26.69	
Wt. of Container (g)	11.15	11.09	13.45	13.34	13.11	
Moisture Content (%) [Wn]	17.69	17.70	47.70	48.88	50.59	

Liquid Limit	**49**
Plastic Limit	**18**
Plasticity Index	**31**
Classification	**CL**

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

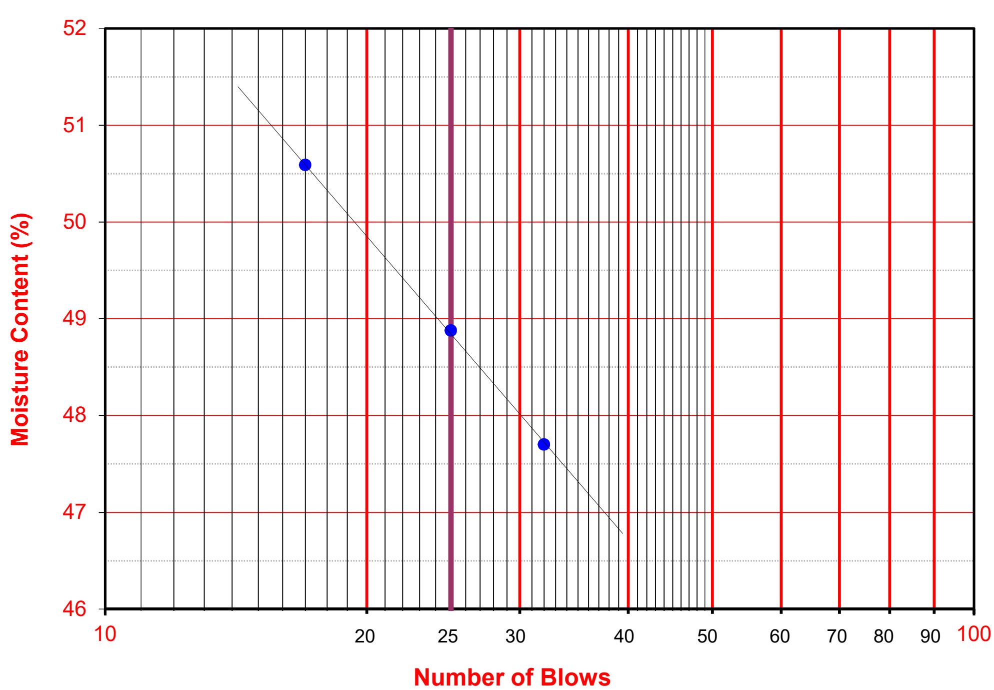
One - Point Liquid Limit Calculation

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



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

Leighton

Project Name:	SMMUSD MMHS Phase 1	Tested By:	Y. Nguyen	Date:	09/03/20
Project No. :	11382.016	Input By:	G. Bathala	Date:	09/04/20
Boring No.:	2020-LB-6	Checked By:	J. Ward		
Sample No.:	S-5	Depth (ft.)	20.0		
Soil Identification:	Strong brown fat clay (CH)				

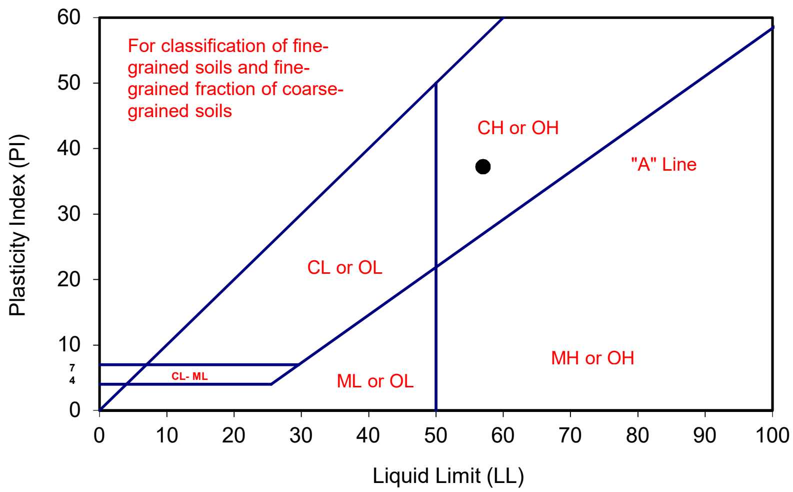
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			28	21	15	
Wet Wt. of Soil + Cont. (g)	9.62	9.54	20.54	19.61	20.44	
Dry Wt. of Soil + Cont. (g)	8.20	8.15	13.48	12.83	13.26	
Wt. of Container (g)	1.01	1.11	1.02	1.09	1.07	
Moisture Content (%) [Wn]	19.75	19.74	56.66	57.75	58.90	

Liquid Limit	**57**
Plastic Limit	**20**
Plasticity Index	**37**
Classification	**CH**

PI at "A" - Line = 0.73(LL-20) | 27.01

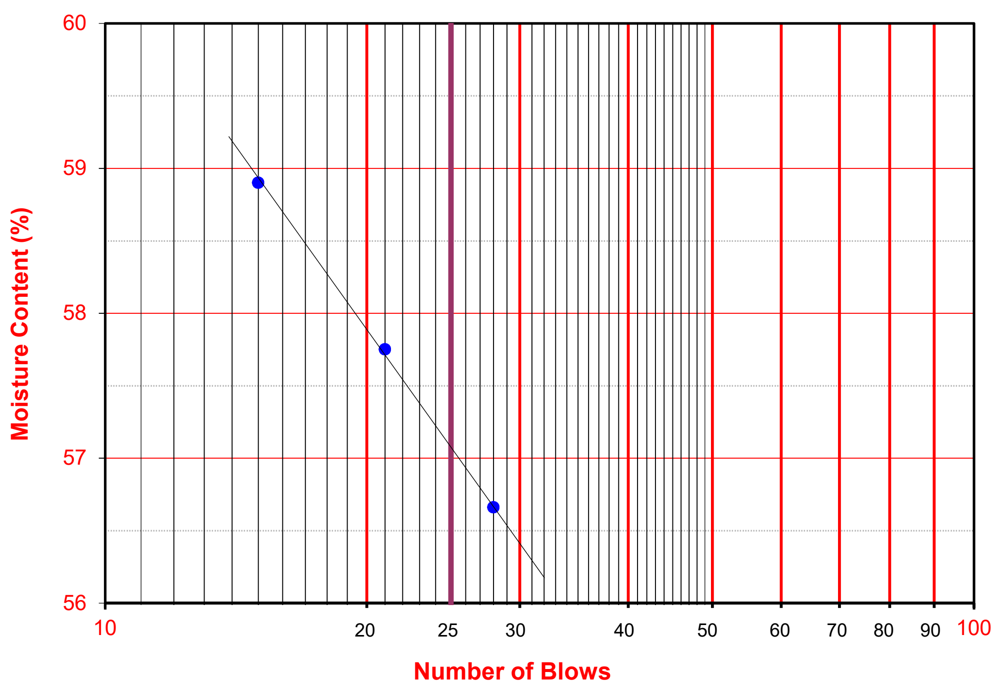
One - Point Liquid Limit Calculation

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



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

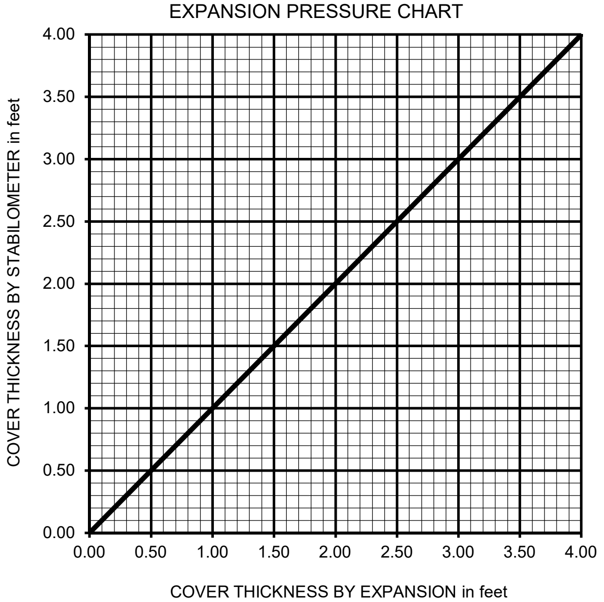
Leighton

PROJECT NAME:	SMMUSD MMHS Phase 1	PROJECT NUMBER:	11382.016
BORING NUMBER:	2020-LB-4	DEPTH (FT.):	0-5
SAMPLE NUMBER:	B-1	TECHNICIAN:	O. Figueroa
SAMPLE DESCRIPTION:	Dark yellowish brown (CL)s	DATE COMPLETED:	8/29/2020

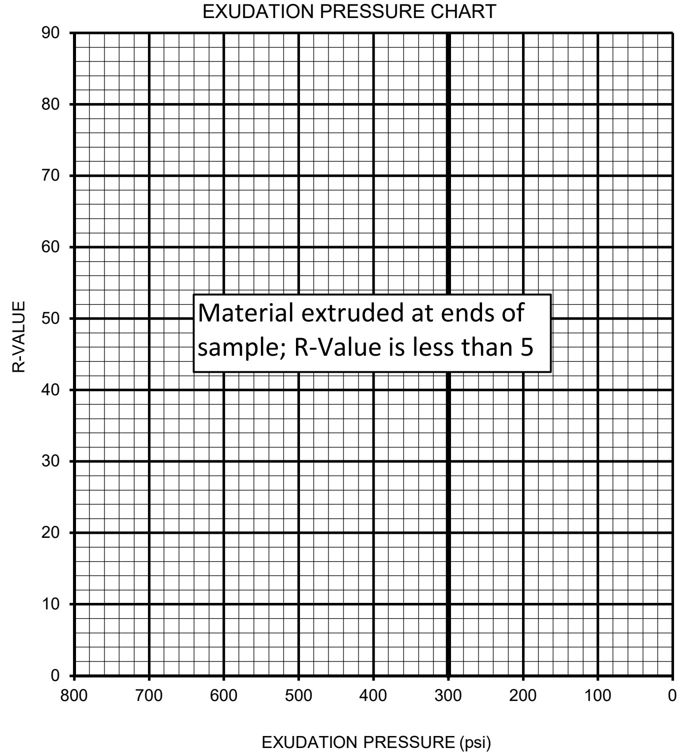
TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %			
HEIGHT OF SAMPLE, Inches			
DRY DENSITY, pcf			
COMPACTOR PRESSURE, psi			
EXUDATION PRESSURE, psi			
EXPANSION, Inches x 10exp-4			
STABILITY Ph 2,000 lbs (160 psi)			
TURNS DISPLACEMENT			
R-VALUE UNCORRECTED	N/A	N/A	N/A
R-VALUE CORRECTED	N/A	N/A	N/A

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

EXPANSION PRESSURE CHART



EXUDATION PRESSURE CHART



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

Leighton

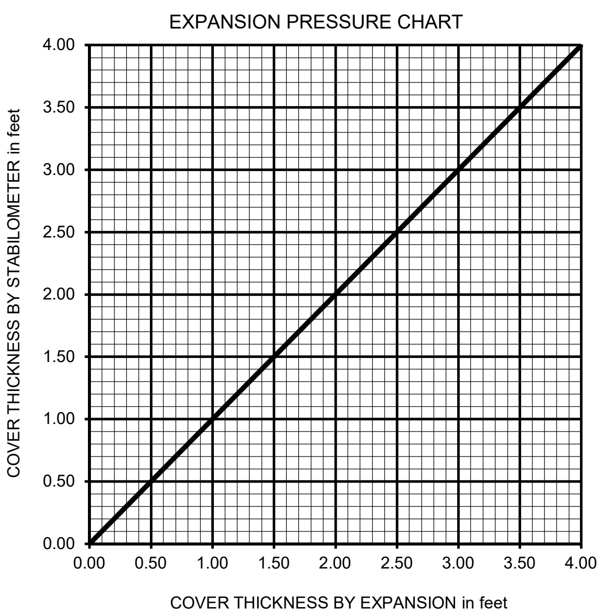
R-VALUE TEST RESULTS

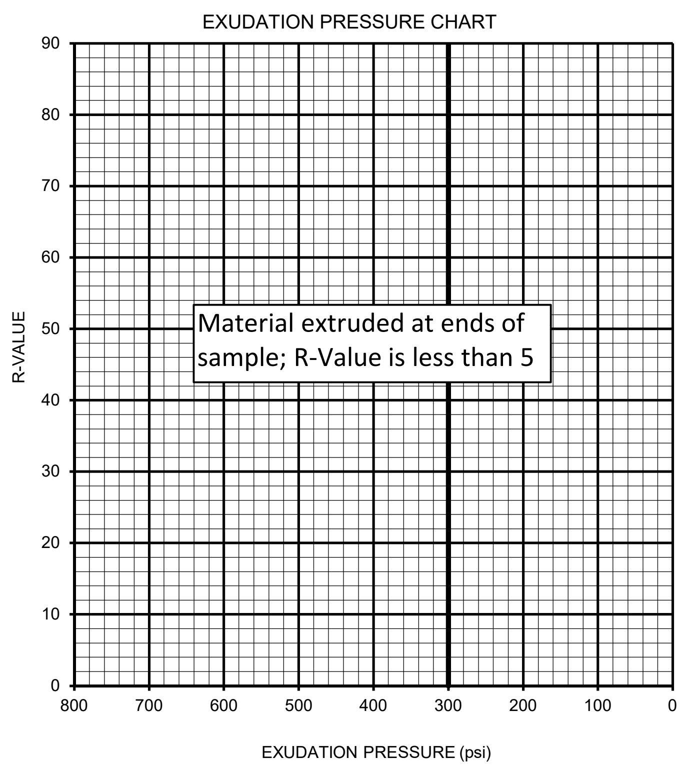
DOT CA Test 301

PROJECT NAME:	SMMUSD MMHS Phase 1	PROJECT NUMBER:	11382.016
BORING NUMBER:	2020-LB-6	DEPTH (FT.):	0-5
SAMPLE NUMBER:	B-1	TECHNICIAN:	O. Figueroa
SAMPLE DESCRIPTION:	Dark yellowish brown (CL)s	DATE COMPLETED:	8/29/2020

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

SEISMICITY DATA

Appendix D - Seismic Design

Code-based seismic design parameters derived from the USGS online tool are appended at the end of this appendix.

Leighton performed a site-specific ground motion study using the computer program Hazard Spectrum Application (OpenSHA, 2020), in accordance with the requirements of the 2019 California Building Code (CBC) and ASCE 7-16. A site-specific seismic hazard analysis was performed and the results processed in accordance with the procedures of ASCE 7-16.

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. Site Class D and an average shear wave velocity in the upper 30 meters equal to 330 meters per second based on the results of our exploration at the site were used with the selected GMPEs.

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 "$v(Z \geq z)$." 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, $v(Z \geq z)$, the probability of exceeding that level $Pr(Z \geq z)$, within any time period of interest, t, is then obtained assuming a Poisson Distribution as follows:

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

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 Hazard Spectrum Application (OpenSHA, 2020) was used to perform the seismic hazard analysis.

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 faults and characteristic magnitudes used to develop the deterministic MCE spectrum are included in the following table. The distances used are based on the deaggregation results obtained from the USGS Unified Hazard Tool website (https://earthquake.usgs.gov/hazards/interactive/).

Fault	Moment of Magnitude (Mw)	Distance (km)
Malibu Coast alt 1	6.63	1.7
Malibu Coast alt 2	7.37	2.4
Ancapa-Dume alt 1	7.26	5.8
Ancapa-Dume alt 2	7.32	5.3
San Pedro Basin	6.77	10.0

The fault magnitudes used in the deterministic analysis were based on the Building Seismic Safety Council (BSSC) 2014 scenario catalog website (https://earthquake.usgs.gov/scenarios/catalog/bssc2014/). The input used in the NGA-West2 deterministic spreadsheet is 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 SEAOC/OSHPD Seismic Design Maps Tool (https://seismicmaps.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 addendum.

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 RotD50 and the maximum component MCE response spectra from the PSHA. The maximum component (MC) factors from Shahi and Baker (2014) are also presented on Figure D.1.

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

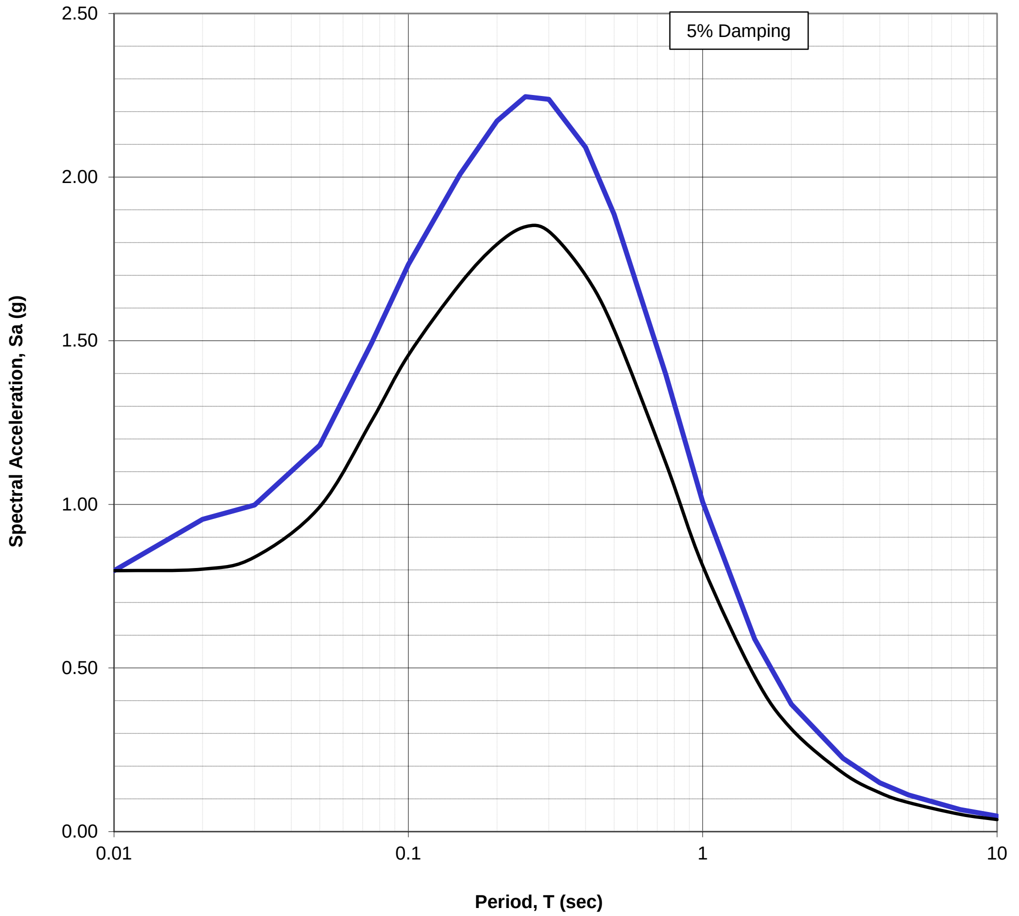
Per Chapter 21.2.3 of ASCE 7-16, the deterministic and probabilistic spectra were compared to establish site-specific maximum component MCE spectra. This step is shown on Figure D.3.

Figure D.4 shows a comparison of site-specific vs. general code-based spectra for the MCE_R spectrum.

The DE spectrum is shown on Figure D.5, which also includes the 80% of the general code-based spectrum floor stipulated in Chapter 21.3 of ASCE 7-16.

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

Project: MMHS Phase I High School Core Project
Project Number: 11382.016
Location: 30237 Morning View Drive, Malibu, California



Deaggregated Probabilistic PGA Magnitude
Mw = 7.0

Period T (s)	MCE GEOMEAN Sa (g)	Maximum Component Factor	MCE MAX COMP Site-Specific Sa (g)
0.01	0.797	1.00	0.797
0.02	0.802	1.19	0.954
0.03	0.839	1.19	0.998
0.05	0.993	1.19	1.181
0.075	1.255	1.19	1.493
0.10	1.456	1.19	1.733
0.15	1.674	1.20	2.009
0.20	1.795	1.21	2.172
0.25	1.849	1.22	2.246
0.30	1.834	1.22	2.238
0.40	1.700	1.23	2.091
0.50	1.534	1.23	1.886
0.75	1.125	1.24	1.395
1.00	0.812	1.24	1.007
1.50	0.475	1.24	0.589
2.00	0.314	1.24	0.389
3.00	0.179	1.25	0.224
4.00	0.119	1.26	0.149
5.00	0.089	1.26	0.112
7.50	0.053	1.28	0.068
10.00	0.037	1.29	0.048

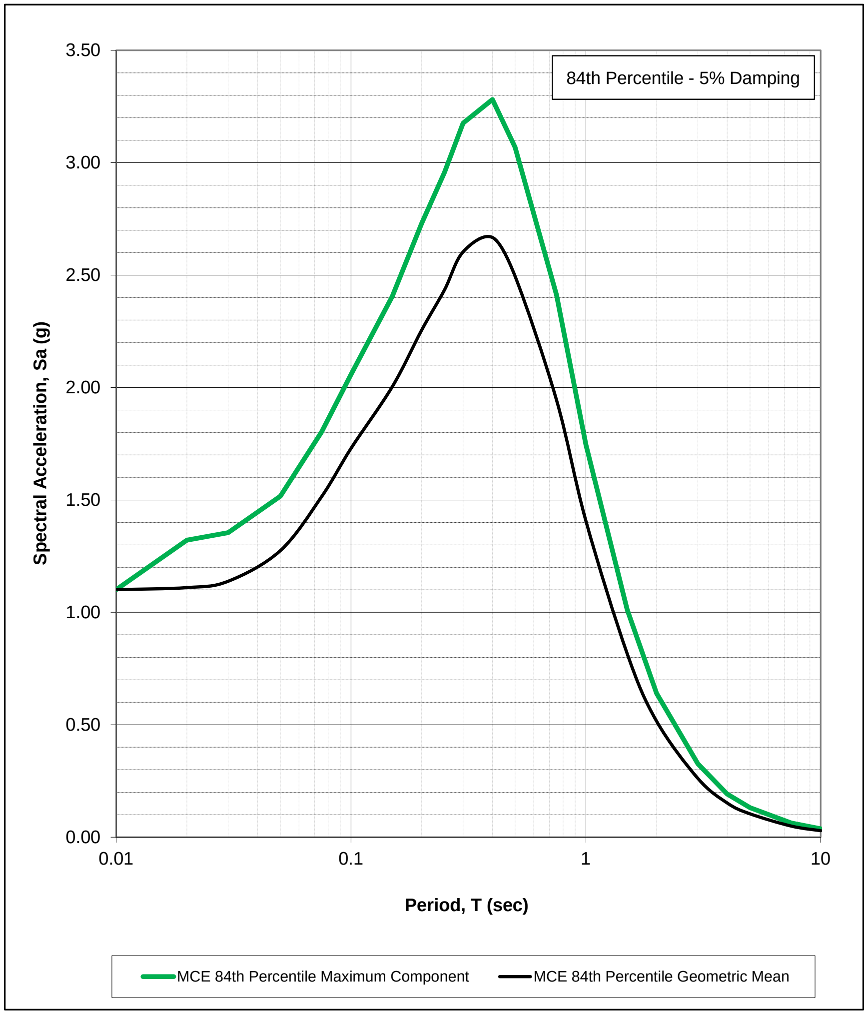
Figure D.1

MCE DETERMINISTIC SPECTRA

Project: MMHS Phase I High School Core Project

Project Number: 11382.016

Location: 30237 Morning View Drive, Malibu, California



DETERMINISTIC PGA MAGNITUDE				
MC FACTOR		DSHA - 84TH PERCENTILE		
Period T (s)	Maximum Component Factor	Period T (s)	MCE GEOMEAN Sa (g)	DETERMINISTIC MCE_R Sa (g)
0.01	1.00	0.01	1.101	1.101
0.02	1.19	0.02	1.110	1.321
0.03	1.19	0.03	1.138	1.355
0.05	1.19	0.05	1.274	1.516
0.075	1.19	0.075	1.515	1.803
0.10	1.19	0.10	1.729	2.058
0.15	1.20	0.15	2.004	2.405
0.20	1.21	0.20	2.256	2.729
0.25	1.22	0.25	2.434	2.957
0.30	1.22	0.30	2.603	3.176
0.40	1.23	0.40	2.668	3.281
0.50	1.23	0.50	2.494	3.068
0.75	1.24	0.75	1.945	2.412
1.00	1.24	1.00	1.408	1.746
1.50	1.24	1.50	0.816	1.012
2.00	1.24	2.00	0.516	0.640
3.00	1.25	3.00	0.261	0.327
4.00	1.26	4.00	0.152	0.192
5.00	1.26	5.00	0.104	0.131
7.50	1.28	7.50	0.049	0.063
10.00	1.29	10.00	0.029	0.037

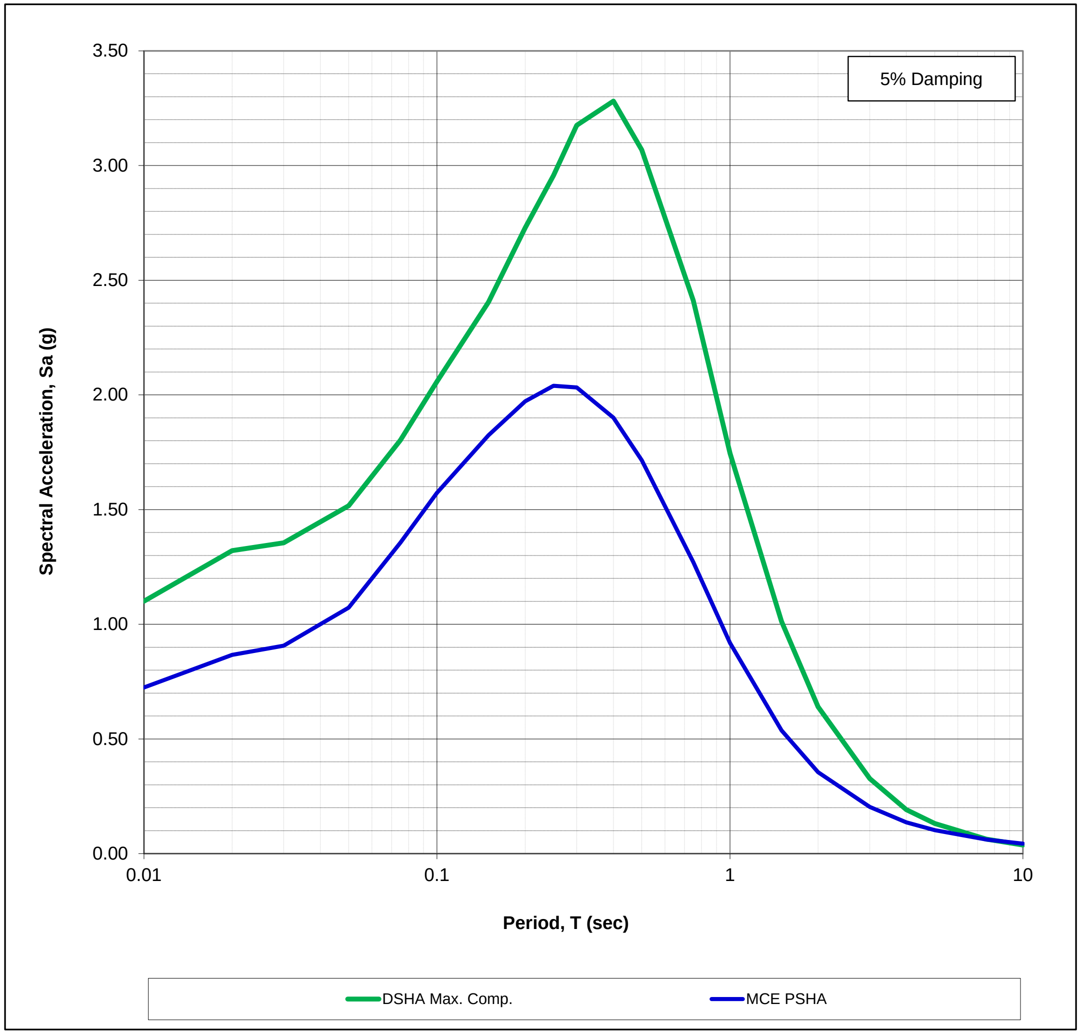
Figure D.2

MCE DETERMINISTIC SPECTRA COMPARISON - MAXIMUM HORIZONTAL COMPONENT

Project: MMHS Phase I High School Core Project

Project Number: 11382.016

Location: 30237 Morning View Drive, Malibu, California



DSHA		PSHA			
Period T (s)	MCE_R Sa (g)	Period T (s)	MCE MAX COMP. Sa (g)	Site Risk Coefficient (C_R)	MCE_R Sa (g)
0.01	1.101	0.01	0.797	0.908	0.724
0.02	1.321	0.02	0.954	0.908	0.867
0.03	1.355	0.03	0.998	0.908	0.906
0.05	1.516	0.05	1.181	0.908	1.073
0.075	1.803	0.075	1.493	0.908	1.356
0.10	2.058	0.10	1.733	0.908	1.573
0.15	2.405	0.15	2.009	0.908	1.824
0.20	2.729	0.20	2.172	0.908	1.972
0.25	2.957	0.25	2.246	0.908	2.040
0.30	3.176	0.30	2.238	0.909	2.033
0.40	3.281	0.40	2.091	0.909	1.901
0.50	3.068	0.50	1.886	0.910	1.716
0.75	2.412	0.75	1.395	0.911	1.271
1.00	1.746	1.00	1.007	0.912	0.919
1.50	1.012	1.50	0.589	0.912	0.537
2.00	0.640	2.00	0.389	0.912	0.355
3.00	0.327	3.00	0.224	0.912	0.204
4.00	0.192	4.00	0.149	0.912	0.136
5.00	0.131	5.00	0.112	0.912	0.102
7.50	0.063	7.50	0.068	0.912	0.062
10.00	0.037	10.00	0.048	0.912	0.043

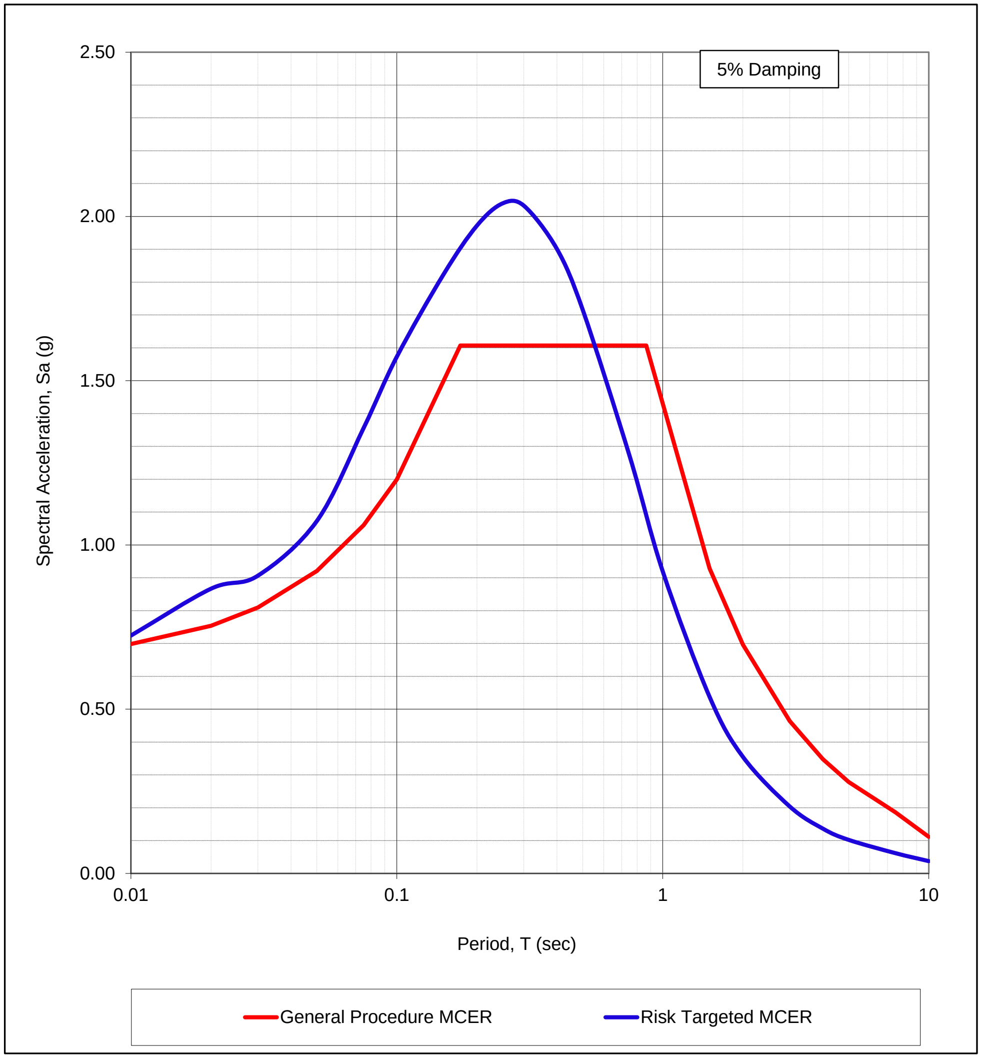
Figure D.3

RISK TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) RESPONSE SPECTRUM

Project: MMHS Phase I High School Core Project

Project Number: 11382.016

Location: 30237 Morning View Drive, Malibu, California



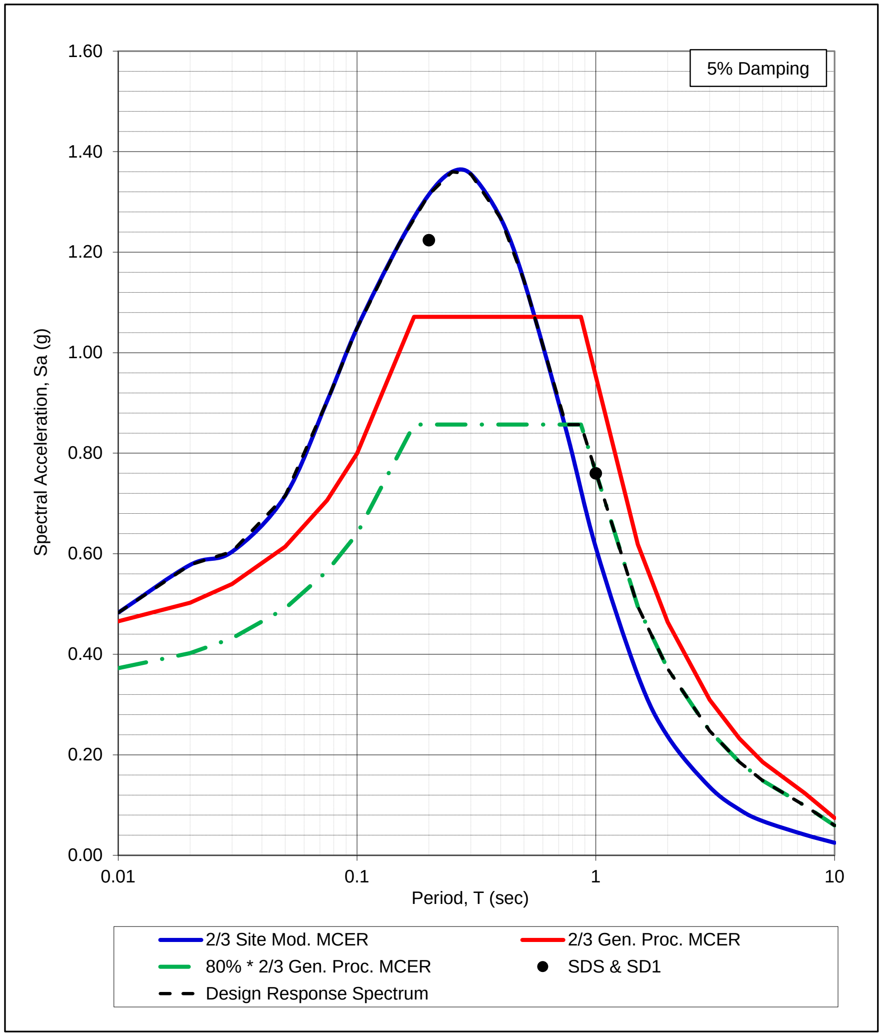
SITE-SPECIFIC vs. GENERAL CODE-BASED SPECTRA

Period T (s)	DETERM. MCE_R Sa (g)	PROB. MCE_R Sa (g)	**Risk TGT MCE_R Sa (g)**	General Procedure MCE_R Sa (g)
0.01	1.101	0.724	**0.724**	0.698
0.02	1.321	0.867	**0.867**	0.754
0.03	1.355	0.906	**0.906**	0.810
0.05	1.516	1.073	**1.073**	0.921
0.075	1.803	1.356	**1.356**	1.060
0.10	2.058	1.573	**1.573**	1.199
0.15	2.405	1.824	**1.824**	1.477
0.20	2.729	1.972	**1.972**	1.607
0.25	2.957	2.040	**2.040**	1.607
0.30	3.176	2.033	**2.033**	1.607
0.40	3.281	1.901	**1.901**	1.607
0.50	3.068	1.716	**1.716**	1.607
0.75	2.412	1.271	**1.271**	1.607
1.00	1.746	0.919	**0.919**	1.425
1.50	1.012	0.537	**0.537**	0.928
2.00	0.640	0.355	**0.355**	0.696
3.00	0.327	0.204	**0.204**	0.464
4.00	0.192	0.136	**0.136**	0.348
5.00	0.131	0.102	**0.102**	0.279
7.50	0.063	0.062	**0.062**	0.186
10.00	0.037	0.043	**0.037**	0.111

Figure D.4

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

Project: MMHS Phase I High School Core Project
Project Number: 11382.016
Location: 30237 Morning View Drive, Malibu, California



	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	0.698	0.466	0.372	0.483	**0.483**
0.02	0.754	0.503	0.402	0.578	**0.578**
0.03	0.810	0.540	0.432	0.604	**0.604**
0.05	0.921	0.614	0.491	0.715	**0.715**
0.075	1.060	0.707	0.565	0.904	**0.904**
0.10	1.199	0.799	0.640	1.049	**1.049**
0.15	1.477	0.985	0.788	1.216	**1.216**
0.20	1.607	1.071	0.857	1.315	**1.315**
0.25	1.607	1.071	0.857	1.360	**1.360**
0.30	1.607	1.071	0.857	1.355	**1.355**
0.40	1.607	1.071	0.857	1.267	**1.267**
0.50	1.607	1.071	0.857	1.144	**1.144**
0.75	1.607	1.071	0.857	0.847	**0.857**
1.00	1.425	0.950	0.760	0.613	**0.760**
1.50	0.928	0.619	0.495	0.358	**0.495**
2.00	0.696	0.464	0.371	0.237	**0.371**
3.00	0.464	0.309	0.248	0.136	**0.248**
4.00	0.348	0.232	0.186	0.091	**0.186**
5.00	0.279	0.186	0.149	0.068	**0.149**
7.50	0.186	0.124	0.099	0.041	**0.099**
10.00	0.111	0.074	0.059	0.025	**0.059**

S_{DS} = 1.224 g
S_{D1} = 0.760 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 $Vs_{30} \le$ 1,200 ft/s ($Vs_{30} \le$ 365.76 m/s). The design Sa shall not be less than 80% of 2/3 of the general procedure (Sec 11.4.6)

Figure D.5

11382.016

Latitude, Longitude: 34.0251, -118.8286

Date	11/17/2020, 1:52:27 PM
Design Code Reference Document	ASCE7-16
Risk Category	IV
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.607	MCE_R ground motion. (for 0.2 second period)
S_1	0.557	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.607	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.071	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.695	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.764	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.607	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.77	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.678	Factored deterministic acceleration value. (0.2 second)
S1RT	0.557	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.611	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.883	Factored deterministic acceleration value. (1.0 second)
PGAd	1.085	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.908	Mapped value of the risk coefficient at short periods
C_{R1}	0.912	Mapped value of the risk coefficient at a period of 1 s

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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 (u…

Latitude

Decimal degrees

34.0251

Longitude

Decimal degrees, negative values for western longitudes

-118.8286

Site Class

259 m/s (Site class D)

Spectral Period

Peak Ground Acceleration

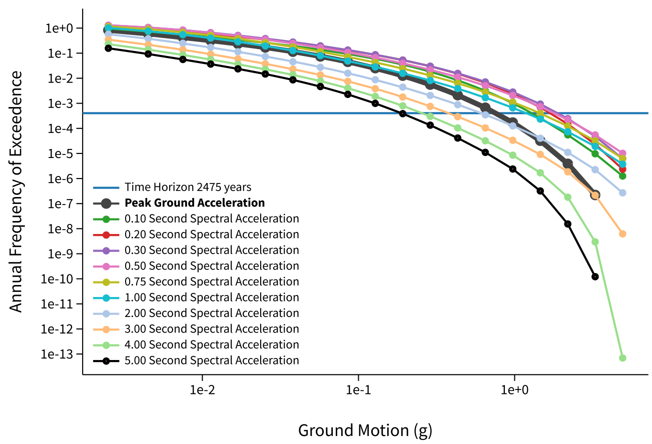
Time Horizon

Return period in years

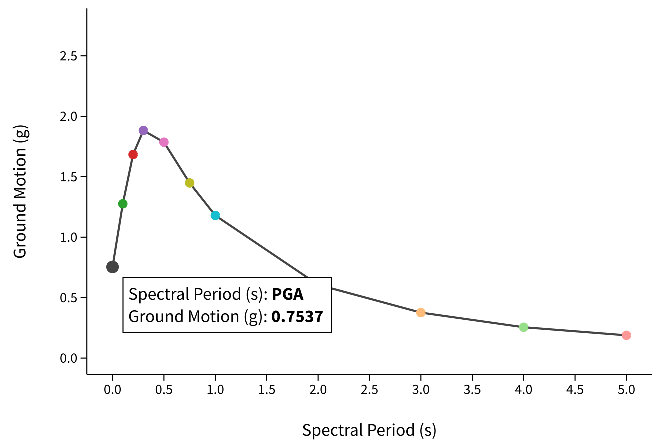
2475

Hazard Curve

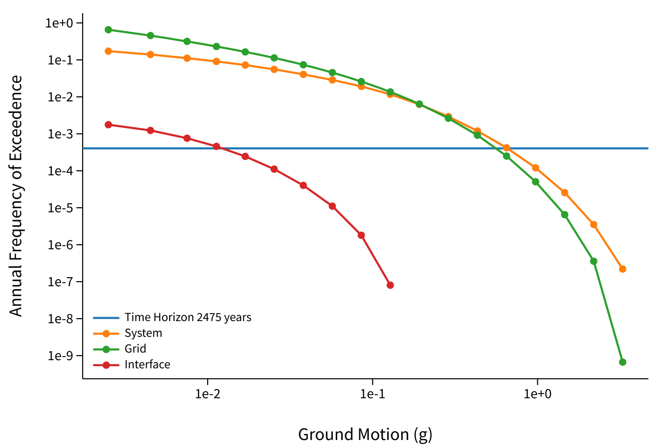
Hazard Curves



Uniform Hazard Response Spectrum



Component Curves for Peak Ground Acceleration

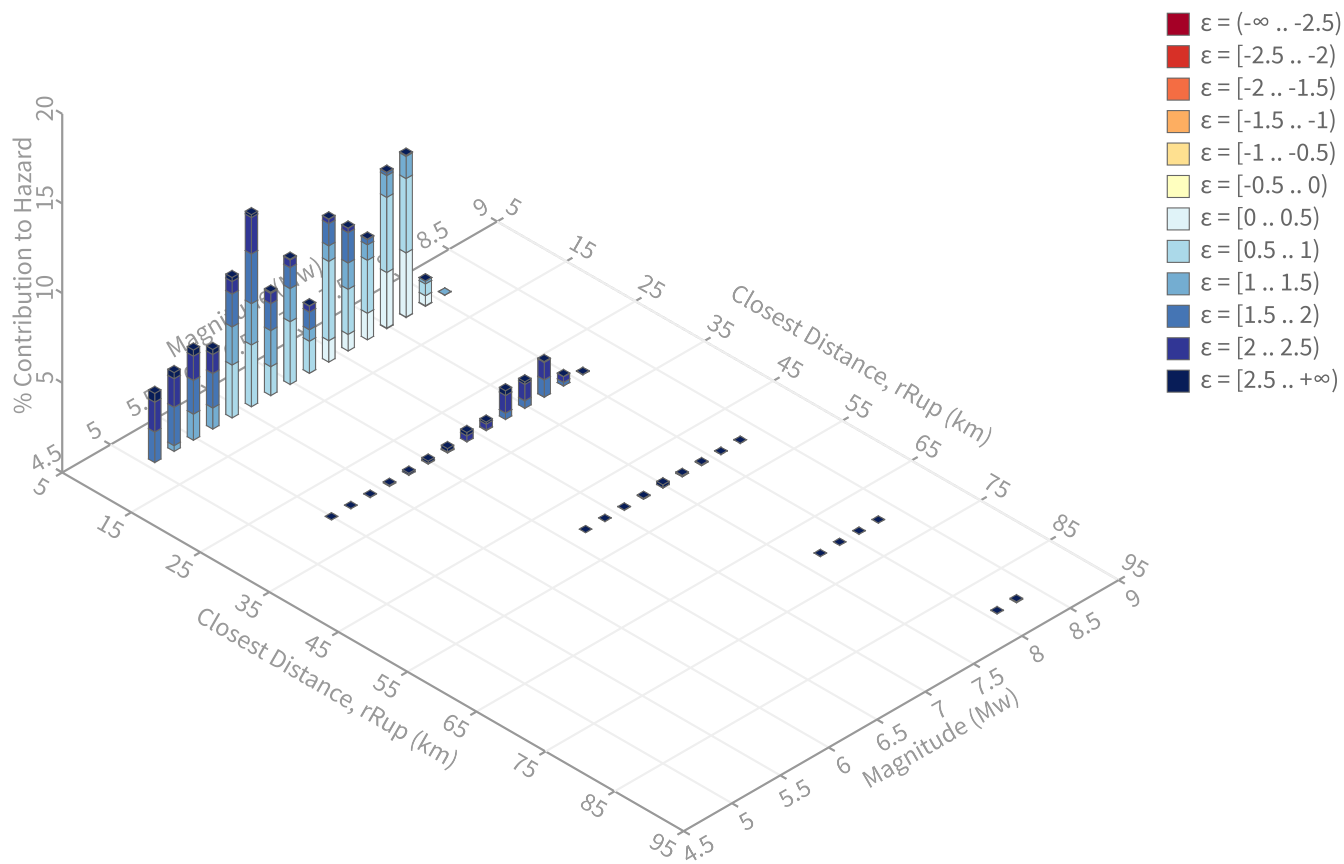


View Raw Data

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.75371658 g

Recovered targets

Return period: 2874.2589 yrs
Exceedance rate: 0.00034791577 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.07 %

Mean (over all sources)

m: 6.62
r: 7.89 km
ε_0: 1.27 σ

Mode (largest m-r bin)

m: 6.12
r: 6.37 km
ε_0: 1.43 σ
Contribution: 10.71 %

Mode (largest m-r-ε_0 bin)

m: 6.9
r: 4.58 km
ε_0: 0.73 σ
Contribution: 4.42 %

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_FM31	System							37.37
Malibu Coast alt 1 [7]		1.67	6.63	0.81	118.831°W	34.038°N	351.30	17.71
Anacapa-Dume alt 1 [1]		5.30	7.32	0.60	118.814°W	33.973°N	166.59	6.71
San Pedro Basin [11]		9.99	6.72	1.69	118.797°W	33.941°N	162.67	4.60
Malibu Coast alt 1 [6]		2.20	6.32	0.97	118.811°W	34.036°N	53.40	1.90
UC33brAvg_FM32	System							29.59
Anacapa-Dume alt 2 [3]		5.81	7.26	0.58	118.816°W	33.969°N	169.75	10.91
Malibu Coast alt 2 [3]		2.40	7.37	0.62	118.831°W	34.041°N	353.11	8.94
San Pedro Basin [11]		9.99	6.77	1.67	118.797°W	33.941°N	162.67	3.57
Anacapa-Dume alt 2 [2]		6.76	6.94	0.90	118.778°W	33.977°N	139.23	1.35
UC33brAvg_FM32 (opt)	Grid							16.70
PointSourceFinite: -118.829, 34.057		6.26	5.59	1.50	118.829°W	34.057°N	0.00	3.39
PointSourceFinite: -118.829, 34.057		6.26	5.59	1.50	118.829°W	34.057°N	0.00	3.39
PointSourceFinite: -118.829, 34.093		8.86	5.66	1.87	118.829°W	34.093°N	0.00	1.60
PointSourceFinite: -118.829, 34.093		8.86	5.66	1.87	118.829°W	34.093°N	0.00	1.60
PointSourceFinite: -118.829, 34.102		9.05	5.90	1.80	118.829°W	34.102°N	0.00	1.05
PointSourceFinite: -118.829, 34.102		9.05	5.90	1.80	118.829°W	34.102°N	0.00	1.05
UC33brAvg_FM31 (opt)	Grid							16.33
PointSourceFinite: -118.829, 34.057		6.22	5.61	1.50	118.829°W	34.057°N	0.00	3.38
PointSourceFinite: -118.829, 34.057		6.22	5.61	1.50	118.829°W	34.057°N	0.00	3.38
PointSourceFinite: -118.829, 34.093		8.77	5.69	1.85	118.829°W	34.093°N	0.00	1.39
PointSourceFinite: -118.829, 34.093		8.77	5.69	1.85	118.829°W	34.093°N	0.00	1.39
PointSourceFinite: -118.829, 34.102		9.10	5.87	1.82	118.829°W	34.102°N	0.00	1.09
PointSourceFinite: -118.829, 34.102		9.10	5.87	1.82	118.829°W	34.102°N	0.00	1.09

Anacapa-Dume 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*

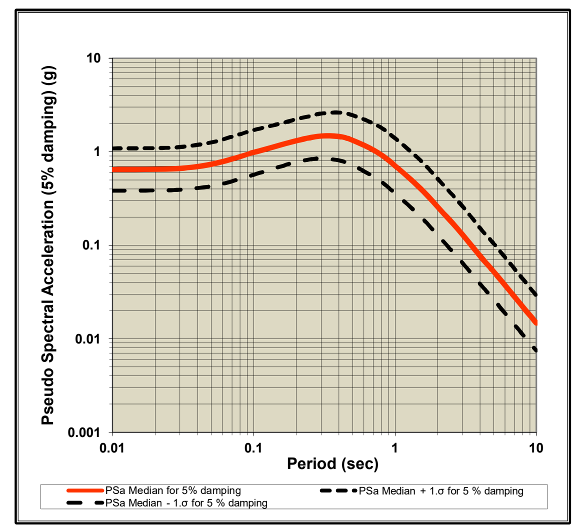
GMPEs	ASK14	BSSA14	CB14	CY14	I14
Weight	0.25	0.25	0.25	0.25	0

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

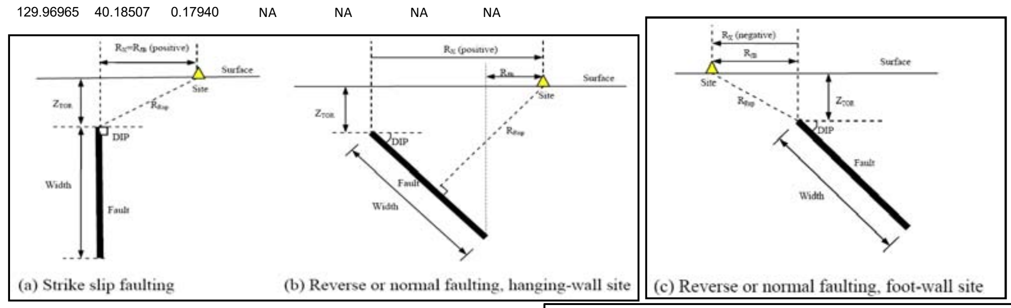
# of std. dev.	1	
Damping ratio (%)	5	*Modification factors are calculated in Sheet DSF*

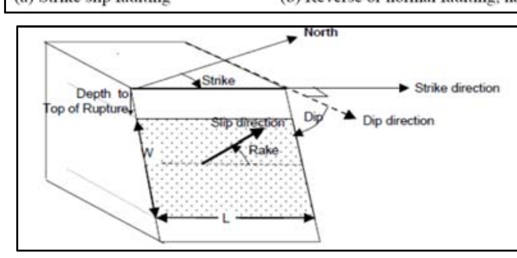
RotD50 Horizontal Component of PGA, PGV and *IMs*

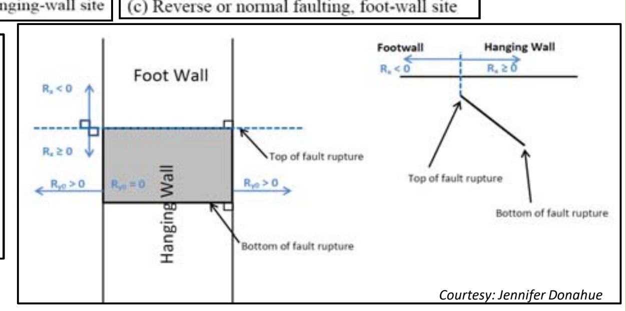
GMP	T (s)	Baseline: 5% Damping PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	User defined: 5% Damping PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	0.01	0.64465	1.08507	0.38300	0.00160	0.64465	1.08507	0.38300	0.00160
	0.02	0.65008	1.09470	0.38605	0.00645	0.65008	1.09470	0.38605	0.00645
	0.03	0.66419	1.12333	0.39272	0.01484	0.66353	1.12220	0.39232	0.01482
	0.05	0.73329	1.25644	0.42796	0.04551	0.73329	1.25644	0.42796	0.04551
	0.075	0.86059	1.49343	0.49591	0.12017	0.86231	1.49641	0.49691	0.12041
	0.1	0.98288	1.70328	0.56718	0.24399	0.98583	1.70839	0.56888	0.24472
	0.15	1.16276	1.97851	0.68335	0.64944	1.16509	1.98247	0.68472	0.65074
	0.2	1.31259	2.22105	0.77572	1.30334	1.31522	2.22549	0.77727	1.30594
	0.25	1.40567	2.39722	0.82425	2.18087	1.41129	2.40681	0.82755	2.18959
	0.3	1.47065	2.55975	0.84493	3.28562	1.47212	2.56231	0.84578	3.28891
	0.4	1.45474	2.62195	0.80714	5.77793	1.45620	2.62457	0.80795	5.78371
	0.5	1.32524	2.45368	0.71577	8.22432	1.32656	2.45613	0.71648	8.23255
	0.75	0.98689	1.90838	0.51035	13.78021	0.98689	1.90838	0.51035	13.78021
	1	0.70726	1.39340	0.35899	17.55689	0.70656	1.39201	0.35863	17.53933
	1.5	0.40915	0.81633	0.20507	22.85214	0.40955	0.81714	0.20527	22.87499
	2	0.25767	0.51643	0.12857	25.58575	0.25716	0.51540	0.12831	25.53458
	3	0.13025	0.26145	0.06489	29.09948	0.13012	0.26119	0.06482	29.07038
	4	0.07654	0.15214	0.03851	30.40079	0.07647	0.15199	0.03847	30.37039
	5	0.05222	0.10402	0.02622	32.40876	0.05207	0.10371	0.02614	32.31154
	7.5	0.02484	0.04935	0.01251	34.69047	0.02479	0.04925	0.01248	34.62109
	10	0.01477	0.02906	0.00750	36.66052	0.01471	0.02895	0.00747	36.51387
PGA (g)	0	0.64099	1.07809	0.38111	0.00159	0.64099	1.07809	0.38111	0.00159
PGV (cm/s)	-1	72.26921	129.96965	40.18507	0.17940	NA	NA	NA	NA



Input variables	Errors and warnings
M_w: 7.2	
R_{RUP} (km): 5.3	
R_{JB} (km): 0	
R_X (km): 5.91	
Ry0 (km): 999	*If unknown use 999*
V_{S30} (m/sec): 330	
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): 999	*If unknown use 999*
Z_{HYP} (km): 999	*If unknown use 999*
$Z_{1.0}$ (km): 0.05	*If unknown use 999*
$Z_{2.5}$ (km): 0.2	*If unknown use 999*
W (km): 19.73	*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*







Courtesy: Jennifer Donahue

Calculated Variables/Flags

ΔDPP	0	*Always 0 for median calcs.*
PGA_r (g)	0.457	
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 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 illustration
- **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)	19.73			21.191		
$Z_{1.0}$ (km)	0.050	0.050			0.433	
$\delta Z_{1.0}$ (km)	-0.383		-0.383			
$Z_{2.5}$ (V_{S30}=1100)(km)	0.200			0.398		
$Z_{2.5}$ (V_{S30})(km)	0.200			1.576		
Z_{hyp} (km)	999.00			11.324		
Z_{tor} (km)	999.00			1.098	1.098	
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.

Anacapa-Dume 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*

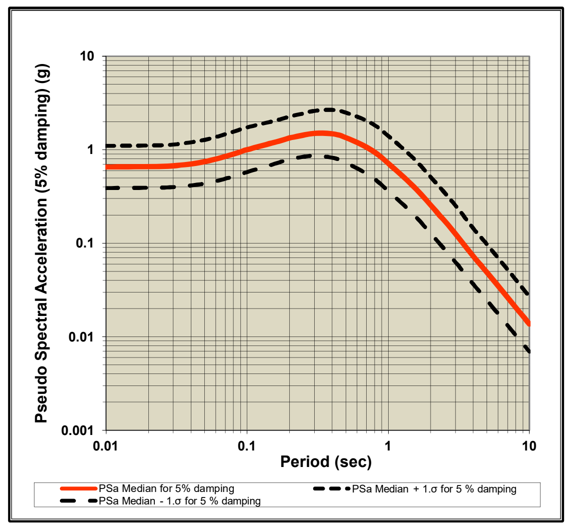
GMPEs	ASK14	BSSA14	CB14	CY14	I14
Weight	0.25	0.25	0.25	0.25	0

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

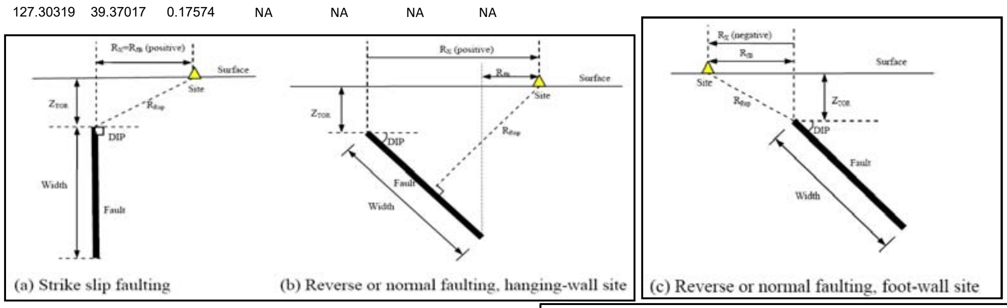
ASK14 Abrahamson & Silva & Kamai 2014 NGA West-2 Model
BSSA14 Boore & Stewart & Seyhan & Atkinson 2014 NGA West-2 Model
CB14 Campbell & Bozorgnia 2014 NGA West-2 Model
CY14 Chiou & Youngs 2014 NGA West-2 Model
I14 Idriss 2014 NGA West-2 Model

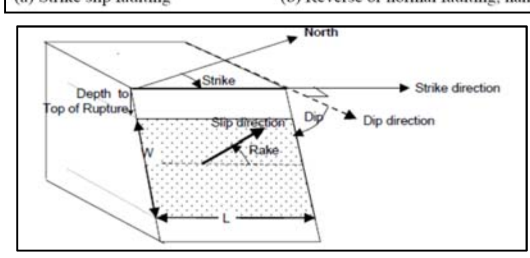
RotD50 Horizontal Component of PGA, PGV and *IMs*

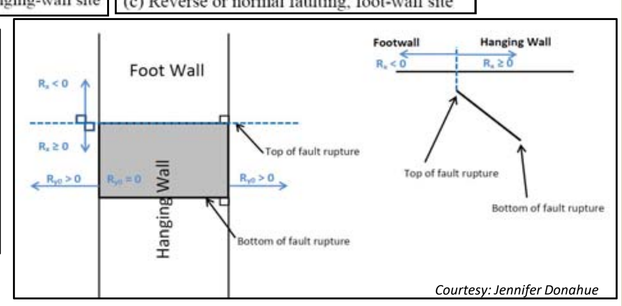
GMP	*T* (s)	Baseline: 5% Damping				User defined: 5% Damping			
		PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	0.01	0.65462	1.10094	0.38923	0.00163	0.65462	1.10094	0.38923	0.00163
	0.02	0.65976	1.11008	0.39212	0.00655	0.65976	1.11008	0.39212	0.00655
	0.03	0.67362	1.13841	0.39859	0.01505	0.67294	1.13728	0.39819	0.01503
	0.05	0.74405	1.27431	0.43444	0.04618	0.74405	1.27431	0.43444	0.04618
	0.075	0.87311	1.51491	0.50321	0.12191	0.87485	1.51794	0.50421	0.12216
	0.1	0.99793	1.72920	0.57590	0.24772	1.00092	1.73439	0.57763	0.24847
	0.15	1.17808	2.00406	0.69253	0.65800	1.18043	2.00807	0.69391	0.65931
	0.2	1.33367	2.25558	0.78857	1.32427	1.33634	2.26009	0.79015	1.32692
	0.25	1.42811	2.43380	0.83799	2.21568	1.43239	2.44110	0.84050	2.22233
	0.3	1.49665	2.60307	0.86051	3.34372	1.49815	2.60567	0.86137	3.34707
	0.4	1.48073	2.66759	0.82193	5.88116	1.48221	2.67026	0.82275	5.88704
	0.5	1.34756	2.49435	0.72801	8.36284	1.34891	2.49685	0.72874	8.37120
	0.75	1.00580	1.94479	0.52018	14.04438	1.00580	1.94479	0.52018	14.04438
	1	0.71465	1.40789	0.36276	17.74017	0.71393	1.40648	0.36239	17.72243
	1.5	0.40879	0.81560	0.20489	22.83202	0.40919	0.81642	0.20509	22.85485
	2	0.25503	0.51114	0.12725	25.32329	0.25452	0.51012	0.12699	25.27264
	3	0.12587	0.25267	0.06271	28.12213	0.12575	0.25242	0.06265	28.09401
	4	0.07260	0.14431	0.03653	28.83585	0.07253	0.14416	0.03649	28.80701
	5	0.04907	0.09775	0.02464	30.45519	0.04893	0.09745	0.02456	30.36382
	7.5	0.02325	0.04618	0.01170	32.46239	0.02320	0.04609	0.01168	32.39746
	10	0.01373	0.02703	0.00698	34.09285	0.01368	0.02692	0.00695	33.95648
PGA (g)	0	0.65095	1.09396	0.38735	0.00162	0.65095	1.09396	0.38735	0.00162
PGV (cm/s)	-1	70.79512	127.30319	39.37017	0.17574	NA	NA	NA	NA



Input variables	Errors and warnings
M_w	
7.16	
R_{RUP} (km)	
5.81	
R_{JB} (km)	
0	
R_X (km)	
6.28	
Ry0 (km)	*If unknown use 999*
999	
V_{S30} (m/sec)	
330	
U (BSSA13)	*1: Unspecified fault mech.*
0	
F_{RV}	*1: reverse fault*
1	
F_{NM}	*1: normal fault*
0	
F_{HW}	*1: hanging wall side*
1	
Dip (deg)	
41	
Z_{TOR} (km)	*If unknown use 999*
999	
Z_{HYP} (km)	*If unknown use 999*
999	
$Z_{1.0}$ (km)	*If unknown use 999*
0.05	
$Z_{2.5}$ (km)	*If unknown use 999*
0.2	
W (km)	*If unknown use 999*
13.99	
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	*Always 0 for median calcs.*
0	
PGA_r (g)	
0.454	
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 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 illustration
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)	0.050	0.050			0.433	
$\delta Z_{1.0}$ (km)	-0.383		-0.383			
$Z_{2.5}$ (V_{S30}=1100)(km)	0.200			0.398		
$Z_{2.5}$ (V_{S30})(km)	0.200			1.576		
Z_{hyp} (km)	999.00			11.429		
Z_{tor} (km)	999.00			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.

Malibu Coast Alt 1 [7] 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*

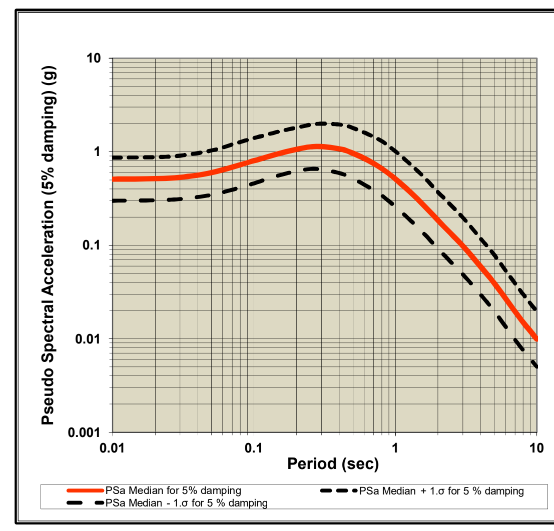
GMPEs	ASK14	BSSA14	CB14	CY14	I14
Weight	0.25	0.25	0.25	0.25	0

# 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*

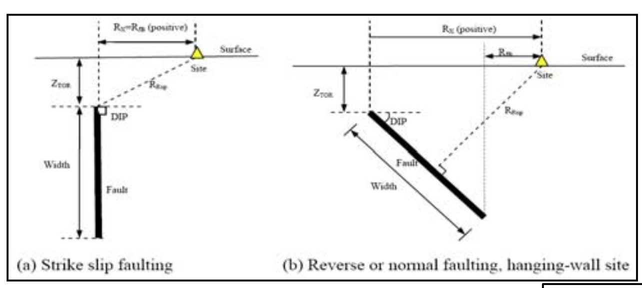
GMP	*T* (s)	Baseline: 5% Damping				User defined: 5% Damping			
		PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	0.01	0.50869	0.86443	0.29935	0.00126	0.50869	0.86443	0.29935	0.00126
	0.02	0.51523	0.87596	0.30305	0.00512	0.51523	0.87596	0.30305	0.00512
	0.03	0.53239	0.90886	0.31186	0.01189	0.53239	0.90886	0.31186	0.01189
	0.05	0.59756	1.03211	0.34596	0.03708	0.59696	1.03108	0.34562	0.03705
	0.075	0.70469	1.23121	0.40334	0.09840	0.70610	1.23367	0.40415	0.09860
	0.1	0.80111	1.39768	0.45917	0.19886	0.80271	1.40047	0.46009	0.19926
	0.15	0.95677	1.64133	0.55773	0.53439	0.95869	1.64461	0.55884	0.53546
	0.2	1.05934	1.81002	0.61999	1.05187	1.06040	1.81183	0.62061	1.05292
	0.25	1.12230	1.93474	0.65102	1.74122	1.12791	1.94442	0.65427	1.74993
	0.3	1.13396	1.99543	0.64441	2.53342	1.13396	1.99543	0.64441	2.53342
	0.4	1.07421	1.95251	0.59099	4.26653	1.07528	1.95447	0.59159	4.27080
	0.5	0.96295	1.79489	0.51662	5.97598	0.96391	1.79669	0.51713	5.98195
	0.75	0.70701	1.37359	0.36391	9.87216	0.70630	1.37221	0.36354	9.86229
	1	0.51171	1.01211	0.25872	12.70264	0.51069	1.01008	0.25820	12.67723
	1.5	0.29190	0.58428	0.14583	16.30341	0.29190	0.58428	0.14583	16.30341
	2	0.18468	0.37118	0.09188	18.33733	0.18431	0.37044	0.09170	18.30066
	3	0.09829	0.19783	0.04884	21.95972	0.09810	0.19744	0.04874	21.91580
	4	0.05915	0.11789	0.02968	23.49216	0.05909	0.11777	0.02965	23.46866
	5	0.03950	0.07889	0.01978	24.51293	0.03934	0.07857	0.01970	24.41488
	7.5	0.01702	0.03391	0.00855	23.77153	0.01704	0.03394	0.00856	23.79530
	10	0.00995	0.01962	0.00504	24.69027	0.00992	0.01957	0.00503	24.61619
PGA (g)	0	0.50576	0.85880	0.29785	0.00126	0.50576	0.85880	0.29785	0.00126
PGV (cm/s)	-1	59.78684	107.65648	33.20251	0.14841	NA	NA	NA	NA

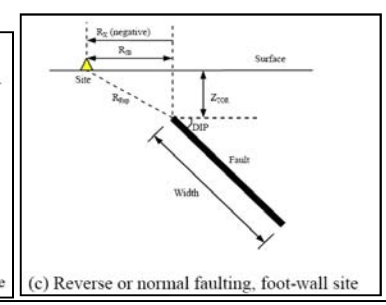


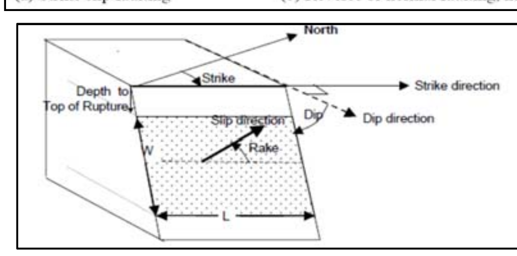
Input variables	Errors and warnings
M_w: 6.64	
R_{RUP} (km): 1.67	
R_{JB} (km): 1.47	
R_X (km): -1.42	
Ry0 (km): 999	*If unknown use 999*
V_{S30} (m/sec): 330	
U (BSSA13): 0	*1: Unspecified fault mech.*
F_{RV}: 0	*1: reverse fault*
F_{NM}: 0	*1: normal fault*
F_{HW}: 0	*1: hanging wall side*
Dip (deg): 75	
Z_{TOR} (km): 999	*If unknown use 999*
Z_{HYP} (km): 999	*If unknown use 999*
$Z_{1.0}$ (km): 0.05	*If unknown use 999*
$Z_{2.5}$ (km): 0.2	*If unknown use 999*
W (km): 7.27	*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*

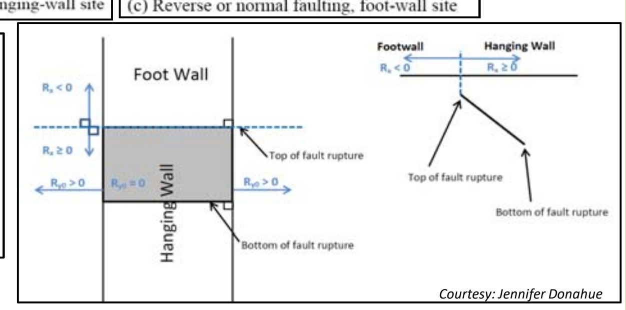
Calculated Variables/Flags

ΔDPP: 0	*Always 0 for median calcs.*
PGA_r (g): 0.423	
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*









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 illustration
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)	7.27			14.906		
$Z_{1.0}$ (km)	0.050	0.050			0.433	
$\delta Z_{1.0}$ (km)	-0.383		-0.383			
$Z_{2.5}$ (V_{S30}=1100)(km)	0.200			0.398		
$Z_{2.5}$ (V_{S30})(km)	0.200			1.576		
Z_{hyp} (km)	999.00			9.780		
Z_{tor} (km)	999.00			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.

Malibu Coast 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*

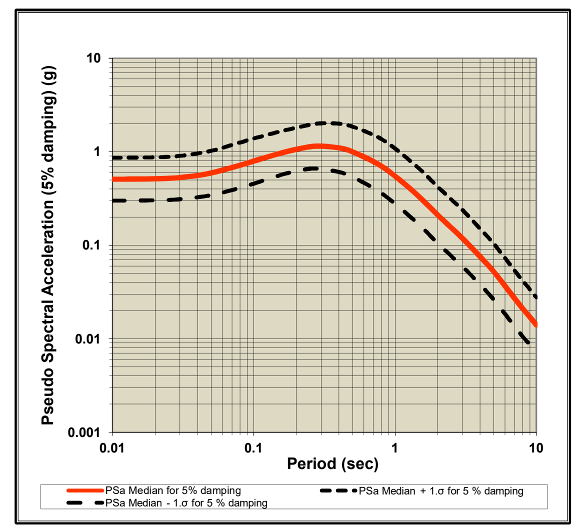
GMPEs	ASK14	BSSA14	CB14	CY14	I14
Weight	0.25	0.25	0.25	0.25	0

# 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*

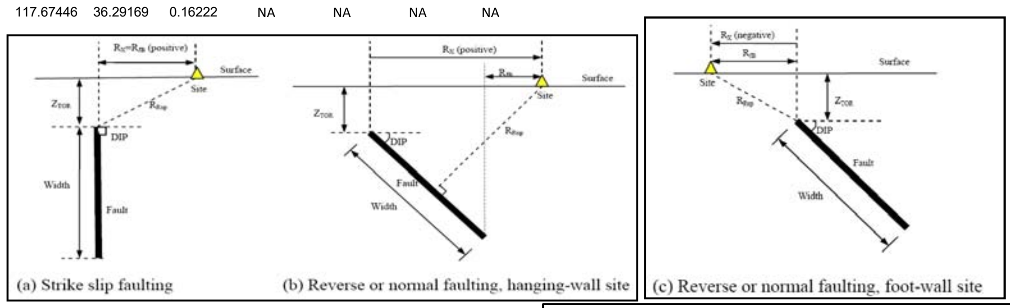
GMP	T (s)	Baseline: 5% Damping				User defined: 5% Damping			
		PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	0.01	0.50801	0.86201	0.29938	0.00126	0.50801	0.86201	0.29938	0.00126
	0.02	0.51390	0.87248	0.30269	0.00510	0.51390	0.87248	0.30269	0.00510
	0.03	0.53025	0.90413	0.31098	0.01185	0.52972	0.90322	0.31066	0.01183
	0.05	0.59259	1.02256	0.34342	0.03678	0.59259	1.02256	0.34342	0.03678
	0.075	0.69796	1.21838	0.39983	0.09746	0.69935	1.22082	0.40063	0.09765
	0.1	0.79410	1.38435	0.45552	0.19713	0.79569	1.38712	0.45643	0.19752
	0.15	0.95187	1.63162	0.55532	0.53165	0.95378	1.63488	0.55643	0.53272
	0.2	1.05882	1.80718	0.62035	1.05135	1.05987	1.80899	0.62097	1.05240
	0.25	1.12646	1.93895	0.65443	1.74768	1.13209	1.94864	0.65771	1.75642
	0.3	1.14570	2.01206	0.65238	2.55965	1.14685	2.01408	0.65303	2.56221
	0.4	1.10046	1.99529	0.60693	4.37079	1.10156	1.99729	0.60754	4.37517
	0.5	0.99795	1.85537	0.53677	6.19320	0.99895	1.85723	0.53731	6.19940
	0.75	0.74458	1.44289	0.38423	10.39680	0.74383	1.44145	0.38384	10.38641
	1	0.54875	1.08263	0.27814	13.62196	0.54765	1.08046	0.27759	13.59471
	1.5	0.32473	0.64838	0.16263	18.13718	0.32473	0.64838	0.16263	18.13718
	2	0.21112	0.42330	0.10530	20.96356	0.21070	0.42245	0.10509	20.92163
	3	0.11891	0.23874	0.05923	26.56648	0.11867	0.23827	0.05911	26.51335
	4	0.07536	0.14983	0.03791	29.93339	0.07529	0.14968	0.03787	29.90345
	5	0.05222	0.10404	0.02621	32.40862	0.05201	0.10362	0.02611	32.27898
	7.5	0.02376	0.04720	0.01196	33.17490	0.02376	0.04720	0.01196	33.17490
	10	0.01412	0.02780	0.00718	35.06014	0.01408	0.02772	0.00715	34.95496
PGA (g)	0	0.50510	0.85643	0.29790	0.00125	0.50510	0.85643	0.29790	0.00125
PGV (cm/s)	-1	65.34986	117.67446	36.29169	0.16222	NA	NA	NA	NA

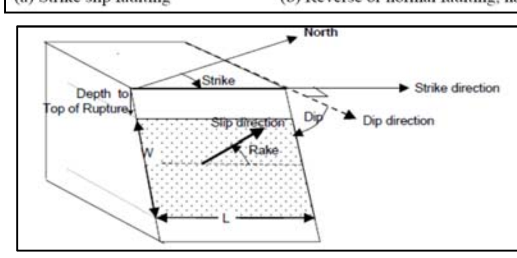


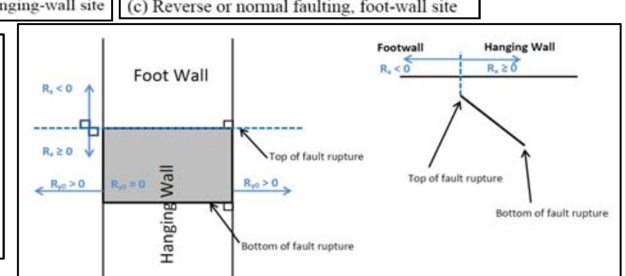
Input variables	Errors and warnings
M_w: 6.97	
R_{RUP} (km): 2.4	
R_{JB} (km): 1.73	
R_X (km): -1.68	
Ry0 (km): 999	*If unknown use 999*
V_{S30} (m/sec): 330	
U (BSSA13): 0	*1: Unspecified fault mech.*
F_{RV}: 0	*1: reverse fault*
F_{NM}: 0	*1: normal fault*
F_{HW}: 0	*1: hanging wall side*
Dip (deg): 74	
Z_{TOR} (km): 999	*If unknown use 999*
Z_{HYP} (km): 999	*If unknown use 999*
$Z_{1.0}$ (km): 0.05	*If unknown use 999*
$Z_{2.5}$ (km): 0.2	*If unknown use 999*
W (km): 15.54	*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): 0.437	
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*







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 illustration
- **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)	15.54			15.437		
$Z_{1.0}$ (km)	0.050	0.050			0.433	
$\delta Z_{1.0}$ (km)	-0.383		-0.383			
$Z_{2.5}$ (V_{S30}=1100)(km)	0.200			0.398		
$Z_{2.5}$ (V_{S30})(km)	0.200			1.576		
Z_{hyp} (km)	999.00			10.387		
Z_{tor} (km)	999.00			0.161	0.161	
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.

San Pedro Basin [11] 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

- ***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

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

RotD50 Horizontal Component of PGA, PGV and *IMs*

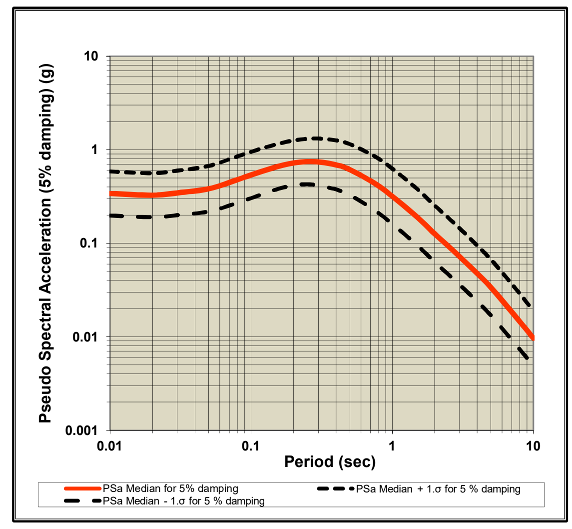
Input variables / **Errors and warnings**

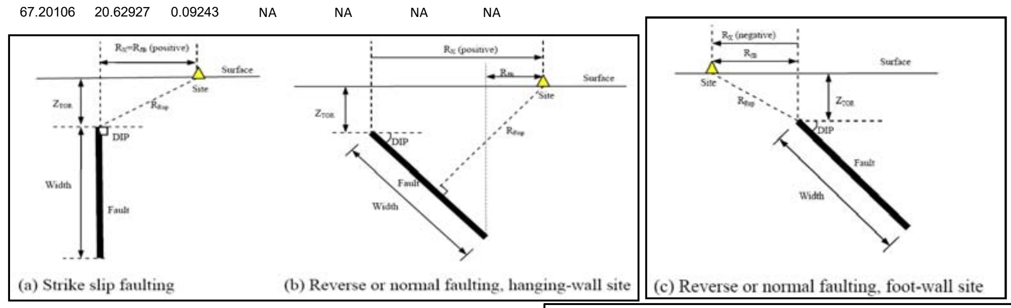
M_w	7.07	
R_{RUP} (km)	9.99	
R_{JB} (km)	9.66	
R_X (km)	6.84	
Ry0 (km)	999	*If unknown use 999*
V_{S30} (m/sec)	330	
U (BSSA13)	0	*1: Unspecified fault mech.*
F_{RV}	0	*1: reverse fault*
F_{NM}	0	*1: normal fault*
F_{HW}	1	*1: hanging wall side*
Dip (deg)	88	
Z_{TOR} (km)	999	*If unknown use 999*
Z_{HYP} (km)	999	*If unknown use 999*
$Z_{1.0}$ (km)	0.05	*If unknown use 999*
$Z_{2.5}$ (km)	0.2	*If unknown use 999*
W (km)	10.36	*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*

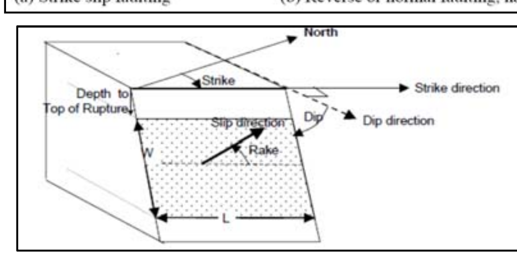
Calculated Variables/Flags

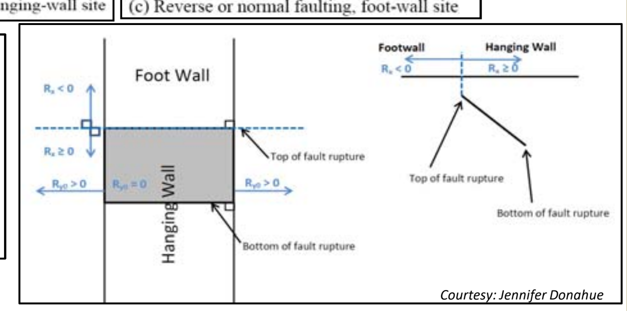
ΔDPP	0	*Always 0 for median calcs.*
PGA_r (g)	0.254	
Z_{BOT} (km) (CB14)	15	*Enter for default W calcs*
SS	1	*auto calculated*
$V_{s30Flag}$	1	*measured*
F_{AS}	0	*Aftershock effect is not applicable.*
Region	0	*California*
Option for Sa value	1	*Weighted average of the natural logarithm of the spectral values*

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	0.33998	0.58487	0.19763	0.00084	0.33998	0.58487	0.19763	0.00084
	0.02	0.32659	0.56235	0.18968	0.00324	0.32659	0.56235	0.18968	0.00324
	0.03	0.34564	0.59828	0.19969	0.00772	0.34564	0.59828	0.19969	0.00772
	0.05	0.38203	0.66942	0.21802	0.02371	0.38203	0.66942	0.21802	0.02371
	0.075	0.46205	0.81860	0.26079	0.06452	0.46297	0.82024	0.26132	0.06465
	0.1	0.53588	0.94860	0.30272	0.13302	0.53748	0.95145	0.30363	0.13342
	0.15	0.65283	1.13847	0.37435	0.36463	0.65414	1.14075	0.37510	0.36536
	0.2	0.72197	1.25442	0.41552	0.71687	0.72341	1.25693	0.41635	0.71831
	0.25	0.74581	1.30549	0.42608	1.15711	0.74730	1.30810	0.42693	1.15943
	0.3	0.73842	1.31581	0.41439	1.64973	0.73916	1.31713	0.41481	1.65138
	0.4	0.68780	1.25855	0.37589	2.73181	0.68849	1.25981	0.37626	2.73454
	0.5	0.61337	1.14770	0.32781	3.80656	0.61399	1.14884	0.32814	3.81036
	0.75	0.43790	0.85144	0.22522	6.11459	0.43790	0.85144	0.22522	6.11459
	1	0.31826	0.62922	0.16097	7.90032	0.31826	0.62922	0.16097	7.90032
	1.5	0.19052	0.38077	0.09532	10.64091	0.19071	0.38115	0.09542	10.65155
	2	0.12641	0.25354	0.06303	12.55194	0.12616	0.25303	0.06290	12.52684
	3	0.07221	0.14496	0.03597	16.13269	0.07214	0.14482	0.03593	16.11656
	4	0.04746	0.09433	0.02388	18.84873	0.04741	0.09423	0.02385	18.82988
	5	0.03376	0.06724	0.01695	20.95074	0.03366	0.06704	0.01690	20.88789
	7.5	0.01644	0.03266	0.00828	22.95797	0.01639	0.03256	0.00825	22.88910
	10	0.00969	0.01907	0.00493	24.06138	0.00965	0.01900	0.00491	23.96513
PGA (g)	0	0.32121	0.55215	0.18686	0.00080	0.32121	0.55215	0.18686	0.00080
PGV (cm/s)	-1	37.23317	67.20106	20.62927	0.09243	NA	NA	NA	NA









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 illustration
- ***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)	10.36			14.926		
$Z_{1.0}$ (km)	0.050	0.050			0.433	
$\delta Z_{1.0}$ (km)	-0.383		-0.383			
$Z_{2.5}$ (V_{S30}=1100)(km)	0.200			0.398		
$Z_{2.5}$ (V_{S30})(km)	0.200			1.576		
Z_{hyp} (km)	999.00			10.309		
Z_{tor} (km)	999.00			0.083	0.083	
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.

APPENDIX E

LIQUEFACTION AND SEISMIC SETTLEMENT

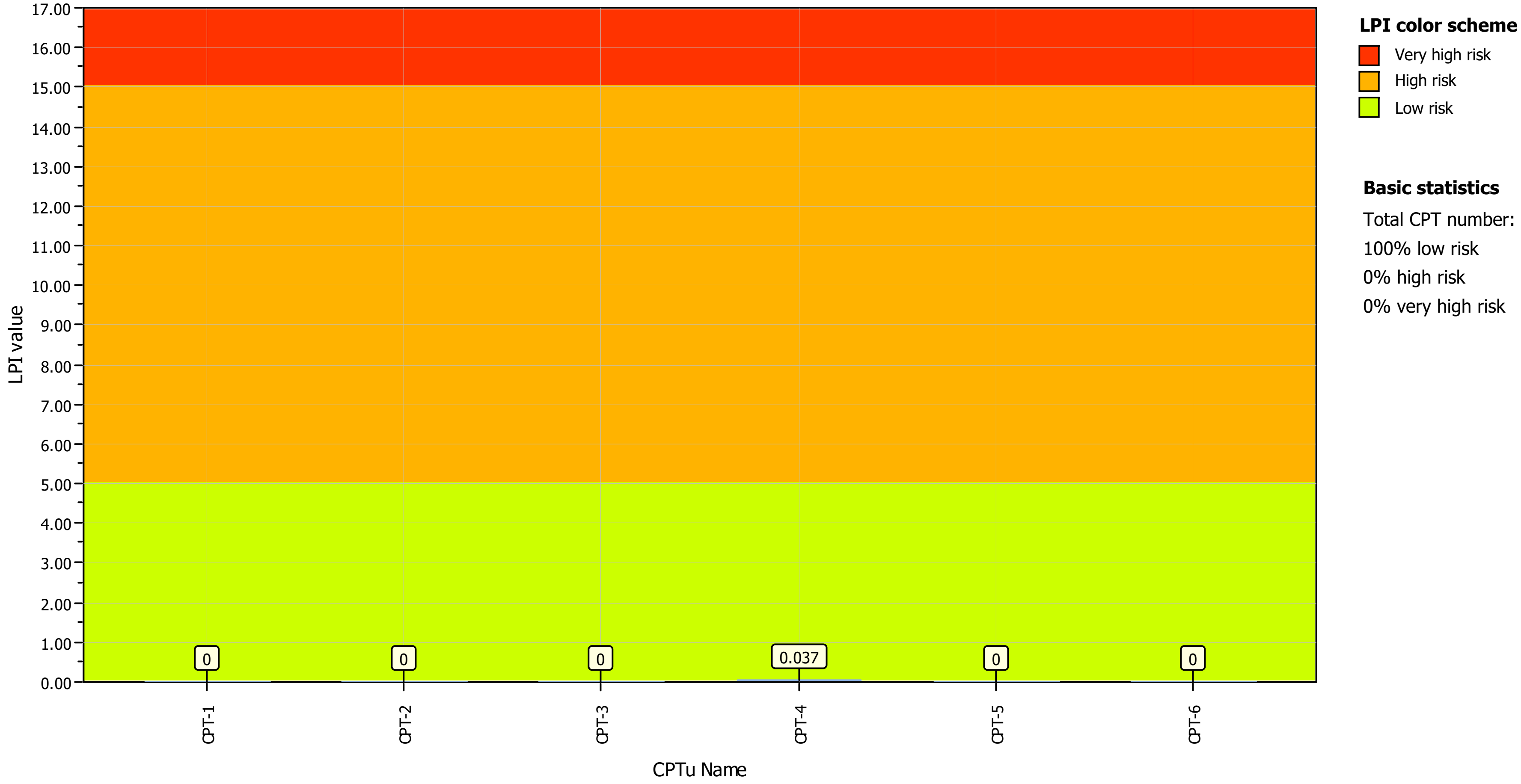
Leighton

Leighton Consulting, Inc.
17781 Cowan
Irvine, CA 92614
http://www.leightongeo.com

Project title : mmhs

Location : morningside drive, malibu, CA

Overall Liquefaction Potential Index report



Basic statistics

Total CPT number: 6

100% low risk

0% high risk

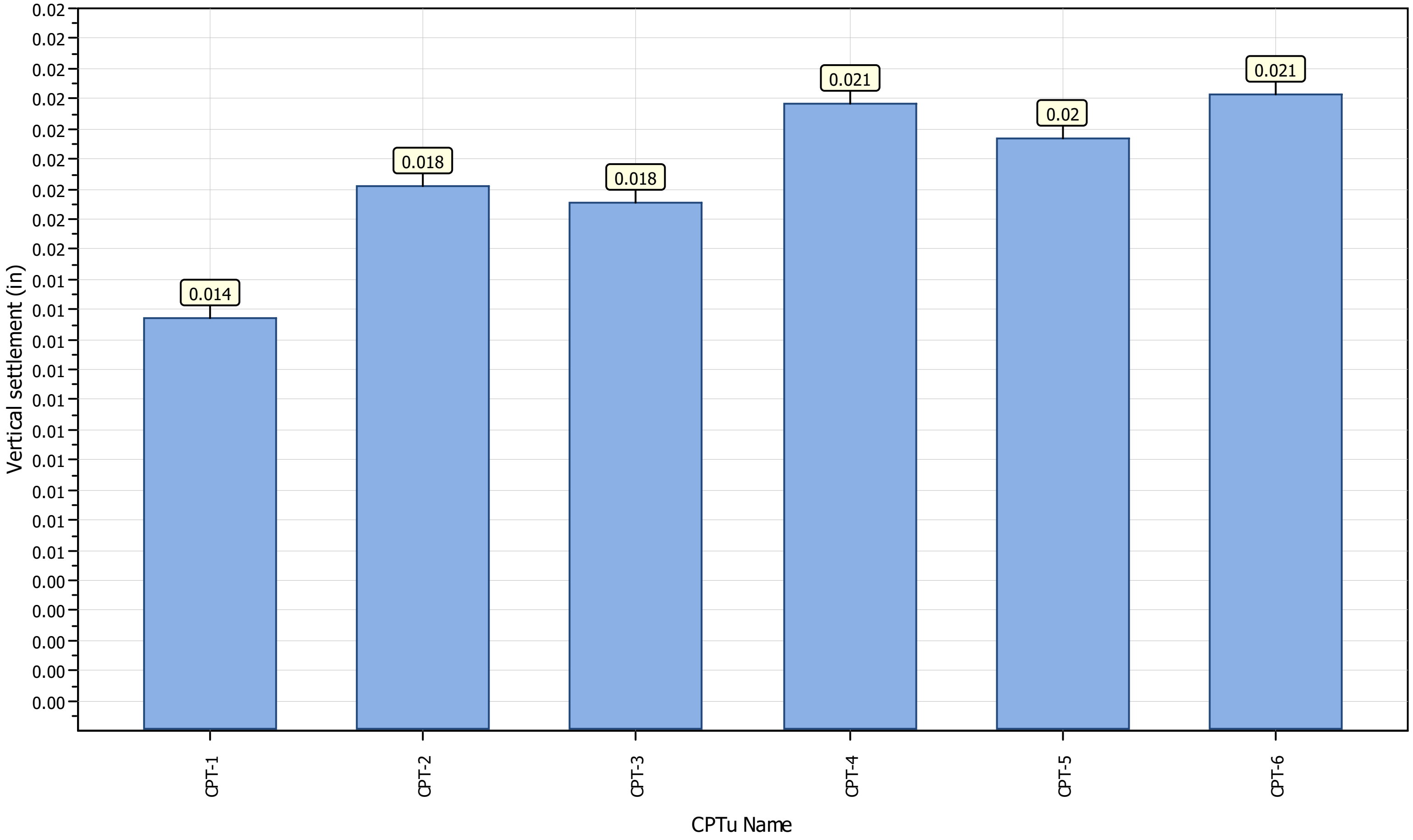
0% very high risk

Leighton Consulting, Inc.
17781 Cowan
Irvine, CA 92614
http://www.leightongeo.com

Project title : mmhs

Location : morningside drive, malibu, CA

Overall vertical settlements report



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Leighton Consulting, Inc.
17781 Cowan
Irvine, CA 92614
http://www.leightongeo.com

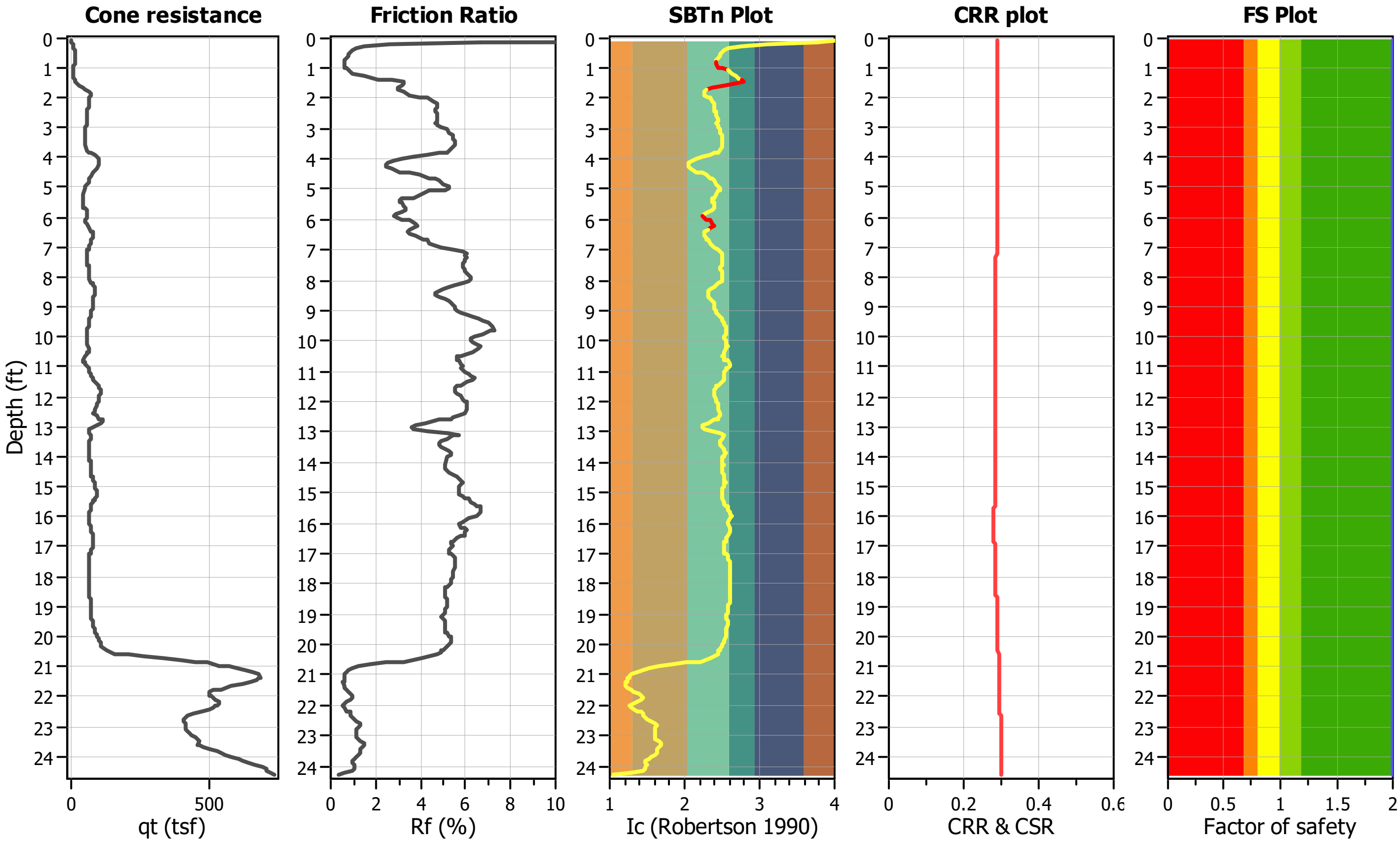
LIQUEFACTION ANALYSIS REPORT

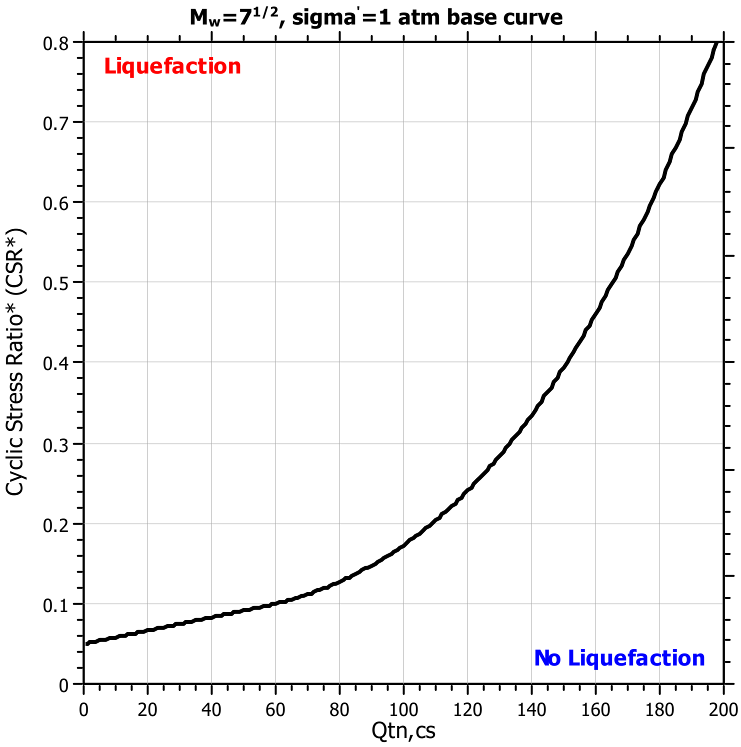
Project title : mmhs **Location : morningside drive, malibu, CA**

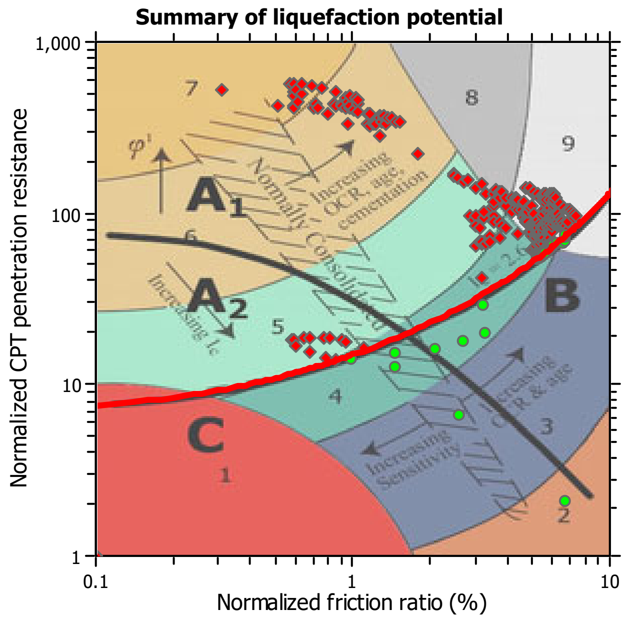
CPT file : CPT-1

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.):	45.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.76	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

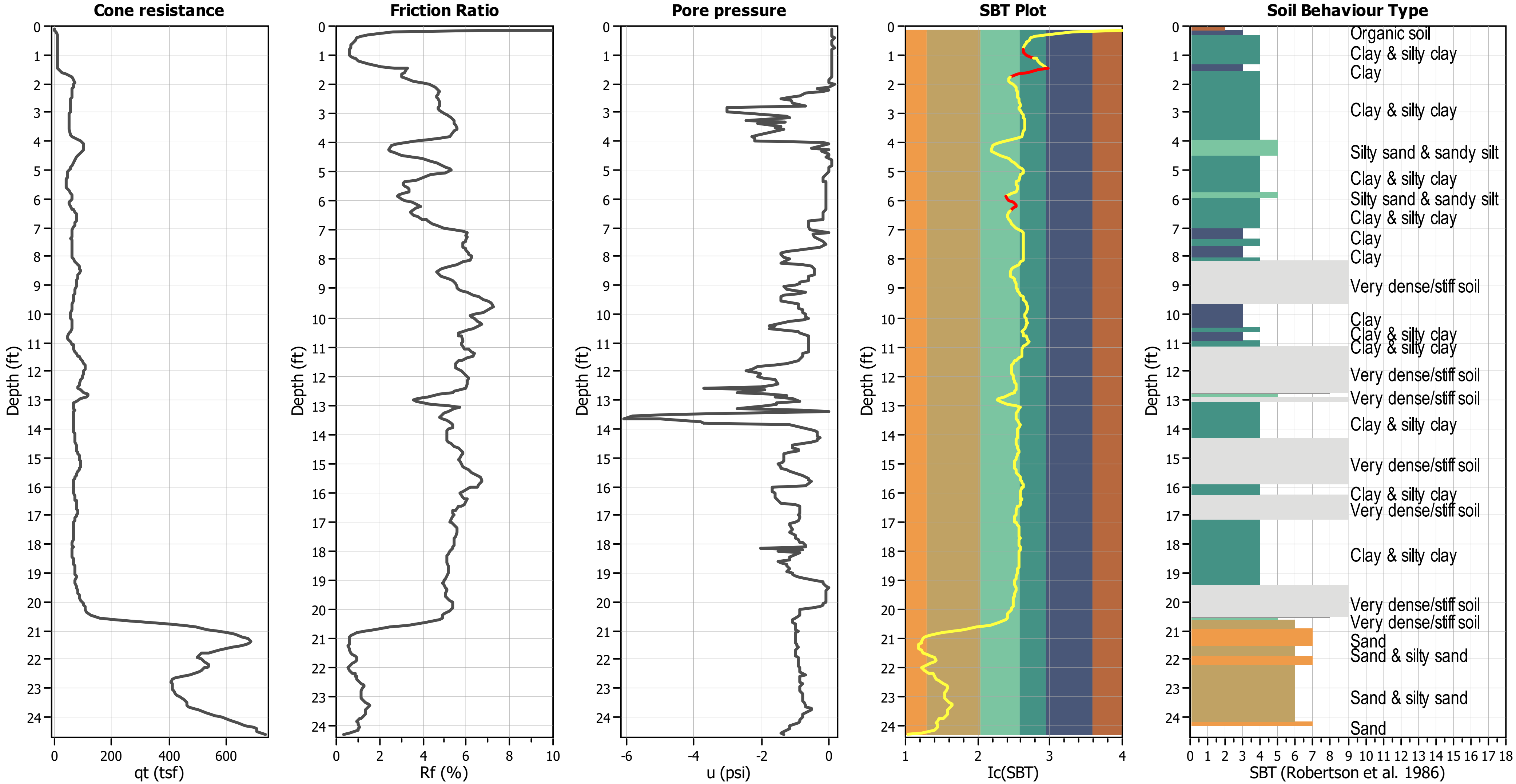






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



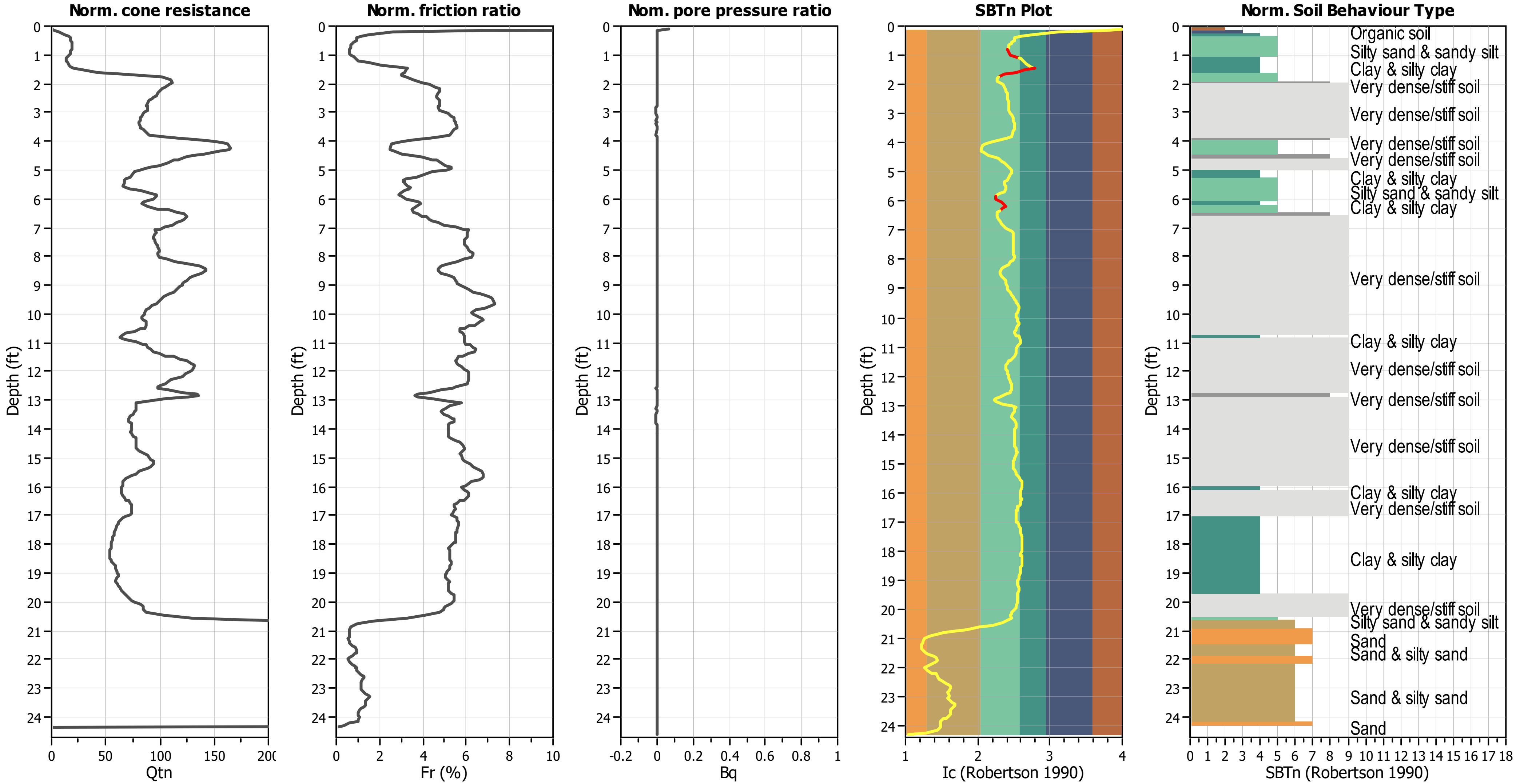
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	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)



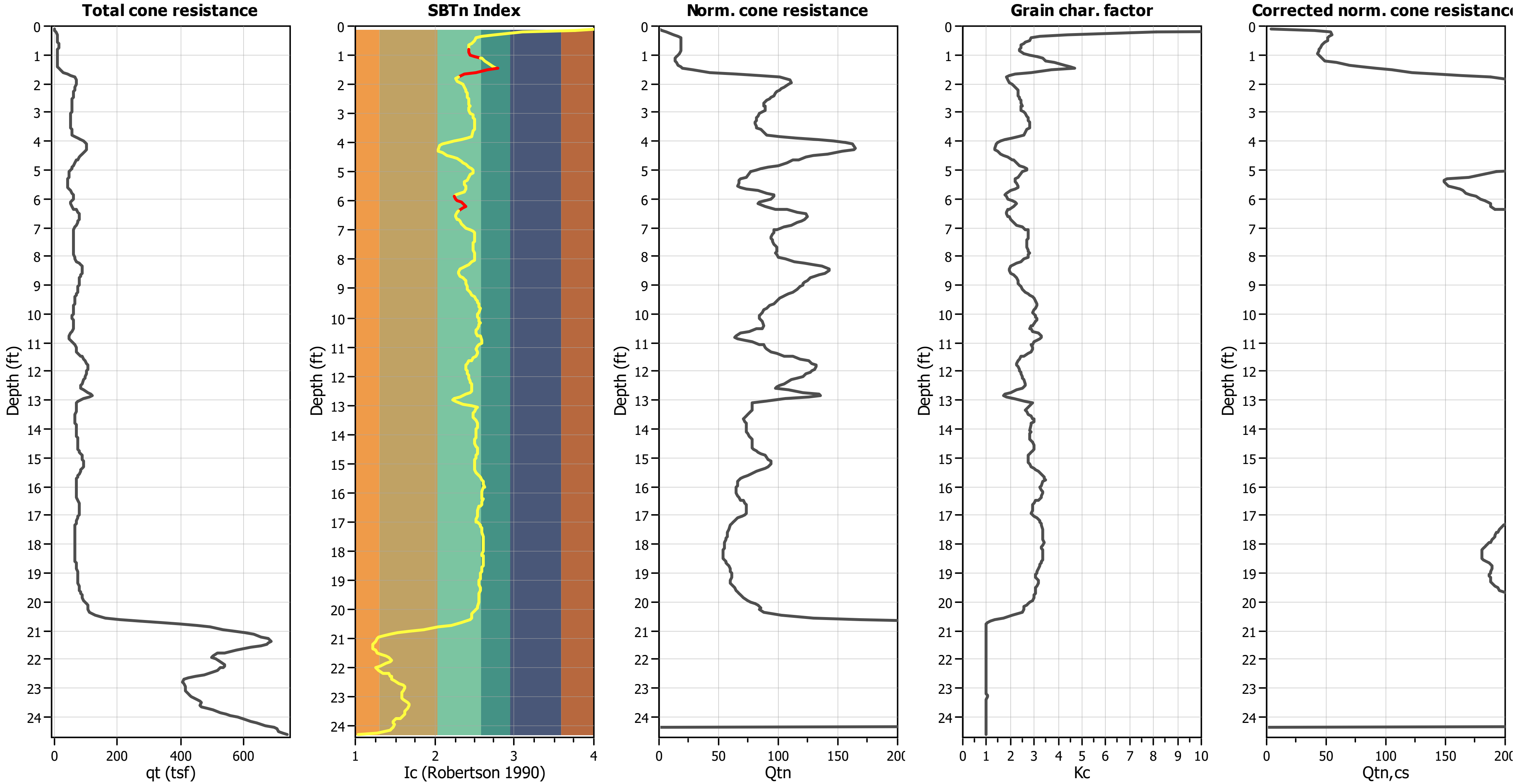
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	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

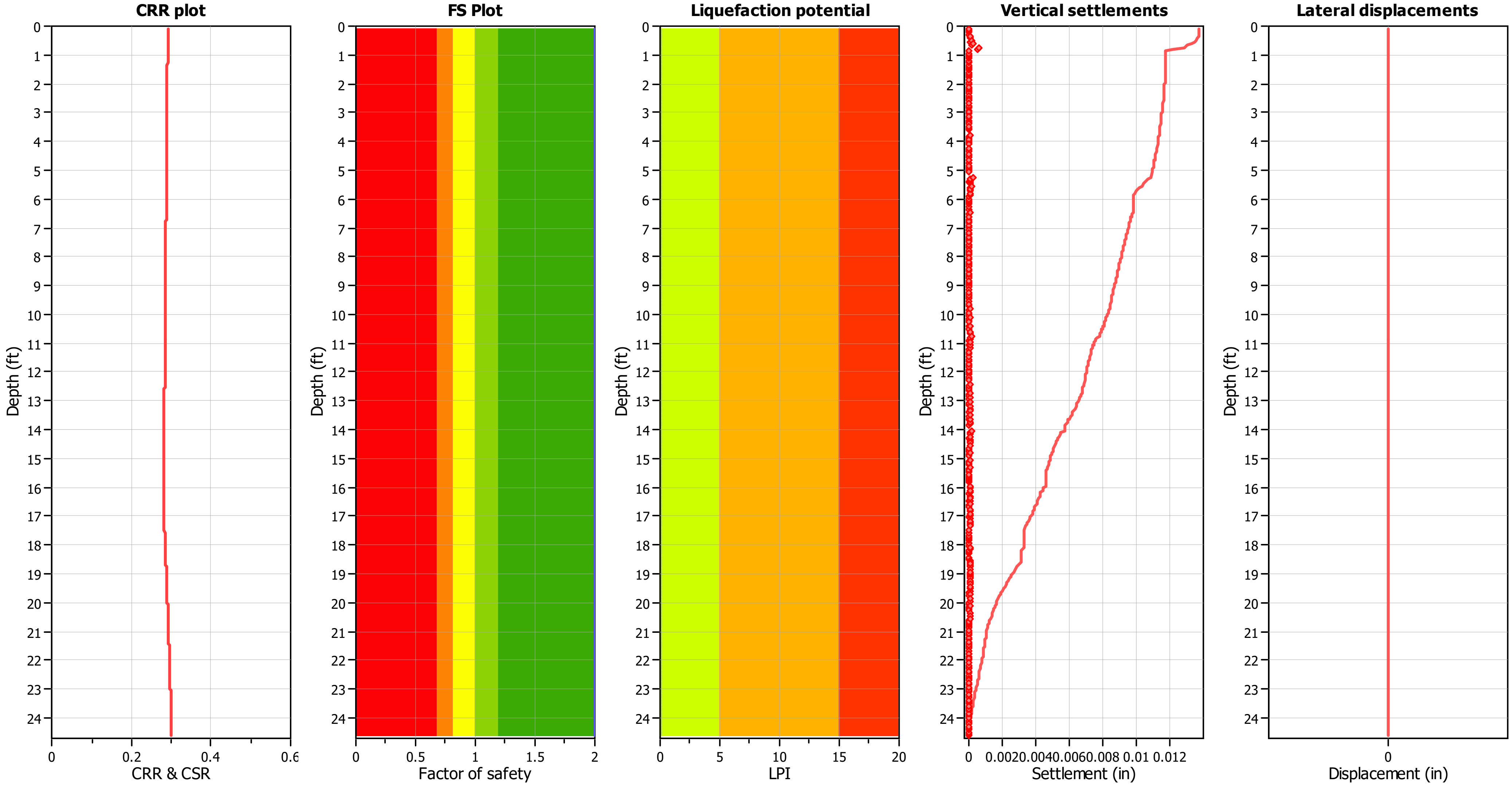
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	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.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	N/A

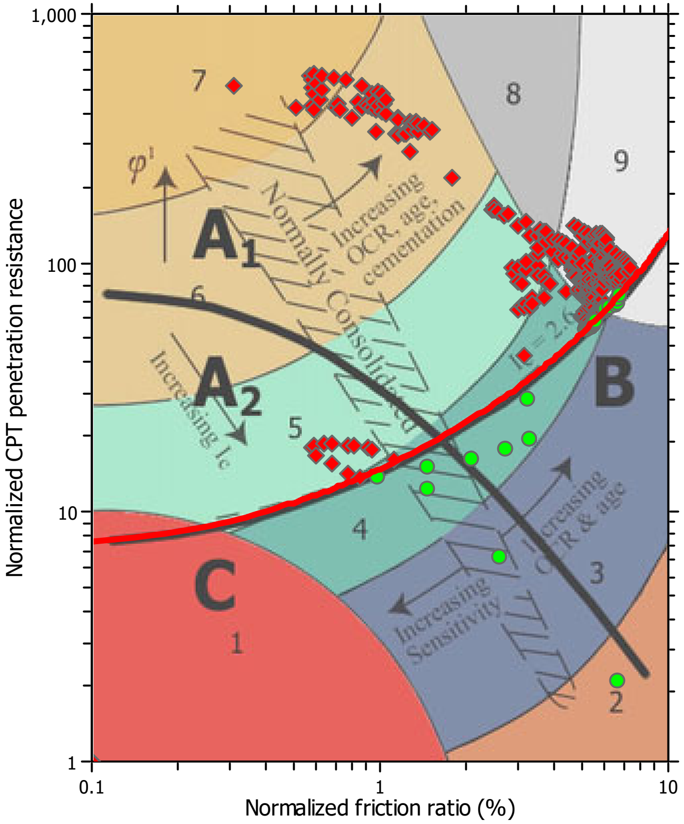
F.S. color scheme

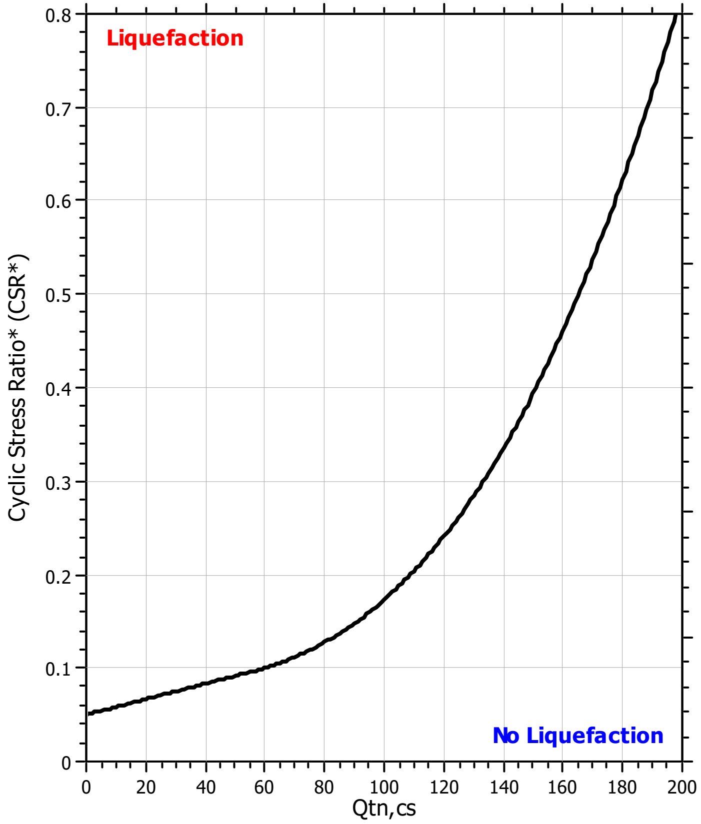
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

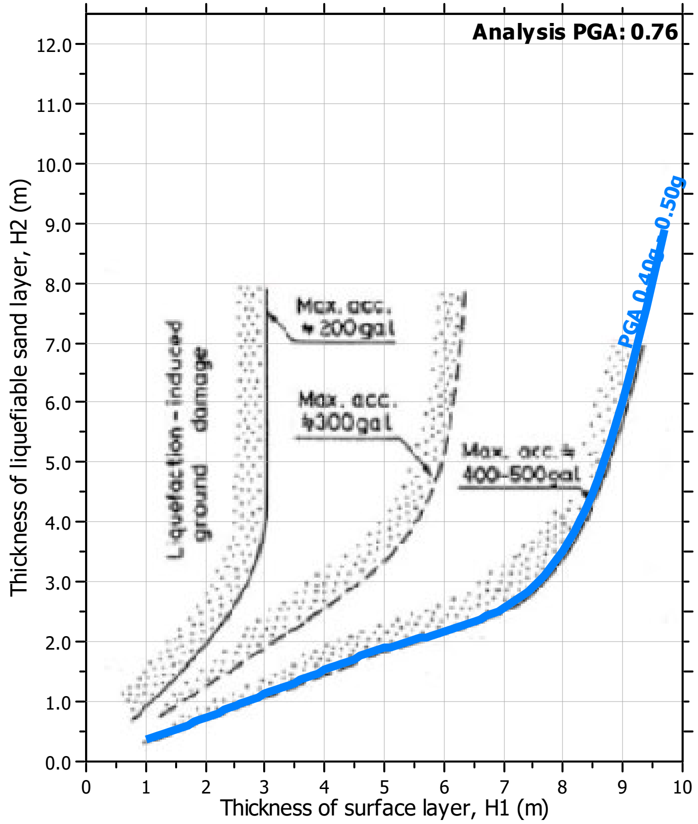
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



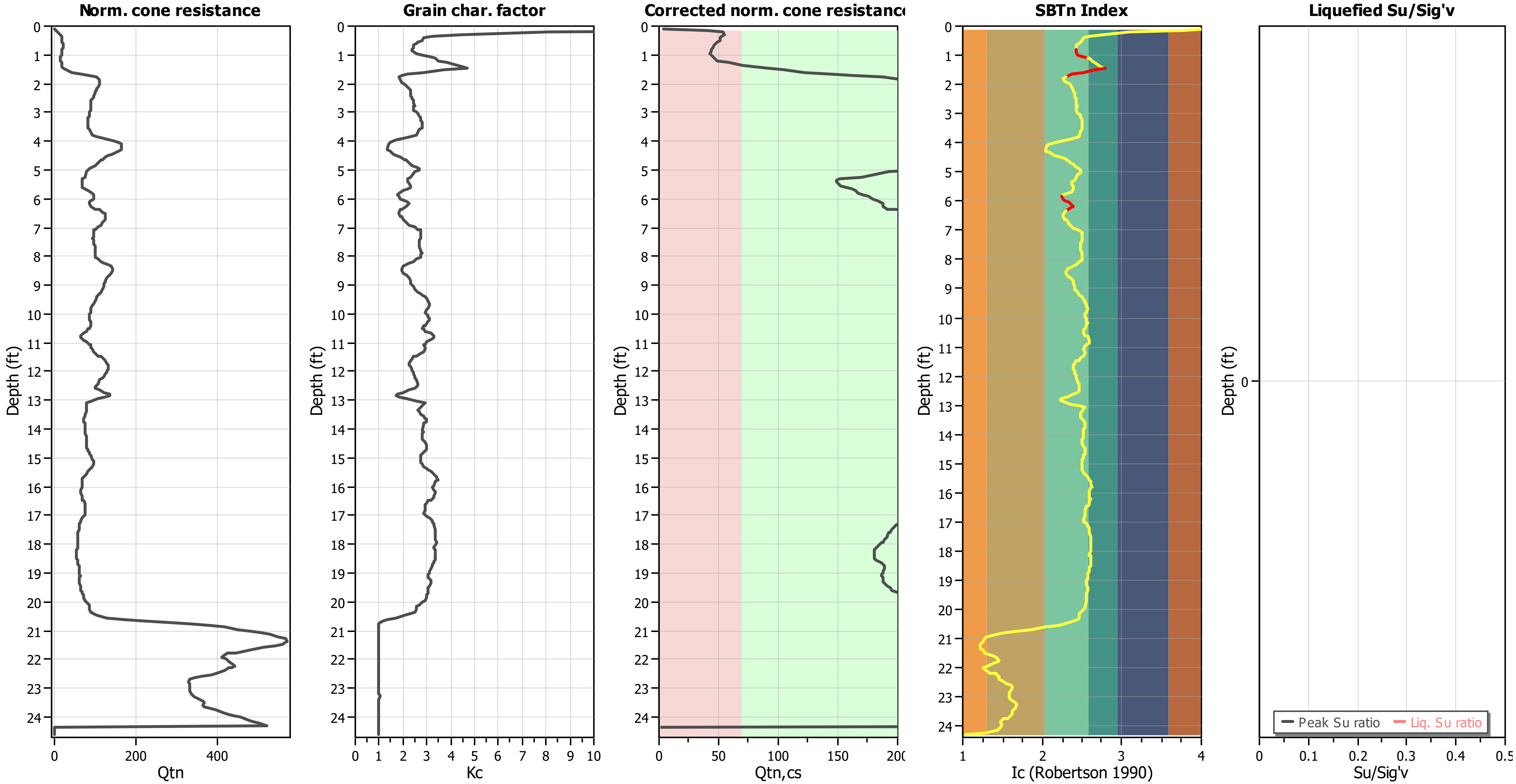




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	N/A

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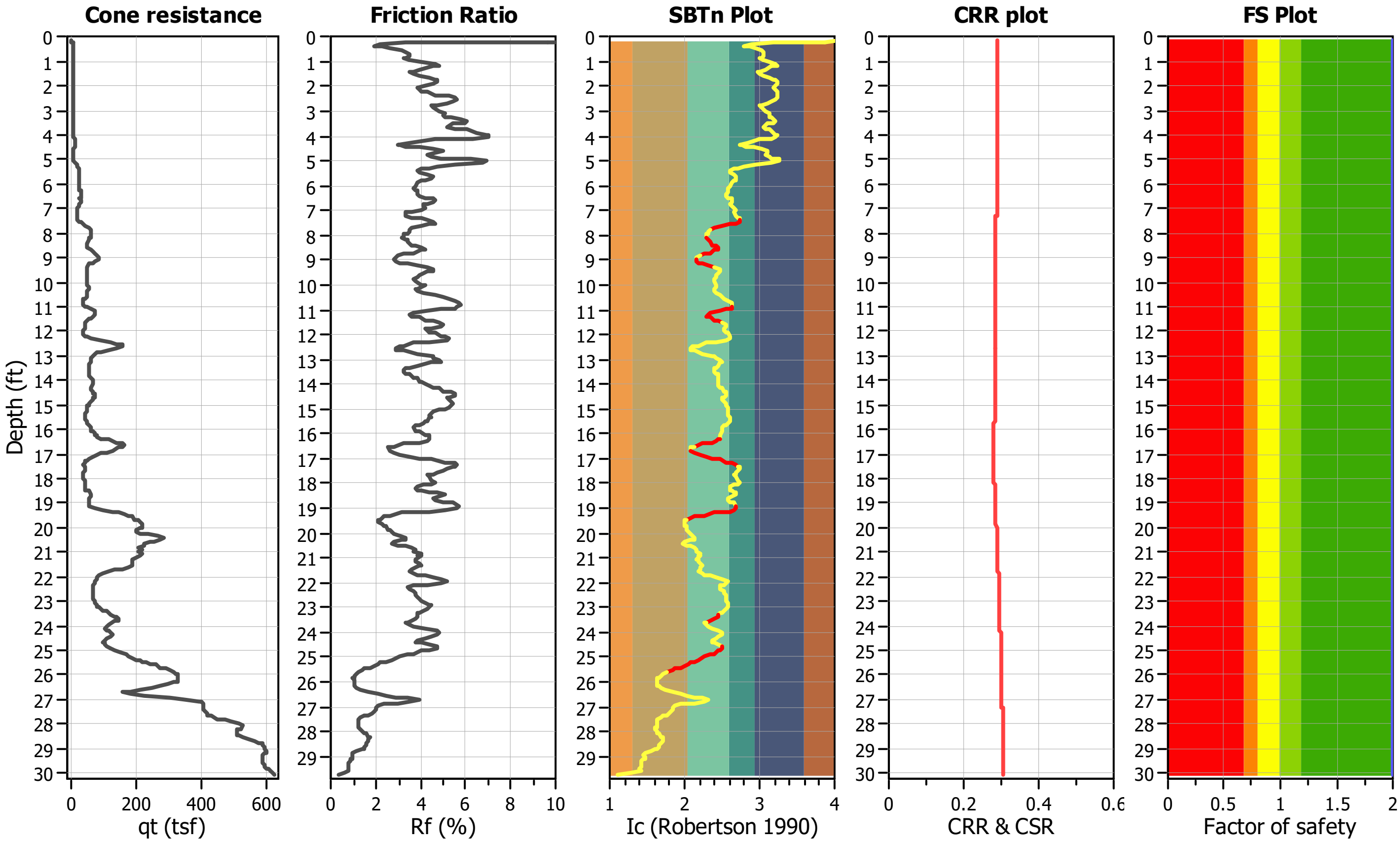
LIQUEFACTION ANALYSIS REPORT

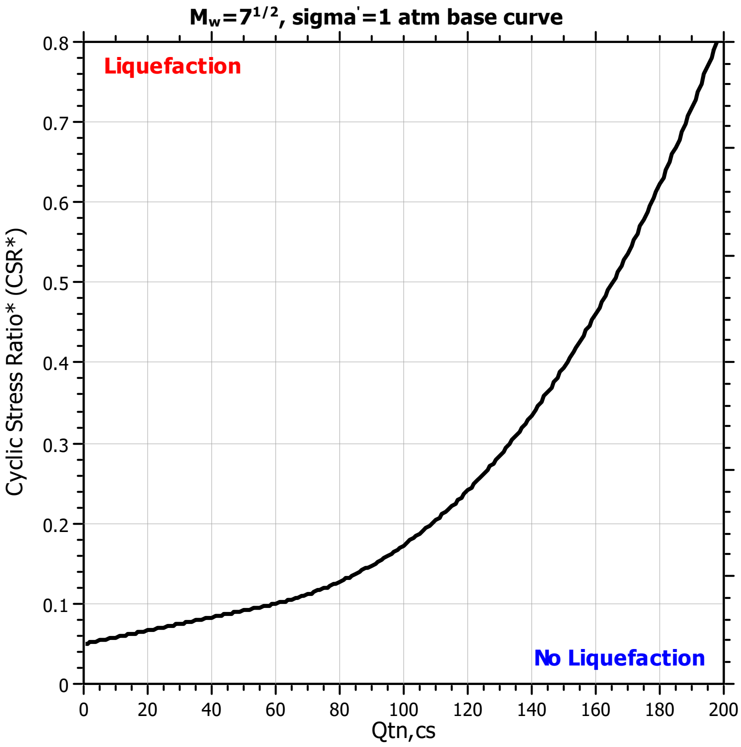
Project title : mmhs **Location : morningside drive, malibu, CA**

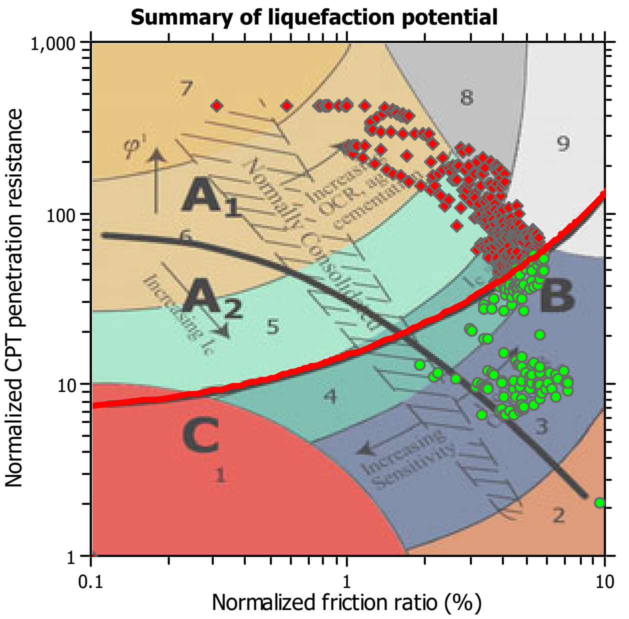
CPT file : CPT-2

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.):	45.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.76	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

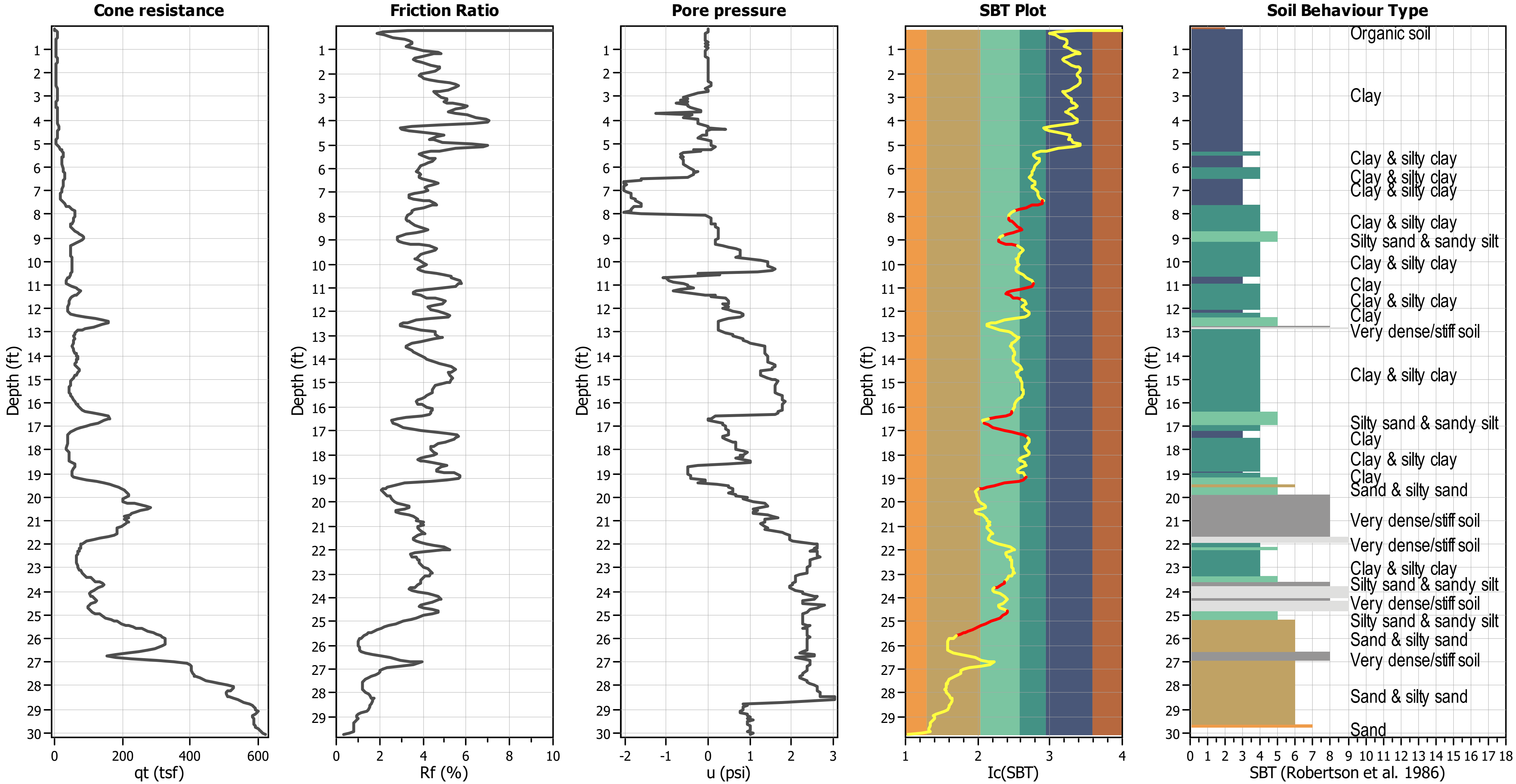






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



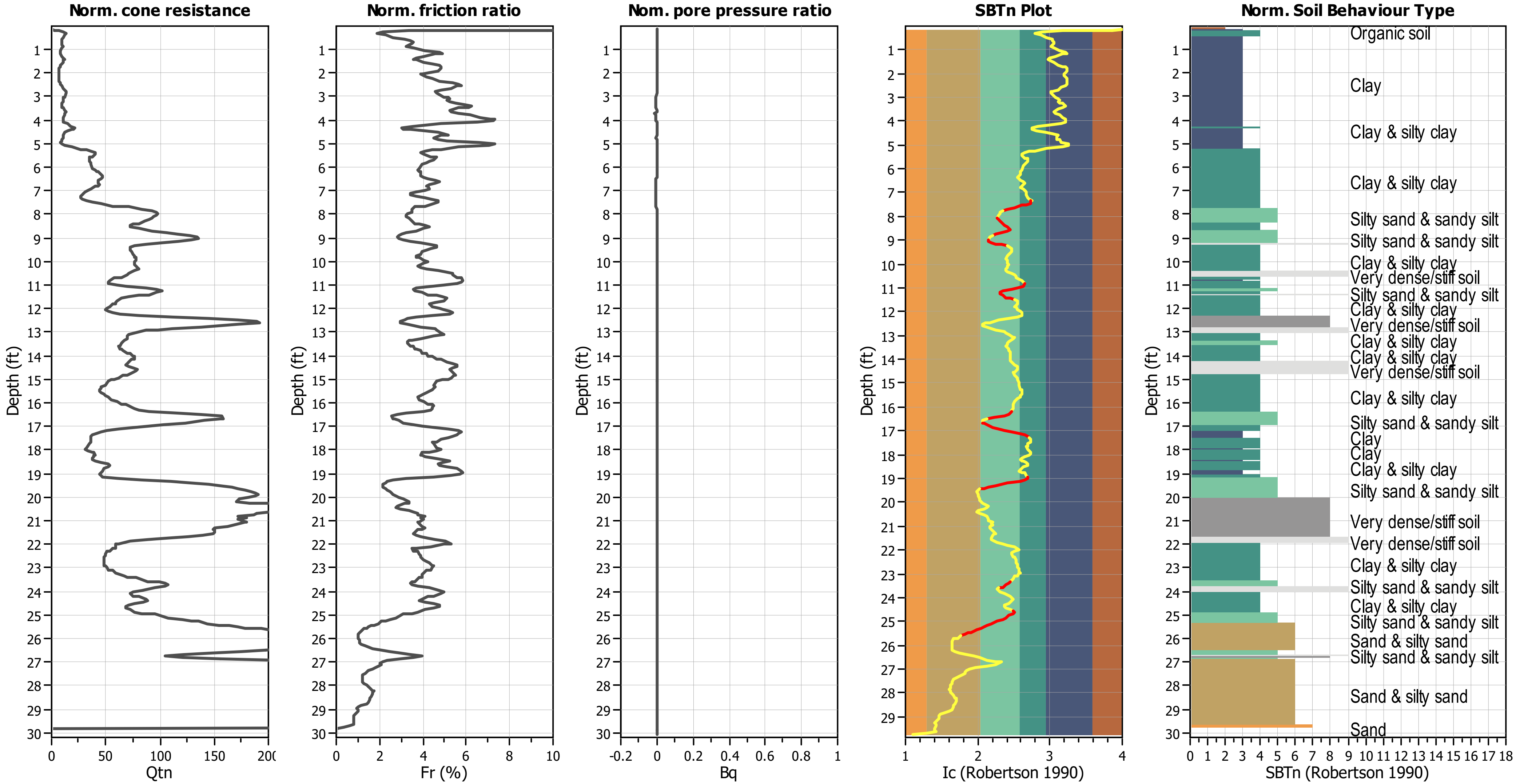
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	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)
Fines correction method: NCEER (1998)
Points to test: Based on Ic value
Earthquake magnitude M_w: 6.10
Peak ground acceleration: 0.76
Depth to water table (insitu): 45.00 ft

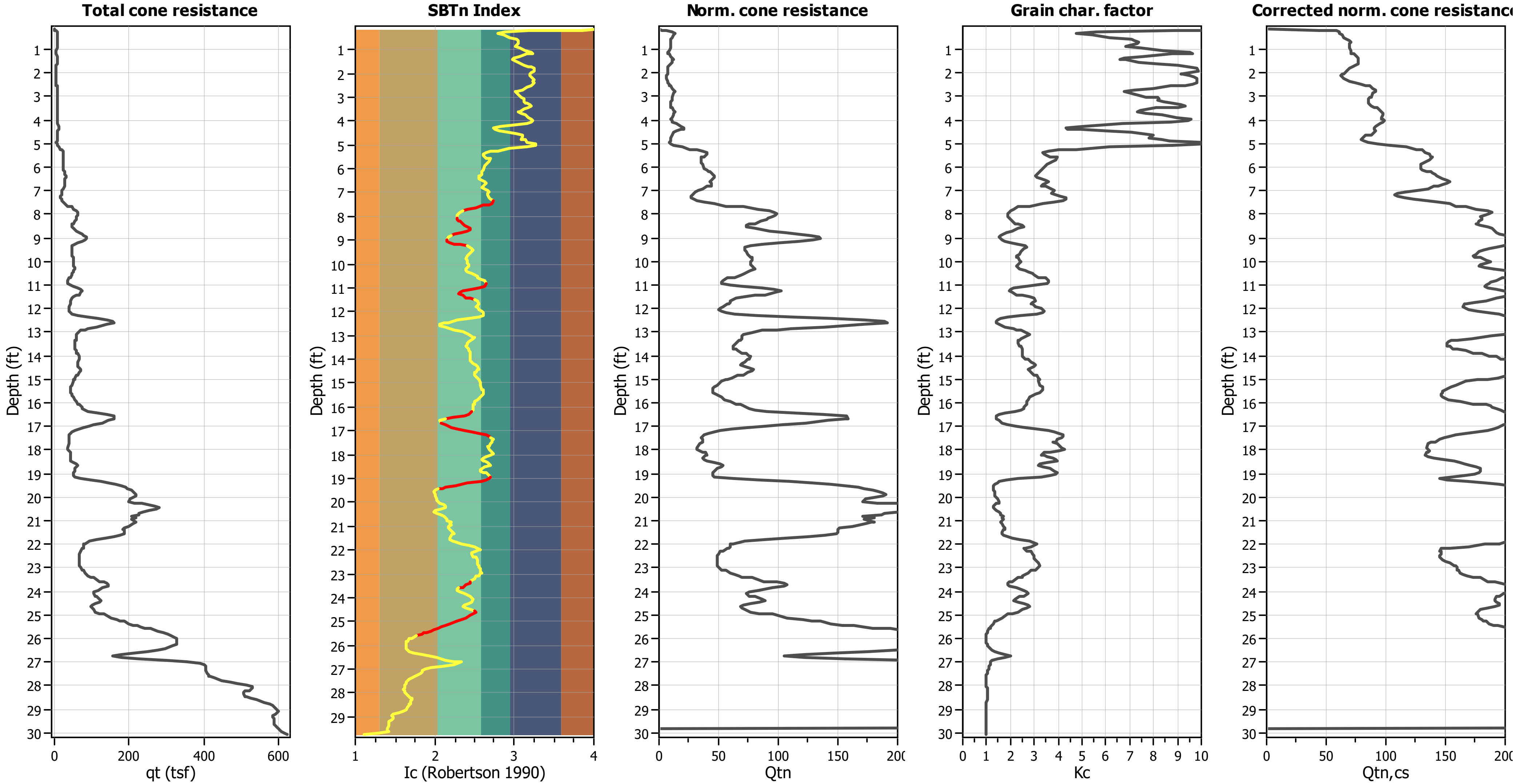
Depth to water table (erthq.): 45.00 ft
Average results interval: 3
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT
Use fill: No
Fill height: N/A

Fill weight: N/A
Transition detect. applied: Yes
K_σ applied: Yes
Clay like behavior applied: Sands only
Limit depth applied: No
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

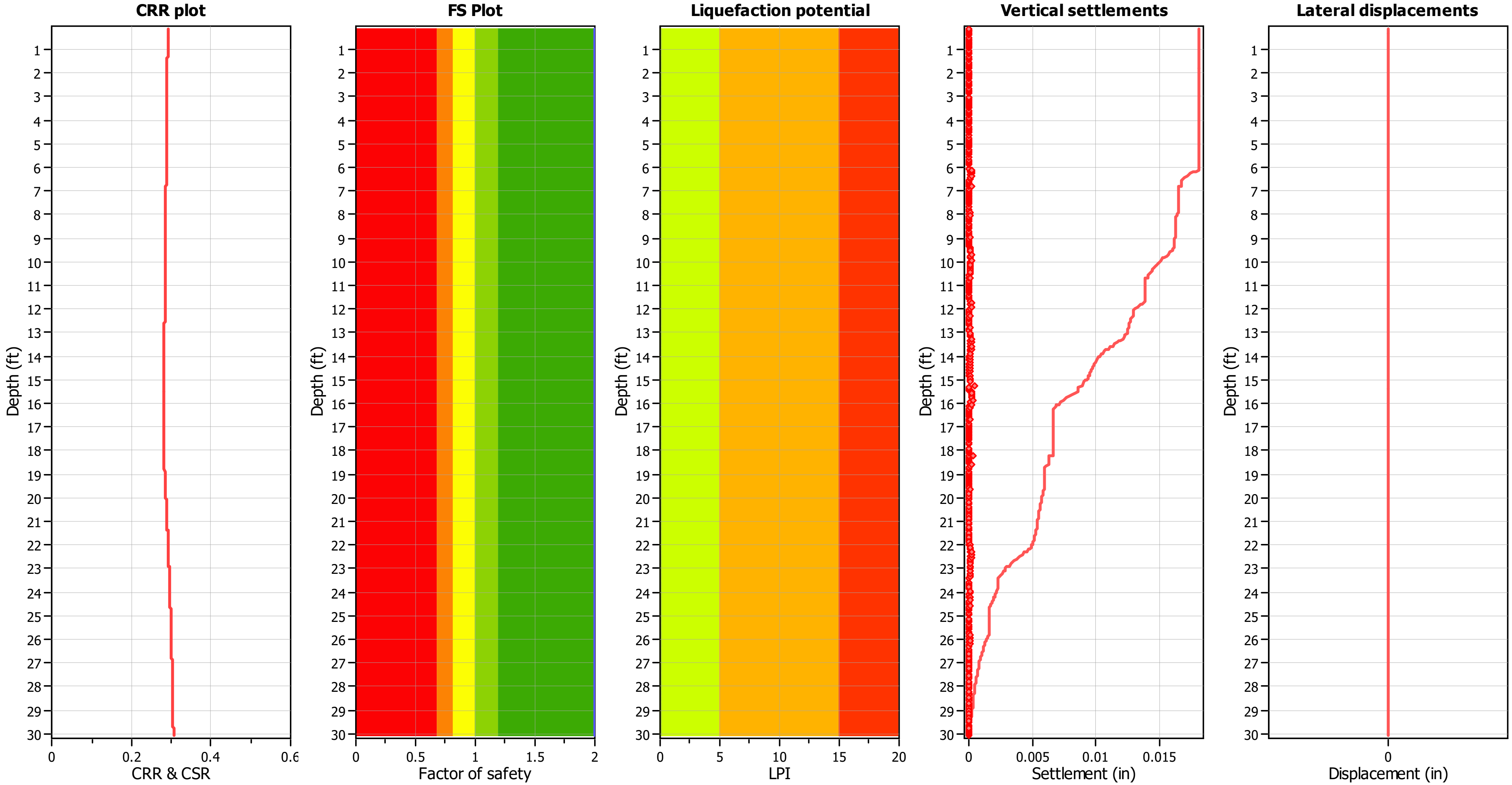
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	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.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	N/A

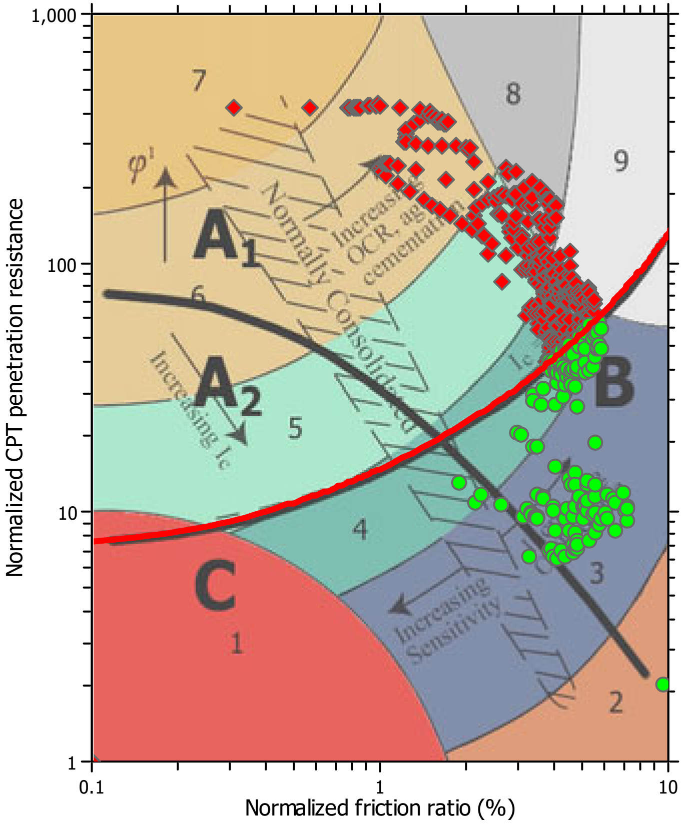
F.S. color scheme

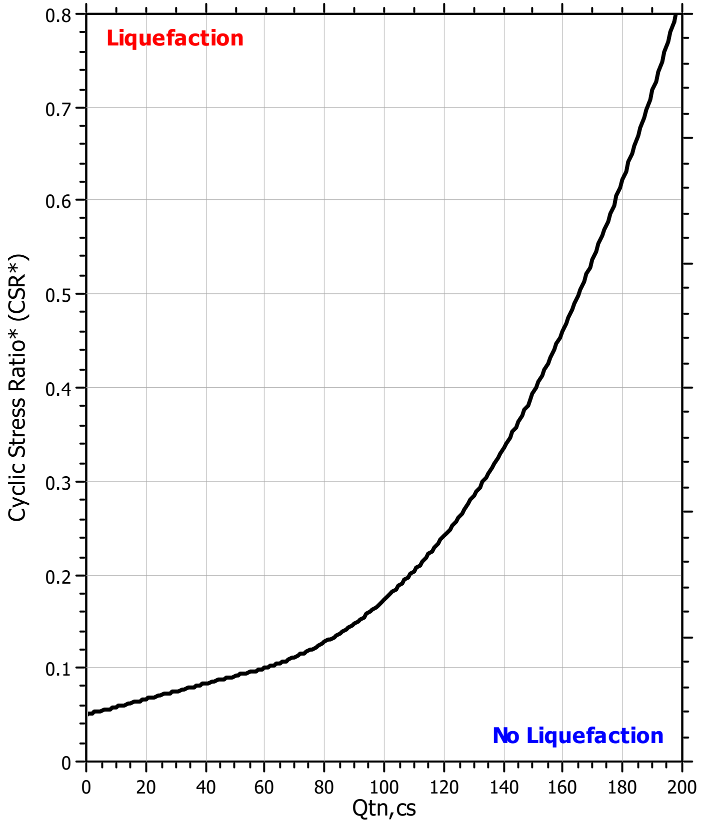
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

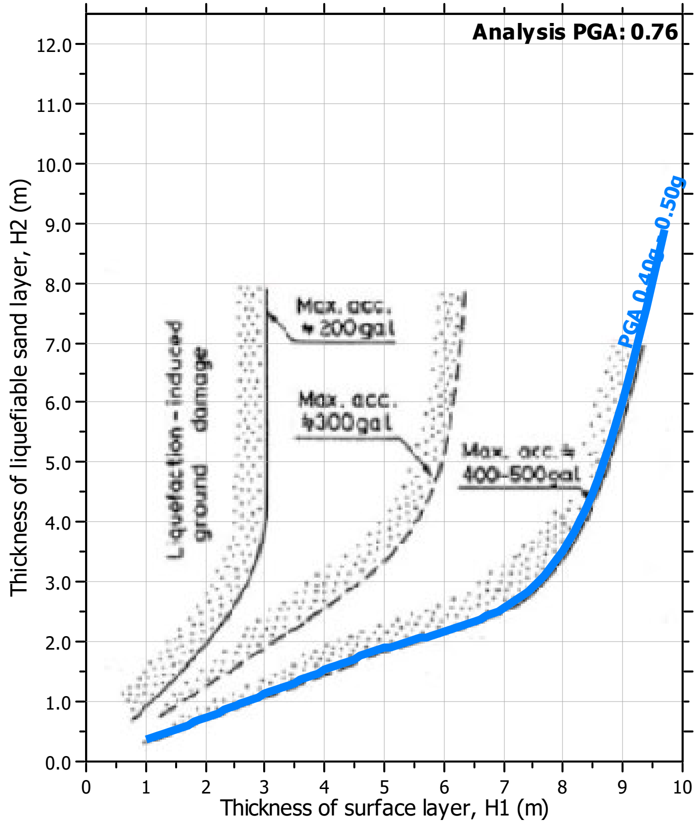
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



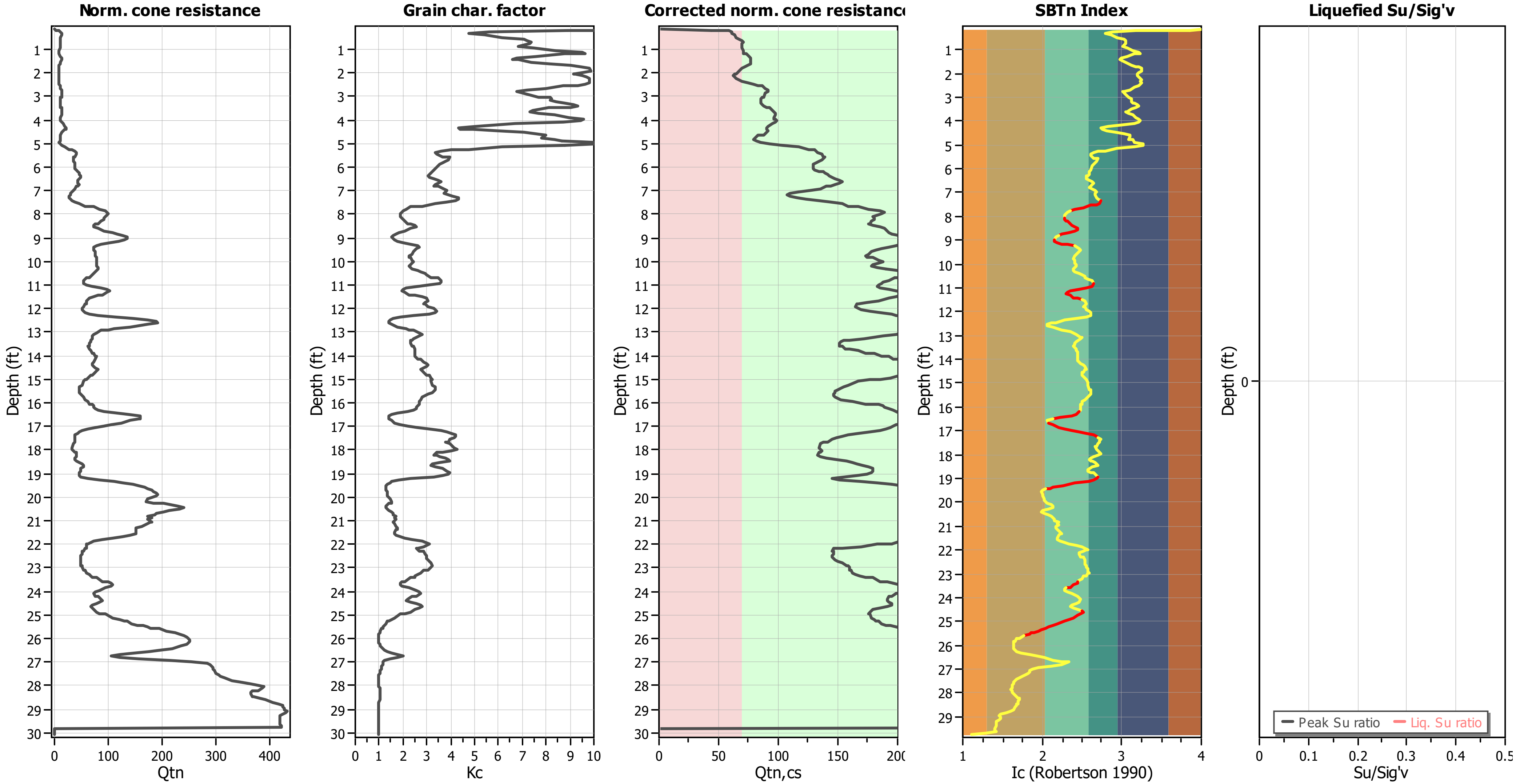




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	N/A

Leighton Consulting, Inc.
17781 Cowan
Irvine, CA 92614
http://www.leightongeo.com

LIQUEFACTION ANALYSIS REPORT

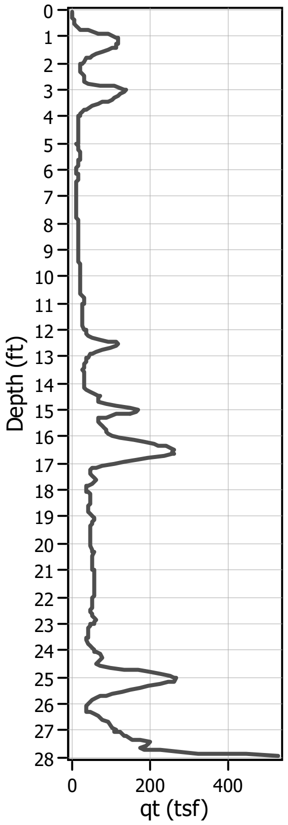
Project title : mmhs **Location : morningside drive, malibu, CA**

CPT file : CPT-3

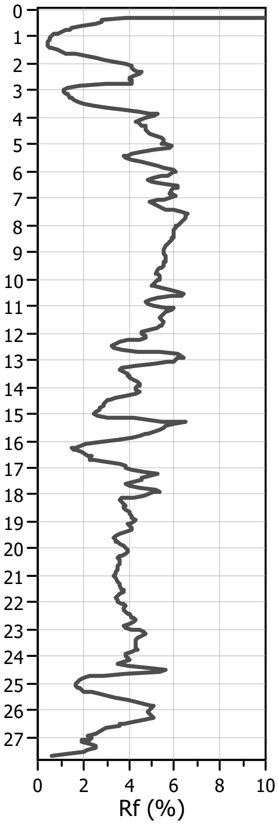
Input parameters and analysis data

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.):	45.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.76	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

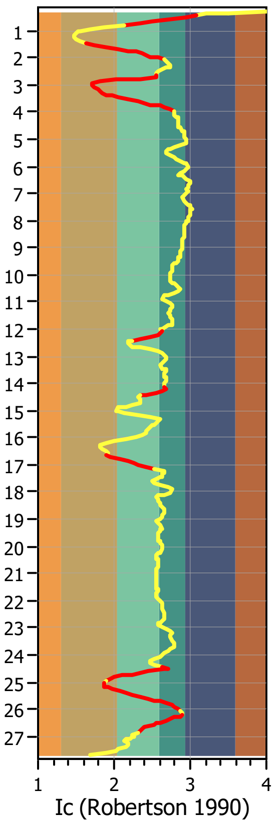
Cone resistance



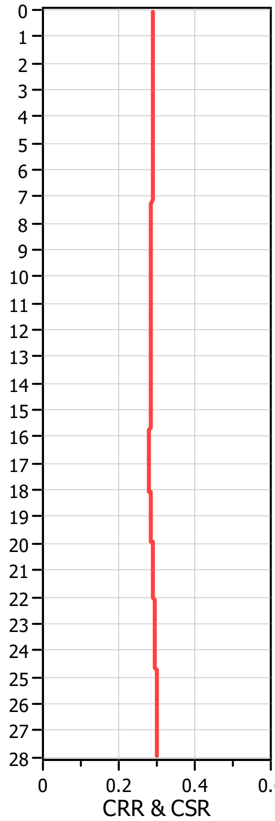
Friction Ratio



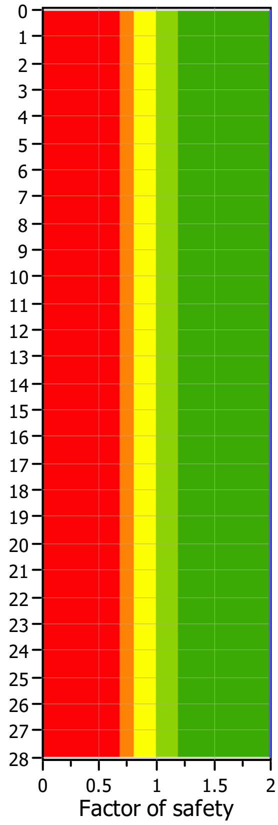
SBTn Plot



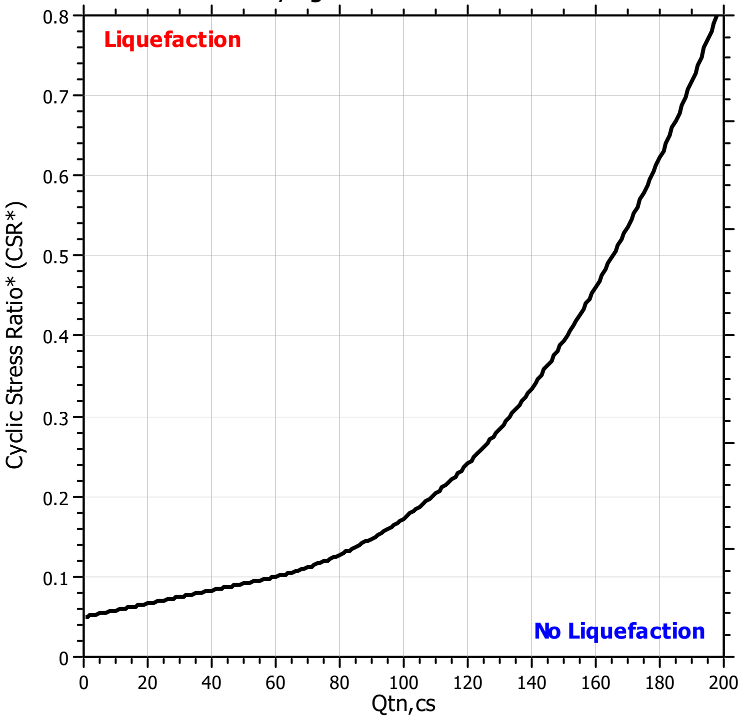
CRR plot



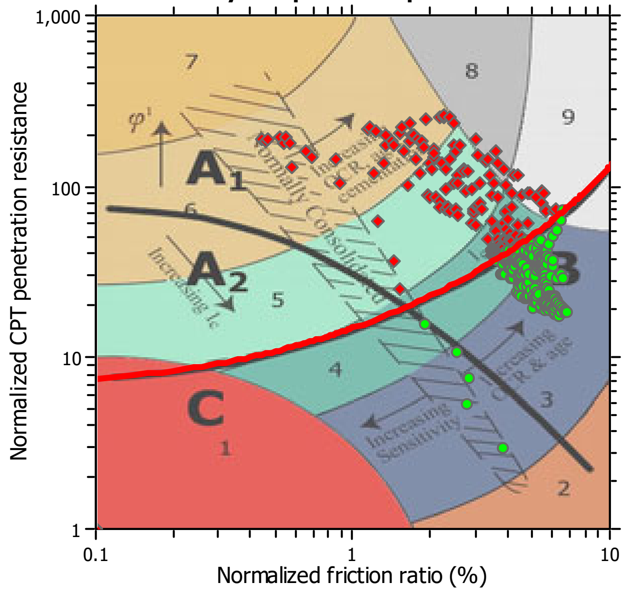
FS Plot



$M_w=7^{1/2}$, sigma'=1 atm base curve

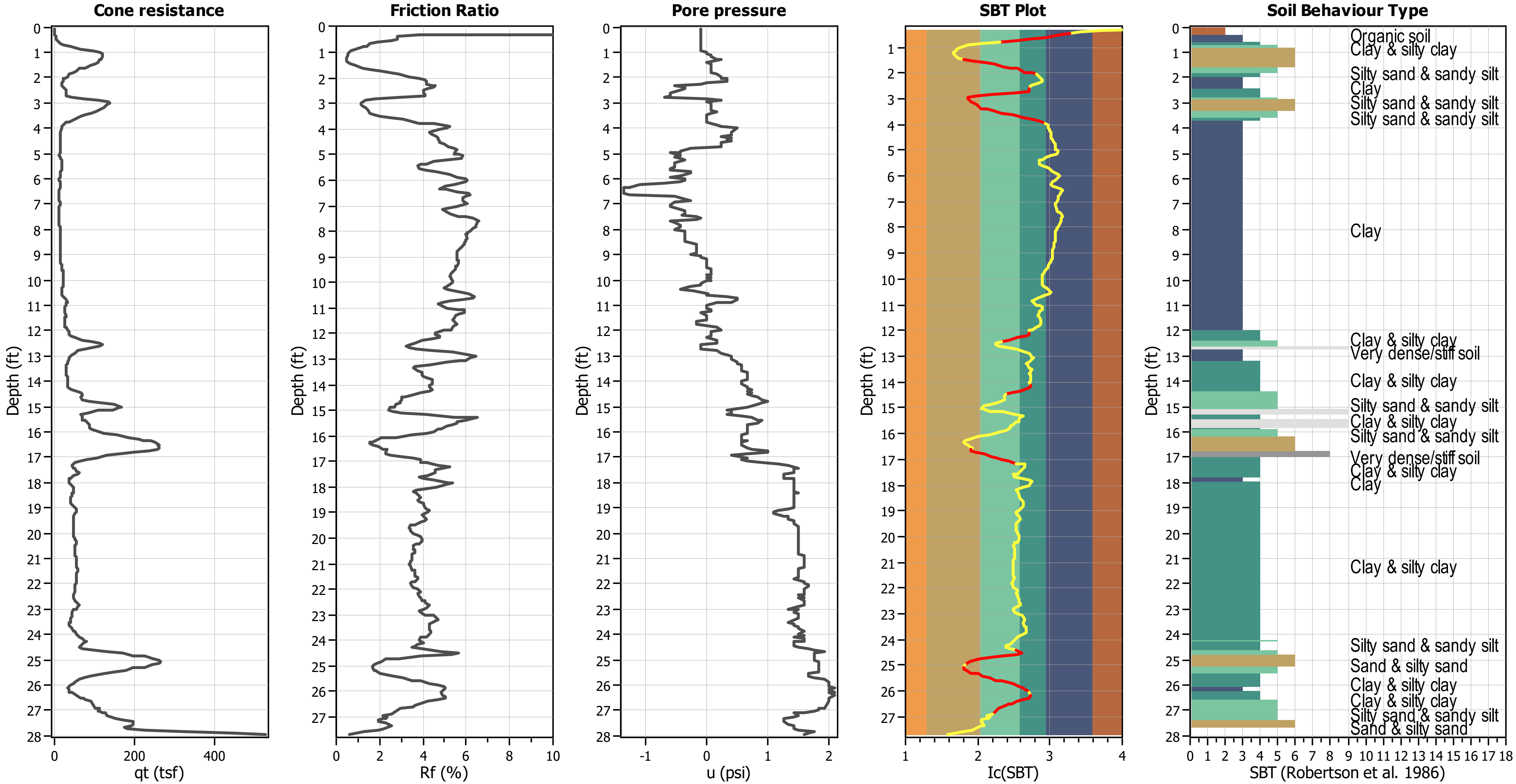


Summary of liquefaction potential



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

CPT basic interpretation plots



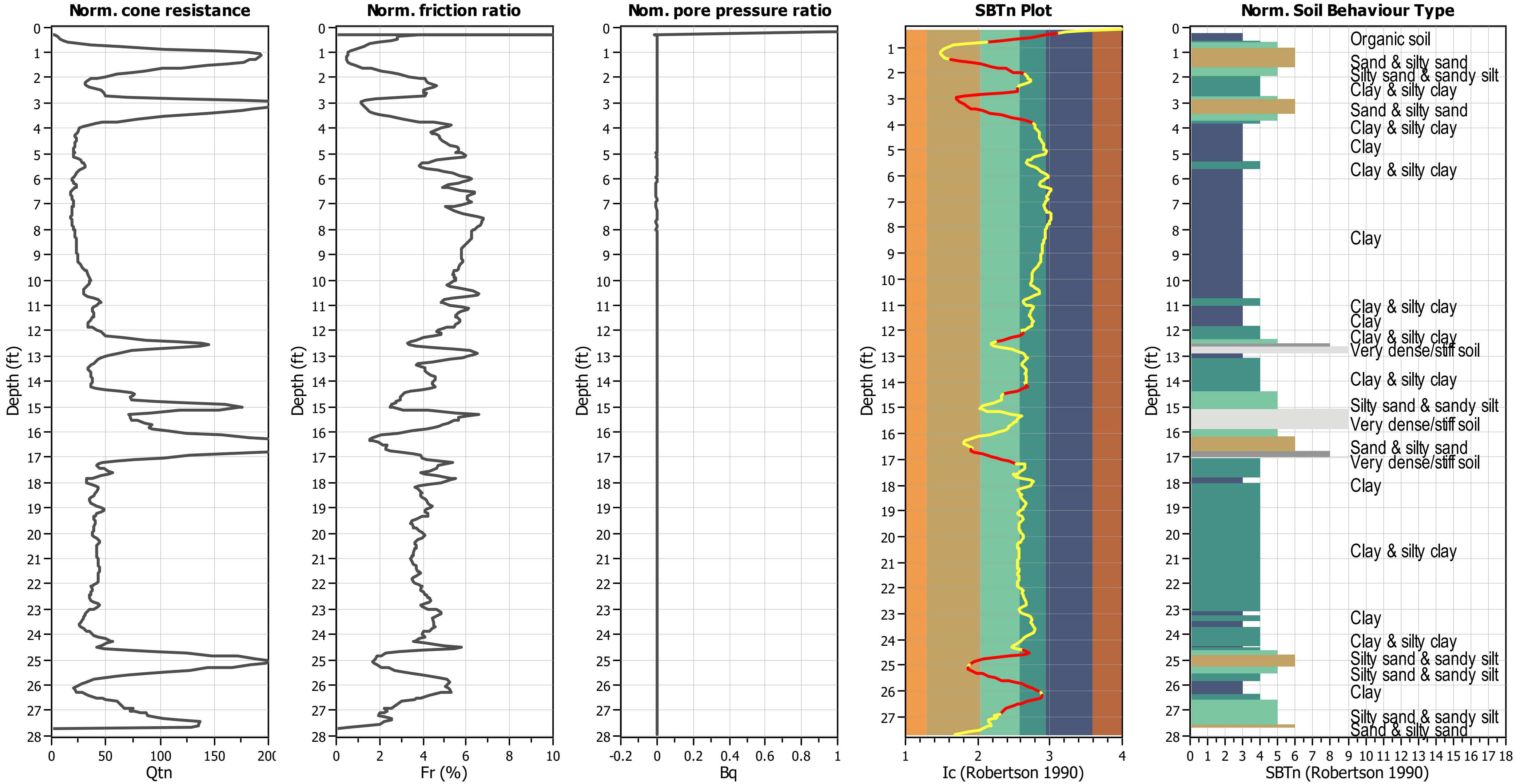
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	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)



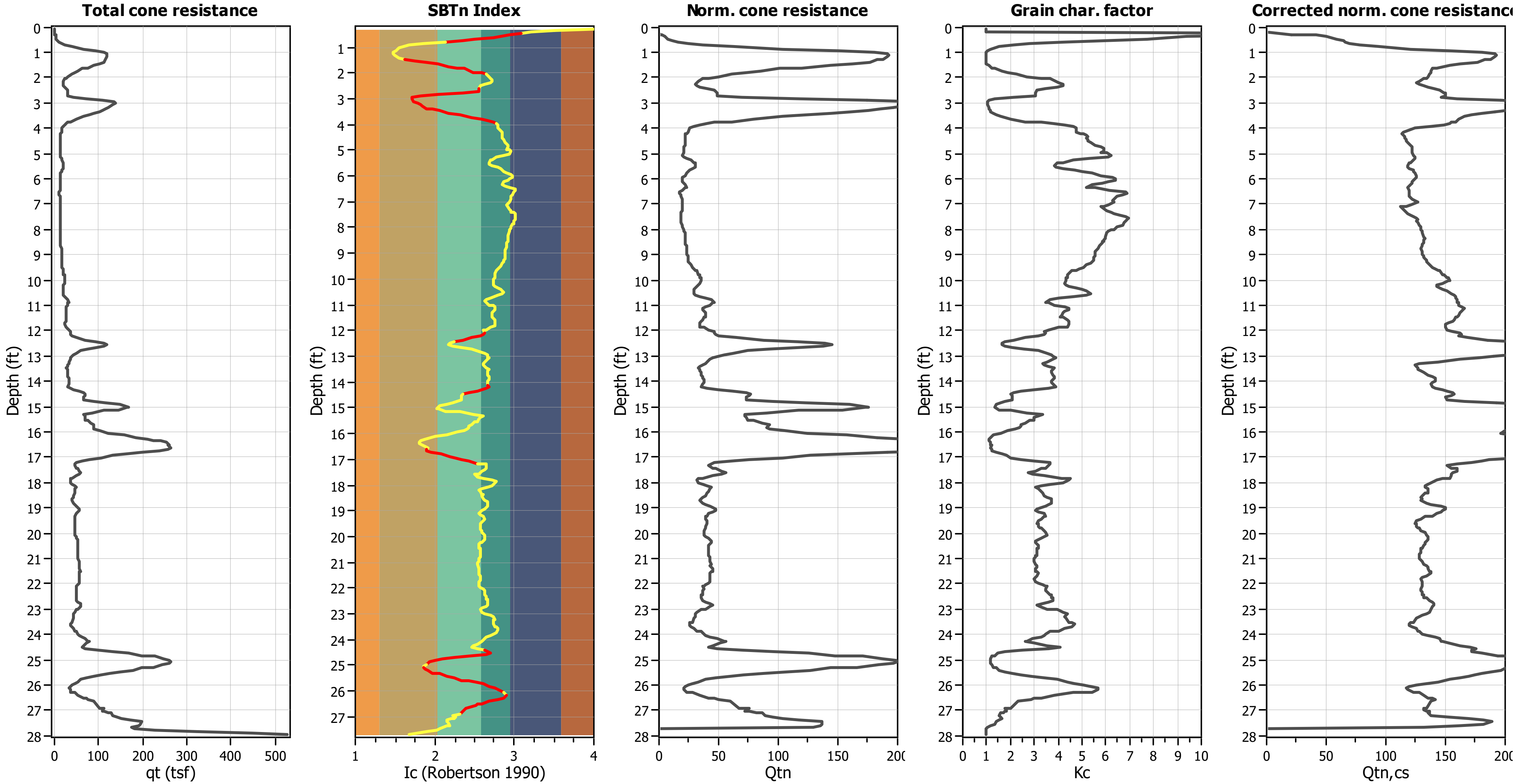
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	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

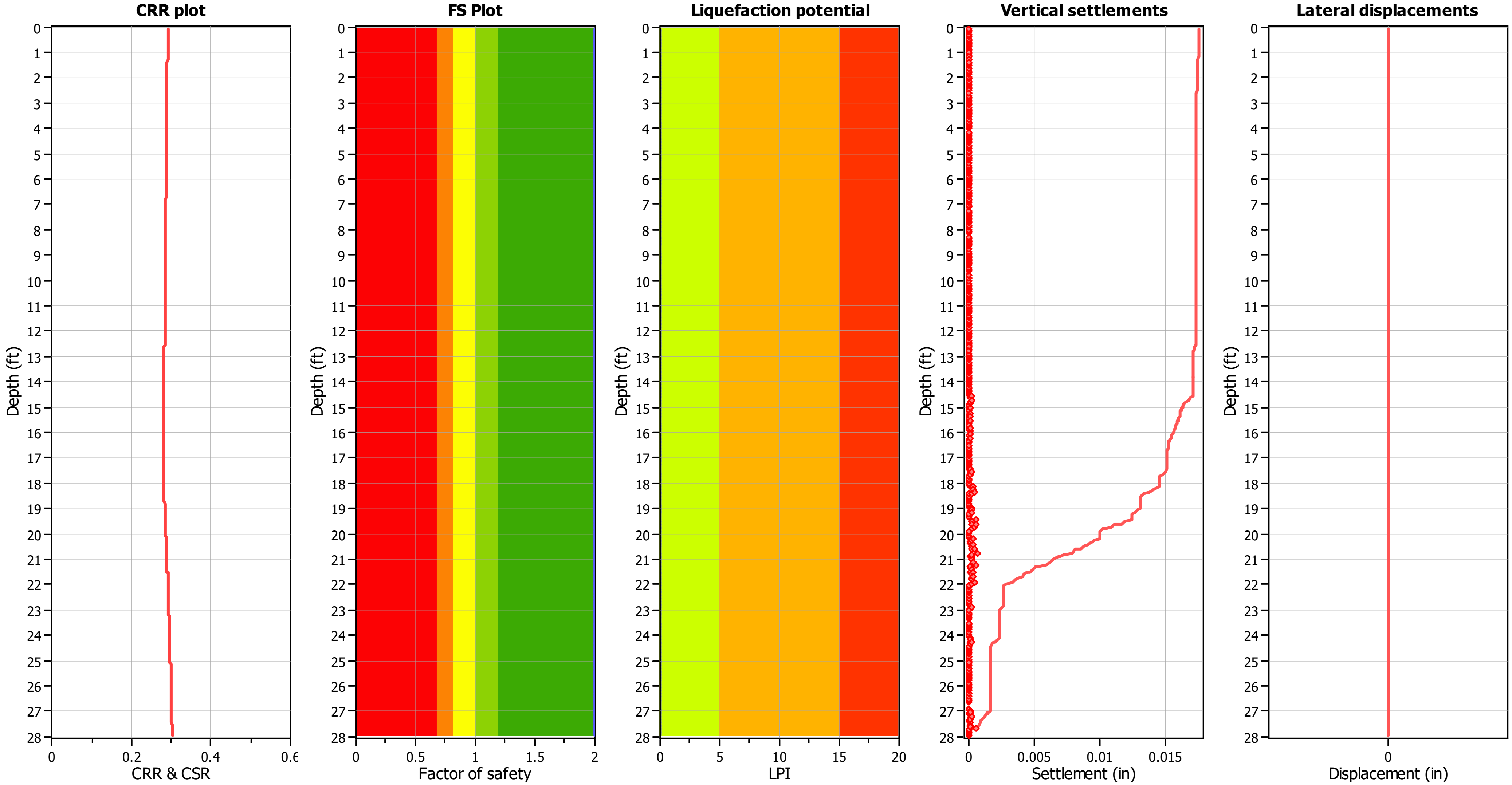
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	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.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	N/A

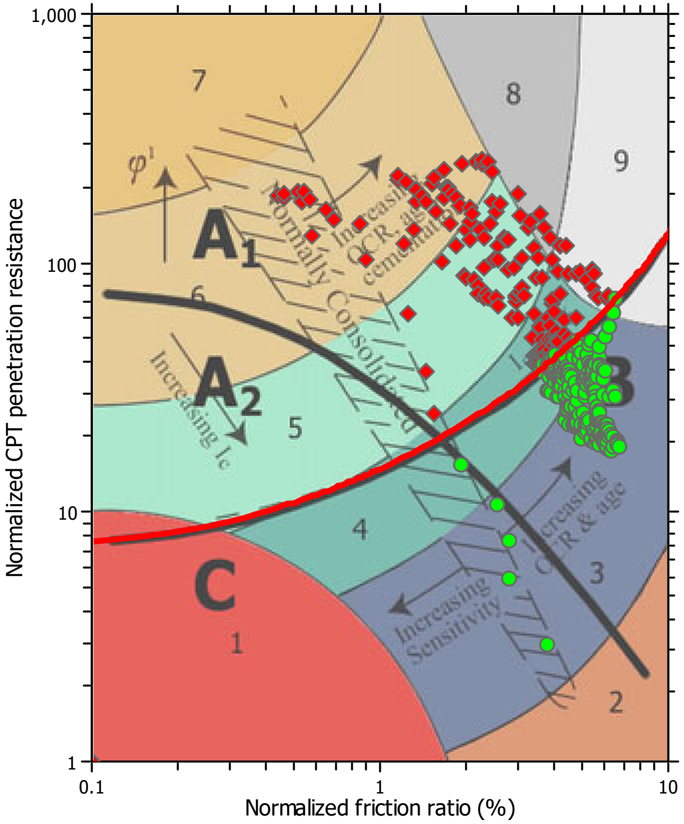
F.S. color scheme

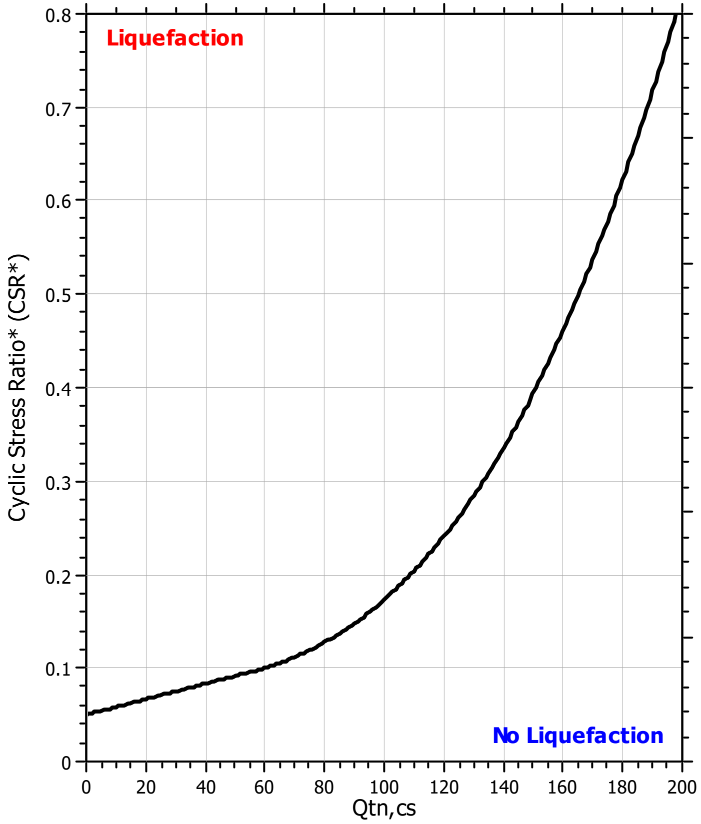
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

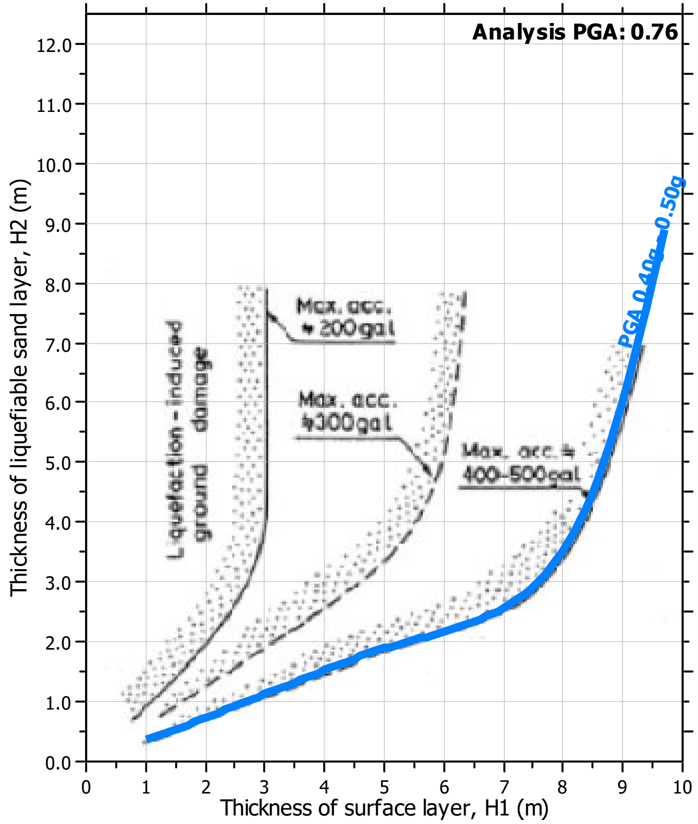
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



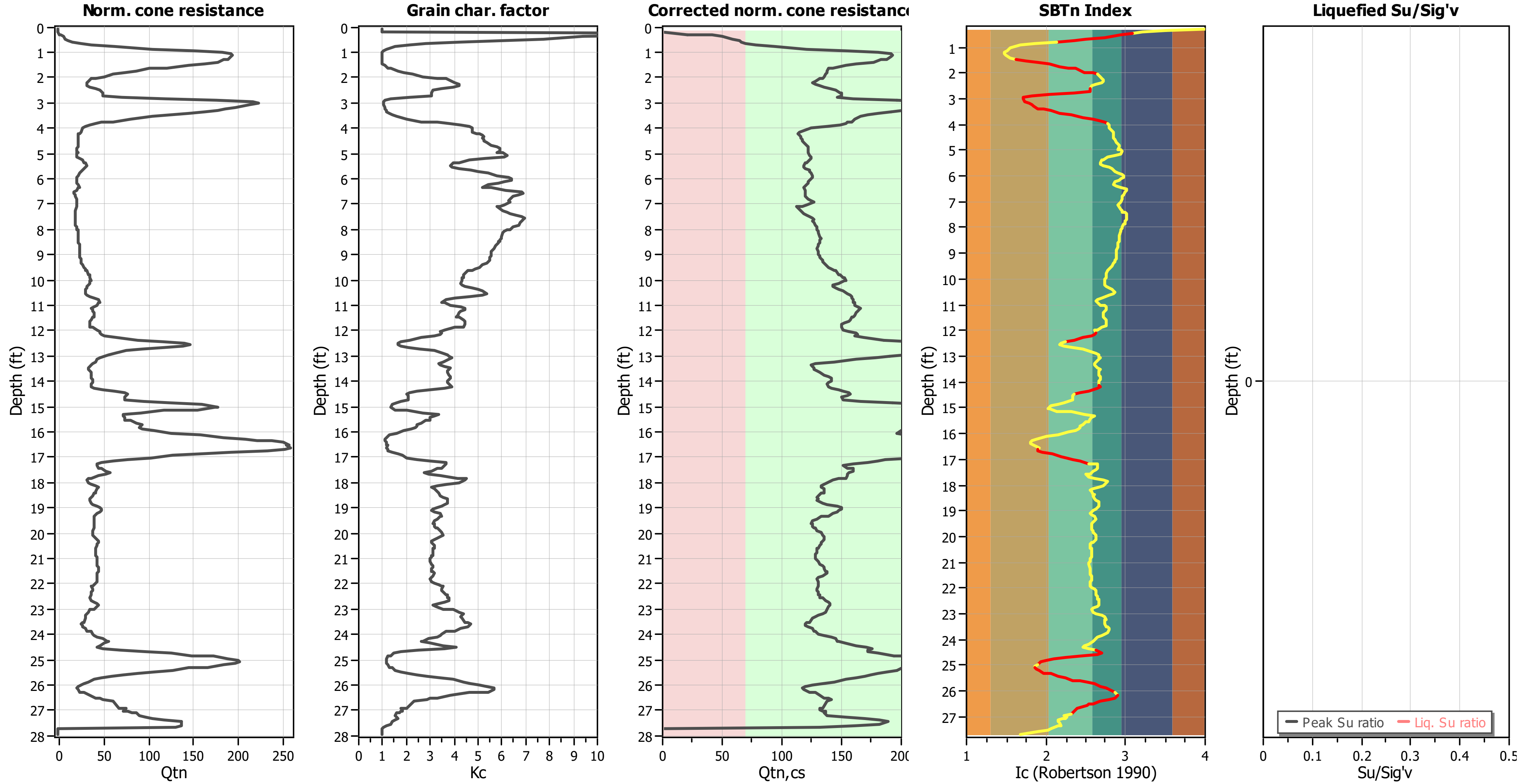




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	N/A

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Irvine, CA 92614
http://www.leightongeo.com

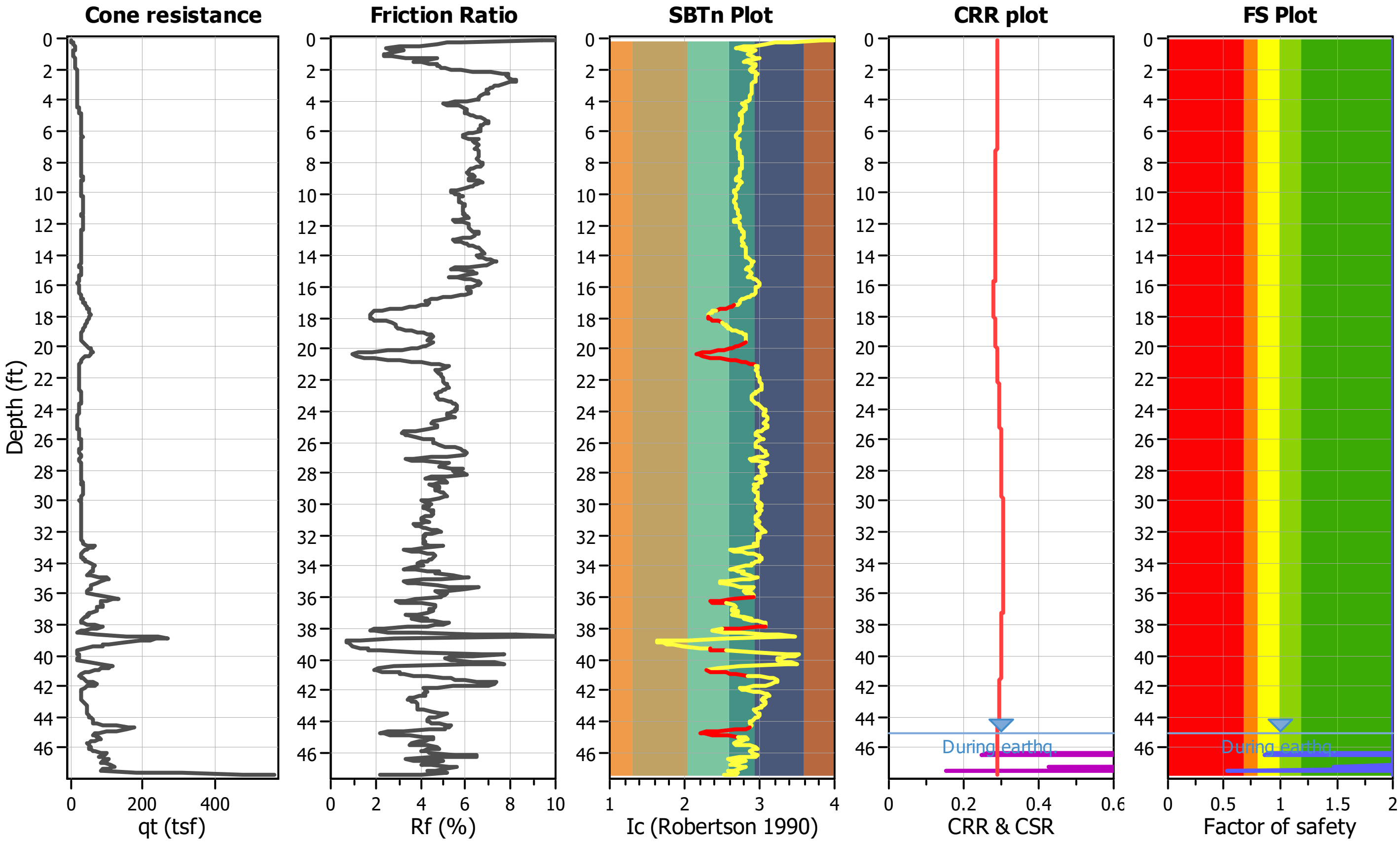
LIQUEFACTION ANALYSIS REPORT

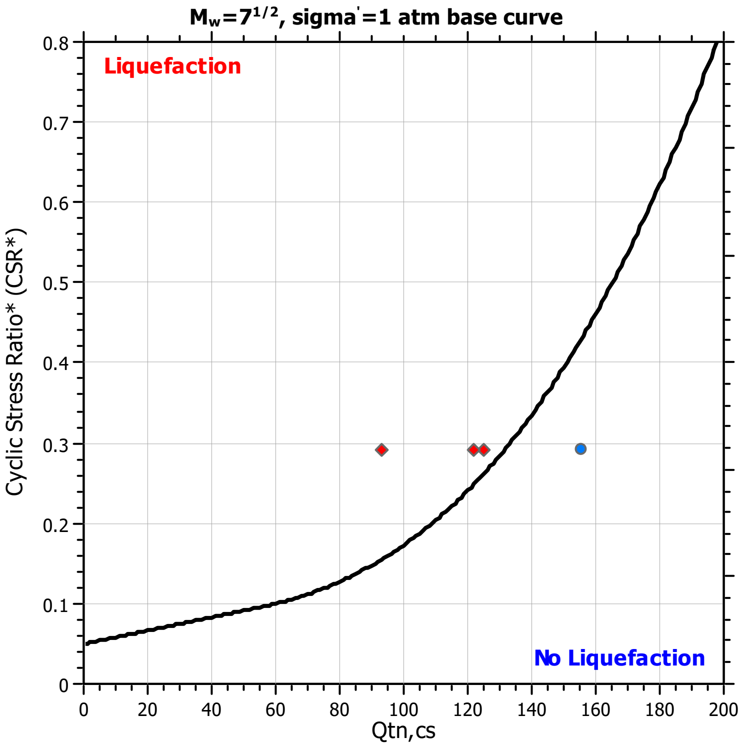
Project title : mmhs **Location : morningside drive, malibu, CA**

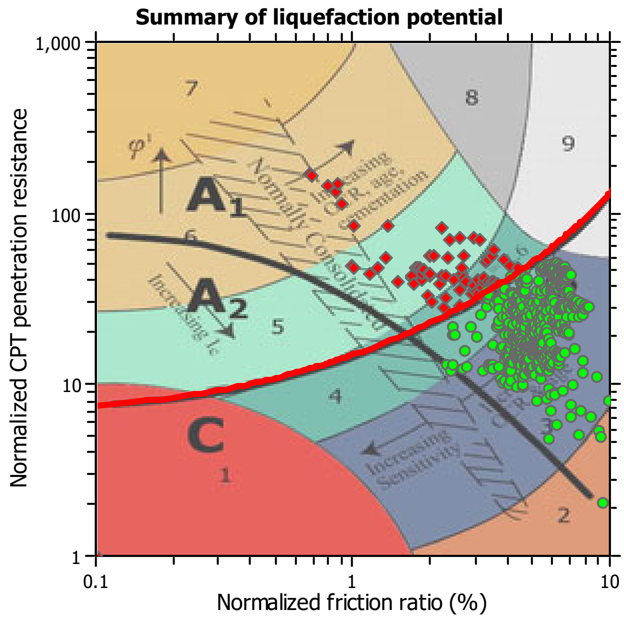
CPT file : CPT-4

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.):	45.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.76	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

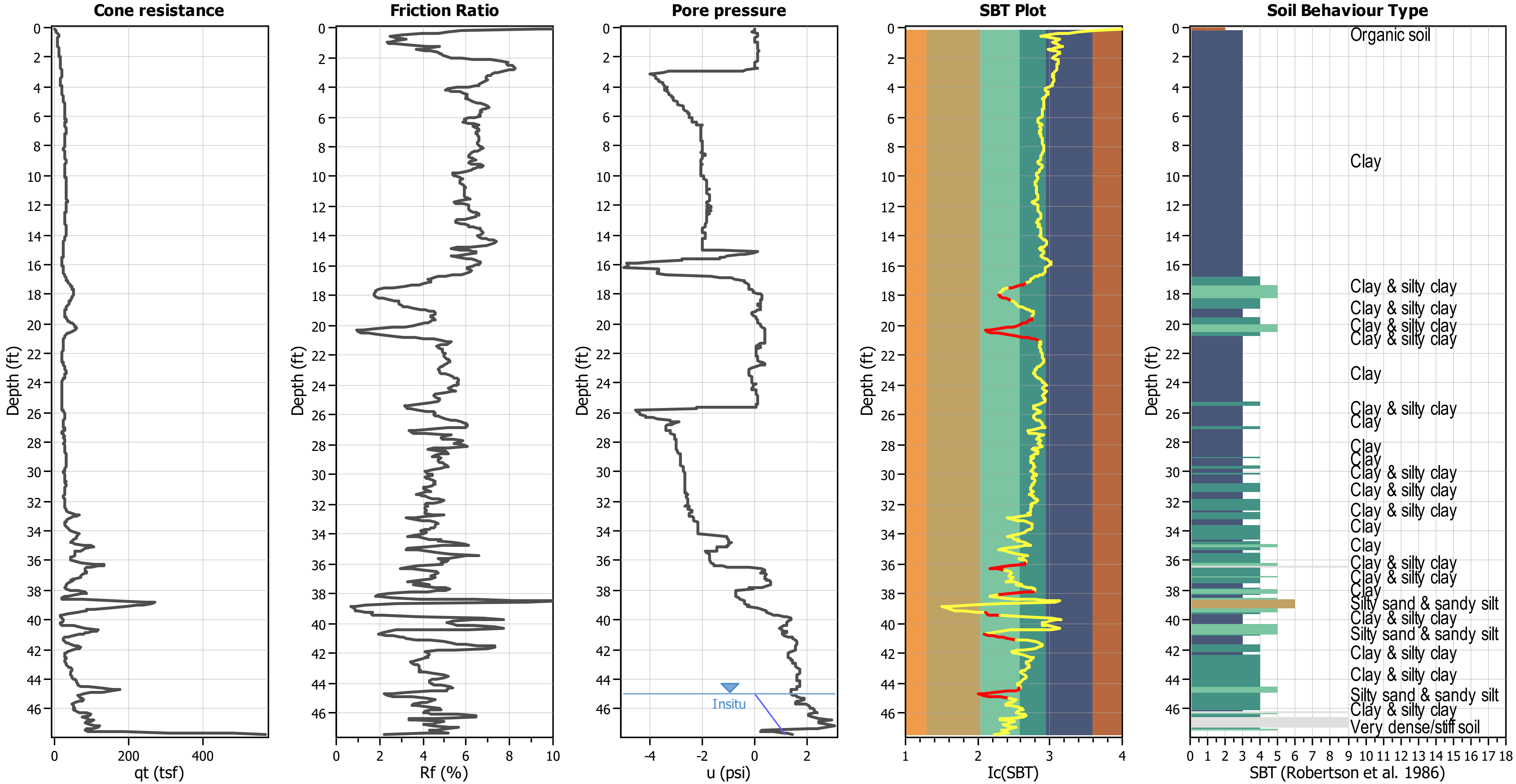






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



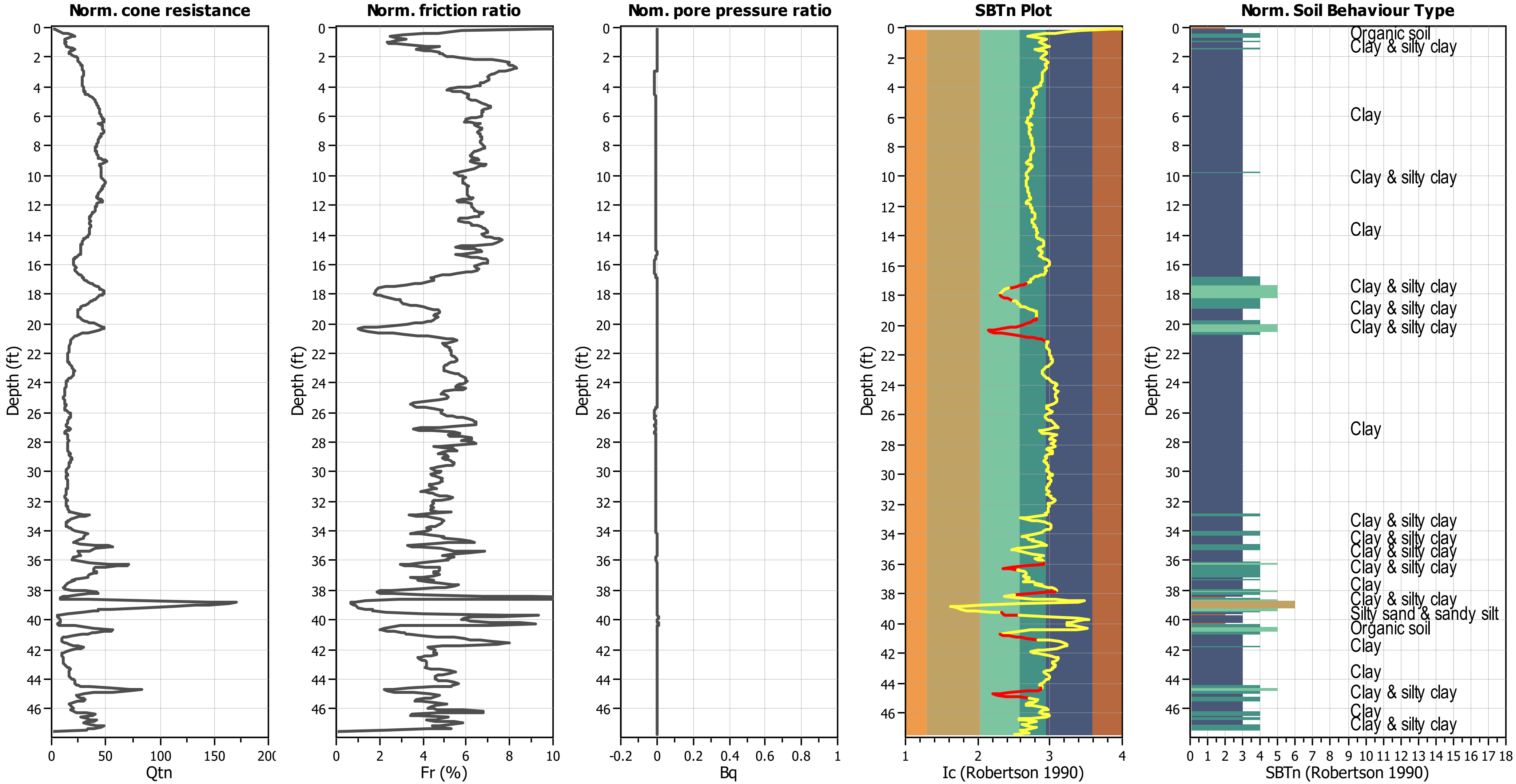
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	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)



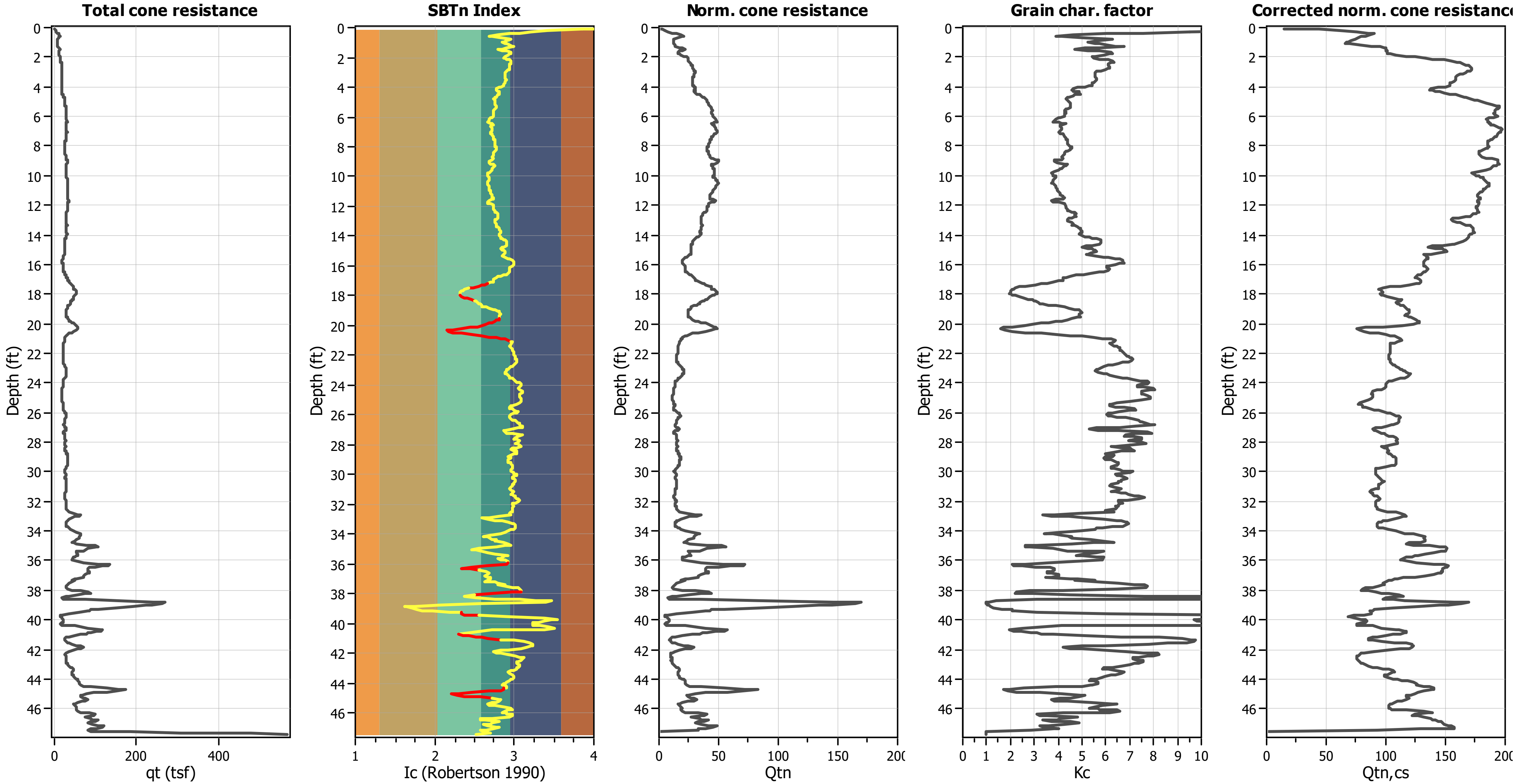
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	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	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

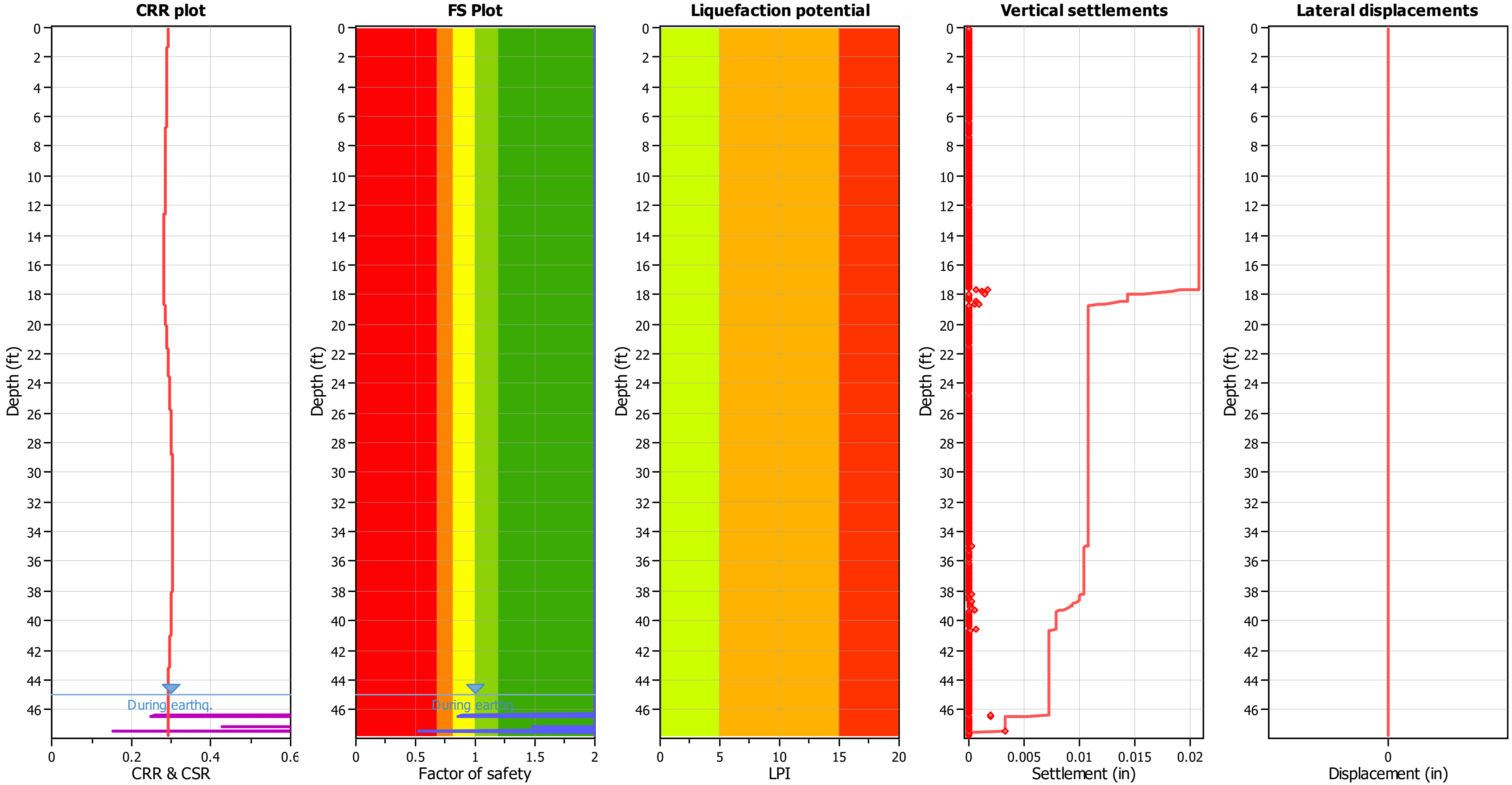
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	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.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	N/A

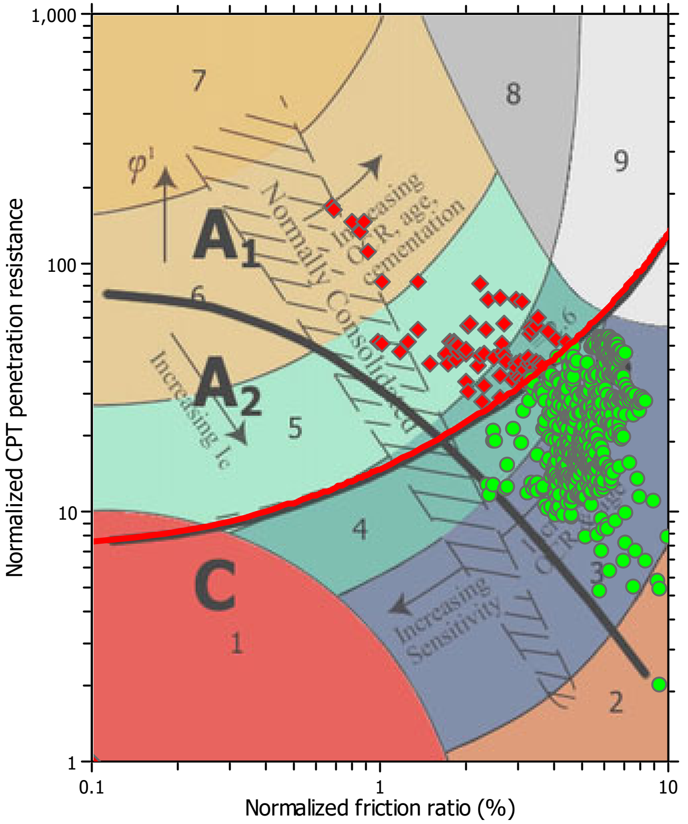
F.S. color scheme

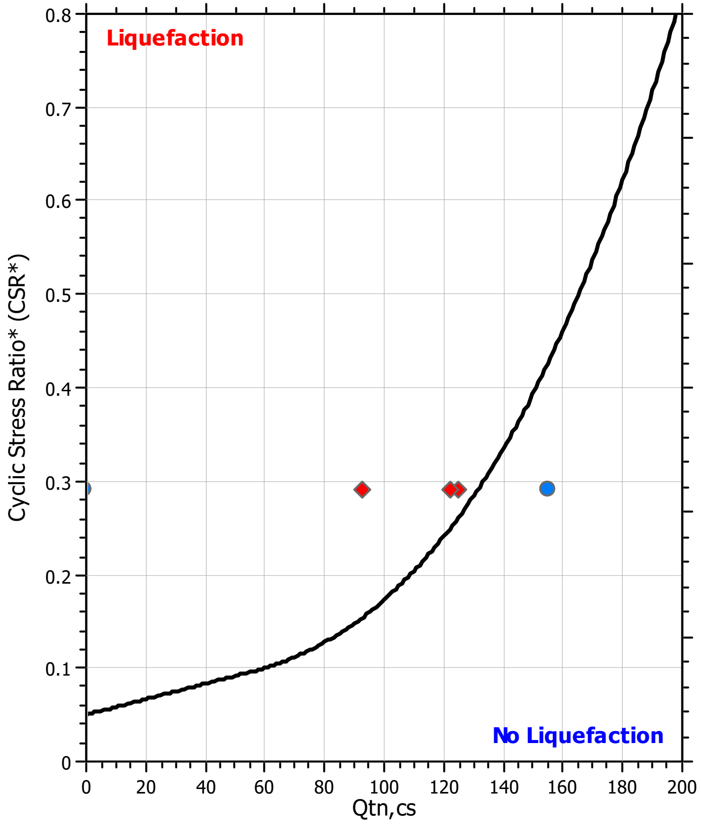
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

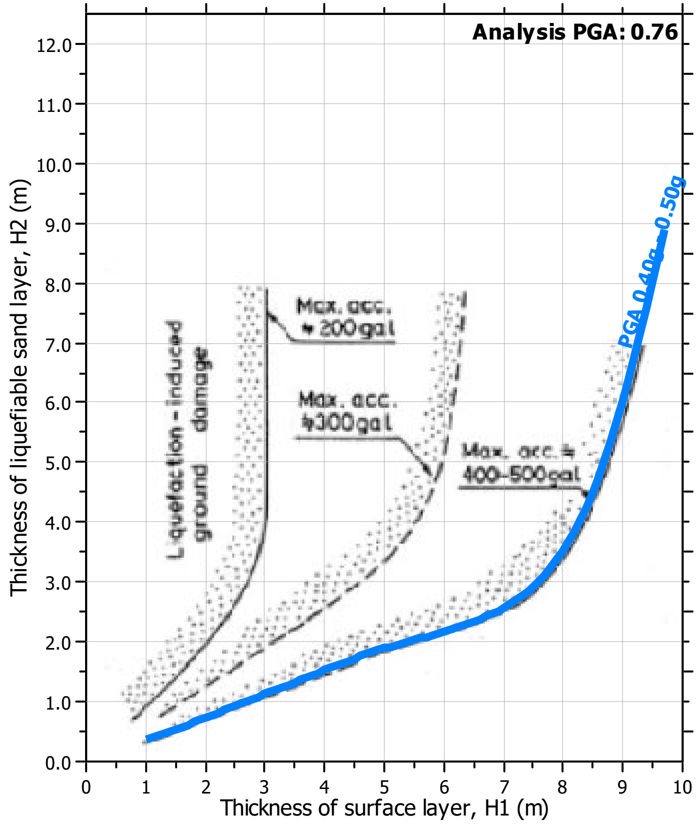
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



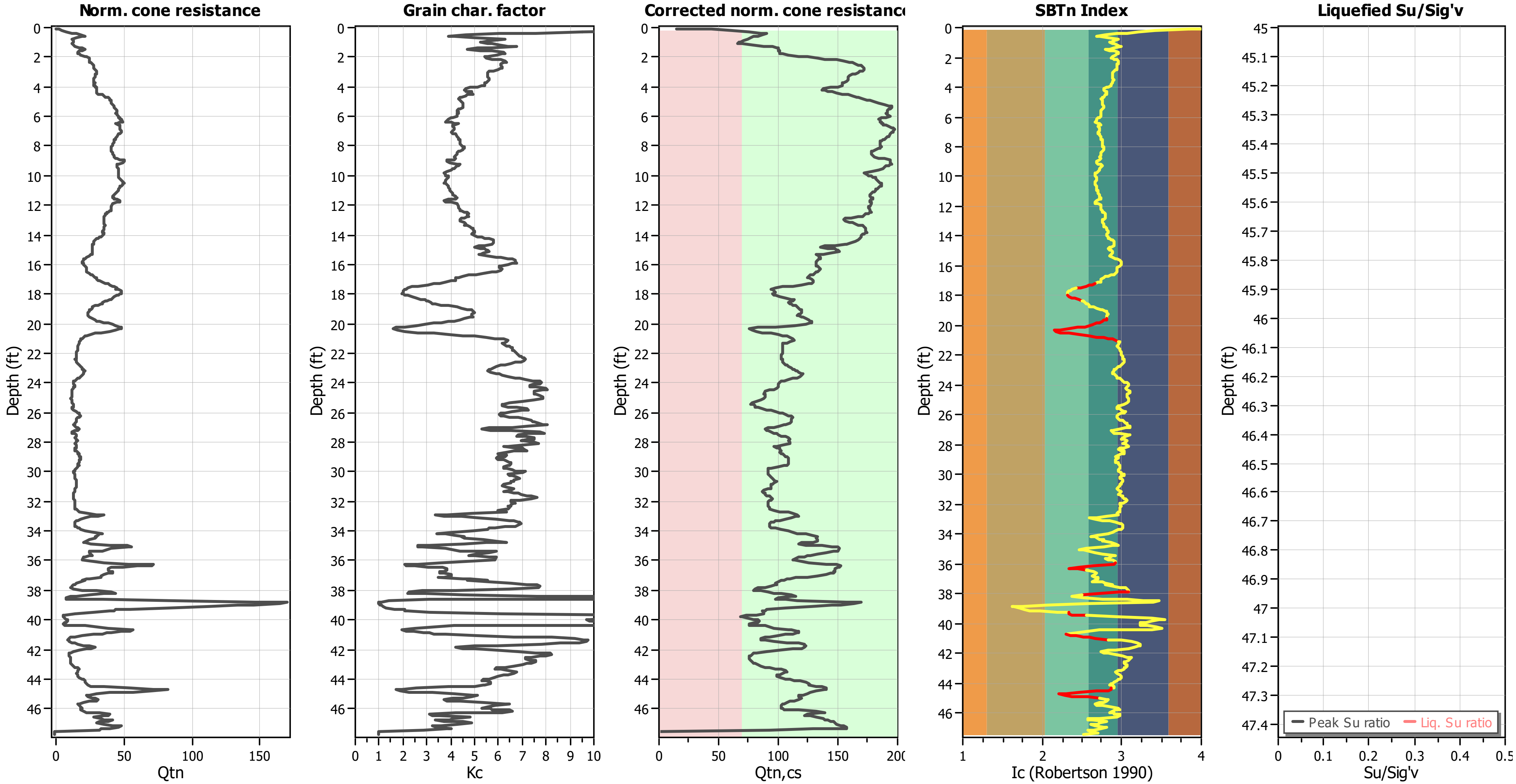




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	N/A

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http://www.leightongeo.com

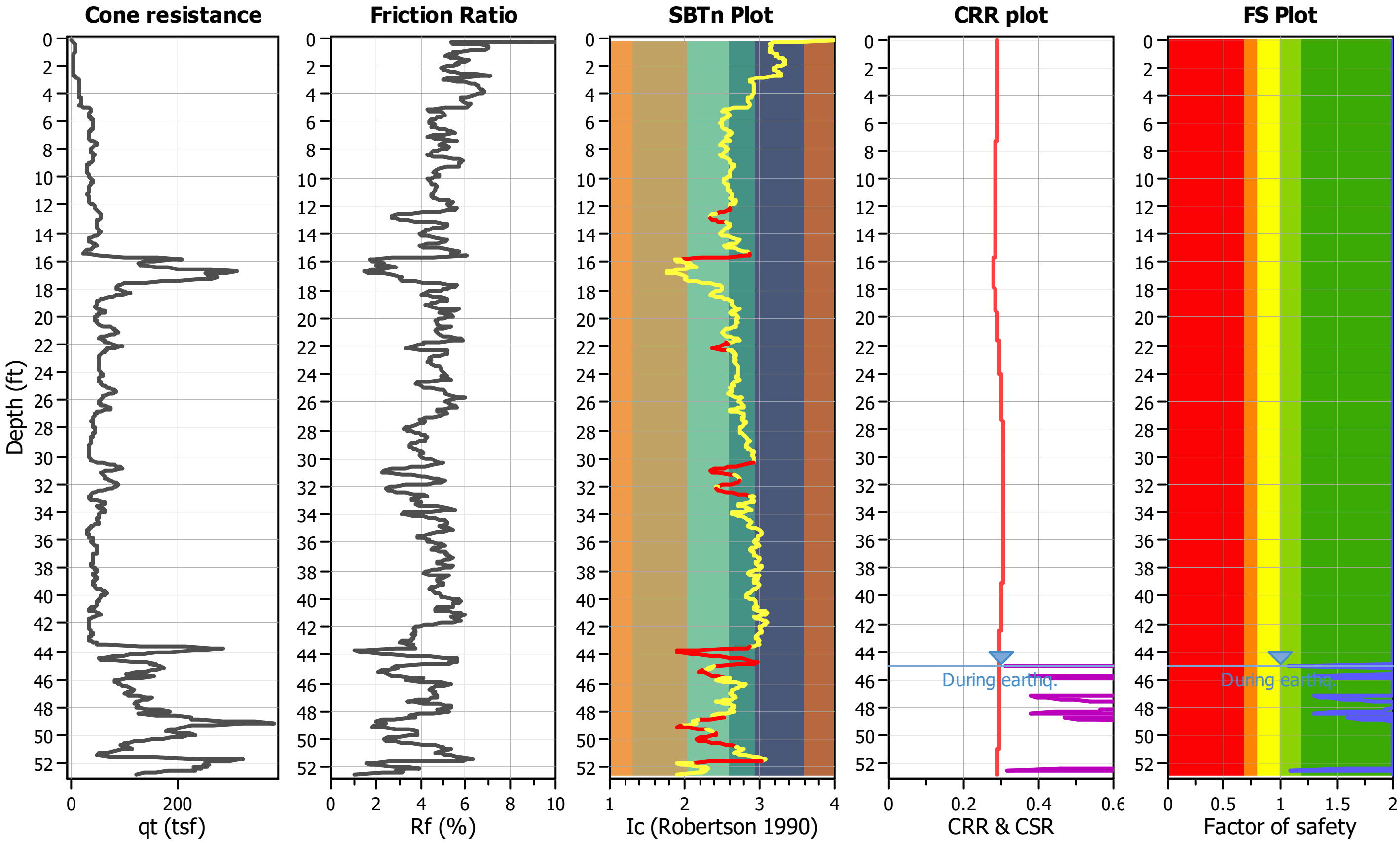
LIQUEFACTION ANALYSIS REPORT

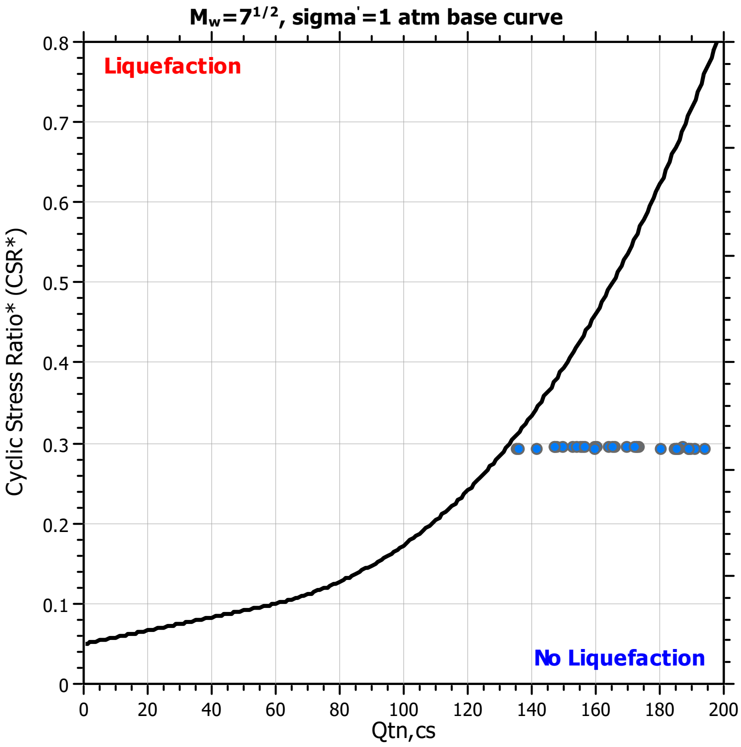
Project title : mmhs **Location : morningside drive, malibu, CA**

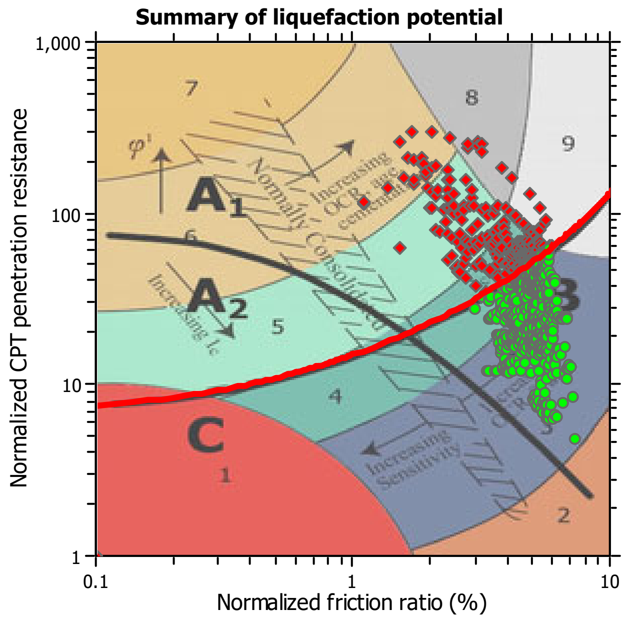
CPT file : CPT-5

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.):	45.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.76	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

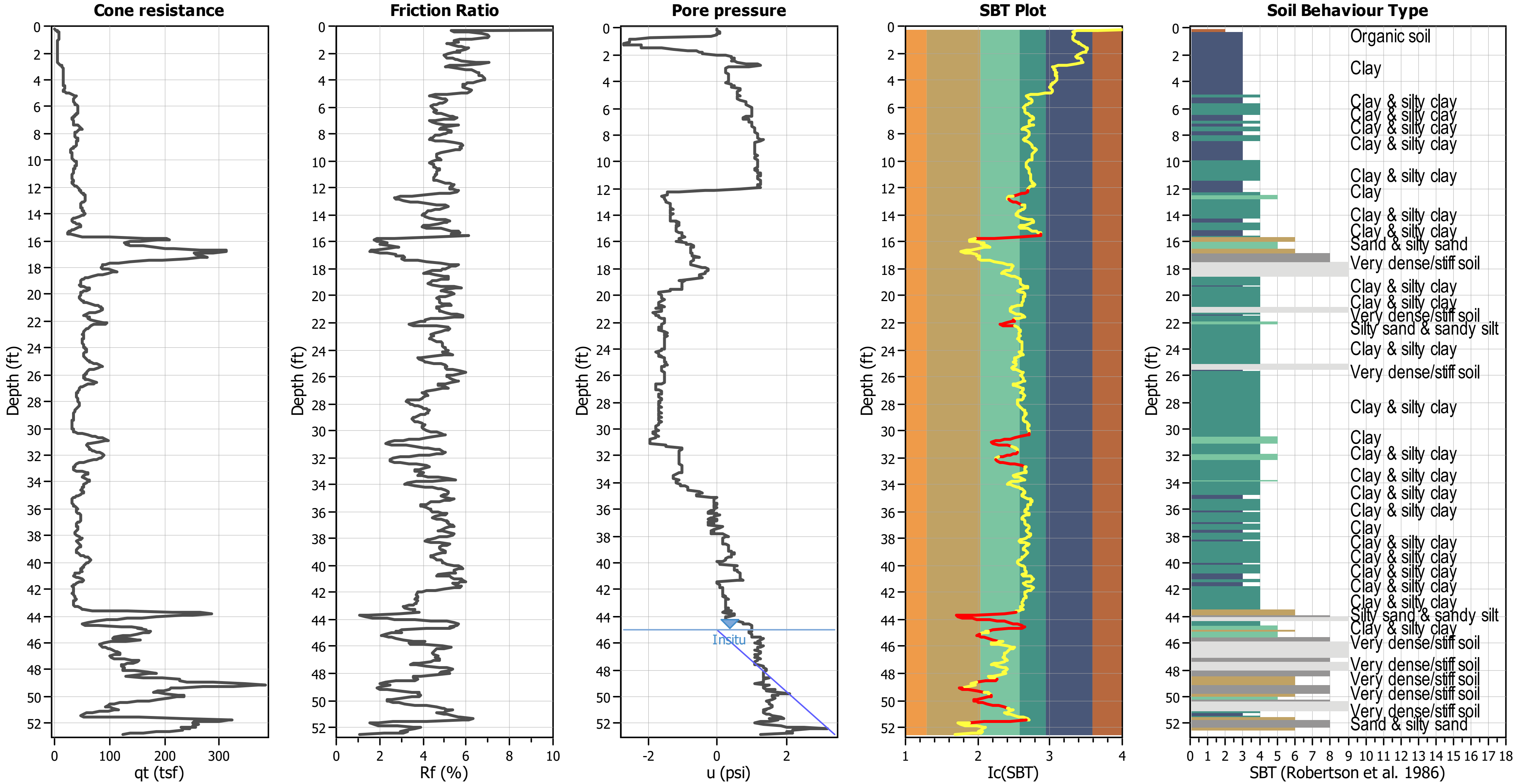






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



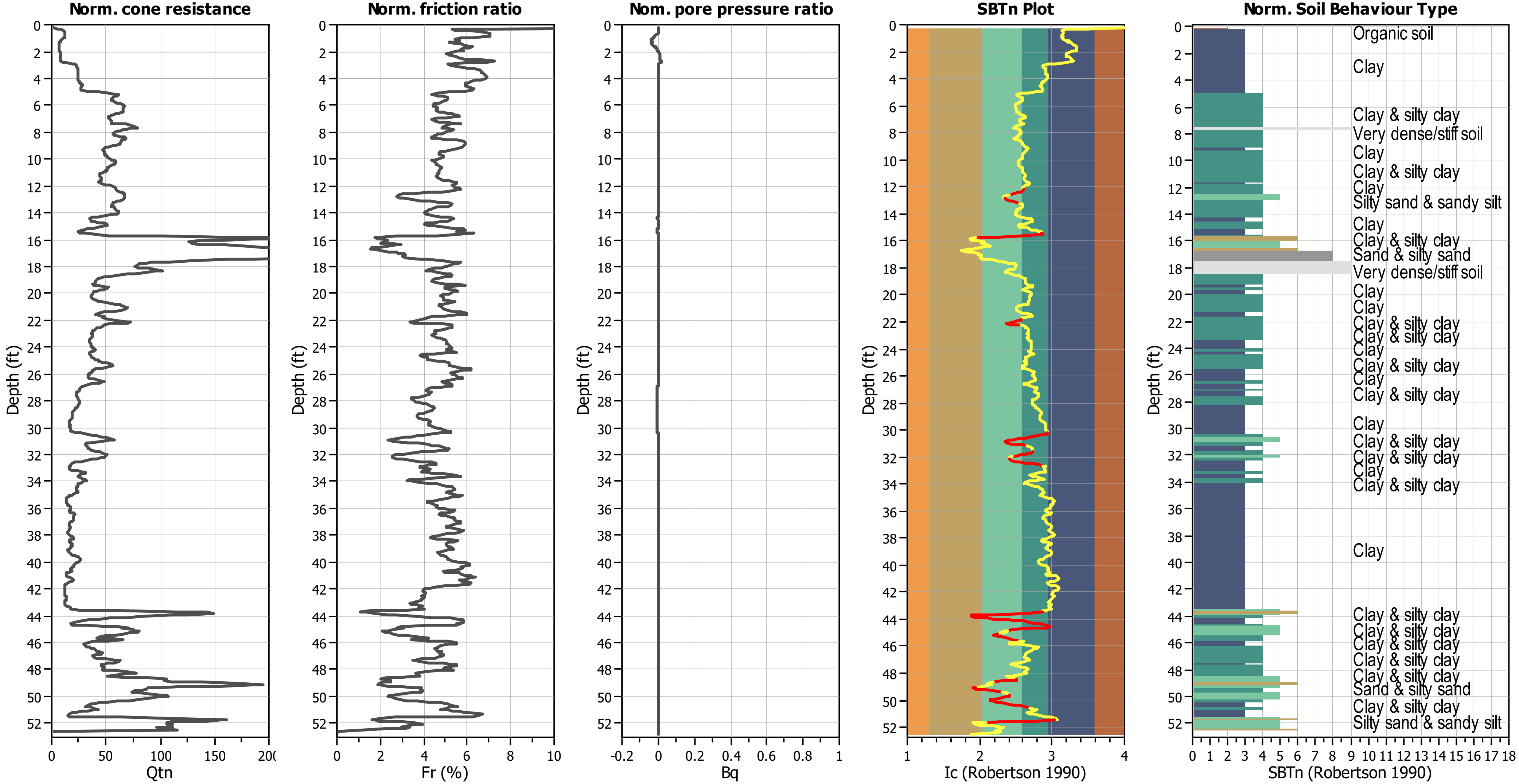
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	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)



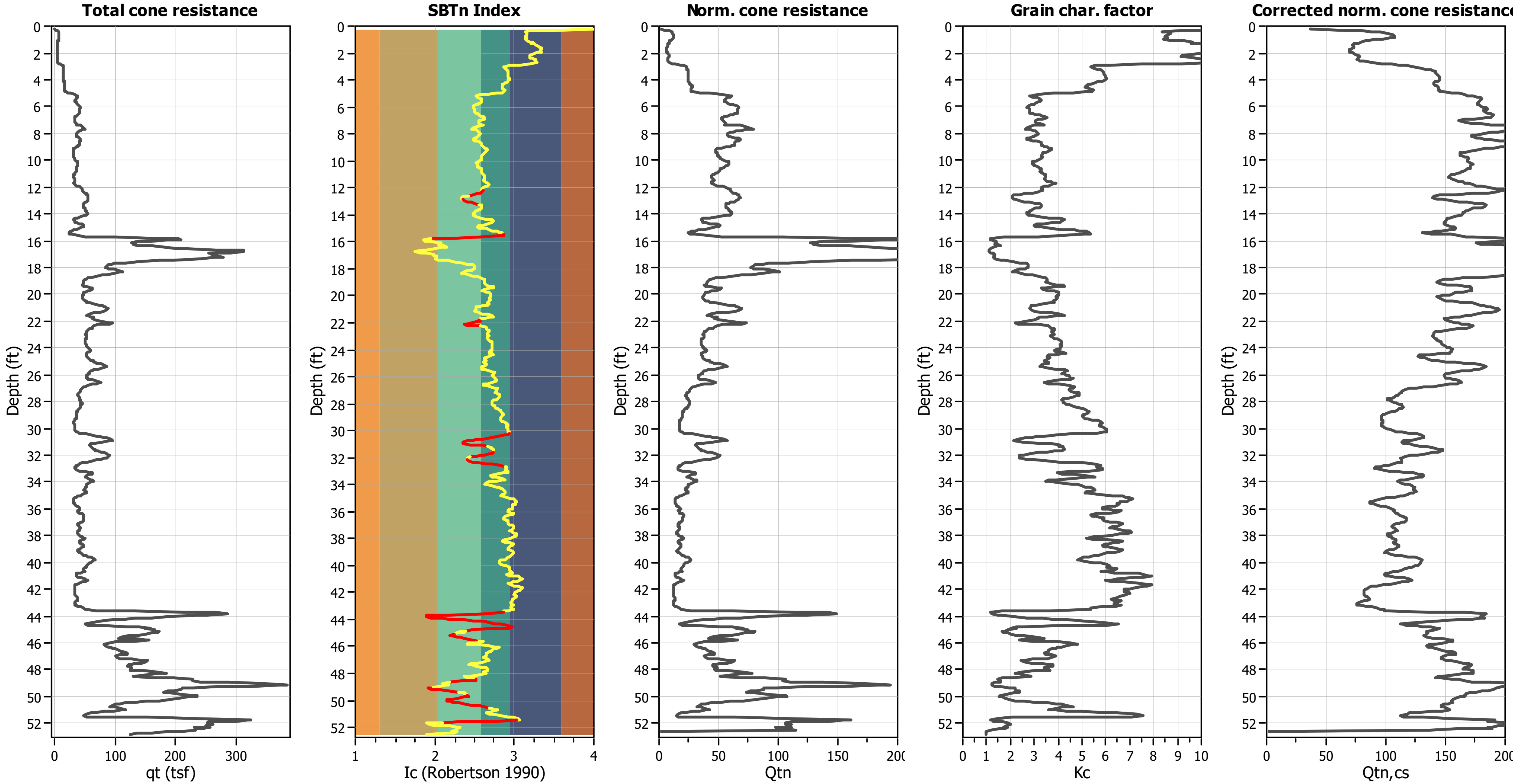
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	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

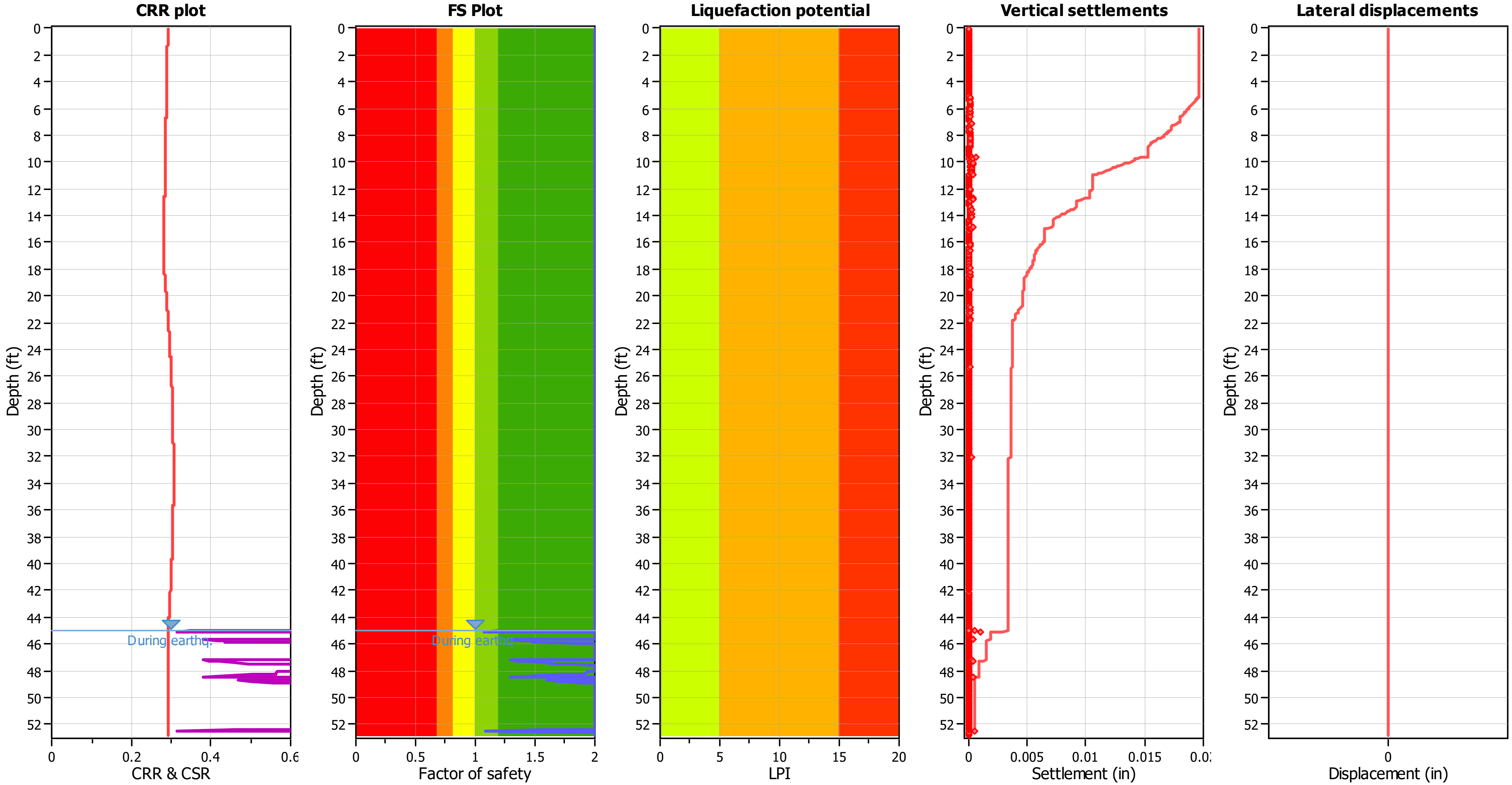
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	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.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	N/A

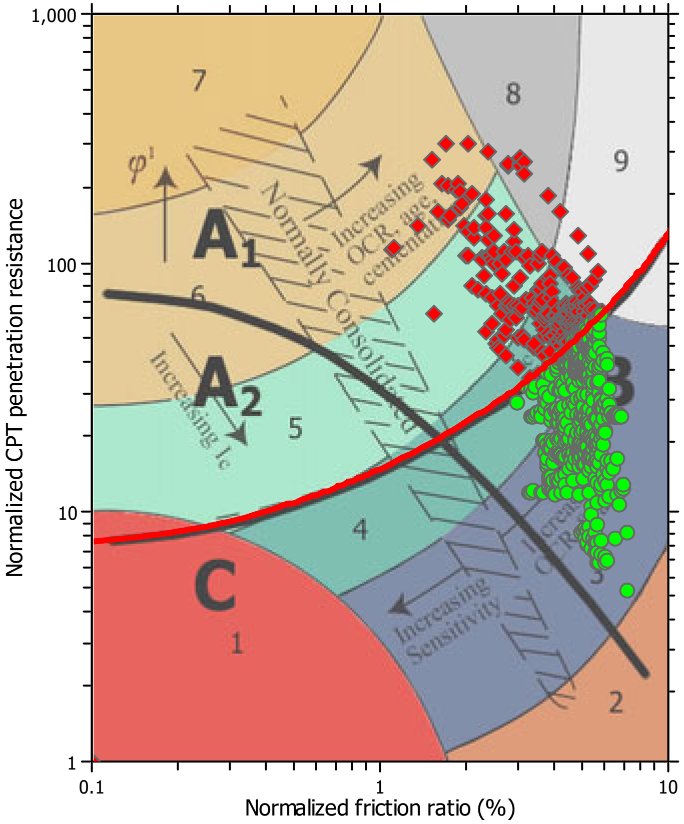
F.S. color scheme

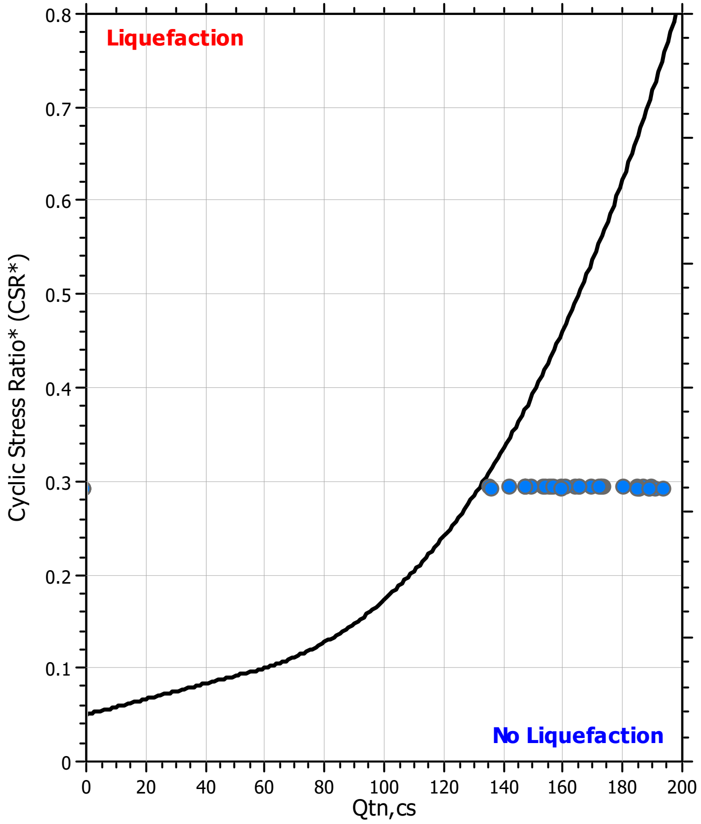
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

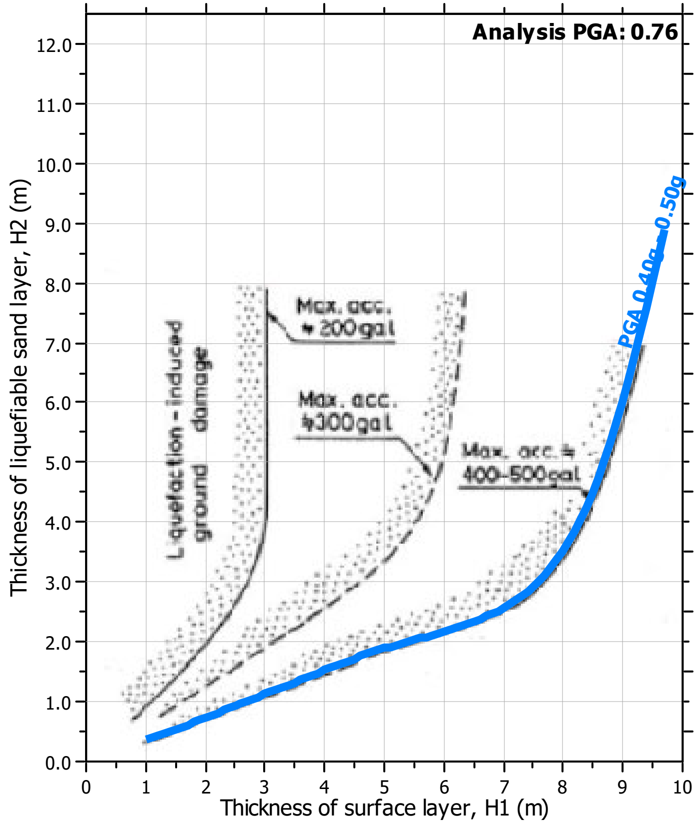
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



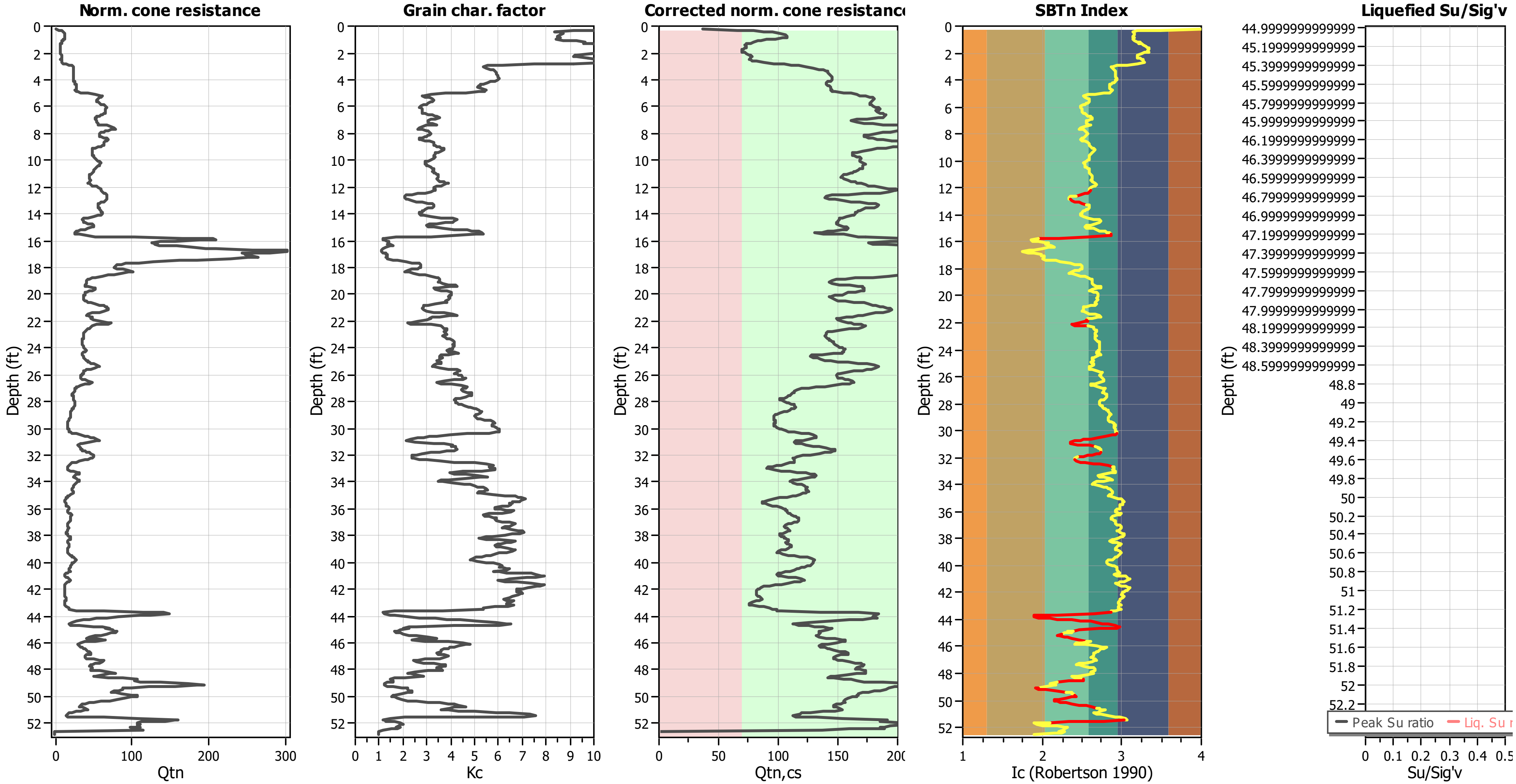




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	N/A

Leighton Consulting, Inc.
17781 Cowan
Irvine, CA 92614
http://www.leightongeo.com

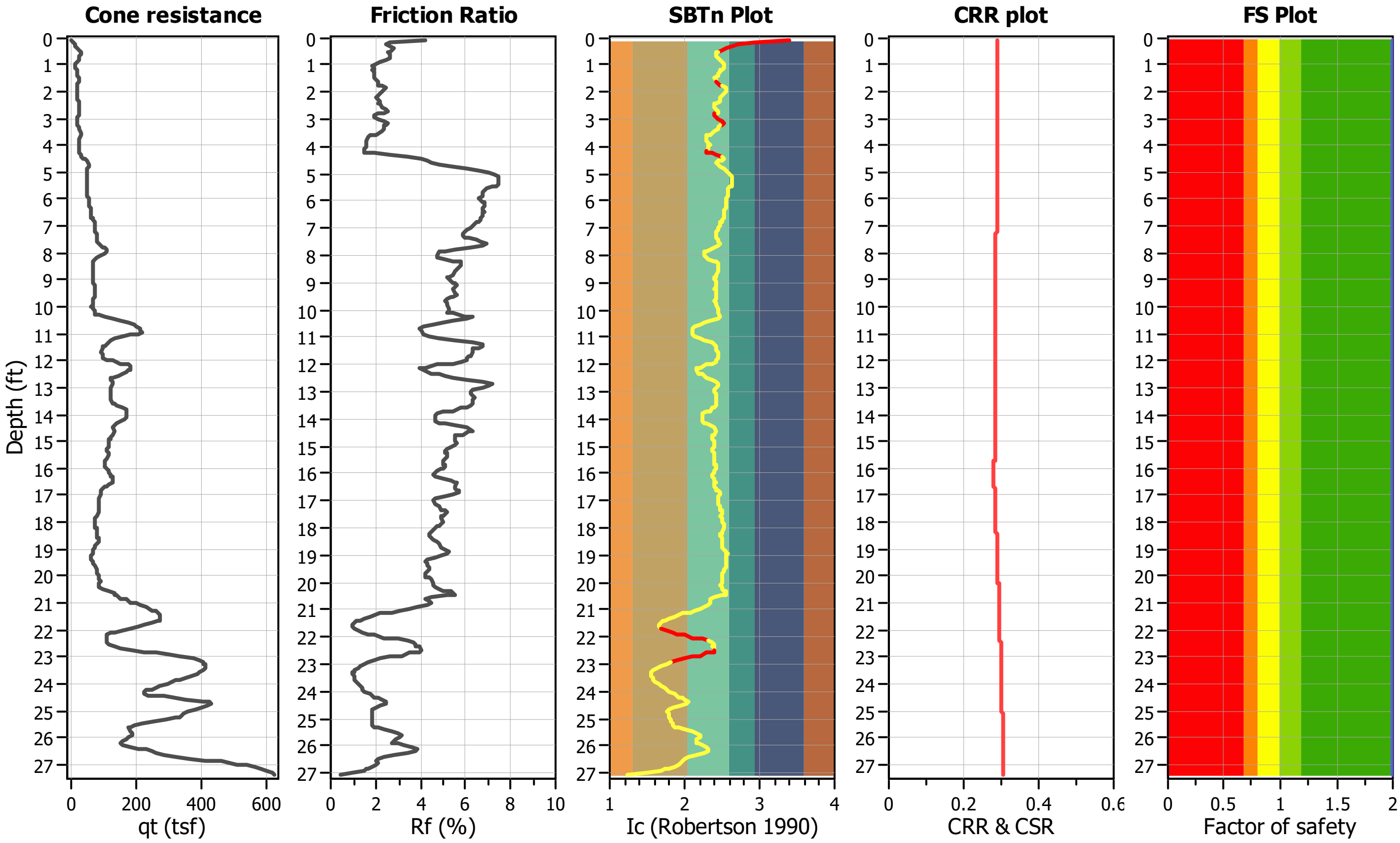
LIQUEFACTION ANALYSIS REPORT

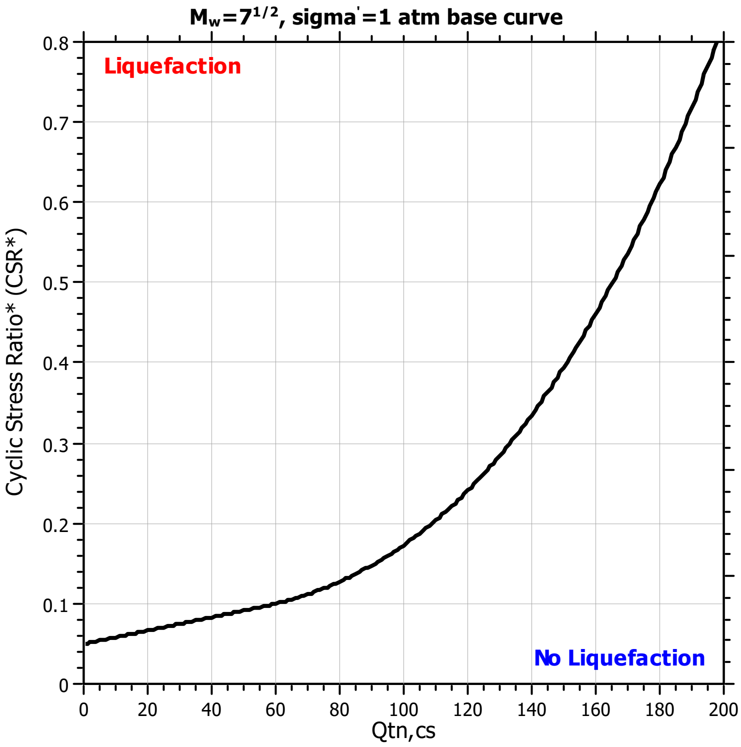
Project title : mmhs **Location : morningside drive, malibu, CA**

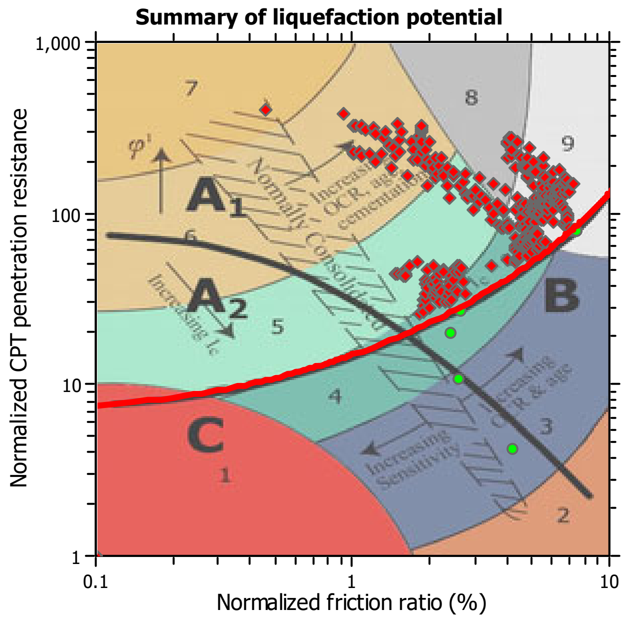
CPT file : CPT-6

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.):	45.00 ft	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	6.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.76	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

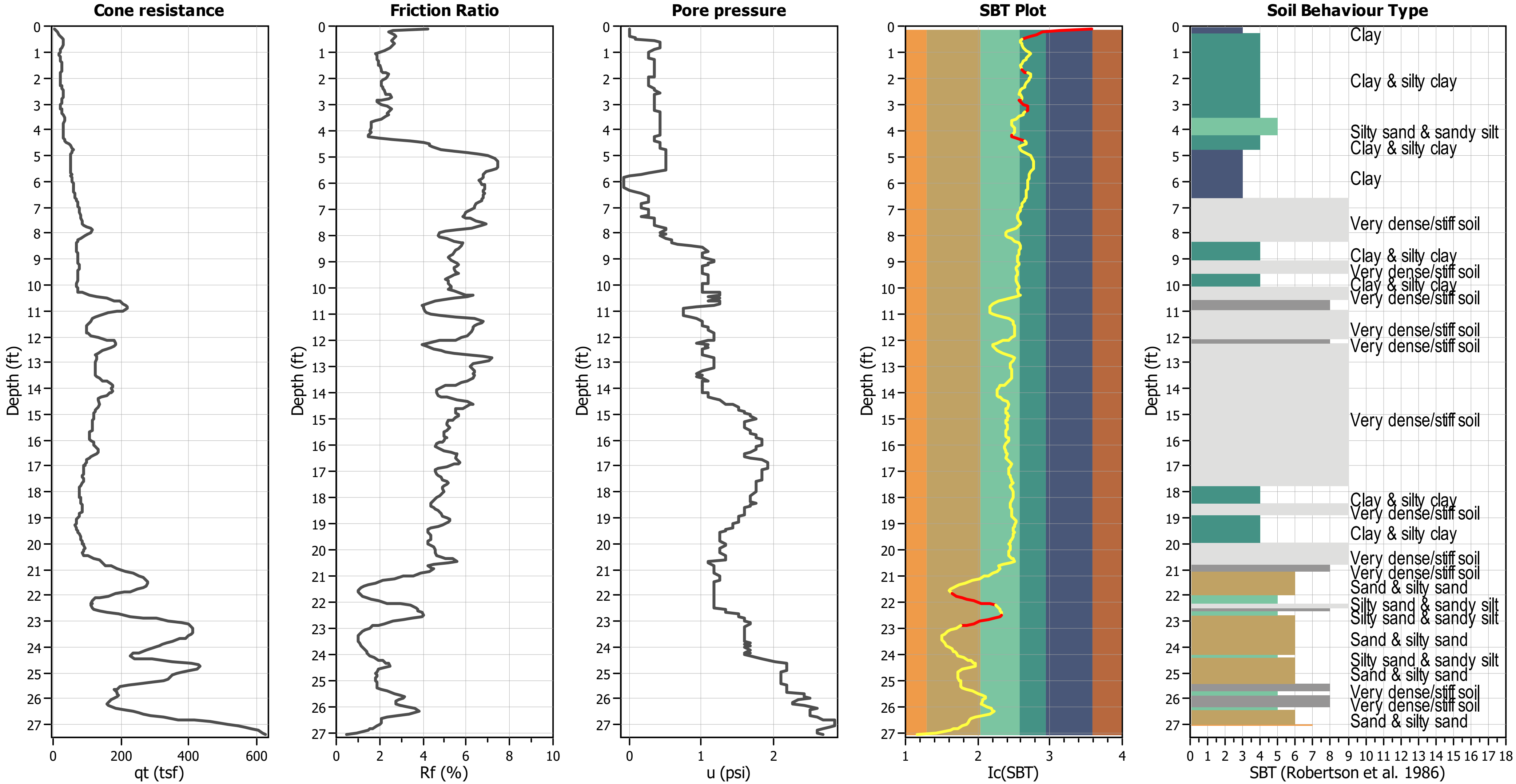






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



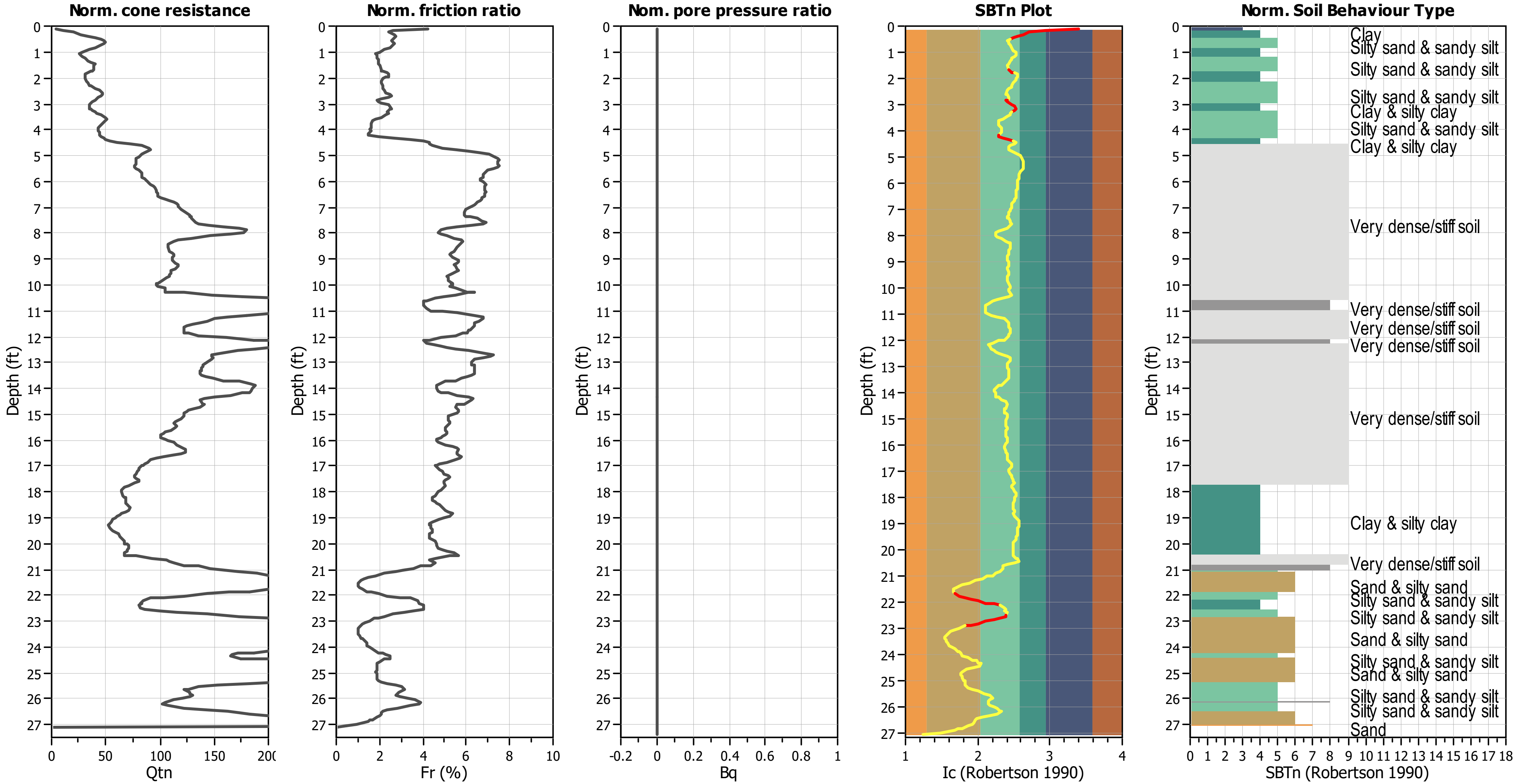
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	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)



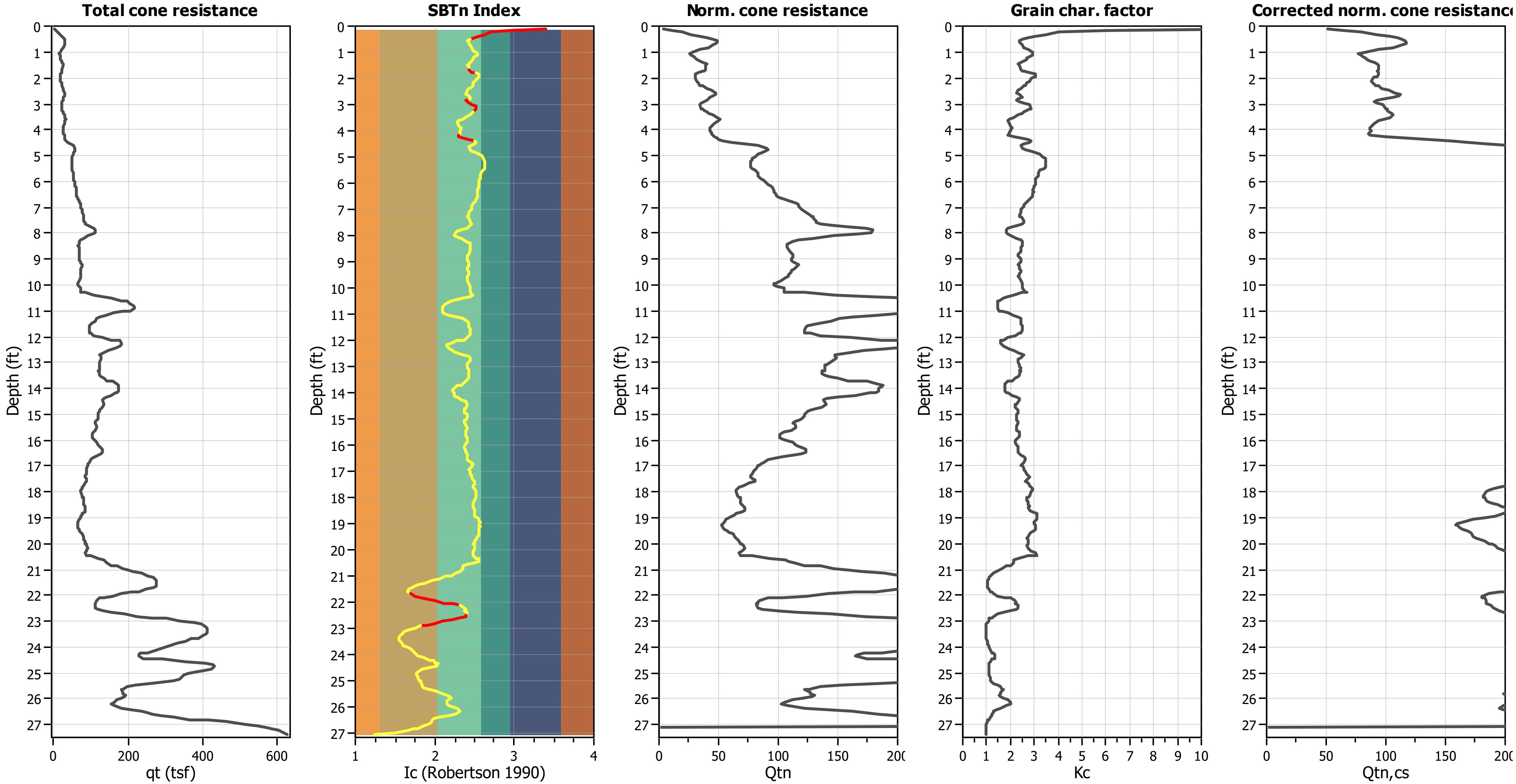
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	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

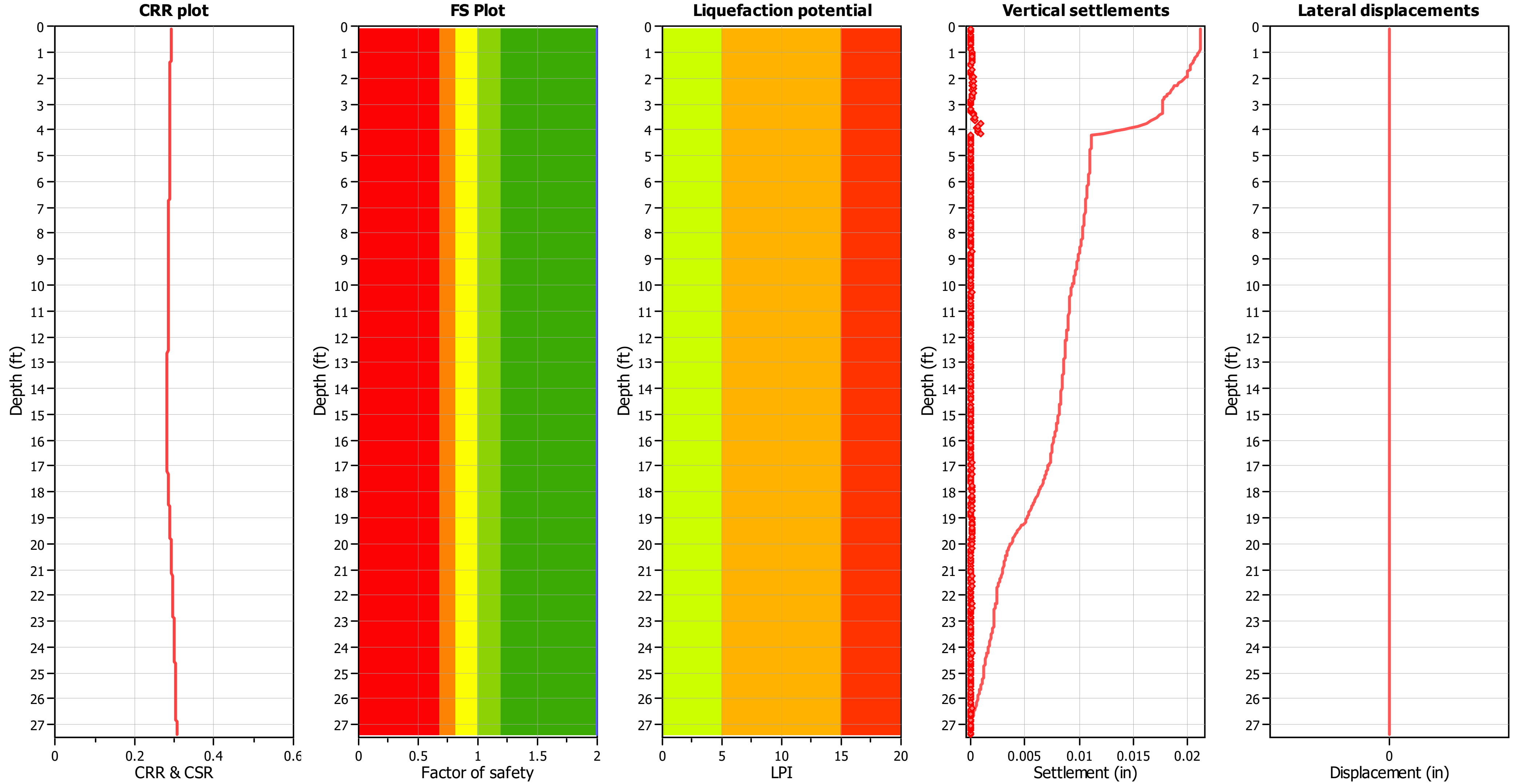
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	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.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	N/A

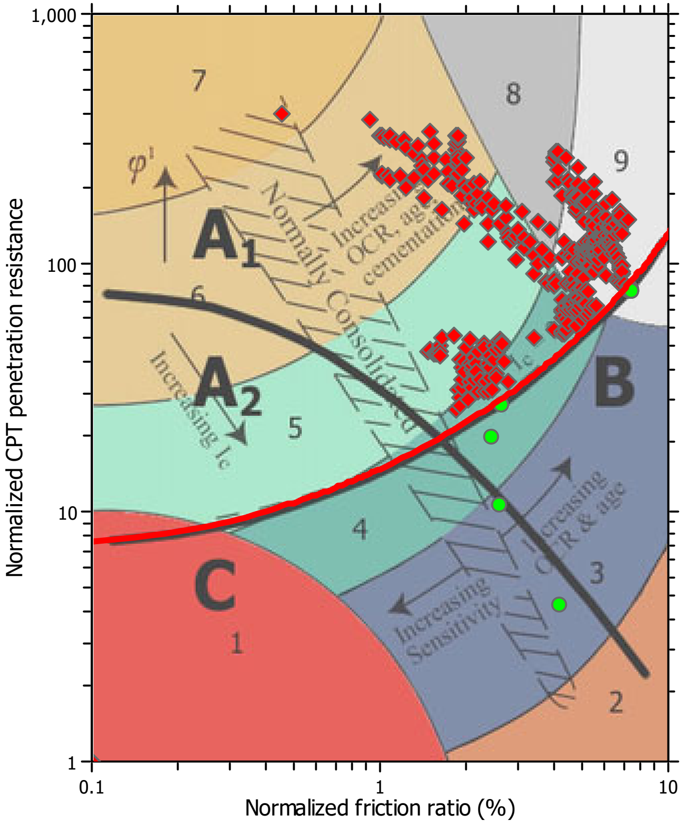
F.S. color scheme

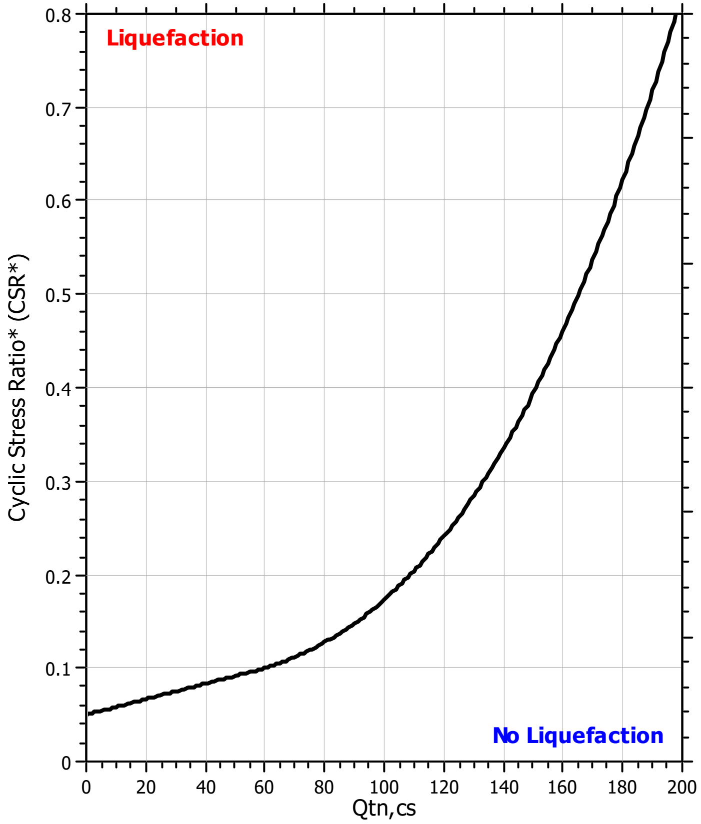
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

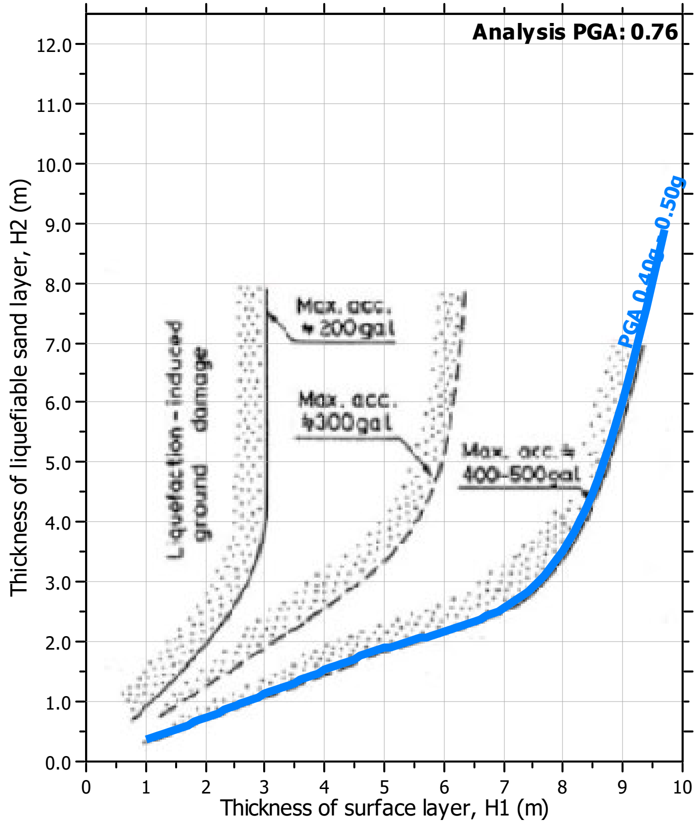
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



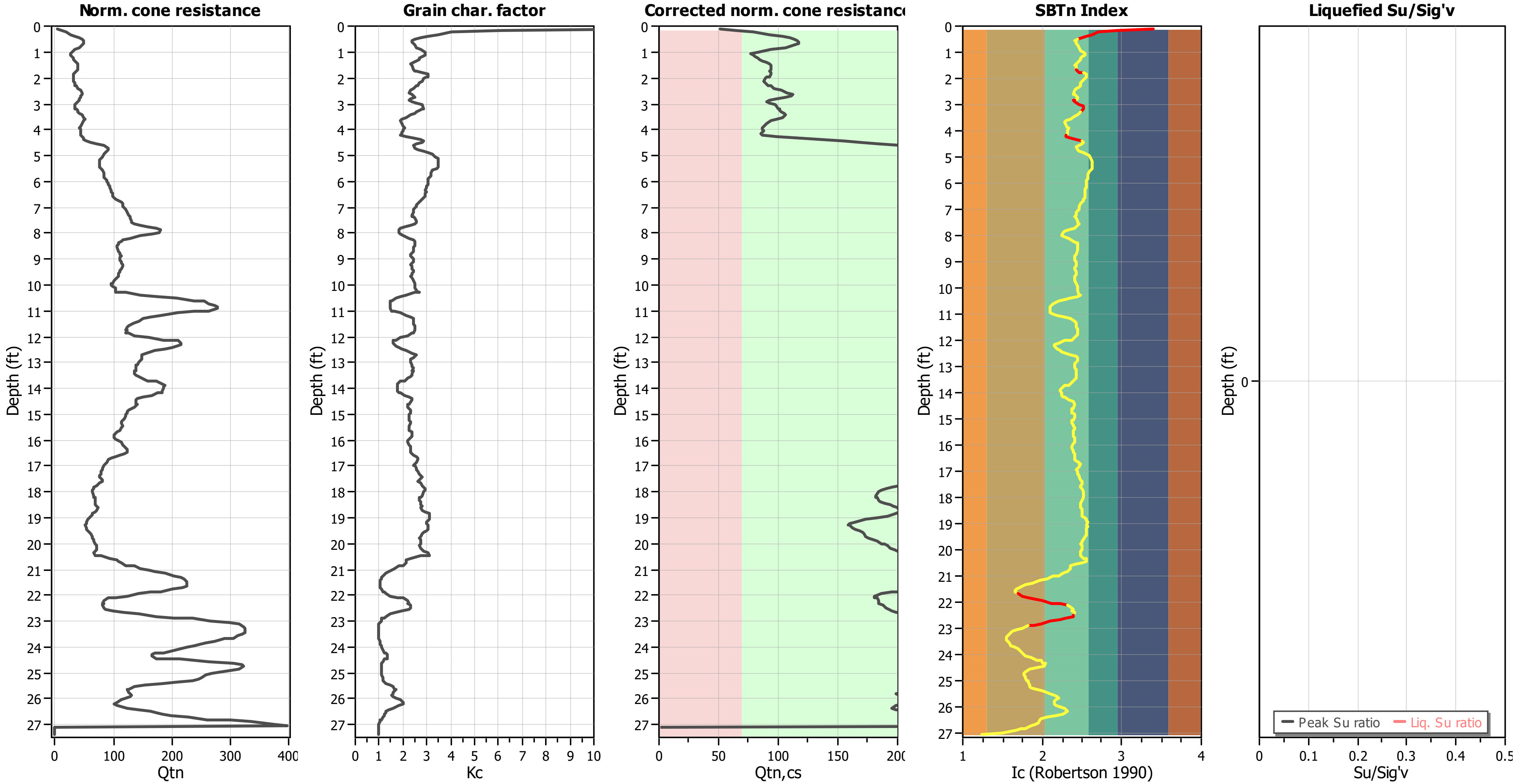




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	45.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.76	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	N/A

APPENDIX F

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

Leighton

APPENDIX F

LEIGHTON CONSULTING, INC.
EARTHWORK AND GRADING GUIDE SPECIFICATIONS

TABLE OF CONTENTS

F-1.0 GENERAL

F-1.1 Intent

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).

F-1.2 Role of Leighton Consulting, Inc.

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.

F-1.3 The Earthwork Contractor

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.

F-2.0 PREPARATION OF AREAS TO BE FILLED

F-2.1 Clearing and Grubbing

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.

F-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 F-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.

F-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

F-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.

F-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.

F-3.0 FILL MATERIAL

F-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.

F-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.

F-3.3 Import

If importing of fill material is required for grading, proposed import material shall meet the requirements of Section F-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.

F-4.0 FILL PLACEMENT AND COMPACTION

F-4.1 Fill Layers

Approved fill material shall be placed in areas prepared to receive fill, as described in Section F-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.

F-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.

F-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.

F-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.

F-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).

F-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.

F-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..

F-6.0 TRENCH BACKFILLS

F-6.1 Safety

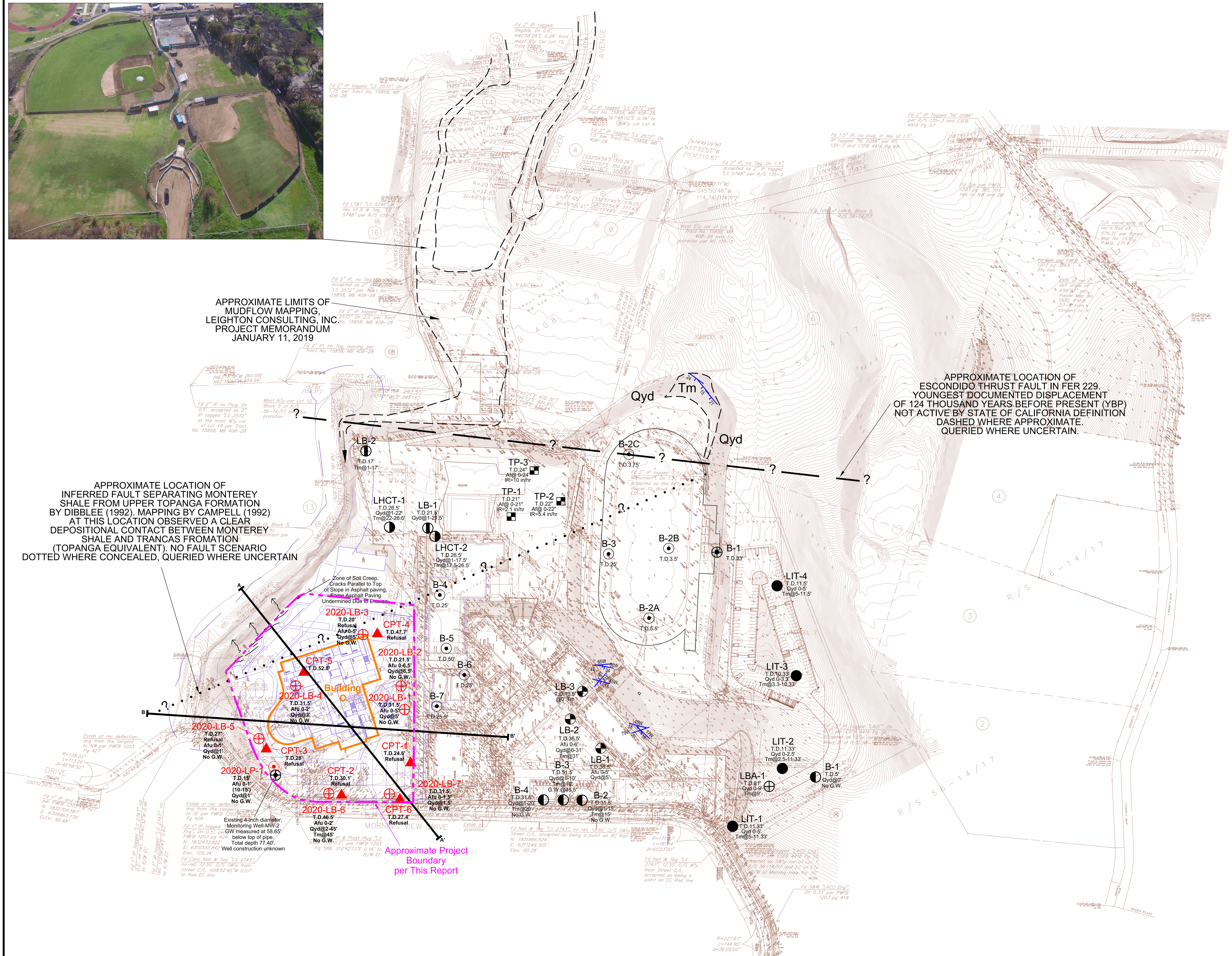
The Contractor shall follow all OSHA and Cal/OSHA requirements for safety of trench excavations. Work should be performed in accordance with Article 6 of the *California Construction Safety Orders*, 2009 Edition or more current (see also: http://www.dir.ca.gov/title8/sb4a6.html).

F-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..

F-6.3 **Lift Thickness**

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.



LEGEND

Afu	ARTIFICIAL FILL, UNDOCUMENTED	B-7 T.D.25.5'	HOLLOW STEM AUGER BORING (ASSOCIATED SOILS ENGINEERS, INC., 1999) SHOWING TOTAL DEPTH (T.D.) IN FEET	LB-3 T.D.13.5' @5' Tm	HOLLOW STEM AUGER BORING (LEIGHTON, 2016b) SHOWING TOTAL DEPTH (T.D.) AND DEPTH TO EARTH UNITS IN FEET BELOW GROUND SURFACE
Qyd	MARINE TERRACE DEPOSITS	B-1 T.D.33'	BUCKET AUGER BORING (ASSOCIATED SOILS ENGINEERS, INC., 1999) SHOWING TOTAL DEPTH (T.D.) IN FEET	LBA-1 T.D.81' Qyd 0-9' Tm@9'	BUCKET AUGER CORE BORING (LEIGHTON, 2016a) SHOWING TOTAL DEPTH (T.D.) AND DEPTH TO EARTH UNITS IN FEET BELOW GROUND SURFACE
Tm	TERTIARY MONTEREY SHALE: SILTSTONE/CLAYSTONE	LB-2 T.D.17' Tm@1-17'	HOLLOW STEM AUGER BORING (LEIGHTON, 2009b), [ADDENDUM 1] SHOWING TOTAL DEPTH (T.D.) AND DEPTH TO EARTH UNITS IN FEET BELOW GROUND SURFACE	B-4 T.D.31.5' Qyd@1-20' Tm@20' No G.W.	HOLLOW STEM AUGER BORING (LEIGHTON, 2009a) SHOWING TOTAL DEPTH (T.D.) AND DEPTH TO EARTH UNITS IN FEET BELOW GROUND SURFACE
65W 42	STRIKE AND DIP OF BEDDING (LEIGHTON, 2009)	LIT-4 T.D.11.5' Qdy@0-5' Tm@5-11.5'	HOLLOW STEM AUGER BORING (LEIGHTON, 2009c), [ADDENDUM 2] SHOWING TOTAL DEPTH (T.D.) AND DEPTH TO EARTH UNITS IN FEET BELOW GROUND SURFACE	G.W.	GROUNDWATER AT TIME OF DRILLING
2020-LB-7	APPROXIMATE LOCATION OF HOLLOW STEM AUGER BORINGS SHOWING TOAL DEPTH (T.D.) AND THICKNESS OF UNDOCUMENTED ARTIFICIAL FILL (Afu) OVER NATIVE MARINE TERRACE (Qyd) OVERLYING BEDROCK OF THE MONTEREY FORMATION (Tm)	LHCT-2 T.D.26.5' Qdy@1-17.5' Tm@17.5-26.5'	HOLLOW STEM AUGER BORING (LEIGHTON, 2009d), [ADDENDUM 3] SHOWING TOTAL DEPTH (T.D.) AND DEPTH TO EARTH UNITS IN FEET BELOW GROUND SURFACE	— — — —	GEOLOGIC CONTACT, DASHED WHERE APPROXIMATE
CPT-6	APPROXIMATE LOCATION OF CONE PENETROMETER TEST (CPT) LOCATIONS SHOWING TOTAL DEPTH (T.D.) IN FEET BELOW GROUND SURFACE. REFUSAL NOTED WHERE APPLICABLE.	TP-3 T.D.24" Af@0-24" IR>10 in/hr	TEST PIT LOCATION (LEIGHTON, 2010) SHOWING TOTAL DEPTH (T.D.) AND DEPTH TO EARTH UNITS IN FEET BELOW GROUND SURFACE, AND INFILLTRATION RATE (IR) IN INCHES PER HOUR (in./hr.)		INDICATES DIRECTION OF SOIL CREEP DOWN SLOPE
LP-1	APPROXIMATE LOCATION OF INFILTRATION TEST WELL SHOWING TOTAL DEPTH (T.D.) IN FEET BELOW GROUND SURFACE TEST DEPTH NOTED IN PARANTHESES			B B'	GEOLOGIC CROSS SECTION. SEE PLATE 2

PLATE 1

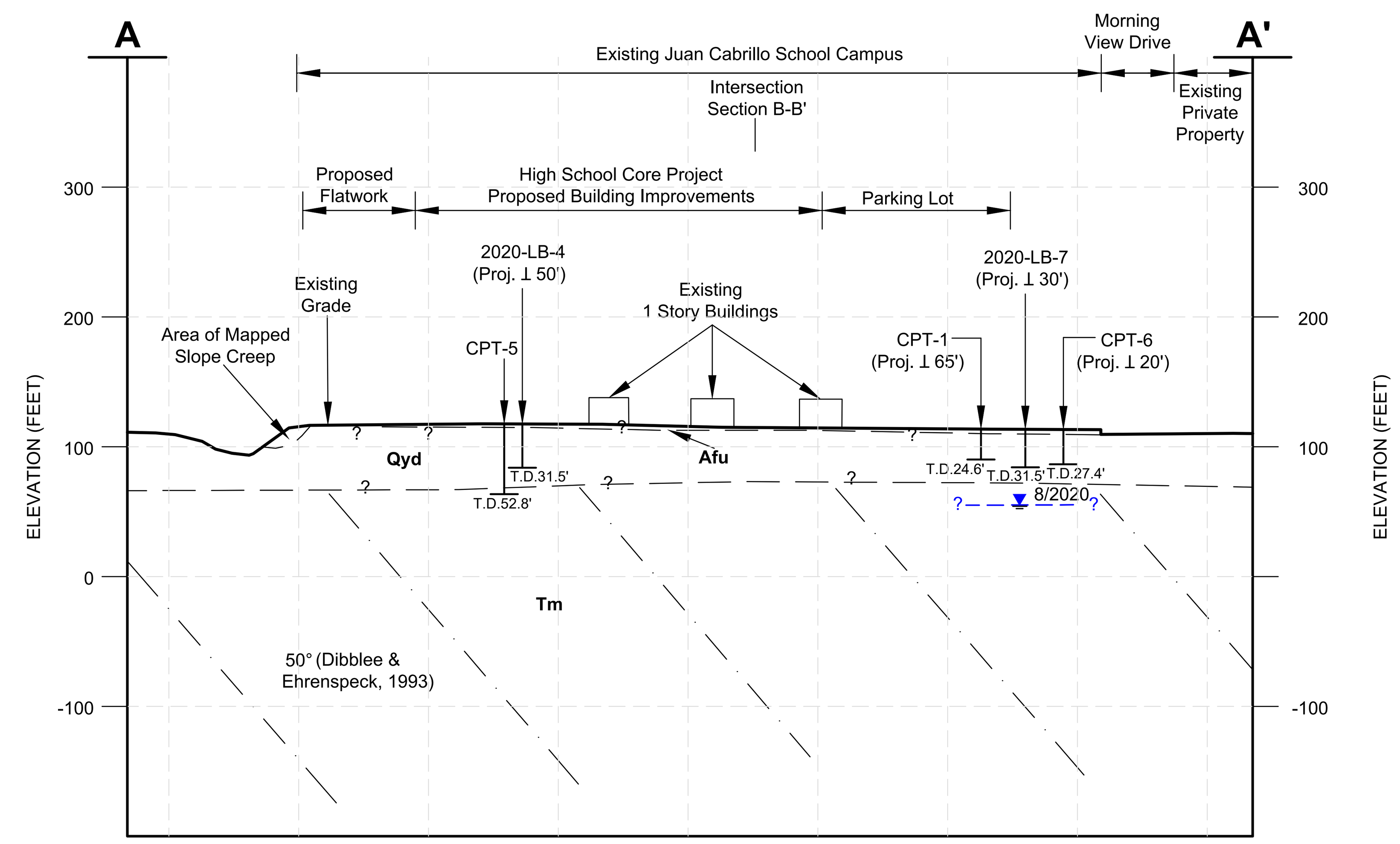
Leighton

GEOTECHNICAL MAP

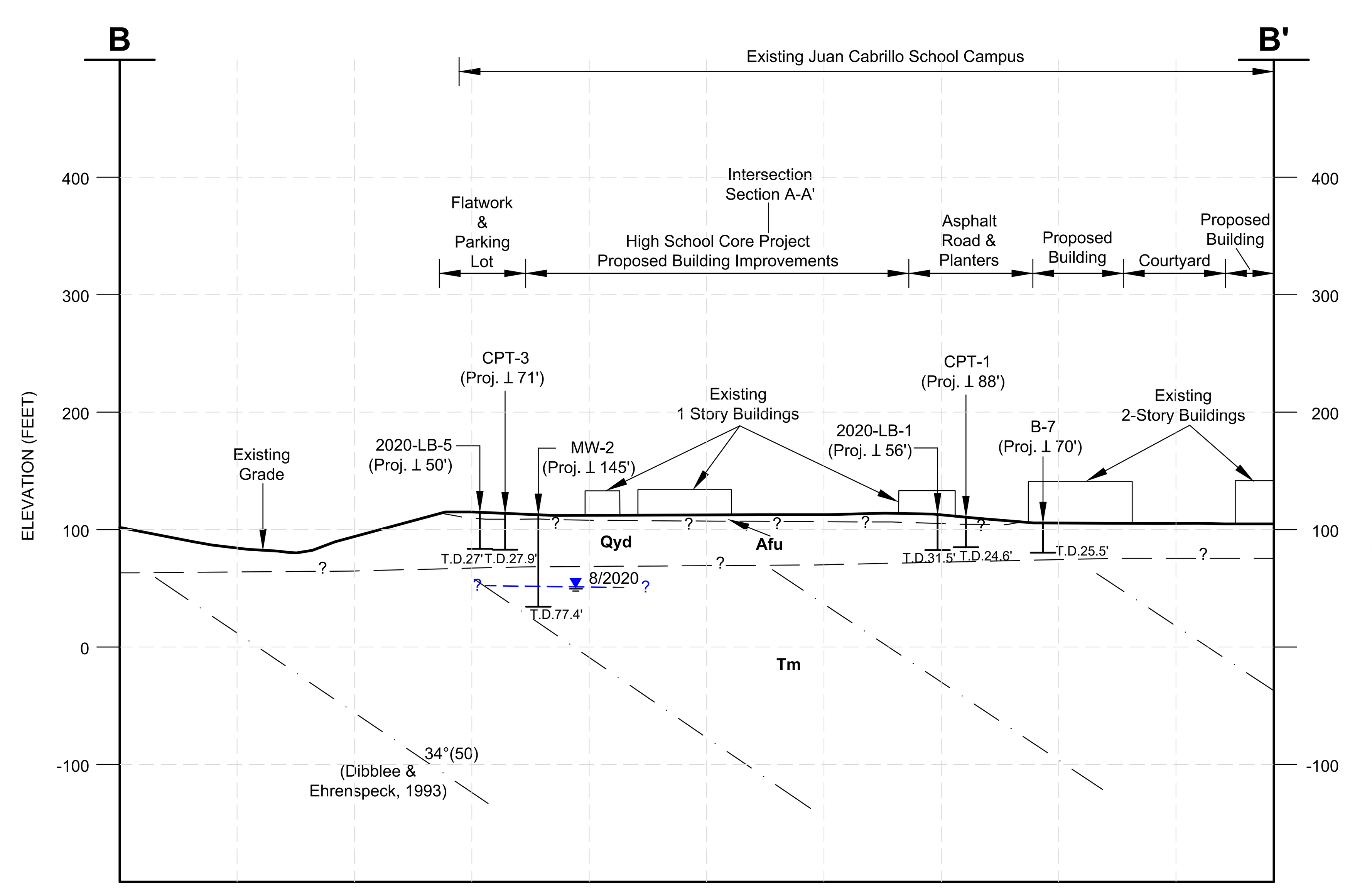
Malibu Middle/High School Building Proposed
Phase I New High School Core Project
30215 Morning View Drive, Malibu, California

Proj: 11382.016	Eng/Geol: CCK/JR
Scale: 1"=100'	Date: January 2021

Drafted By: BQT | Checked By: BQT | V:\DRAFTING\11382\016\CAD\2020-06-10\11382-016_P01_GM_2021-01-15.DWG (01-15-21 2:36:47PM) Plotted by: btran



S9°E



S57°E

PLATE 2	**GEOTECHNICAL CROSS SECTIONS A-A' AND B-B'** Malibu Middle/High School Building Proposed Phase I New High School Core Project 30215 Morning View Drive, Malibu, California
Leighton	Proj: 11382.016 / Eng/Geol: CCK/JR
	Scale: 1"=100' / Date: November 2020

Drafted By: BQT Checked By: BQT