Appendix H Geotechnical Report

Appendices

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GEOTECHNICAL EXPLORATION REPORT
NEW CLASSROOM BUILDING
MCKINLEY ELEMENTARY SCHOOL
2401 SANTA MONICA BOULEVARD
SANTA MONICA, LOS ANGELES COUNTY
CALIFORNIA

Prepared For

SANTA MONICA-MALIBU
UNIFIED SCHOOL DISTRICT
2828 FOURTH STREET
SANTA MONICA, CALIFORNIA 90405-4308

Prepared By

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Project No. 11428.036

November 19, 2021

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Santa Monica-Malibu Unified School District
2828 Fourth Street
Santa Monica, California 90405-4308

Attention: Mr. Kevin Klaus

Subject: Geotechnical Exploration Report
New Classroom
McKinley Elementary School
2401 Santa Monica Boulevard
Santa Monica, Los Angeles County, California

Per our April 4, 2021 proposal, authorized on October 5, 2021; Leighton Consulting, Inc. (Leighton) is pleased to present this geotechnical exploration report for the subject project. This report is intended to meet requirements of Section 1803A.2 of the 2019 California Building Code (CBC) and the CGS's Note 48 checklist for review of engineering geology and seismology reports for California public schools.

This site is **not** located within a currently designated Alquist-Priolo Special Studies Zone for surface fault rupture. This site is **not** located within a currently designated liquefaction hazard zone. As is the case for most of Southern California, strong ground shaking has and will occur at this site.

Specific recommendations for site grading, foundations, and other geotechnical aspects of the project are presented in this report.

We appreciate this opportunity to be of service. If you have any questions regarding this report or if we can be of further service, please call us at **(866) *LEIGHTON***, directly at the phone extensions or e-mail addresses listed below:

Respectfully submitted,

LEIGHTON CONSULTING, INC.

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1.0 INTRODUCTION

1.1 Site Description and Proposed Development

McKinley Elementary is an active K through 5th grade school located at 2401 Santa Monica Boulevard in the City of Santa Monica, situated within a densely developed residential and commercial neighborhood. The school campus location (latitude 34.0324°, longitude -118.4768°) and immediate vicinity are shown on Figure 1, *Site Location Map*.

The campus is a rectangular parcel of land developed with one to two story classroom buildings, a playfield, asphalt concrete (AC) blacktop, and a parking lot fronting Chelsea Avenue. Overall, the campus is bounded on the northwest by Arizona Avenue, the northeast by Chelsea Avenue, the southeast by Santa Monica Boulevard, and the southwest by 23rd Court. According to the United States Geological Survey (USGS) 7.5-Minute Beverly Hills Quadrangle (USGS, 1981), the site surface is relatively flat with an approximate elevation (El.) of ±155 to El.170 feet mean sea level (msl).

Our understanding of the proposed development is based on review of your *Request for Qualifications/Proposal for Geotechnical Services, SMMUSD Elementary and Middle School Assessment Projects* issued on July 15, 2021; and the associated *Existing Site Plan Sheet A1.01* prepared by Johnson Favaro, dated January 20, 2021. As currently conceived, the project consists of the construction of a new 2-story, 12,500-square-foot classroom building with outdoor classrooms, proposed in the northeastern region of the overall school campus site. No subterranean levels are currently planned. The footprint of the proposed new classroom building is shown on Figure 2, *Exploration Location Map.*

1.2 Purpose and Scope of Exploration

The purpose of our geotechnical exploration was to evaluate the soil and groundwater conditions at the new classroom site through review of available data and subsurface explorations in order to provide geotechnical recommendations to aid in design and construction for the project as currently proposed (see Section 1.1). The scope of this geotechnical exploration included the following tasks:

- ***Background Review*** – A background review was performed of readily available and relevant geotechnical, civil, and geological documents pertinent to the project site. References reviewed in preparation of this report are listed in Section 8.0.

- ***Field Exploration*** – Our field exploration was performed October 6, 2020 and consisted of three (3) hollow-stem auger borings (designated LB-1 through LB-3) drilled to approximate depths ranging from of 31½ feet to 51½ feet below ground surface (bgs). In addition, six (6) cone penetrometer test (CPT) soundings (designated CPT-1 through CPT-5 and CPT-3A) were each advanced to an approximate depth of 50 feet bgs.

  Prior to the field exploration, the borings and CPT's were marked and Underground Service Alert (USA) was notified for utility clearance. In addition, a private utility locator was utilized to locate any unknown or unmarked utilities in the areas of the proposed boring locations prior to drilling.

  During drilling of the hollow-stem auger borings (LB-1 through LB-3), bulk and relatively undisturbed drive samples were obtained from the borings for geotechnical laboratory testing. Relatively undisturbed samples were collected from the borings using a Modified California Ring sampler conducted in accordance with ASTM Test Method D3550. Standard Penetration Tests (SPT) were also performed within the hollow-stem auger borings in accordance with ASTM Test Method D1586. The samplers were driven for a total penetration of 18 inches, unless practical refusal was encountered, using a 140-pound automatic hammer falling freely for 30 inches. The number of blows per 6 inches of penetration was recorded on the boring logs.

  The borings were logged in the field by a member of our technical staff. Each soil sample collected was reviewed and described in accordance with the Unified Soil Classification System (USCS). The samples were sealed and packaged for transportation to our laboratory. After completion of drilling, all of the borings were backfilled with excess soils generated during the exploration. The boring logs are presented in Appendix A, *Field Exploration Logs*. The approximate locations of the explorations are shown on Figure 2, *Exploration Location Map*.

- ***Laboratory Testing*** – Geotechnical laboratory tests were conducted on selected bulk and undisturbed soil samples obtained from our borings. This laboratory testing program was designed to evaluate geotechnical (physical) characteristics of site soil. A description of geotechnical laboratory test-procedures and results are presented in Appendix B, *Laboratory Test Results*. The following laboratory tests were performed:

— In-situ Moisture Content and Dry Density (ASTM D2216 and ASTM D2937);

— Expansion Index (ASTM D4829);

— Modified Proctor Compaction Test (ASTM D1557);

— Direct Shear (ASTM D 3080);

— R Value (DOT CA Test 301);

— Consolidation (ASTM D2435); and

— Corrosivity (Soluble Sulfate ASTM C1580, Soluble Chloride ASTM C1411-09, pH ASTM D4972, and Resistivity ASTM G187-12a).

The in-situ moisture and density of soil samples at depths are shown on the borings logs included in Appendix A. The results of the remaining laboratory tests are presented in Appendix B.

- ***Engineering Analysis*** – Data obtained from field explorations and geotechnical laboratory testing was evaluated and analyzed to develop geotechnical conclusions and provide recommendations in accordance with the 2019 California Building Code and the California Geological Survey's (CGS) Note 48 (November 2019 version). Geologic cross sections prepared for this campus presented on Plate 1, *Geotechnical Cross Sections AA' and BB'* (in pocket).

- ***Shear Wave Velocity*** - Shear wave velocities were profiled at 5-foot intervals to a depth of 50 feet bgs in CPT-3 (Figure 2) to estimate average S-wave velocities of the upper 100 feet (Vs_{100}) and 30 meters (Vs_{30}). The average sear wave velocity recorded onsite is approximately 1238 feet per second (ft/sec). The shear wave velocity report is included in Appendix A. Based on collected velocities and in accordance with the 2019 California Building Code, the soils at this site classified as Seismic Site Class D.

- ***Report Preparation*** - Results of our geologic hazards review and geotechnical exploration have been summarized in this report, presenting our findings, conclusions and geotechnical design recommendations for design and construction of the new McKinley Elementary School Classroom as currently proposed. *Once building loads are known and bearing pressure diagrams prepared they should be provided to Leighton Consulting, Inc. (Leighton) for* *review to ensure our recommendations remain appropriate for the project as currently proposed*.

It should be noted that the recommendations in this report are subject to the limitations presented in Section 7.0 of the report.

2.0 GEOTECHNICAL FINDINGS

2.1 Geologic Setting

The site is in the Santa Monica Plain, an uplifted and inclined alluvial surface within the southwestern block of the Los Angeles Basin (Hoots, 1931; Poland and Piper, 1956). The Los Angeles Basin (Basin), a structural trough, is a northwest-trending, alluviated lowland plain approximately 50 miles long and 20 miles wide. Mountains and hills that generally expose Late Cretaceous to Late Pleistocene-age sedimentary and igneous rocks bound the Basin along the north, northeast, east and southeast (Yerkes, 1965). The Basin is part of the Peninsular Ranges geomorphic province of California characterized by sub parallel blocks sliced longitudinally by young, steeply dipping northwest-trending fault zones. The Basin, located at the northerly terminus of the Peninsular Ranges, is the site of active sedimentation and the strata are interpreted to be as much as 31,000 feet thick in the center of the synclinal trough of the Central Block of the Los Angeles Basin.

The Santa Monica Plain formed during the Pleistocene epoch by continental aggradation and has since been uplifted and heavily incised by both current and former drainage patterns (Hoots, 1931). As shown on Figure 3, *Regional Geology Map,* the area of the Santa Monica Plain where the McKinley Elementary School campus is located is mapped as being underlain by Quaternary old alluvial fan deposits and infilled with Holocene age alluvial deposits.

2.2 Local Geologic Units and Subsurface Conditions

Presented below are brief descriptions of the geologic units encountered in the exploratory borings completed at the site by Leighton. Detailed descriptions of the geologic units encountered are presented on the boring logs in Appendix A. Geotechnical conditions described on the logs represent the conditions at the actual exploratory excavation locations. Other variations may occur beyond and/or between the excavations. Lines of demarcation between the geologic units and the various earth materials on the logs represent approximated boundaries, and (unless otherwise noted) actual transitions may be gradual. The locations of the subsurface explorations are shown on Figure 2, *Exploration Location Map* and a subsurface profile based on data obtained and interpreted from the borings and CPTs is shown on Plate *1, Geotechnical Cross-Section A-A' and B-B'.*

Artificial fill (Afu) materials were encountered underlying existing pavements within the exploratory borings and interpreted in the CPTs. Local geology was interpreted from published regional geologic maps of the area (Yerkes and Campbell, 2005; Dibblee, 1991). Figure 3, *Regional Geology Map*, illustrates the approximate distribution of geologic units at the site. Native geologic units underlying the artificial fill materials consist of Quaternary young alluvial valley deposits age (map symbol: Qya) overlying Quaternary old alluvial fan deposits (map symbol: Qof), correlative to Dibblee's (DF-31, 1991) Quaternary Old Marine and in part non marine sediments derived from the Santa Monica Mountains. Existing site improvement likely removed the thin veneer of younger alluvium at the site as these deposits were not encountered within our borings.

Undocumented Artificial Fill: (Map Symbol: Afu): Artificial fill materials were encountered to a depth of approximately 4 feet. Fill, as encountered, is characterized as medium reddish brown sandy lean clay and clay. No documentation or records related to fill placement was available at the time of this report preparation. Therefore, for purposes of this report, all fill encountered onsite and anticipated in future explorations is considered undocumented and unsuitable for support of new improvements in its current condition.

Quaternary Old Alluvial Fan Deposits (Map Symbol: Qof): The Pleistocene alluvial fan deposits encountered directly beneath the artificial fill materials generally consist of reddish brown to dark reddish brown, stiff to hard, silty to sandy lean clay with gravel, with interbeds of medium dense to dense silty sand and varying amounts of Jurassic age slaty gravels.

The stratigraphy of the subsurface soils encountered in each soil boring is presented in the boring logs (Appendix A). The general subsurface conditions across the site, interpreted from the boring and CPT data are shown on Plate 1.

2.3 Corrosion

Corrosion: 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 1 below.

Table 1 - 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 2019 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 1A - Sulfate Concentration and Exposure

Soluble Sulfate in Water (parts-per-million)	Water-Soluble Sulfate (SO_4) 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

A representative composite, near surface (0-5 feet) bulk soil sample collected from LB-2 was tested to evaluate corrosion potential. The chemical analysis test results for the onsite soil from our geotechnical exploration are included in Appendix B of this report and are summarized below.

Table 2 - Corrosivity Test Results

Test Parameter	Test Results LB-2 0-5'	General Classification of Hazard
Water-Soluble Sulfate-SO_4 in Soil (ppm)	177	Negligible sulfate exposure to buried concrete
Percent by Weight SO_4	0.0177	
Water-Soluble Chloride in Soil (ppm)	80	Non-corrosive to buried concrete (per Caltrans Specifications)
Percent by Weight (Cl^-)	0.0080	
pH	8.47	Mildly alkaline
Minimum Resistivity (saturated, ohm-cm)	1149	Severely Corrosive to buried ferrous pipes

Additional corrosion testing should be performed upon completion of grading to confirm the findings and conclusions presented above.

2.4 Expansive Soils

Expansion Index (EI) testing of one representative bulk sample collected from boring LB-2 within the upper 5 feet indicates an expansion index (EI) of 42, corresponding to a low potential for expansion. Given the clayey nature of the near surface soils expansion potential is anticipated to vary, and for purposes of this report, the expansion properties of the soil below the proposed new classroom should be considered as medium (EI=51 to 90). Additional testing of soils upon completion of grading should be performed to confirm the results of the initial testing.

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 3 below. Geotechnical laboratory testing results are presented in Appendix B, *Laboratory Test Results.*

Table 3 – Soil Geotechnical Properties Synopsis

Parameters	Soil Properties
In-situ Moisture:	Dry to very moist
In-situ Density:	Stiff to hard/Medium dense to dense
Swell/Expansion Potential:	swell/expansion potential is **low to medium.**
Collapse Potential:	Not susceptible to collapse when wetted
Strength:	Adequate to provide structural support
Corrosivity:	No sulfate attack of concrete but **severely corrosive to ferrous metals**.

2.5 Groundwater

Groundwater was not encountered in our borings or CPTs to the maximum depth explored of 51½ feet bgs. Historic groundwater levels, as interpreted from the Beverly Hills 7.5 Minute Quadrangle, Los Angeles County, California (CGS, 1998) indicate historic high groundwater was at a level of approximately 40 feet below ground surface.

Review of environmental data reported through the State Water Resources Control Board (see http://geotracker.waterboards.ca.gov/) shows that a series of eight monitoring wells were installed in association with a leaking underground storage tank remediation at Providence St. Johns medical Center; located approximately 600 feet southwest of the project site. Groundwater levels as measured within these monitoring wells was documented at depths ranging from approximately 110 to 132 feet bgs. Groundwater is not expected to pose a constraint to the proposed development as currently planned.

3.0 GEOLOGIC/SEISMIC HAZARDS

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

3.1 Faulting

Based on our site reconnaissance and review of available geologic literature and aerial photographs, we find no evidence that suggests active faults have been mapped across the site, and the site is **not** located within a currently established *Alquist-Priolo (AP) Earthquake Fault Zone* (Bryant and Hart, 2007, CGS, 2018). The limit of the AP Zone for the Santa Monica Fault Zone (SMFZ), as mapped by CGS (2018), is located approximately 370 feet northwest of the proposed classroom building footprint. The AP Zone was established based on recommendations provided in the Fault Evaluation Report 259 (FER 259) prepared by CGS and dated June 28, 2017 (CGS, 2017). Therefore, a fault hazard assessment is not mandated by the State for the proposed development.

The site is, however, located within a City of Santa Monica Fault Hazard Management Zone (City Safety Element, Leighton 1994). The City Fault Hazard Management Zone is defined roughly as the area located between the active Northern strand and inactive Southern strand (Dolan and Pratt, 1997) of the Santa Monica Fault Zone. The campus is located approximately 1,300 feet northwest of the mapped Southern strand, characterized as a structurally inverted Miocene normal fault that was active as a reverse fault during Miocene and latest Pliocene time (circa 1.5 to 5 mya); Quaternary strata are not deformed by this strand (Wright, 1991, Tsutsumi et al, 2000). Conversely, the campus is located approximately 3,300 feet southeast of the mapped Northern strand, which is considered active, the location of which is presented in the Safety Element of the City of Santa Monica (1994).

Investigations by academia (Dolan, J.F., Sieh, K., and Rockwell, T.K., 2000) have mapped the 40-km long, oblique left-lateral reverse Santa Monica fault zone as extending through Los Angeles, Santa Monica and offshore paralleling the Malibu coastline. Their work indicates the SMFZ has undergone at least six surface ruptures in the past 50 k.y. Based on poorly constrained soil age estimates, at least two or three probable events are interpreted to have occurred after burial of a well-dated prominent paleosol circa 16-16 k.a. This data led academia researchers to assign a 7-8 k.y Pleistocene-Holocene recurrence interval for large surface rupture events, which is much longer than the hypothetical 1.9-3.3 k.y recurrence interval calculated for a 6.9-7.0 Mw event generated by rupture of the entire SMFZ. The younger recurrence interval is predicated on a postulated fault (F4 in Dolan et al., 2000) that does not break Holocene soil or offset buried paleosols, however this interpreted fault was obscured by a utility trench during fault trenching operations. It is highly likely given the steep dip angle of the faults recorded both at the Veterans Hospital (Dolan, et al., 2000) and at University High School (Mactec, 2004) are upper plate faults and not the actual Santa Monica Fault proper. Not all researchers agree as to the activity of various segments. Investigations conducted along the north branch suggest the north branch, which may be a series of upper plate boundary faults may be active. Investigations conducted on the southern branch have either concluded lack of faulting or Pleistocene faulting capped by unbroken soils of middle to early Pleistocene age.

Based on our review of geologic literature (references) and subsurface exploration, the potential for surface fault rupture at the site is considered low. Plate 1, *Geotechnical Cross-Sections A-A' & B-B',* presents our interpretation of the subsurface stratigraphy. Based predominantly on the CPT transect, our interpretation of subsurface stratigraphy shows multiple laterally continuous stratum extending across the footprint of the new classroom footprint within the underlying Pleistocene age alluvial fan deposits.

Several active and potentially active faults are mapped within approximately 10 km (6.2 miles) of the site. Figure 4, *Regional Fault and Historical Seismicity Map*, shows the proximity of known active and potentially active faults within the region.

Santa Monica Fault: The State of California Geological Survey (CGS, 2018) has zoned the Santa Monica Fault, which is the closest known fault to the site, at a distance of approximately 800 feet (0.15 miles) northeast of the site. The SMFZ is considered, but not proven to be active, mapped as being located primarily north of Santa Monica Boulevard. This fault zone trends southeast-northwest along the southern boundary of the Santa Monica Mountains for more than 24.8 miles (40 km) and is included as part of the Transverse Ranges Southern Boundary fault system, which consists of east-west trending, left-lateral and oblique-reverse movements along several active faults. The SMFZ consists of one or more strands, is about 40 km (24.8 miles) in length, and is one of a series of reverse, left-lateral oblique-slip structures that extend more than 200 km (125 miles) across southern California and accommodate westward motion of the Transverse Ranges

(Dolan *et al*., 1997). Pleistocene or Holocene movement has been postulated, but not directly proven along some upper plate secondary fault segments related to the SMFZ (Dolan *et al.*, 2000). Recurrence interval and recency of movement along many fault segments are neither well documented nor understood, mainly because intense urbanization has modified or destroyed any surface traces of the fault (Hill *et al*., 1979). Southern California Earthquake Center (SCEC) identifies the most recent rupture as Late Quaternary with intervals between events unknown.

The State of California Geological Survey (CGS, 2018) has established an Earthquake fault Zone based on the criteria of “sufficiently active” and “well defined” (Bryant and Hart, 2007) in their FER 259 dated June 28, 2017.

Malibu Coast Fault: Located approximately 2.8 miles (4.5 km) northeast of the project site. 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 magnitude earthquake of 6.7 (Petersen et al., 1996).

Newport-Inglewood Fault: The onshore southeast-trending Newport-Inglewood fault zone (NIFZ) is discontinuous at the surface, consisting of a series of primarily left-stepping *en echelon* fault strands, each up to 6.5 km (4 miles) long that extend from near Beverly Hills south to Newport Beach, a distance of approximately 65 km (41 miles). At Newport Beach, the fault continues offshore where it lines up with the deeply incised Newport Submarine Canyon and is comprised of five strands and three step overs. To the south, back onshore, the fault continues as the Rose Canyon fault, extending in a southeasterly direction through San Diego and the international border to Baja California, where it continues as the Agua Blanca fault. Overall, from Beverly Hills to Baja California, the fault zone is more than 300 km (185 miles) long. At least five earthquakes of magnitude 4.9 or larger have been associated with the NIFZ since 1920 (Barrows, 1974). Estimated maximum deterministic magnitude earthquake is generally modeled between magnitude 6.5 and 7.5.

Hollywood Fault: Located approximately 5.4 miles (8.7 km) northeast of the site, the Hollywood Fault begins near the Los Angeles River and eastern edge of the

Santa Monica Mountains and extends westward for approximately 9½ miles where it is thought to shift its locus of active deformation to the area near the West Beverly Hills Lineament (WBHL), where faulting takes a left step to the Santa Monica Fault. The Hollywood Fault is deemed capable of producing a magnitude 6.4 to 6.6 earthquake (Dolan et al., 1997). Investigators have estimated the lateral slip rate to be about 1.0 ±0.5 mm/year, with a vertical slip rate to be 0.25 mm/year (Dolan et al., 1997). Conversely, a lower slip rate of 0.04 - 0.4 mm/year (Ziony and Yerkes, 1985) leads to a long return period.

Recent detailed geologic and geotechnical studies have provided cumulative physical evidence for Holocene displacements resulting in an Alquist-Priolo Special Study Zone being established for the Hollywood Fault (CGS, 2014). Exposures identified in prior explorations (Crook and Proctor, 1992), coupled with bulk-soil radiocarbon ages provide scant evidence for an early to mid-Holocene age for the most recent surface rupture approximately 6,000 years to 11,000 years ago; suggesting a long period of quiescence between surface rupturing on the Hollywood Fault (Dolan, 1997, 2000) (Ziony and Yerkes, 1985).

Palos Verdes Fault: The main trace of the onshore Palos Verde Hills (PVH) fault is recognized as a general topographic escarpment along the northeast margin of Palos Verdes Hills, based on the presence of linear drainages, saddles, and tilted or uplifted surfaces (Fischer and others, 1987). The PVH fault is reportedly a high-angle southwest-dipping dextral oblique fault (with reverse component) which forms the southwestern boundary of the Los Angeles basin at the Palos Verdes uplift (Wright, 1991, McNeilan and others, 1996). The sense of movement is dominantly right-lateral as interpreted by Stephenson et.al. (1995). The ratio of horizontal to vertical offset is on the order of 7:1 to 8:1, as estimated by McNeilan and others (1996). Most of the PVH section may have a larger reverse component than the other sections due to the change in strike of the fault.

The PVFZ is classified as a Class A Fault (No. 128b) by the California Geological Survey (CGS) (Treiman, 2015). Class A Faults are defined as those exhibiting geologic evidence for Quaternary tectonic offset, whether exposed or inferred from liquefaction or other deformational features. 3.

3.2 Historical Seismicity

An evaluation of historical seismicity from significant past earthquakes related to the site was performed (see Figure 4, *Regional Fault and Historical Seismicity*

Map). Peak ground accelerations (PGA) at the site resulting from significant past earthquakes between 1800 to 2018, with magnitudes 4.0 or greater, were estimated using the EQSEARCH computer program (Blake, 2000) with 2018 updates. This historical seismicity search was performed for a 100-kilometer (62-mile) radius from the project site, and is included in Appendix C, *Seismicity Data*. The largest earthquake magnitude found in the search was the magnitude 7.7 earthquake, known as the Arvin-Tehachapi quake that occurred on July 21, 1952 approximately 74 miles (118 kilometers) from the site producing an estimated PGA of approximately 0.05g at the site. The largest estimated PGA found in the search was approximately 0.2g from the 1994 magnitude 6.7 Northridge Earthquake located approximately 13 miles (20 kilometers) north of the site.

Review of additional data publicly available from the Center for Engineering Strong Motion Data (CESMD) website (http://strongmotioncenter.org/) was reviewed for stations near the project site. The data reviewed indicates that a site (CSMIP Station 24202-Providence St. Johns Hospital) less than ¼ mile to the southwest of the project site experienced a PGA of 0.03g from the magnitude 5.4 Chino Hills Earthquake on July 29, 2008. Another station (CGS Station 24048) located near the corner of 19th Street and Wilshire, approximately 0.4 mile southwest of the project site, experienced a PGA of 0.15g from the March 17, 2014 magnitude 4.4 Encino Earthquake. We are unaware of any reported damage to this campus as a result of earthquakes occurring over the last century.

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.

Review of both the Beverly Hills Quadrangle Seismic Hazard Zone Map (CGS, 1999) and the City of Santa Monica Geologic Hazards map (City of Santa Monica, 2014) indicates that the site is not within an area potentially susceptible to liquefaction (Figure 5, *Seismic Hazard Map*). The site is mapped within an area identified on the City of Santa Monica Geologic Hazards as a low to medium Liquefaction Risk.

The site is underlain by stiff to hard clays interbedded with medium dense to dense sand and silty sand and groundwater is anticipated below a depth of 50 feet. Given these factors, the potential for liquefaction and lateral spreading to affect the site is 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.

Based on our analysis, the total seismically-induced settlement is expected to be on the order of ½ inch or less. Accordingly, seismically-induced differential settlement is expected to be on the order of ¼ inch over 40 feet.

3.5 Seismically-Induced Landslides

The proposed project site is not located in an area mapped as potentially susceptible to seismically-induced landslides (Figure 5, *Seismic Hazard Map*). No landslides are mapped or known to exist at the project site or vicinity. The site is relatively flat and is not located adjacent to a significant slope. The potential for seismically induced landslides to affect the site is low.

3.6 Flooding

As shown on Figure 6, *Flood Hazard Zone Map*, the site is located outside of areas recognized by the Federal Emergency Management Agency (FEMA) to within 0.2% annual flood potential (FEMA, 2008). Earthquake-induced flooding can be caused by failure of dams or other water-retaining structures as a result of an earthquake. As shown on Figure 7, *Dam inundation Map*, the site is located outside of a dam inundation area due to the absence of such structures near the site, therefore the potential for earthquake-induced flooding at the site is considered low.

3.7 Seiches and Tsunamis

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Tsunamis are sea waves generated by large-scale disturbance of the ocean floor that induces a rapid displacement of the water column above. The most frequent causes of tsunamis are shallow underwater earthquakes and submarine landslides.

The site is not located within the tsunami run up area as mapped on the Los Angeles Tsunami Hazard: Maximum Run-up map (CalEMA, 2010). The run up area indicates zones along the Pacific Coast below an elevation of 42 feet (msl) are susceptible to tsunami inundation. The project site is topographically at least 120 feet above the areas identified to have a potential for Tsunamis impact. In addition, the site is not located within a tsunami inundation area as mapped by the State of California (CGS, 2009).

Based on the site's elevation of approximately 160 feet above sea level and the lack of nearby enclosed water bodies, the risks associated with tsunamis and seiches are considered negligible.

4.0 FINDINGS AND CONCLUSIONS

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

- This site is **not** located within a currently designated Alquist-Priolo Special Studies Zone (CGS, 2018) for surface fault rupture and is also **not** within a designated (March 25, 1999) liquefaction hazard zone. The site is not located in any geologic or seismic hazard zone that could preclude the development of the proposed project. As is the case for most of Southern California, strong ground shaking has and will occur at this site.

- The site is underlain by undocumented artificial fill to a depth of approximately 4 feet overlying native alluvial valley deposits generally consisting of stiff to hard clays interbedded with medium dense to dense sands; with varying proportions of predominantly slate gravels.

- Groundwater was not encountered during the current exploration. Groundwater is not expected to pose a constraint to construction. The historic high groundwater level at the site was reported to be on the order of 40 feet bgs.

- 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 on the order of ½ inch or less.

- Based on our observations and testing, the onsite soils that will be in contact with the planned structures are expected to have a low to moderate expansion potential. Additional testing is recommended during design stage or at completion of grading. For purposes of design we recommend using a moderate expansion index EI=51 to 90.

- Concrete in contact with the onsite soil is expected to have negligible 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 spread footings established on engineered fill or undisturbed natural soils.

Based on the results of this study, it is our opinion that the subject site is suitable for the proposed project from a geotechnical viewpoint. Geotechnical recommendations for the proposed development are presented in the following sections and are intended to provide sufficient geotechnical information to develop the project plans in accordance with the 2019 edition of the California Building Code (CBC) requirements.

5.0 RECOMMENDATIONS

The following recommendations have been developed based on the exhibited engineering properties of the onsite soils and their anticipated behavior during and after construction. Recommendations are specifically provided for design of foundations, seismic design considerations, floor slabs, retaining structures, paving, and grading. The proposed structure may be supported on spread-type shallow footing foundation systems established on engineered fill or undisturbed natural soils. Leighton should review the grading plan, foundation plans, shoring plans and specifications when they are available to verify that the recommendations presented in this report have been properly interpreted and incorporated.

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 Grading

Project earthwork is expected to include complete demolition/removal of existing surface pavements, landscaping, utilities 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

After the site is cleared, the soils should be carefully observed for the removal of all unsuitable deposits. We recommend that after removal of pavements, hardscape, and existing utilities, all undocumented fill soils should be removed and recompacted within the proposed improvement footprint. Undocumented fill was encountered as deep as 4 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. Overexcavation is not required for footings established directly on undisturbed natural soils. Any underground obstructions encountered should be removed. Utility lines should be removed or rerouted where interfering with proposed construction. *It is essential that excavation not undermine foundations of the existing buildings and structures that will remain in place along the* *boundaries project. As-Built details of any structure to remain should be provided to Leighton and the structural engineer prior to incorporation into the new design.*

Areas outside the classroom footprint limits, planned for new asphalt and/or concrete pavement, should be over-excavated to a minimum depth of 24 inches below existing or finish grade, or 18 inches below proposed pavement sections; whichever is deeper.

Resulting removal excavation bottom-surfaces should be observed by Leighton prior to placement of any backfill or new construction. After these over-excavations are completed, and prior to fill placement, exposed surfaces should be scarified to a minimum depth of 8 inches, moisture-conditioned to 2 percent above optimum moisture content, and recompacted (proof rolled) to a minimum 90 percent relative compaction as determined by ASTM D 1557 (modified Proctor compaction curve).

5.1.2 <u>Earthwork Observation and Testing</u>

Leighton Consulting, Inc. should observe and test all grading and earthwork, to check that the site is properly prepared, the selected fill materials are satisfactory, and that placement and compaction of fills has been performed in accordance with our recommendations and the project specifications. Sufficient notification to us prior to earthwork is essential. Project plans and specifications should incorporate recommendations contained in the text of this report.

Variations in site conditions are possible and may be encountered during construction. To confirm correlation between soil data obtained during our field and laboratory testing and actual subsurface conditions encountered during construction, and to observe conformance with approved plans and specifications, it is essential that we be retained to perform continuous or intermittent review during earthwork, excavation and foundation construction phases. Therefore, conclusions and recommendations presented in this report are contingent upon us performing construction observation services.

5.1.3 Fill Placement and Compaction

Onsite soils free of organics, debris and oversized material (greater-than 6 inches in largest dimension) are suitable for use as compacted structural fill. However, any soil to be placed as fill, whether onsite or imported material, should be first viewed by Leighton and then tested if and as necessary, prior to approval for use as compacted fill. All structural fill must be free of hazardous materials.

All fill soil should be placed in thin, loose lifts, moisture-conditioned, as necessary, to 2 percent above optimum moisture content 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.

Fill Materials: The onsite soils, less any deleterious material or organic matter, 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 a very low Expansion Index (EI<20). All proposed import materials should be approved by the geotechnical engineer of record prior to being placed at the site.

Surface Drainage: Water should not be allowed to pond or accumulate anywhere except in detention basins. Pad 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 and/or percolation should not be allowed for at least 10 feet horizontally around buildings.

5.1.4 Reuse of Concrete and Asphalt in Fill

Pulverized demolition concrete free of rebar and other materials and demolished asphalt pavement can be pulverized to particles no-larger-than (≤) 3-inches and mixed with site soils for use in compacted fill. Blended pulverized concrete and asphalt should be mixed with at least 25% soils by weight. Such materials must be free of and segregated from any hazardous materials and/or organic material of any kind.

5.1.5 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 within the proposed structural depths 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.

5.1.6 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**:* Any proposed pipe should be placed on properly placed bedding materials. Pipe bedding should extend to a depth in accordance to the pipe manufacturer's specification. The pipe bedding should extend to least 1 foot over the top of the conduit. The bedding material may consist of compacted free-draining sand, gravel, or crushed rock. If sand is used, the sand should have a sand equivalent greater than 30. As an alternate, the pipe bedding zone can 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 2021 Edition of the Standard Specifications for Public Works Construction (Greenbook). CLSM bedding should be placed to 1 foot over the top of the conduit, and vibrated. CLSM should not be jetted.

  Where granular backfill is used in utility trenches adjacent moisture sensitive subgrades and foundation soils, we recommend that a cut-off "plug" of impermeable material be placed in these trenches at the perimeter of buildings, and at pavement edges adjacent to irrigated landscaped areas. A "plug" can consist of a 5-foot long section of clayey soils with more than 35-percent passing the No. 200 sieve, or a Controlled Low Strength Material (CLSM) consisting of one sack of Portland-cement plus one sack of bentonite per cubic-yard of sand. CLSM should generally conform to Section 201-6 of the "Greenbook". This is intended to reduce the likelihood of water permeating trenches from landscaped areas, then seeping along permeable trench backfill into the building and pavement subgrades, resulting in wetting of moisture sensitive subgrade earth materials under buildings and pavements.

- ***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. 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 ASTM D 1557 laboratory maximum dry density within building footprints. The upper 12-inches under hardscape, parking, paver etc. should be compacted to 95% relative compaction. 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.

5.1.7 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 subsurface explorations, concrete structures in contact with the onsite soil will likely have **"negligible"** 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, 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.

5.2 Foundations

The proposed new structures may be supported on a shallow spread footing foundation system established on engineered fill or undisturbed natural soils.

5.2.1 Shallow Spread Footings

Footings for proposed structures should have a minimum embedment of 3 feet and have a minimum width of 18 inches. Footings for proposed temporary structures may be supported directly on grade.

Bearing Value: Footings or post-tensioned concrete slabs with thickened edges established on engineered fill or undisturbed natural soils may be designed to impose an allowable bearing pressure of 3,000 pounds per square foot (psf).

The excavations should be deepened as necessary to extend into satisfactory soils.

The ultimate bearing capacity can be taken as 9,000 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 column loads not exceeding 300 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 0.35. The passive resistance may be computed using an equivalent fluid pressure of 300 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.

5.2.2 Modulus of Subgrade Reaction

For foundations established in undisturbed natural soil or engineered fill, an initial unit modulus of subgrade reaction (k_1) value of 150 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.

5.2.3 Flagpole-Type Foundations

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.

5.3 Seismic Design Parameters

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 4 - 2019 CBC Mapped Seismic Parameters

Categorization/Coefficients	Code-Based (1) (2)
Site Longitude (decimal degrees) West	-118.4768
Site Latitude (decimal degrees) North	34.0324
Site Class	D
Mapped Spectral Response Acceleration at 0.2s Period, S_s	1.955
Mapped Spectral Response Acceleration at 1s Period, S_1	0.698
Short Period Site Coefficient at 0.2s Period, F_a	1.0
Long Period Site Coefficient at 1s Period, F_v	null*
Adjusted Spectral Response Acceleration at 0.2s Period, S_{MS}	1.955
Adjusted Spectral Response Acceleration at 1s Period, S_{M1}	null*
Design Spectral Response Acceleration at 0.2s Period, S_{DS}	1.303
Design Spectral Response Acceleration at 1s Period, S_{D1}	null*
Design Peak Ground Acceleration, PGA_M	0.918

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.

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 C.

Table 5 - Site-Specific 2019 CBC Seismic Design Parameters

Categorization/Coefficients	Design Value
Adjusted Spectral Response Acceleration at 0.2s Period, S_{MS}	2.305g
Adjusted Spectral Response Acceleration at 1s Period, S_{M1}	1.531g
Design Spectral Response Acceleration at 0.2s Period, S_{DS}	1.537g
Design Spectral Response Acceleration at 1s Period, S_{D1}	1.02g
Design Peak Ground Acceleration, PGA_M	0.93g

5.4 Slabs-on-Grade

Concrete slabs-on-grade should be designed by the structural engineer in accordance with 2019 CBC requirements for soils with a moderate expansion potential. 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 characterized as lean clay, 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 24 inches of relatively non-expansive fill (EI<20). Slab-on-grade subgrade soil should be moisture conditioned to 2% 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. Onsite soil may be suitable for this use; however additional expansion testing should be performed upon completion of grading to verify expansive properties of onsite soil.

- ***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 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

5.4.1 Utilities and Trenches

Open or backfilled trenches paralleling any new or existing footings to remain shall not be below a 1:1 projection from outer lowest edge of footings or slab on grade. Where pipes cross under footings the footings shall be specifically designed by the engineer in charge. Pipe sleeves shall be provided where pipes cross through footings or footing walls and sleeve clearances shall be designed to account for potential settlement of not less than 1 inch around the pipe. Alternate and approved clearances can be provided by the design professional in charge of the utility.

5.5 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 may be suitable to be used as retaining wall backfill due to its low expansion potential (Appendix B), however, field and laboratory verification are recommended before use. Site soils can be variable in composition and expansive characteristics, See Section 2.4. Should site soil be desired for reuse behind retaining walls the material should be tested to ensure Expansion potential is less than 20 (EI<20). Recommended lateral earth pressures for retaining walls backfilled with sandy soils with drained conditions as shown on Figure 8 are as follows:

Table 6 - Retaining Wall Design Earth Pressures

Retaining Wall Condition (Level Backfill)	Equivalent Fluid Pressure (pounds-per-cubic-foot)*
Active (cantilever)	35
At-Rest (braced)	55
Passive Resistance (compacted fill)	300
Seismic Increment (add to active pressure)	30

*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 seismic condition, the pressure should be distributed as an inverted triangular distribution and the dynamic thrust should be applied at a height of 0.6H above the base of the wall.

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.

5.5.1 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.

5.5.2 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), 2021 Edition. This pervious backfill should extend at least 2 feet out from the wall and 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), 2021 Edition. The subdrain outlet should be connected to a free-draining outlet or sump.

Miradrain, Geotech Drainage Panels, or Enkadrain drainage geocomposites, or similar, 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.

5.6 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 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.

5.6.1 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.

5.6.2 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 or comparable soils with an R-value of at least 10 (Appendix B) 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	4	8
Heavy Truck	6	4	10½
Main Drives	7	5	12½

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.

5.6.3 Portland Cement Concrete Paving

Portland Cement Concrete (PCC) paving and walks supported on clayey onsite soils should be underlain by at least 18 inches of engineered fill consisting of relatively non-expansive (EI < 20) soils. We have assumed that such a subgrade will have an R-value of at least 40, which will need to be verified during grading. Onsite soils are anticipated to have an EI>20, therefore, we expect that relatively non-expansive (EI<20) will need to be imported for PCC paving.

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.

6.0 CONSTRUCTION CONSIDERATIONS

6.1 Excavations

Based on our field observations, caving of cohesionless strata and loose fill soils will likely be encountered in unshored excavations. To protect workers entering excavations, excavations should be performed in accordance with OSHA and Cal-OSHA requirements, and the current edition of the California Construction Safety Orders, see:

http://www.dir.ca.gov/title8/sb4a6.html

Contractors should be advised that fill soils should be considered Type C soils as defined in the California Construction Safety Orders. As indicated in Table B-1 of Article 6, Section 1541.1, Appendix B, of the California Construction Safety Orders, excavations less-than (<) 20 feet deep within Type C soils should be sloped back no steeper than 1½:1 (horizontal:vertical), where workers are to enter the excavation. This may be impractical near adjacent existing utilities and structures; so shoring may be required depending on trench depth and locations. Stiff undisturbed native clays will stand steeper.

During construction, soil conditions should be regularly evaluated to verify that conditions are as anticipated. The contractor is responsible for providing the "competent person" required by OSHA standards to evaluate soil conditions. Close coordination between the competent person and Leighton Consulting, Inc. should be maintained to facilitate construction while providing safe excavations.

Excavations must not undermine foundations for existing buildings. Excavations must not encroach within a 1:1 (horizontal:vertical) wedge extending down and out from existing shallow footings to remain. Shoring or underpinning of existing building foundations may be required depending upon final footprint and floor elevations.

6.2 Geotechnical Services During Construction

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 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, foundation, and shoring 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 utility trench backfilling and compaction,
- During pavement subgrade and base preparation, and/or
- If and when any unusual geotechnical conditions are encountered.

7.0 LIMITATIONS

Leighton's work was performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. No other warranty, express or implied, is made as to the conclusions and professional opinions included in this report. As in many projects, conditions revealed in excavations may be at variance with our current findings. If this occurs, the changed conditions must be evaluated by the geotechnical consultant and additional recommendations be obtained, as warranted.

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

This report is issued with the understanding that it is the responsibility of the owner or 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 contracts 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 testes. 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 and/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 express 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 express written consent of Santa Monica - Malibu Unified 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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Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer will <u>not</u> likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will <u>not</u> be adequate to develop geotechnical design recommendations for the project.

Do <u>not</u> rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do <u>not</u> rely on an executive summary. Do <u>not</u> read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept* *responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are <u>not</u> final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals' plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for information purposes only.* To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are <u>Not</u> Covered

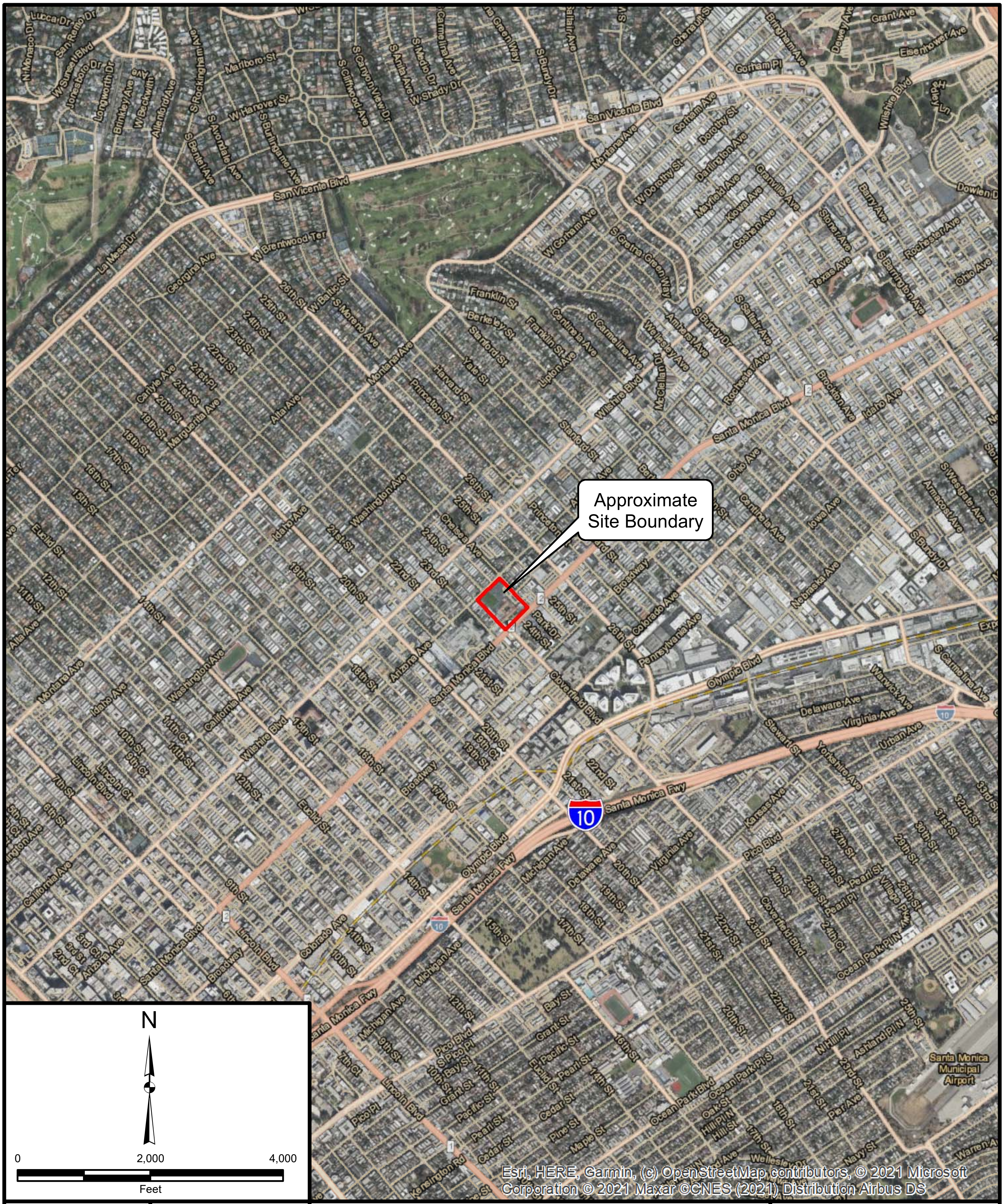
The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer's services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will <u>not</u> of itself be sufficient to prevent moisture infiltration. Confront the risk of moisture infiltration*** by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are <u>not</u> building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org



Project: 11428.036	Eng/Geol: EMH
Scale:1 " = 2,000 '	Date: October 2021

Base Map: ESRI ArcGIS Online 2021
Thematic Information: Leighton
Author: Leighton Geomatics (btran)

SITE LOCATION MAP

McKinley Elementary School
2401 Santa Monica Boulevard
Santa Monica, California

FIGURE 1

Leighton

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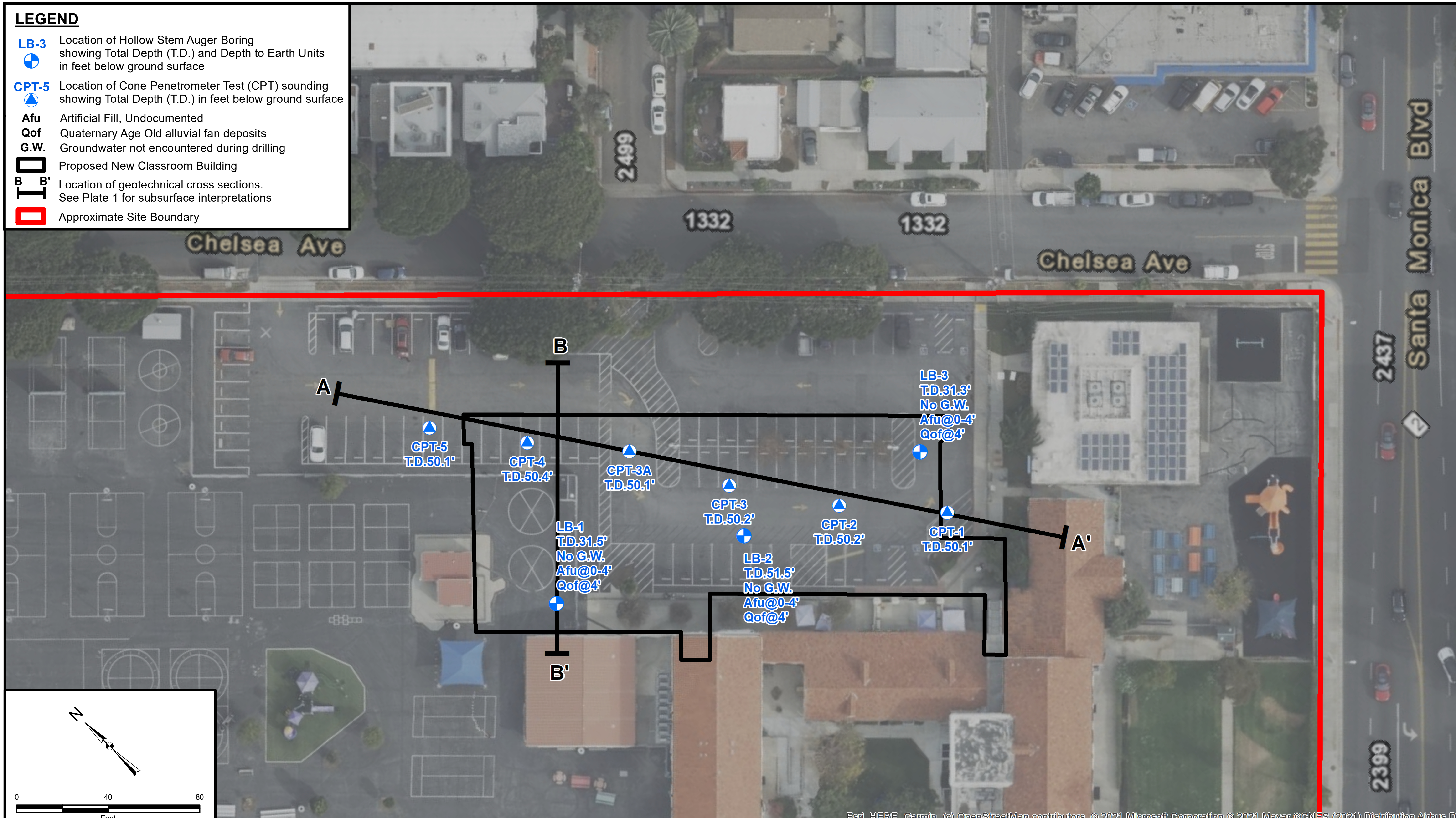


Project: 11428.036	Eng/Geol: EMH
Scale: 1 " = 40 '	Date: November 2021

Base Map: ESRI ArcGIS Online 2021

Author: Leighton Geomatics (btran)

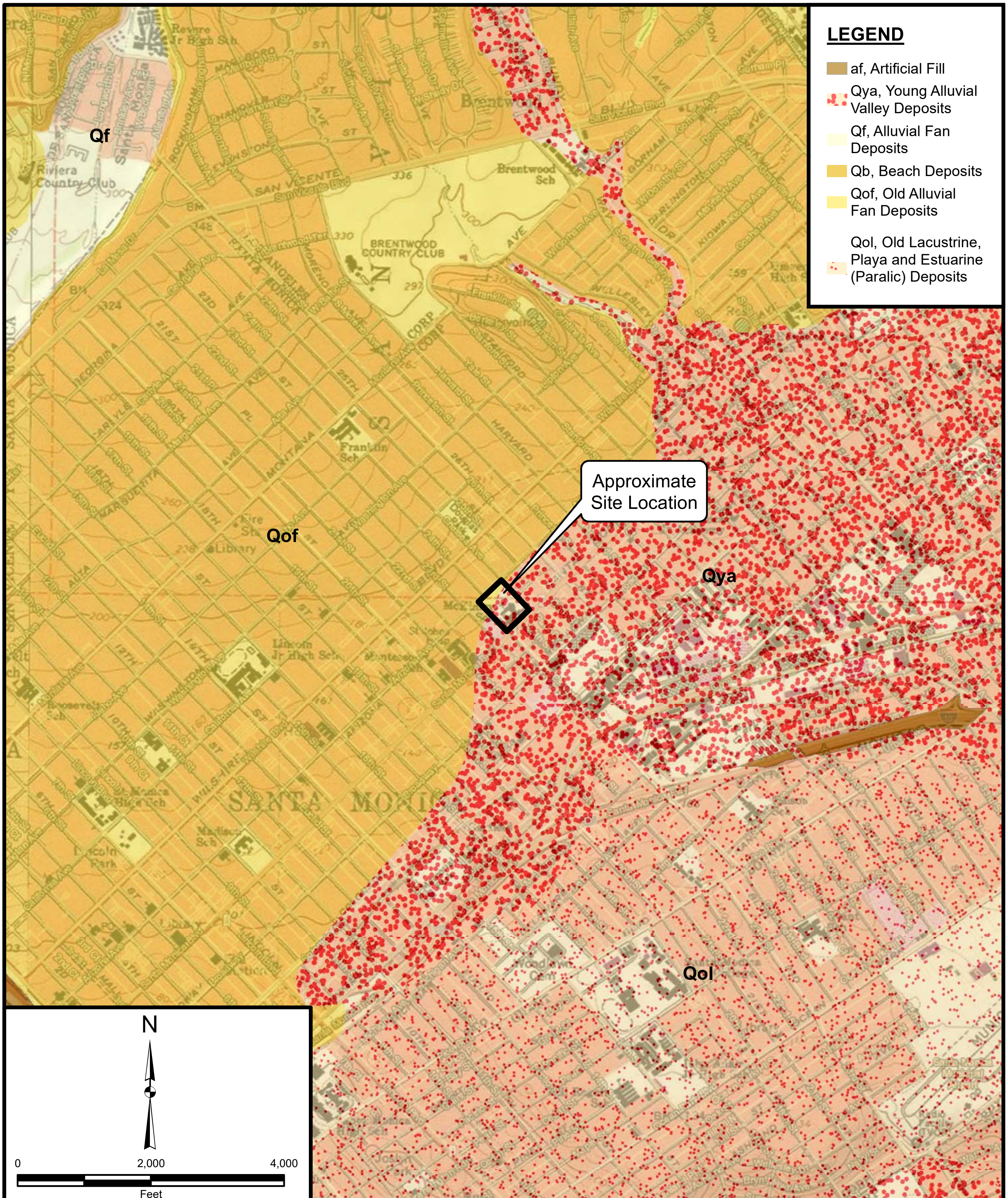
EXPLORATION LOCATION MAP

McKinley Elementary School
2401 Santa Monica Boulevard
Santa Monica, California

FIGURE 2

Leighton

Map Saved as V:\Drafting\11428\036\Maps\11428-036_F02_ELM_2021-11-17.mxd on 11/16/2021 4:23:59 PM



Project: 11428.036	Eng/Geol: EMH
Scale:1 " = 2,000 '	Date: October 2021

Base Map: ESRI ArcGIS Online 2021
Socal Preliminary Geology, 2010

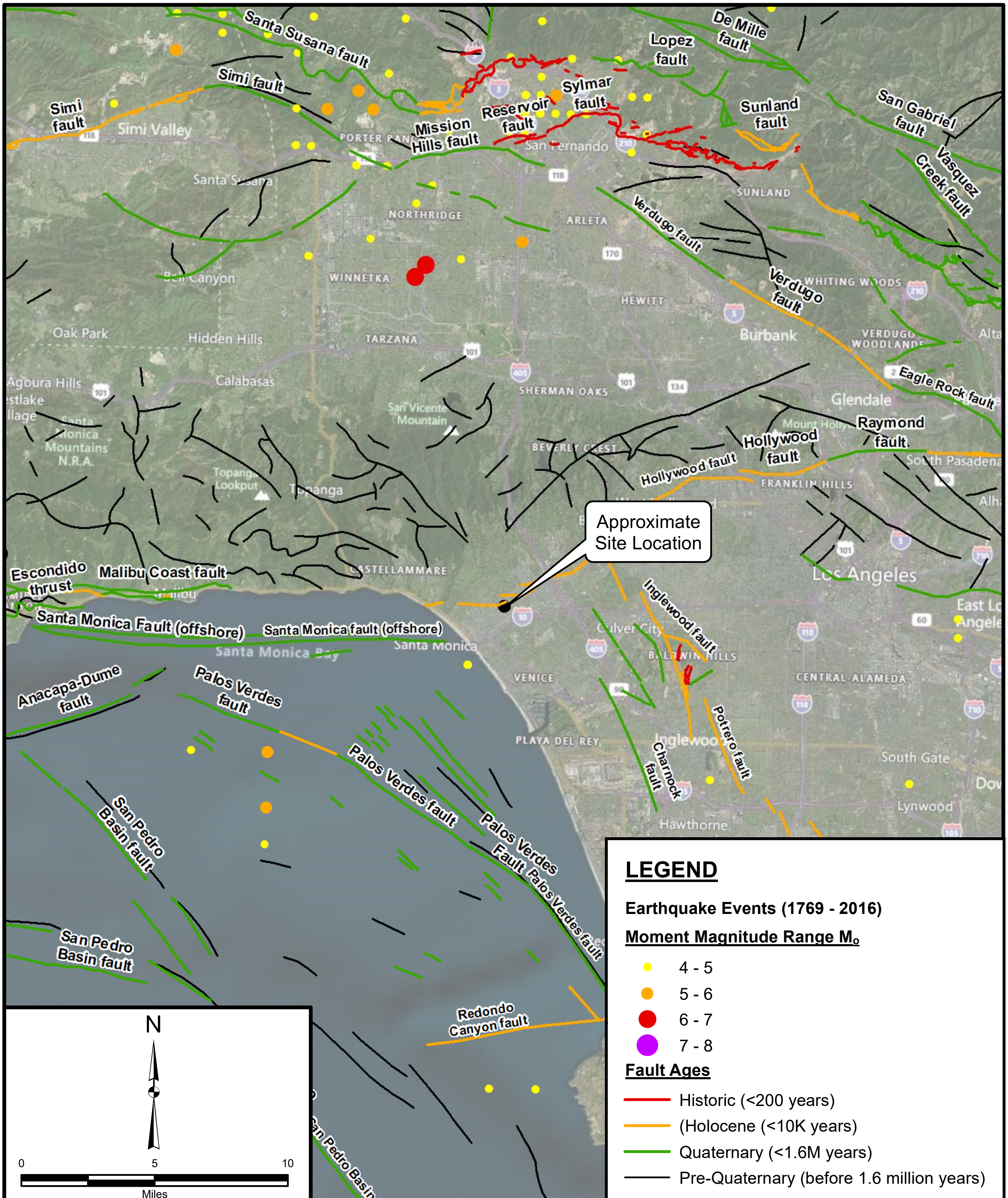
Author: Leighton Geomatics (btran)

REGIONAL GEOLOGY MAP

McKinley Elementary School
2401 Santa Monica Boulevard
Santa Monica, California

FIGURE 3

Leighton



Project: 11428.036	Eng/Geol: EMH	REGIONAL FAULT AND HISTORICAL SEISMICITY MAP McKinley Elementary School 2401 Santa Monica Boulevard Santa Monica, California	FIGURE 4
Scale:1 " = 5 miles	Date: October 2021		Leighton
Reference: ESRI ArcGIS Online 2021 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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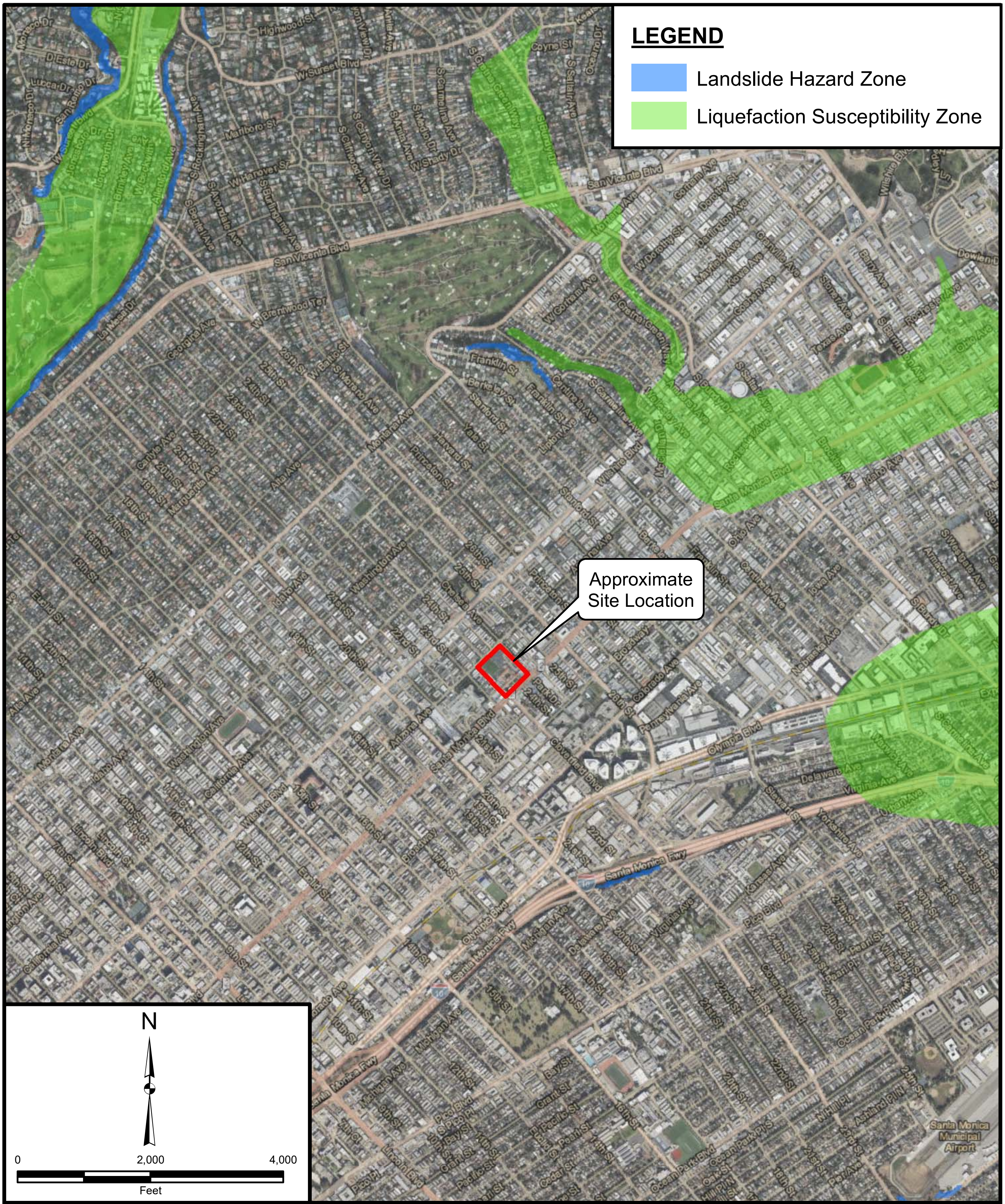

Project: 11428.036	Eng/Geol: EMH
Scale:1 " = 2,000 '	Date: October 2021

Base Map: ESRI ArcGIS Online 2021

Author: Leighton Geomatics (btran)

SEISMIC HAZARD MAP

McKinley Elementary School
2401 Santa Monica Boulevard
Santa Monica, California

FIGURE 5

Leighton

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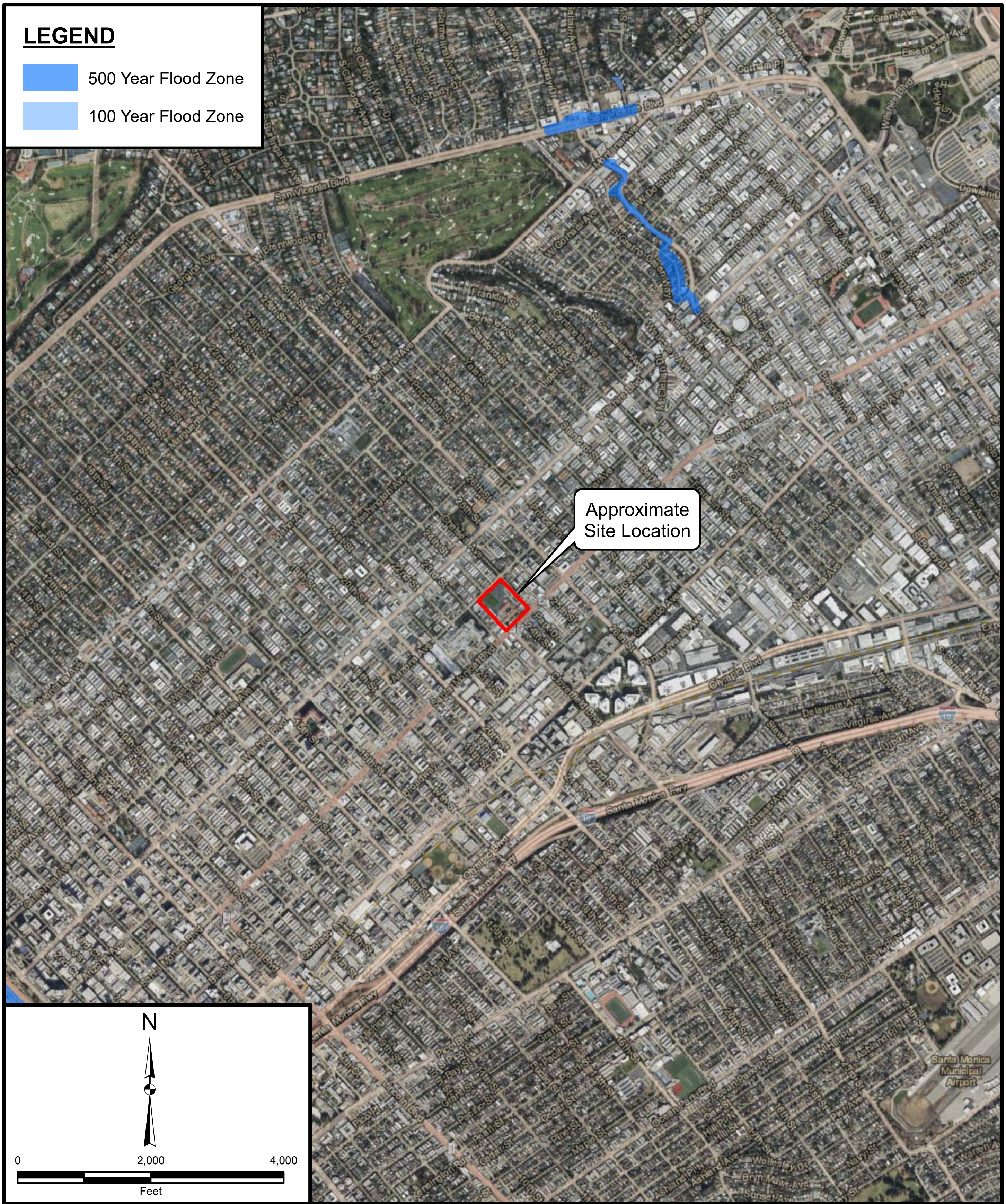

Project: 11428.036	Eng/Geol: EMH
Scale:1 " = 2,000 '	Date: October 2021

Base Map: ESRI ArcGIS Online 2021
Reference: CA DWR, FEMA
Author: Leighton Geomatics (btran)

FLOOD HAZARD ZONE MAP

McKinley Elementary School
2401 Santa Monica Boulevard
Santa Monica, California

FIGURE 6

Leighton

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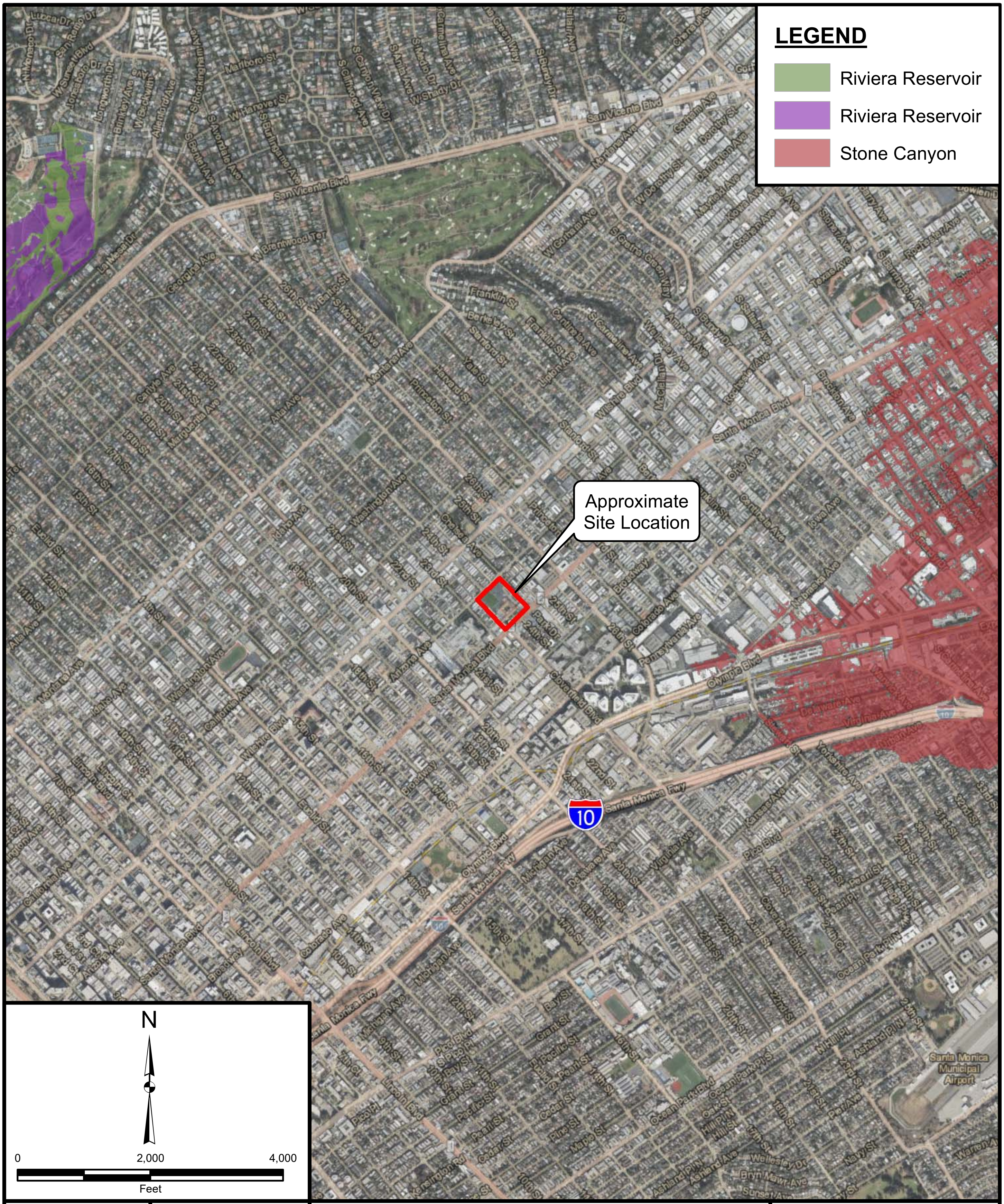


Project: 11428.036	Eng/Geol: EMH
Scale:1 " = 2,000 '	Date: October 2021

Base Map: ESRI ArcGIS Online 2021

Author: Leighton Geomatics (btran)

DAM INUNDATION MAP

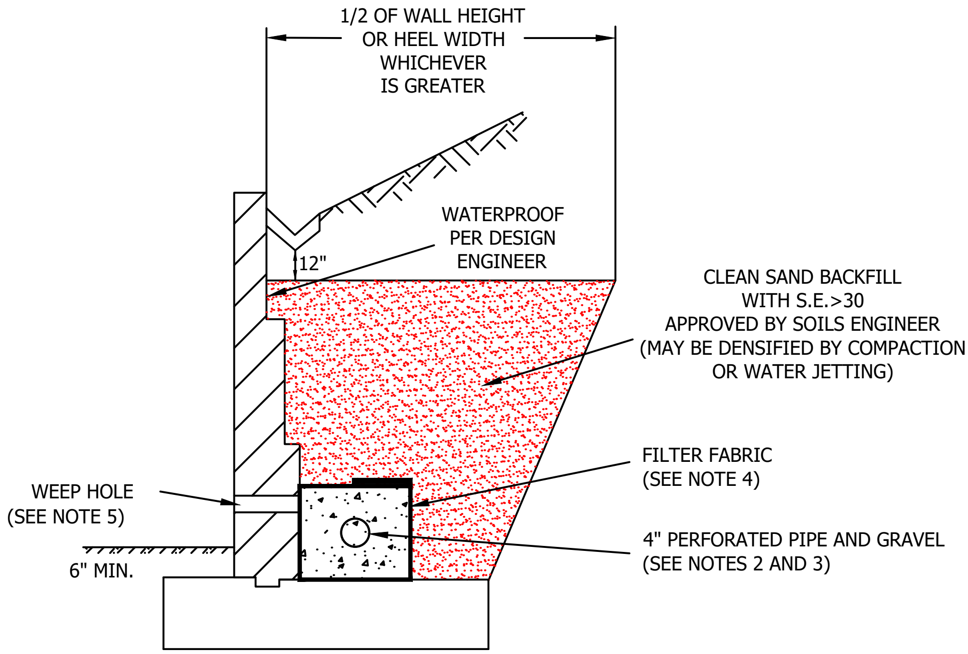
McKinley Elementary School
2401 Santa Monica Boulevard
Santa Monica, California

FIGURE 7

Leighton

Map Saved as V:\Drafting\11428\036\Maps\11428-036_F07_DIM_2021-10-15.mxd on 10/15/2021 12:02:43 PM

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



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

GENERAL NOTES:

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

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

RETAINING WALL BACKFILL AND SUBDRAIN DETAIL

WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF >50

Leighton

FIGURE 8

APPENDIX A

Field Exploration Logs

GEOTECHNICAL BORING LOG LB-1

Project No.	11428.036	**Date Drilled**	9-16-21
Project	McKinley ES	**Logged By**	EMH
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	162'
Location	See Figure 2 - *Exploration Location Map*	**Sampled By**	EMH

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 *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0			BB-1				CL	**Artificial fill, undocumented: (Afu)** @0': 4-inches Asphalt Concrete (AC) over Subgrade @4-inches: Sandy CLAY (CL), medium reddish brown, slightly moist, fine sand, trace medium to coarse sand and fine gravel	
160										
									Quaternary Old Alluvial Fan Deposits (Qof)	
	5			R-1	12 20 33	116	16	CL	@5': Silty CLAY (CL), hard, medium reddish brown, slightly moist, with fine sand and trace fine slate gravels, low to medium plasticity, PP > 4.50	
155										
				R-2	8 16 20	125	12		@7.5': Sandy CLAY with gravel (CL), very stiff, medium reddish brown, moist, fine to medium sand, trace coarse sand, fine slate gravels with occasional subrounded granitic gravel, PP > 4.50	
	10			R-3	4 9 14	110	19		@10': Silty CLAY (CL), stiff, reddish brown, moist, some fine sand, trace fine slate gravels, medium plasticity, PP > 4.50	
150										
	15			R-4	6 13 13	110	13	ML	@15': Sandy SILT with clay (ML), very stiff, reddish brown, moist, high fine sand content, trace slate gravel, low plasticity, PP > 4.50	
145										
	20			S-1	3 3 4		15		@20': Laminated with Silty SAND and discrete fine slate gravel laminations, firm, moist	
140										
	25			R-5	7 20 27	128	7	SM	@25': Silty SAND (SM), dense, dark brown, moist, fine sand, with gravel lamination at 26.3 feet	
135										
	30									

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

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

Leighton

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

GEOTECHNICAL BORING LOG LB-1

Project No.	11428.036	**Date Drilled**	9-16-21
Project	McKinley ES	**Logged By**	EMH
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	162'
Location	See Figure 2 - *Exploration Location Map*	**Sampled By**	EMH

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
	30			S-2		3 7 12		13	CL	@30': CLAY with gravel (CL), very stiff, dark reddish brown, moist, fine slate gravels, medium to high plasticity	
130										Total Depth of Boring: 31.5 feet No groundwater encountered during drilling Boring backfilled with tamped soil cuttings, and surface patched with asphalt cold patch on 9-16-2021.	
	35										
125											
	40										
120											
	45										
115											
	50										
110											
	55										
105											
	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
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- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG LB-2

Project No.	11428.036	**Date Drilled**	9-16-21
Project	McKinley ES	**Logged By**	EMH
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	160'
Location	See Figure 2 - *Exploration Location Map*	**Sampled By**	EMH

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
160	0			BB-1	Bulk				CL	**Artificial fill, undocumented: (Afu)** @0': 4-inches Asphalt Concrete (AC) over Subgrade @4-inches: LEAN CLAY (CL), reddish brown, hard, slightly moist, trace fine to medium sand, medium plasticity	EI, MD, CN, CR DS, RV
										Quaternary Old Alluvial Fan Deposits (Qof)	
155	5			R-1	Driven	16 33 45	119	15	CL	@5': LEAN CLAY (CL), reddish brown, hard, slightly moist, trace fine to medium sand, medium plasticity	CN, DS
				R-2	Driven	11 16 29	118	13		@7.5': LEAN CLAY (CL), reddish brown, hard, slightly moist, trace silt, fine to medium sand, and fine gravel, low to medium plasticity, PP > 4.50 tsf	CN, DS
150	10			R-3	Driven	14 34 32				@10': Silty CLAY with Gravel and Sand (CL), hard, reddish brown, slightly moist, fine sand, fine slate gravels, low plasticity	
145	15			S-1	Driven	2 3 6		18		@15': Interlaminated Clay and Sandy CLAY (CL), stiff, reddish brown, moist, fine sand, some silt, medium plasticity	
140	20			R-4	Driven	4 9 13	106	14	SM	@20': Silty SAND (SM), medium dense, reddish brown, moist, fine sand, faintly laminated, grades to Sandy SILT (ML), reddish brown, slightly moist, fine sand	
135	25			S-2	Driven	2 2 2		14	ML	@25': Sandy SILT (ML), soft, reddish brown, moist, fine sand, some clay with discrete slate pebble and clay rich laminations	
130	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
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- 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 LB-2

Project No. 11428.036
Project McKinley ES
Drilling Co. Martini Drilling
Drilling Method Hollow Stem Auger - 140lb - Autohammer - 30" Drop
Location See Figure 2 - *Exploration Location Map*

Date Drilled 9-16-21
Logged By EMH
Hole Diameter 8"
Ground Elevation 160'
Sampled By EMH

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
130	30			R-5		6 22 28	126	11	CL	@30': Silty CLAY with gravel (CL), hard, reddish brown, moist, with fine to medium sand, fine slate gravels, low plasticity, chaotic assemblage, PP > 4.50	
125	35			S-3		4 5 8		17	CL	@35': CLAY with gravel (CL), stiff, reddish brown, moist, grades finer with less gravel, medium plasticity, fine slate gravels	
120	40			R-6		5 11 19	116	14		@40': CLAY to Silty CLAY (CL), very stiff, reddish brown, moist, trace to little fine slate gravels, medium plasticity	
115	45			S-4		2 2 11		24		@45': Stiff, increase in moisture, abundant slate gravels @46.3' to 46.5'	
110	50			R-7		4 8 21	114	18		@50': CLAY with sand (CL), very stiff, reddish brown, moist, fine sand, medium plasticity, occasional fine slate gravel, PP > 4.50	
										Total Depth of Boring: 51.5 feet No groundwater encountered during drilling Boring backfilled with tamped soil cuttings, and surface patched with asphalt cold patch on 9-16-2021.	
105	55										
100	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:
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- CN CONSOLIDATION
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- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

Leighton

GEOTECHNICAL BORING LOG LB-3

Project No.	11428.036	**Date Drilled**	9-16-21
Project	McKinley ES	**Logged By**	EMH
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	159'
Location	See Figure 2 - *Exploration Location Map*	**Sampled By**	EMH

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		BB-1				CL	**Artificial fill, undocumented: (Afu)** @0': 3.5-inches Asphalt Concrete (AC) over Subgrade @3.5-inches: Sandy Silty CLAY (CL), reddish brown, moist, fine to medium sand, occasional fine gravel	
155								**Quaternary Old Alluvial Fan Deposits (Qof)**	
	5		R-1	6 17 26	114	18	CL	@5': CLAY (CL), very stiff, reddish brown, hard, moist, medium plasticity, little fine sand and silt, PP = 4.50	
			R-2	5 10 19	121	14		@7.5': Silty CLAY with gravel (CL), very stiff, medium reddish brown, moist, little fine sand and fine slate gravels, PP > 4.50 tsf	
150	10		R-3	7 14 17	123	10		@10': Sandy CLAY with gravel (CL), very stiff, reddish brown, moist, fine to medium sand, fine slate gravels, chaotic assemblage	
145	15		S-1	3 4 3		10		@15': Sandy CLAY with gravel (CL), firm, reddish brown, soft, moist, fine to medium sand, fine slate gravels	
140	20		R-4	21 50/6"	133	4	GP-GC	@20': GRAVEL with sand and clay (GP-GC), reddish brown matrix, slightly moist, fine to coarse sand, fine slate gravels	
135	25		S-2	10 14 15		7	SM	@25': Silty SAND (SM), medium dense, dark brown, slightly moist, mostly fine sand, few medium sand, trace coarse sand and fine gravel, grades to sandy clay at tip	
130	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
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GEOTECHNICAL BORING LOG LB-3

Project No.	11428.036	**Date Drilled**	9-16-21
Project	McKinley ES	**Logged By**	EMH
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	159'
Location	See Figure 2 - *Exploration Location Map*	**Sampled By**	EMH

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-5		10 20 50/4"	126	8	SC	@30': Clayey SAND with gravel (SC), dense, reddish brown, slightly moist, fine to coarse sand, fine slate gravels, grades coarser with abundant gravels	
										Total Depth of Boring: 31.3 feet No groundwater encountered during drilling Boring backfilled with tamped soil cuttings, and surface patched with asphalt cold patch on 9-16-2021.	
125	35										
120	40										
115	45										
110	50										
105	55										
100	60										

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.

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

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
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- CR CORROSION
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- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

Leighton

SUMMARY

OF

CONE PENETRATION TEST DATA

Project:

McKinley Elementary School
Santa Monica, CA
September 16, 2021

Prepared for:

Mr. Eric Holliday
Leighton Consulting
17781 Cowan
Irvine, CA 92614-6009
Office (800) 253-4567/Fax (949) 250-1114

Prepared by:

KTE
KEHOE TESTING & ENGINEERING
5415 Industrial Drive
Huntington Beach, CA 92649-1518
Office (714) 901-7270 / Fax (714) 901-7289
www.kehoetesting.com

TABLE OF CONTENTS

SUMMARY

OF

CONE PENETRATION TEST DATA

1. INTRODUCTION

This report presents the results of a Cone Penetration Test (CPT) program carried out for the McKinley Elementary School project located in Santa Monica, California. The work was performed by Kehoe Testing & Engineering (KTE) on September 16, 2021. The scope of work was performed as directed by Leighton Consulting personnel.

2. SUMMARY OF FIELD WORK

The fieldwork consisted of performing CPT soundings at six locations to determine the soil lithology. A summary is provided in **TABLE 2.1**.

LOCATION	DEPTH OF CPT (ft)	COMMENTS/NOTES:
CPT-1	50	
CPT-2	50	
CPT-3	50	
CPT-3A	50	
CPT-4	50	
CPT-5	50	

TABLE 2.1 - Summary of CPT Soundings

3. FIELD EQUIPMENT & PROCEDURES

The CPT soundings were carried out by **KTE** using an integrated electronic cone system manufactured by Vertek. The CPT soundings were performed in accordance with ASTM standards (D5778). The cone penetrometers were pushed using a 30-ton CPT rig. The cone used during the program was a 15 cm^2 cone with a cone net area ratio of 0.83. The following parameters were recorded at approximately 2.5 cm depth intervals:

- Cone Resistance (qc)
- Sleeve Friction (fs)
- Dynamic Pore Pressure (u)
- Inclination
- Penetration Speed

At location CPT-3, shear wave measurements were obtained at approximately 5-foot intervals. The shear wave is generated using an air-actuated hammer, which is located inside the front jack of the CPT rig. The cone has a triaxial geophone, which recorded the shear wave signal generated by the air hammer.

The above parameters were recorded and viewed in real time using a laptop computer. Data is stored at the KTE office for up to 2 years for future analysis and reference. A complete set of baseline readings was taken prior to each sounding to determine temperature shifts and any zero load offsets. Monitoring base line readings ensures that the cone electronics are operating properly.

4. CONE PENETRATION TEST DATA & INTERPRETATION

The Cone Penetration Test data is presented in graphical form in the attached Appendix. These plots were generated using the CPeT-IT program. Penetration depths are referenced to ground surface. The soil behavior type on the CPT plots is derived from the attached CPT SBT plot (Robertson, "Interpretation of Cone Penetration Test…", 2009) and presents major soil lithologic changes. The stratigraphic interpretation is based on relationships between cone resistance (qc), sleeve friction (fs), and penetration pore pressure (u). The friction ratio (Rf), which is sleeve friction divided by cone resistance, is a calculated parameter that is used along with cone resistance to infer soil behavior type. Generally, cohesive soils (clays) have high friction ratios, low cone resistance and generate excess pore water pressures. Cohesionless soils (sands) have lower friction ratios, high cone bearing and generate little (or negative) excess pore water pressures.

The CPT data files have also been provided. These files can be imported in CPeT-IT (software by GeoLogismiki) and other programs to calculate various geotechnical parameters.

It should be noted that it is not always possible to clearly identify a soil type based on qc, fs and u. In these situations, experience, judgement and an assessment of the pore pressure data should be used to infer the soil behavior type.

If you have any questions regarding this information, please do not hesitate to call our office at (714) 901-7270.

Sincerely,

KEHOE TESTING & ENGINEERING

Steven P. Kehoe
President

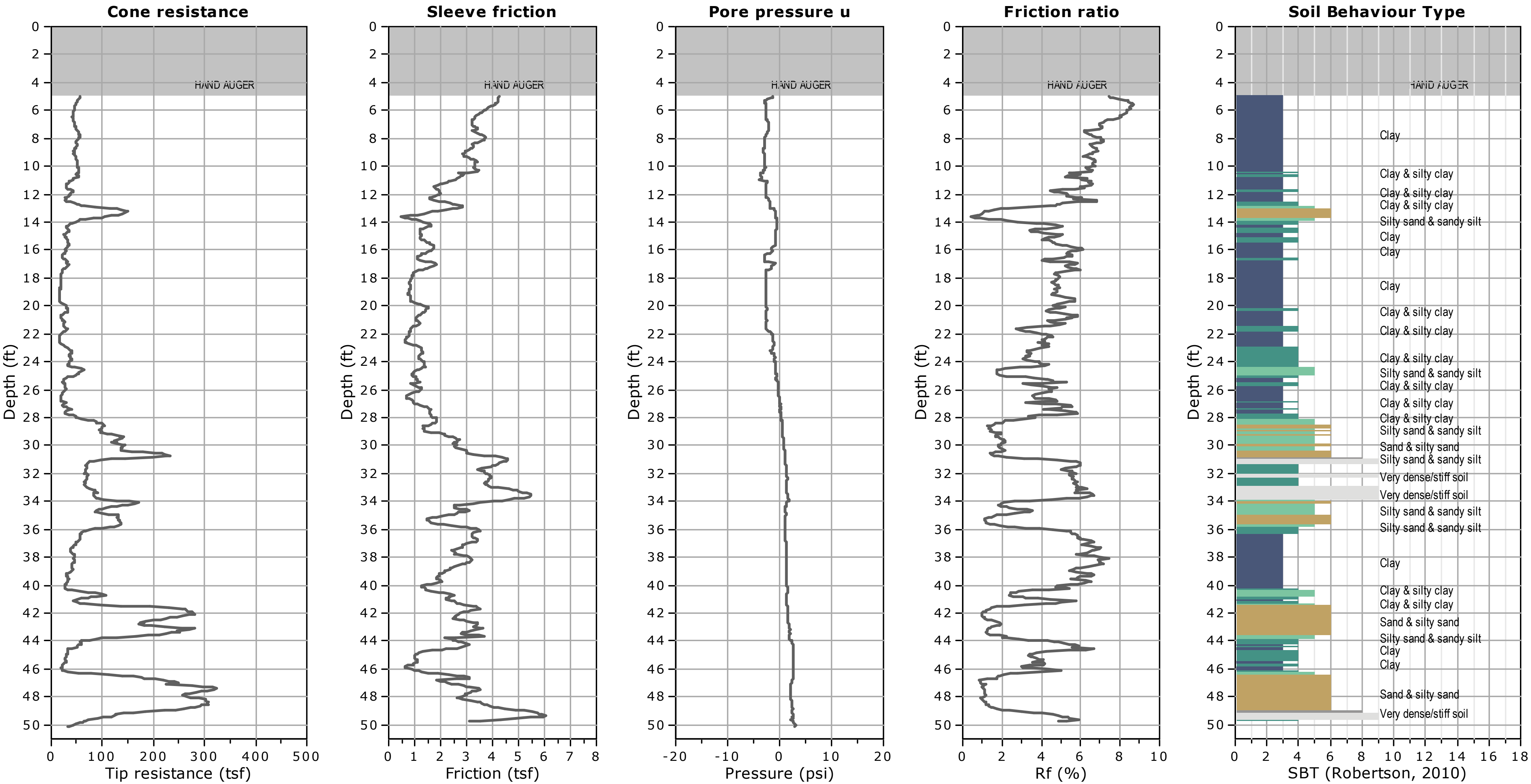
09/22/21-hh-3195

APPENDIX

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

Project: Leighton Consulting / Mckinley Elementary School
Location: Santa Monica, CA

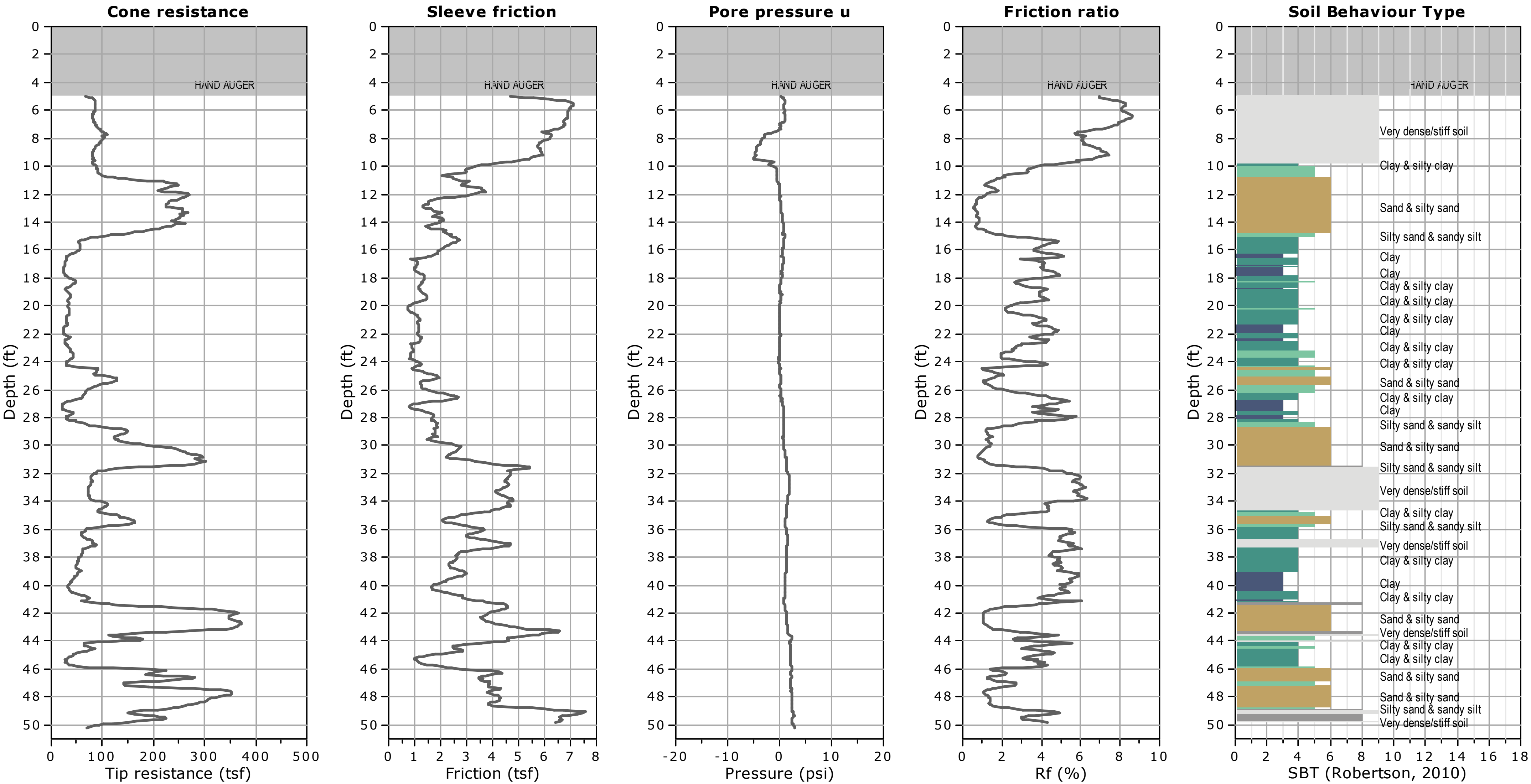
CPT-1
Total depth: 50.13 ft, Date: 9/16/2021



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

Project: Leighton Consulting / Mckinley Elementary School
Location: Santa Monica, CA

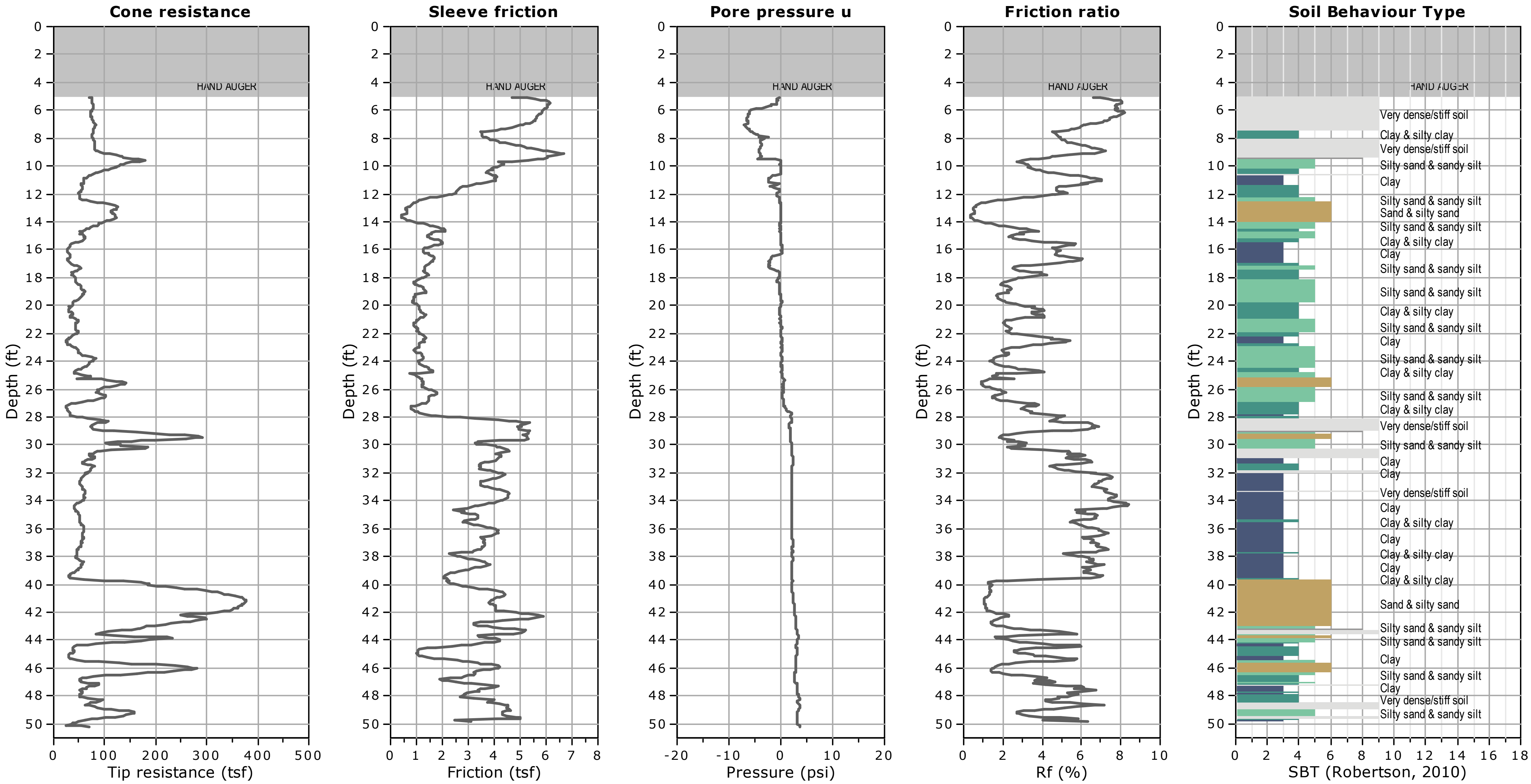
CPT-2
Total depth: 50.20 ft, Date: 9/16/2021



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

Project: Leighton Consulting / Mckinley Elementary School
Location: Santa Monica, CA

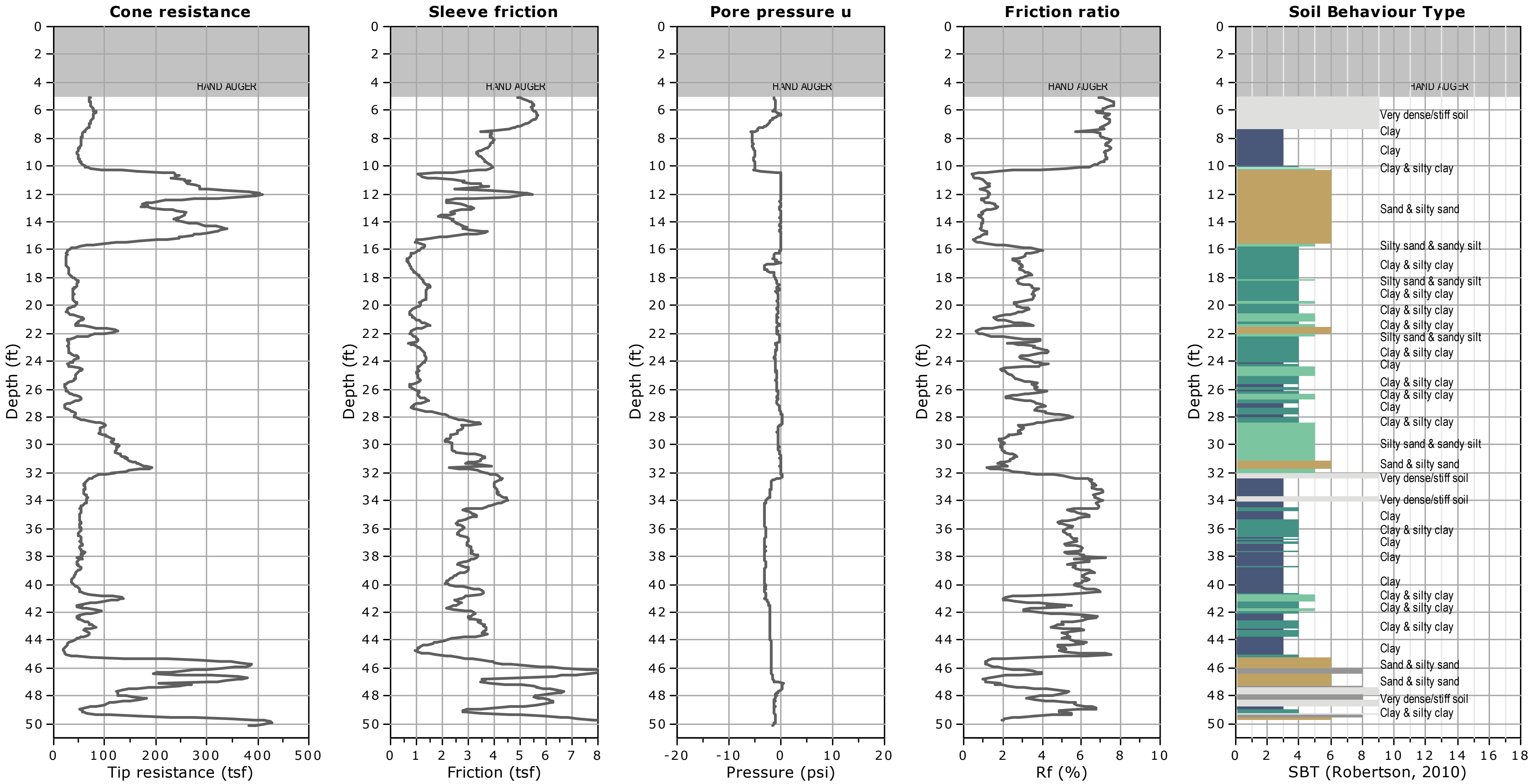
CPT-3
Total depth: 50.20 ft, Date: 9/16/2021



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

Project: **Leighton Consulting / Mckinley Elementary School**
Location: **Santa Monica, CA**

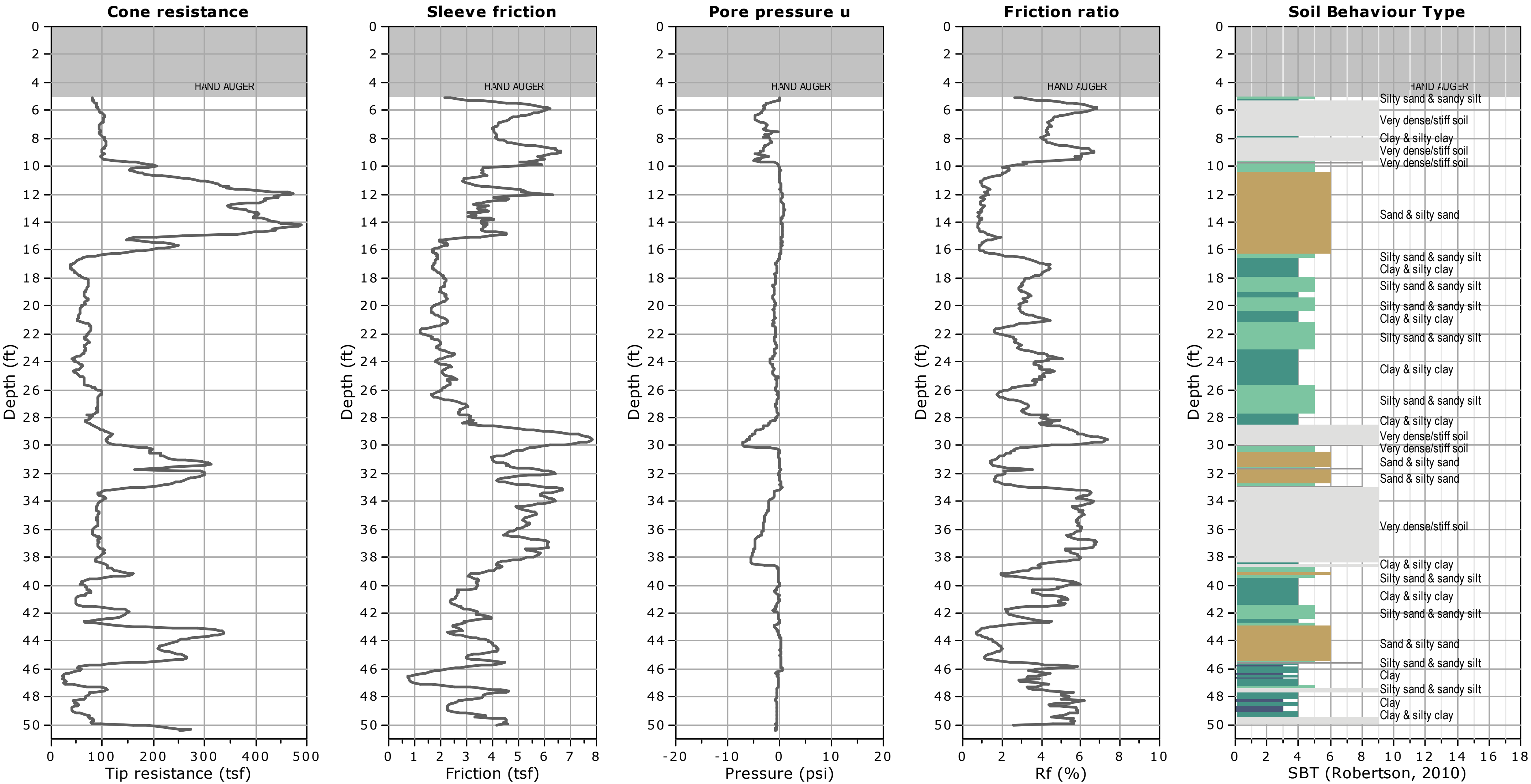
CPT-3A
Total depth: 50.14 ft, Date: 9/16/2021



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

Project: Leighton Consulting / Mckinley Elementary School **CPT-4**

Location: Santa Monica, CA Total depth: 50.40 ft, Date: 9/16/2021



Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
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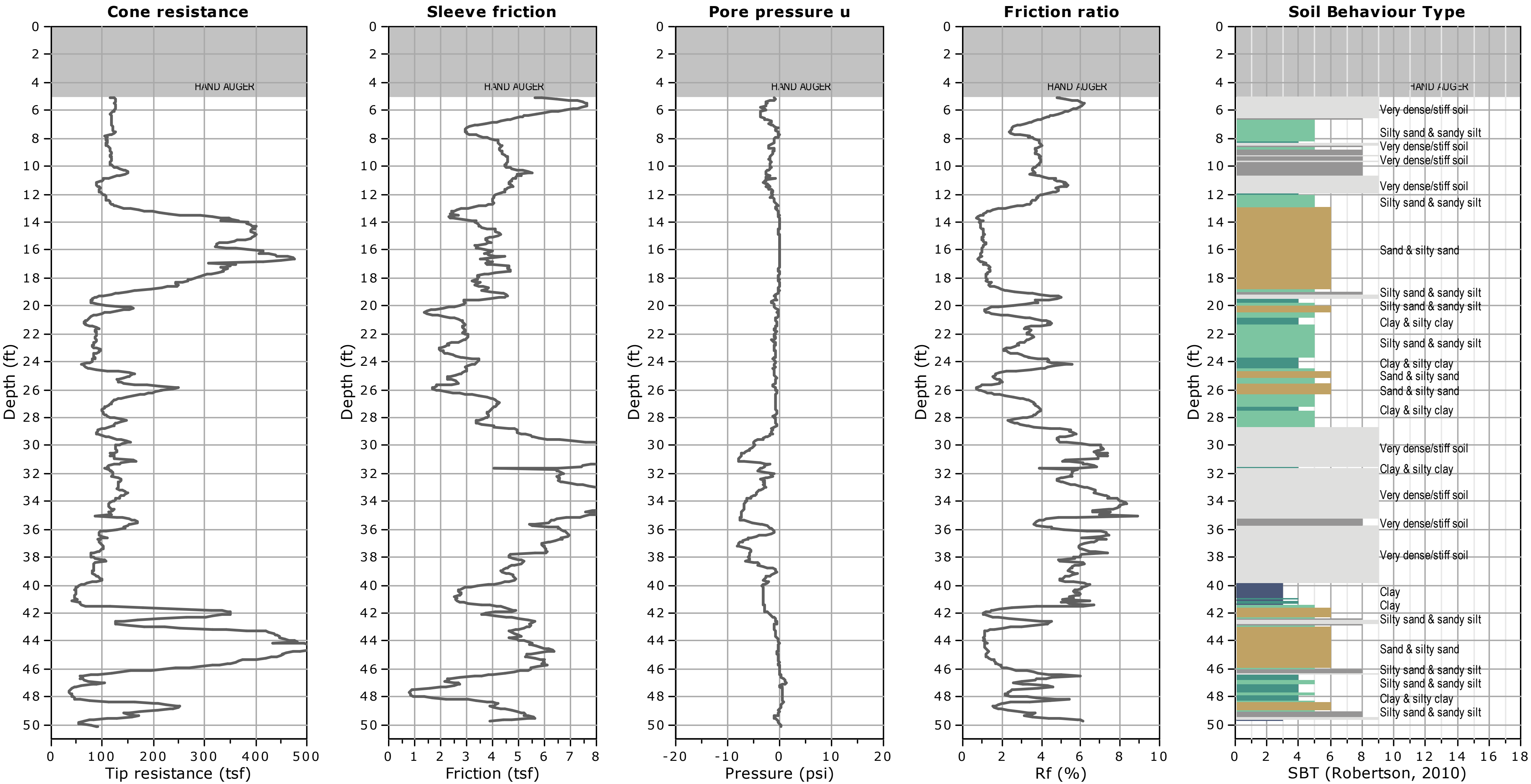
Project: Leighton Consulting / Mckinley Elementary School
Location: Santa Monica, CA

CPT-5
Total depth: 50.14 ft, Date: 9/16/2021



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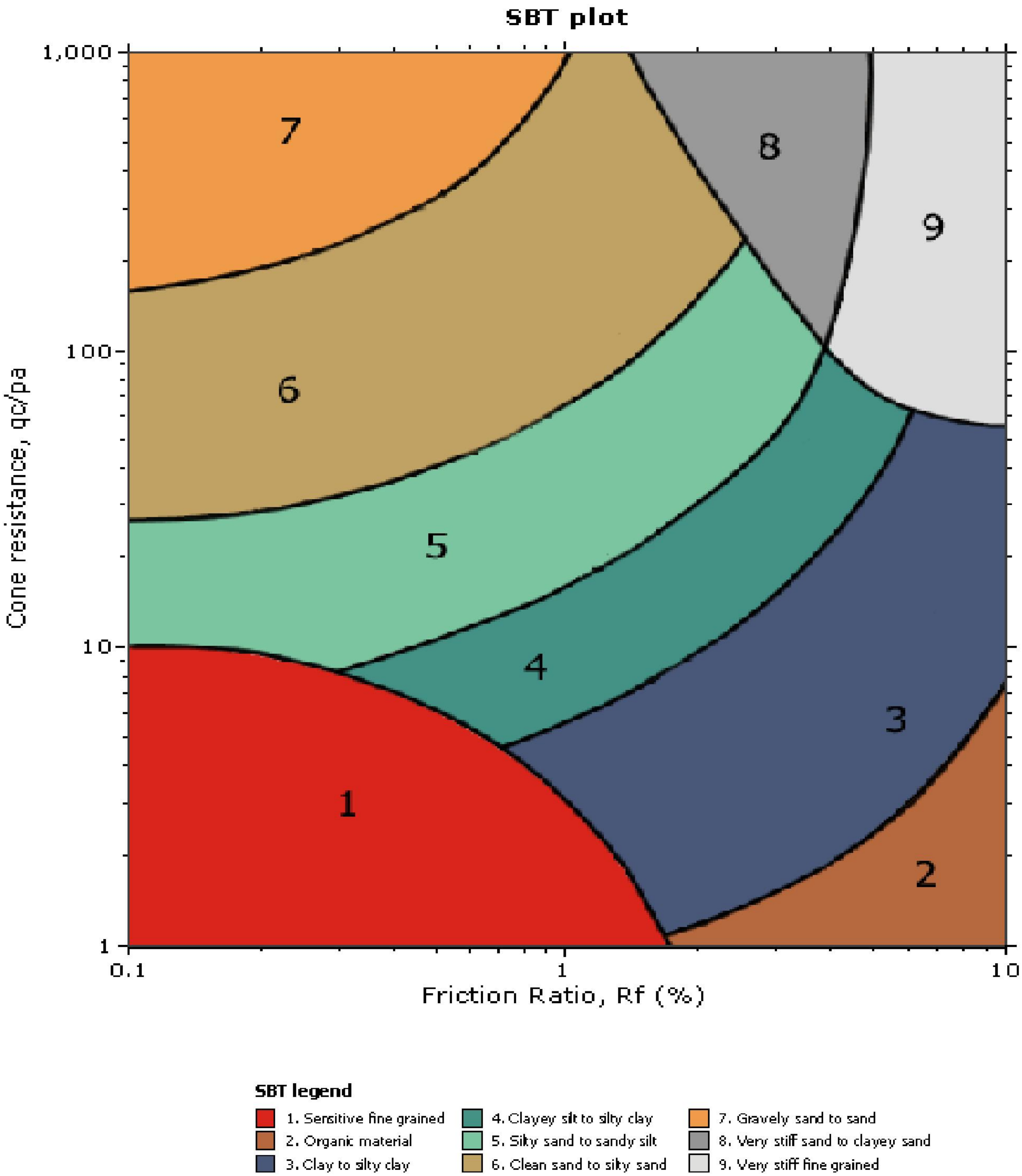
steve@kehoetesting.com
www.kehoetesting.com



Leighton Consulting
McKinley Elementary School
Santa Monica, 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-3	5.02	4.02	4.49	3.08	1458	
	10.01	9.01	9.23	8.00	1154	963
	15.06	14.06	14.20	12.72	1116	1053
	20.08	19.08	19.18	17.44	1100	1056
	25.20	24.20	24.28	22.88	1061	937
	30.02	29.02	29.09	25.68	1133	1717
	35.04	34.04	34.10	29.22	1167	1415
	40.03	39.03	39.08	32.96	1186	1332
	45.05	44.05	44.10	36.80	1198	1306
	50.03	49.03	49.07	40.44	1213	1367

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

Laboratory Test Results

APPENDIX B - GEOTECHNICAL LABORATORY TESTING

Our geotechnical laboratory testing program was directed toward a quantitative and qualitative evaluation of physical and mechanical properties of soils underlying this campus at proposed improvements, and to aid in verifying soil classification. This geotechnical testing was performed at our Irvine laboratory (DSA LEA 63).

Modified Proctor Compaction Curve: Laboratory modified Proctor compaction curves (ASTM D 1557) were established for bulk soil-samples to determine sample-specific modified Proctor laboratory maximum dry density and optimum moisture content. Results of these tests are presented on the following "*Modified Proctor Compaction Test*" sheets in this appendix.

Direct Shear Tests: Direct shear tests were performed, in general accordance with ASTM Test Method D 3080, on remolded soil samples remolded to 90% of the ASTM D 1557 laboratory maximum density. Remolded specimens were soaked for a minimum of 24 hours under a surcharge equal to the applied normal force during testing. After transfer of the sample to the shear box, and reloading the sample, pore pressures set up in the sample due to the transfer were allowed to dissipate for a period of approximately 1 hour prior to application of shearing force. These specimens were tested under various normal loads with a motor-driven, strain-controlled, direct-shear testing apparatus at a strain rate of 0.05 inches per minute (depending upon the soil type). Test results are presented on the *Direct Shear Test Results* sheets which follow in this appendix.

Consolidation: Consolidation tests on relatively undisturbed drive samples from our borings were performed in accordance with ASTM D 2435. Results are included in this appendix on the *One-Dimensional Consolidation Properties of Soils* sheets.

Corrosivity Tests: To evaluate corrosion potential of subsurface soils at the site, we tested a bulk sample collected during our subsurface exploration for pH, electrical resistivity (CTM 532/643), soluble sulfate content (CTM 417 Part II) and soluble chloride content (CTM 422) testing. Results of these tests are enclosed at the end of this appendix.

R-Value Tests: Selected samples were tested in accordance with DOT CA Test 301. The R-Value test measures the response of a compacted sample of soil or aggregate to a vertically applied pressure under specific conditions. This test is used by Caltrans for pavement design, replacing the California bearing ratio test. The R-value of a material is determined when the material is in a state of saturation such that water will be exuded from the compacted test specimen when a 16.8 kN load (2.07 MPa) is applied to test a series of specimens prepared at different moisture contents. R-Value is used in pavement design, with the thickness of each layer dependent on the R-value of the layer below and the expected level of traffic loading, expressed as a Traffic Index. Results of these tests are enclosed at the end of this appendix.

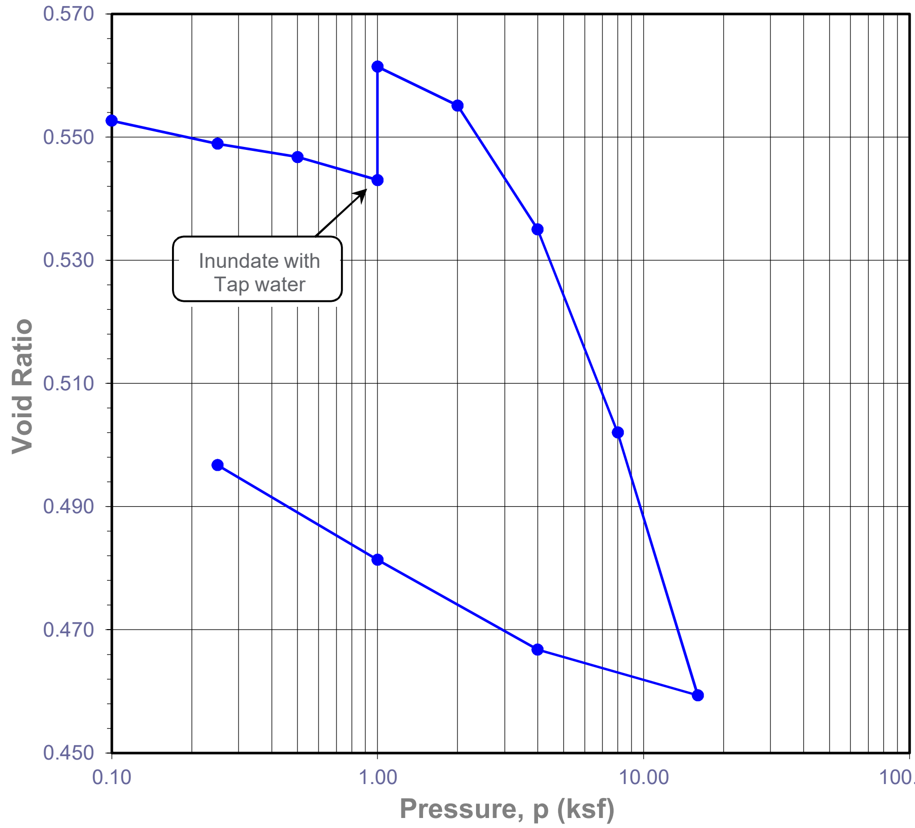
Expansion Tests: In accordance with ASTM D 4829 the specimen is compacted into a metal ring so that the degree of saturation is between 40 and 60 % and the specimen and the ring are placed in a consolidometer. A vertical confining pressure of 1 psi is applied to the specimen and then the specimen is inundated with distilled water. The deformation of the specimen is recorded for 24 hours or until the rate of deformation becomes less than 0.005 mm/hour. The Expansion Index, EI, is used to measure a basic index property of soil and therefore, the EI is comparable to other indices such as the liquid limit, plastic limit, and plasticity index of soils. Results of these tests are enclosed at the end of this appendix.

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name:	McKinley ES	Tested By: GB/YN	Date:	09/23/21
Project No.:	11428.036	Checked By: A. Santos	Date:	10/14/21
Boring No.:	LB-2	Depth (ft.): 7.5		
Sample No.:	BB-1	Sample Type:	90% Remold	
Soil Identification:	Brown lean clay (CL)			

Sample Diameter (in.)	2.415
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	193.07
Weight of Ring (g)	45.52
Height after consol. (in.)	0.9642
Before Test	
Wt.Wet Sample+Cont. (g)	211.85
Wt.of Dry Sample+Cont. (g)	194.83
Weight of Container (g)	64.01
Initial Moisture Content (%)	13.0
Initial Dry Density (pcf)	108.6
Initial Saturation (%)	64
Initial Vertical Reading (in.)	0.3292
After Test	
Wt.of Wet Sample+Cont. (g)	268.92
Wt. of Dry Sample+Cont. (g)	245.26
Weight of Container (g)	69.47
Final Moisture Content (%)	18.16
Final Dry Density (pcf)	112.4
Final Saturation (%)	98
Final Vertical Reading (in.)	0.2902
Specific Gravity (assumed)	2.70
Water Density (pcf)	62.43

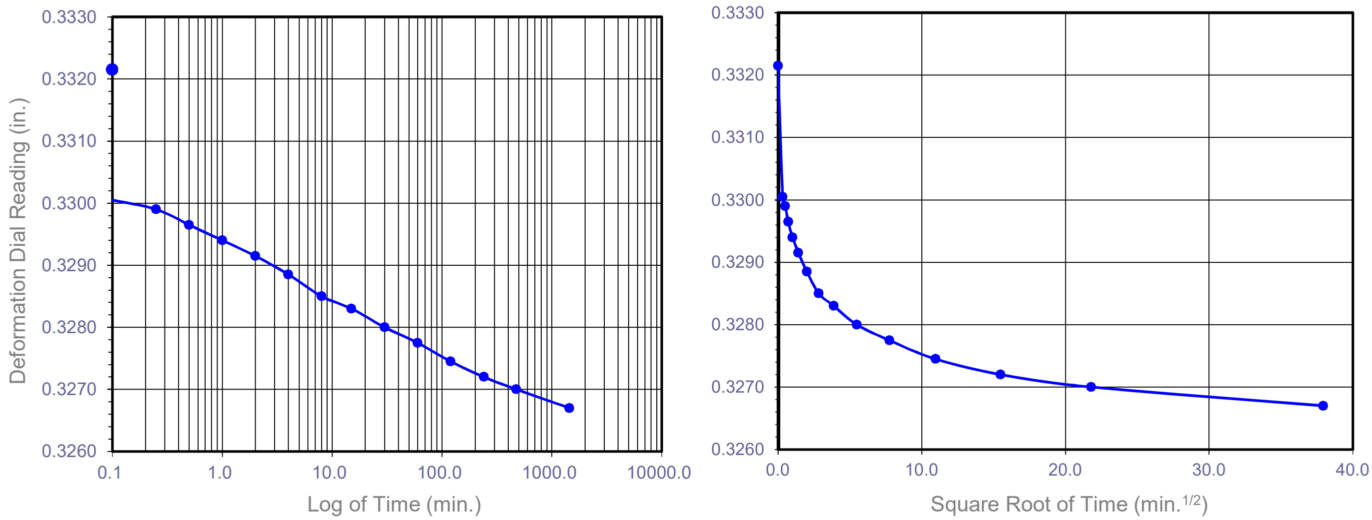


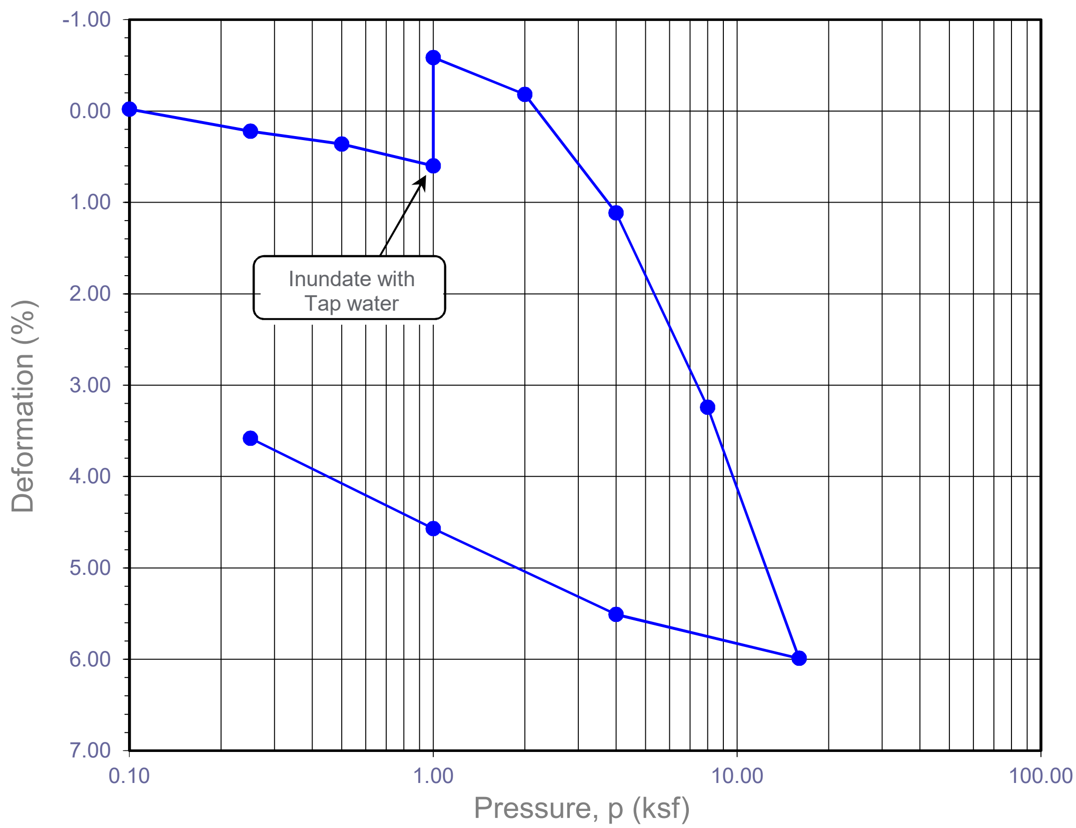
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.3294	1.0002	0.00	-0.02	0.553	-0.02
0.25	0.3261	0.9969	0.09	0.31	0.549	0.22
0.50	0.3238	0.9946	0.18	0.54	0.547	0.36
1.00	0.3203	0.9911	0.29	0.89	0.543	0.60
1.00	0.3322	1.0030	0.29	-0.30	0.561	-0.59
2.00	0.3267	0.9975	0.43	0.25	0.555	-0.18
4.00	0.3119	0.9827	0.62	1.74	0.535	1.12
8.00	0.2884	0.9592	0.84	4.08	0.502	3.24
16.00	0.2586	0.9294	1.07	7.06	0.459	5.99
4.00	0.2658	0.9366	0.83	6.34	0.467	5.51
1.00	0.2781	0.9489	0.54	5.11	0.481	4.57
0.25	0.2902	0.9610	0.32	3.90	0.497	3.58

Time Readings @ 2 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
9/27/21	11:20:00	0.0	0.0	0.3322
9/27/21	11:20:06	0.1	0.3	0.3301
9/27/21	11:20:15	0.2	0.5	0.3299
9/27/21	11:20:30	0.5	0.7	0.3297
9/27/21	11:21:00	1.0	1.0	0.3294
9/27/21	11:22:00	2.0	1.4	0.3292
9/27/21	11:24:00	4.0	2.0	0.3289
9/27/21	11:28:00	8.0	2.8	0.3285
9/27/21	11:35:00	15.0	3.9	0.3283
9/27/21	11:50:00	30.0	5.5	0.3280
9/27/21	12:20:00	60.0	7.7	0.3278
9/27/21	13:20:00	120.0	11.0	0.3275
9/27/21	15:20:00	240.0	15.5	0.3272
9/27/21	19:15:00	475.0	21.8	0.3270
9/28/21	11:20:00	1440.0	37.9	0.3267

Time Readings @ 2 ksf





Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-2	**BB-1**	**7.5**	**13.0**	**18.2**	**108.6**	**112.4**	**0.552**	**0.497**	**64**	**98**

Soil Identification: Brown lean clay (CL)

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 11428.036

McKinley ES

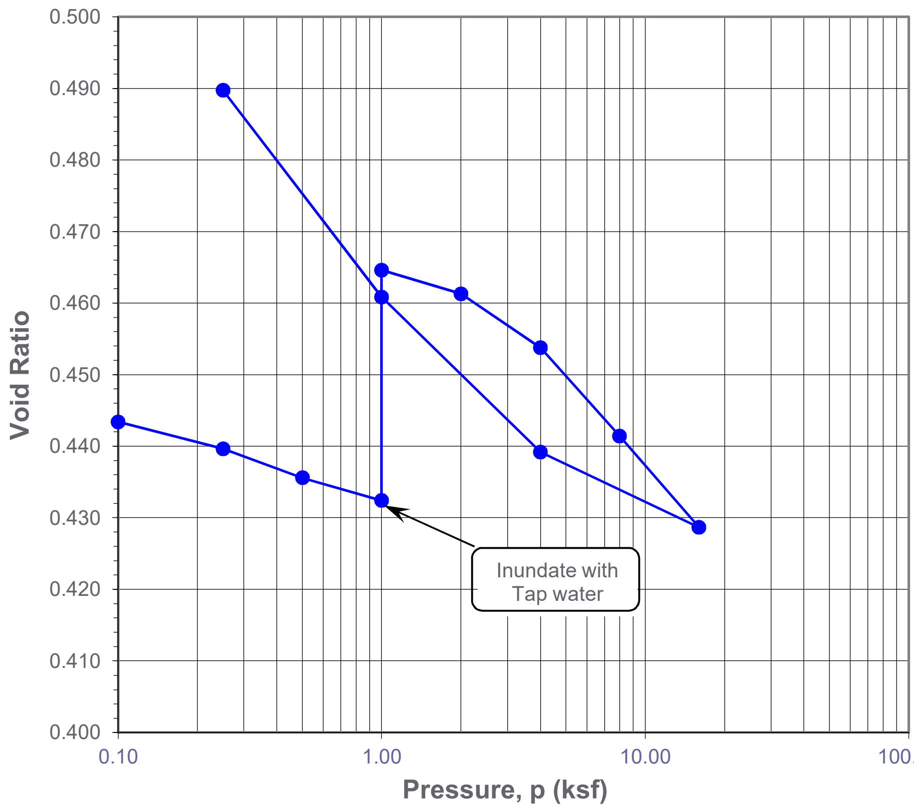
10-21

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name:	McKinley ES	Tested By: GB/YN	Date: 09/21/21
Project No.:	11428.036	Checked By: A. Santos	Date: 10/12/21
Boring No.:	LB-2	Depth (ft.): 5.0	
Sample No.:	R-1	Sample Type:	Ring
Soil Identification:	Light olive brown lean clay (CL)		

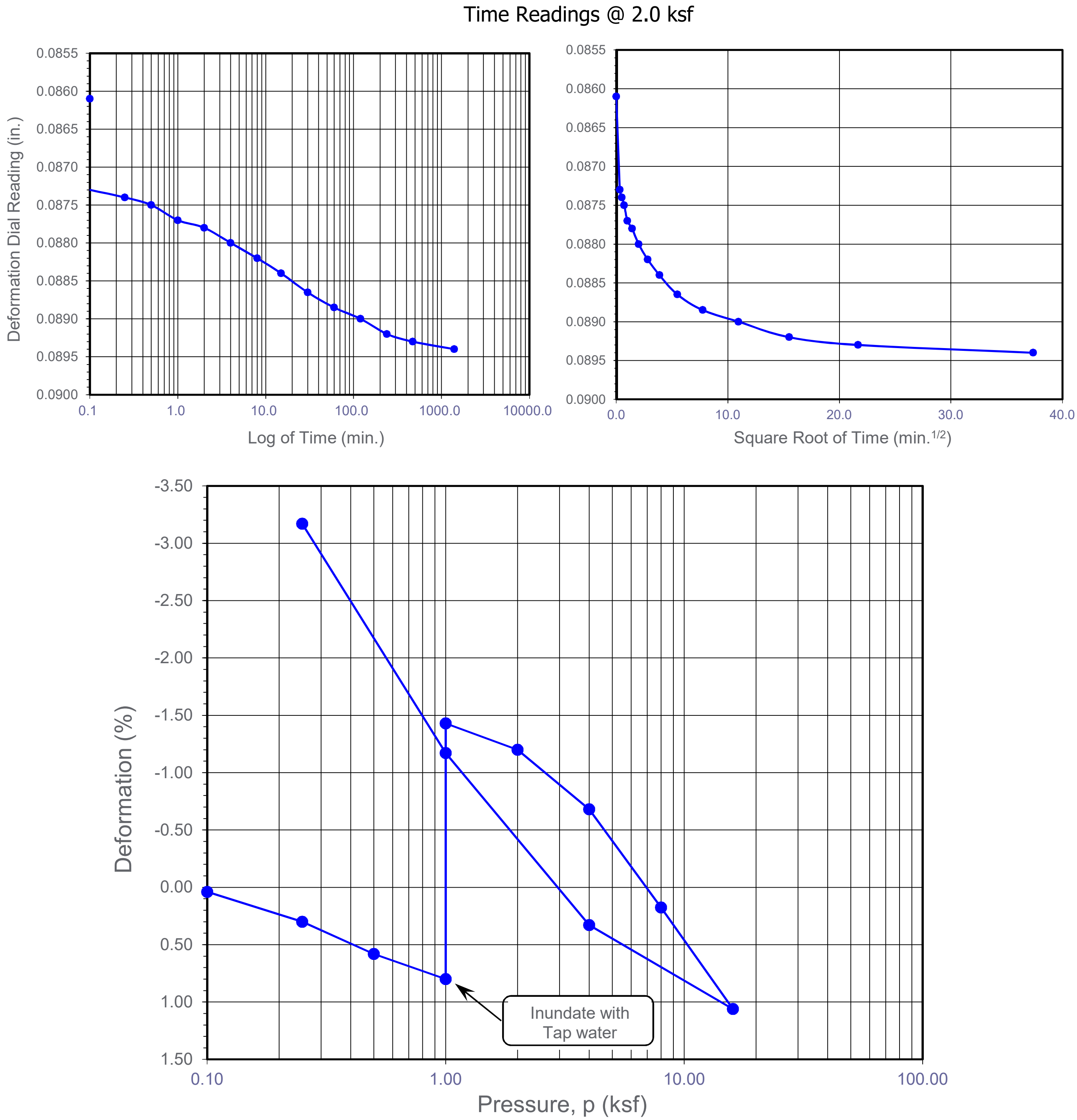
Sample Diameter (in.):	2.415
Sample Thickness (in.):	1.000
Weight of Sample + ring (g):	209.10
Weight of Ring (g):	44.68
Height after consol. (in.):	1.0317
Before Test	
Wt. of Wet Sample+Cont. (g):	217.70
Wt. of Dry Sample+Cont. (g):	196.76
Weight of Container (g):	57.24
Initial Moisture Content (%)	15.0
Initial Dry Density (pcf)	118.9
Initial Saturation (%):	93
Initial Vertical Reading (in.)	0.0986
After Test	
Wt. of Wet Sample+Cont. (g):	271.46
Wt. of Dry Sample+Cont. (g):	245.68
Weight of Container (g):	59.18
Final Moisture Content (%)	18.18
Final Dry Density (pcf):	114.3
Final Saturation (%):	100
Final Vertical Reading (in.)	0.0700
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	0.0990	0.9996	0.00	0.04	0.443	0.04
0.25	0.1022	0.9964	0.06	0.36	0.440	0.30
0.50	0.1055	0.9931	0.11	0.69	0.436	0.58
1.00	0.1084	0.9902	0.18	0.98	0.432	0.80
1.00	0.0861	1.0125	0.18	-1.25	0.465	-1.43
2.00	0.0894	1.0092	0.28	-0.92	0.461	-1.20
4.00	0.0957	1.0029	0.39	-0.29	0.454	-0.68
8.00	0.1056	0.9931	0.52	0.70	0.441	0.18
16.00	0.1157	0.9829	0.65	1.71	0.429	1.06
4.00	0.1072	0.9914	0.53	0.86	0.439	0.33
1.00	0.0911	1.0075	0.42	-0.75	0.461	-1.17
0.25	0.0700	1.0286	0.31	-2.86	0.490	-3.17

Time Readings @ 2.0 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
9/23/21	11:25:00	0.0	0.0	0.0861
9/23/21	11:25:06	0.1	0.3	0.0873
9/23/21	11:25:15	0.2	0.5	0.0874
9/23/21	11:25:30	0.5	0.7	0.0875
9/23/21	11:26:00	1.0	1.0	0.0877
9/23/21	11:27:00	2.0	1.4	0.0878
9/23/21	11:29:00	4.0	2.0	0.0880
9/23/21	11:33:00	8.0	2.8	0.0882
9/23/21	11:40:00	15.0	3.9	0.0884
9/23/21	11:55:00	30.0	5.5	0.0887
9/23/21	12:25:00	60.0	7.7	0.0889
9/23/21	13:25:00	120.0	11.0	0.0890
9/23/21	15:25:00	240.0	15.5	0.0892
9/23/21	19:15:00	470.0	21.7	0.0893
9/24/21	10:42:00	1397.0	37.4	0.0894



Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-2	**R-1**	**5**	**15.0**	**18.2**	**118.9**	**114.3**	**0.444**	**0.490**	**93**	**100**

Soil Identification: Light olive brown lean clay (CL)

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 11428.036

McKinley ES

10-21

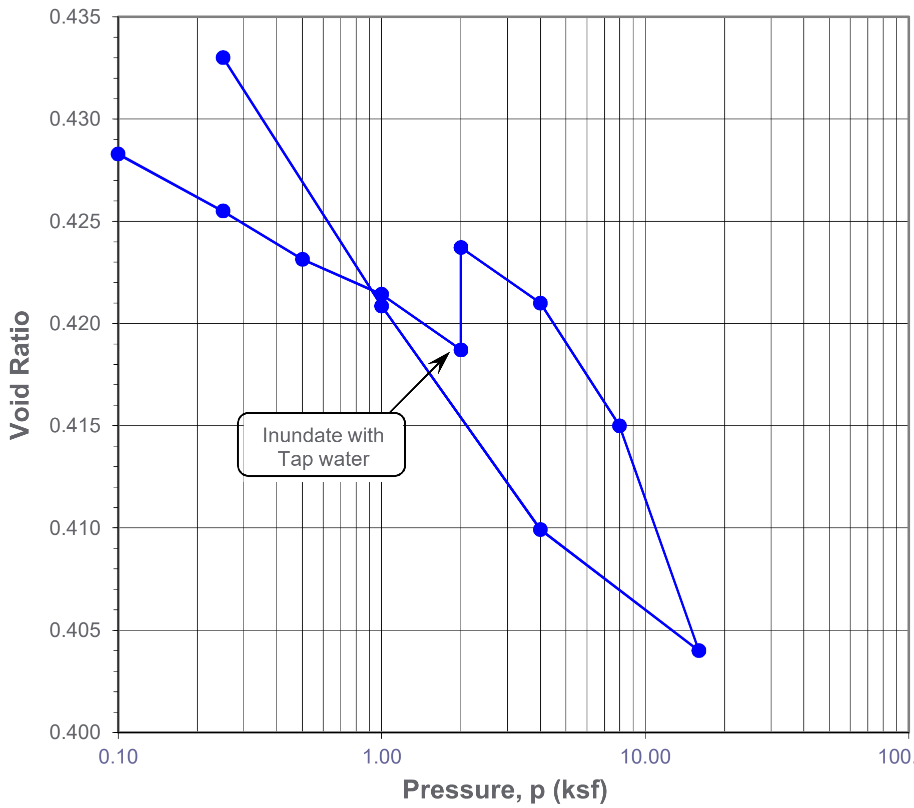
ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name: McKinley ES
Project No.: 11428.036
Boring No.: LB-2
Sample No.: R-2
Soil Identification: Brown lean clay (CL)

Tested By: GB/YN Date: 09/23/21
Checked By: A. Santos Date: 10/15/21
Depth (ft.): 7.5
Sample Type: Ring

Sample Diameter (in.):	2.415
Sample Thickness (in.):	1.000
Weight of Sample + ring (g):	206.35
Weight of Ring (g):	45.54
Height after consol. (in.):	1.0031
Before Test	
Wt. of Wet Sample+Cont. (g):	178.96
Wt. of Dry Sample+Cont. (g):	164.62
Weight of Container (g):	53.69
Initial Moisture Content (%)	12.9
Initial Dry Density (pcf)	118.4
Initial Saturation (%):	82
Initial Vertical Reading (in.)	0.1311
After Test	
Wt. of Wet Sample+Cont. (g):	270.78
Wt. of Dry Sample+Cont. (g):	249.03
Weight of Container (g):	61.74
Final Moisture Content (%)	15.34
Final Dry Density (pcf):	117.5
Final Saturation (%):	95
Final Vertical Reading (in.)	0.1313
Specific Gravity (assumed):	2.71
Water Density (pcf):	62.43

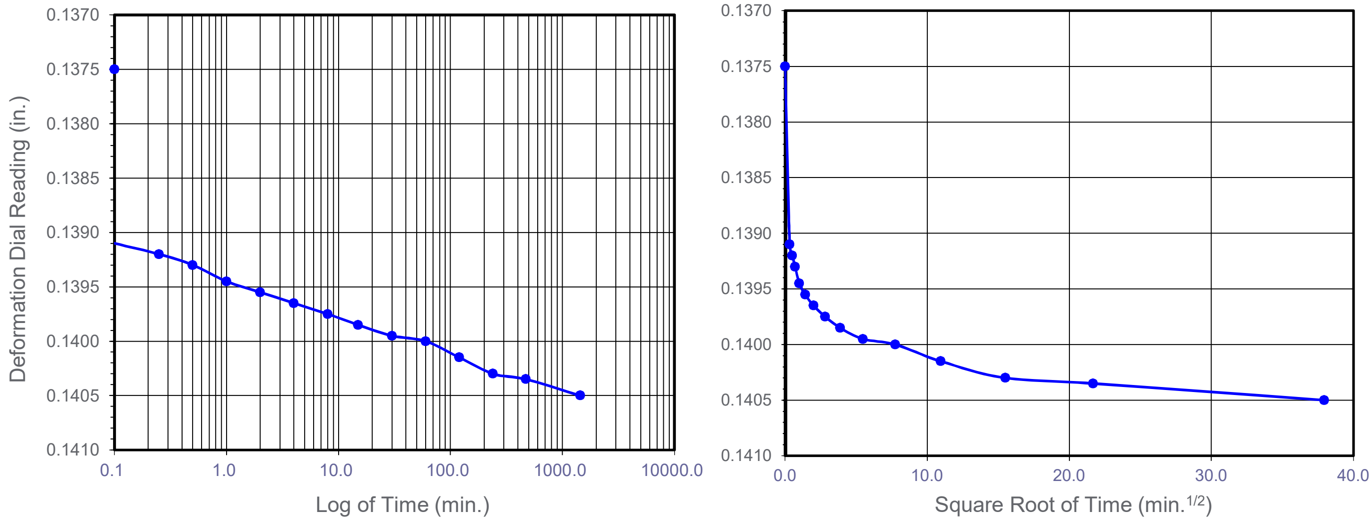


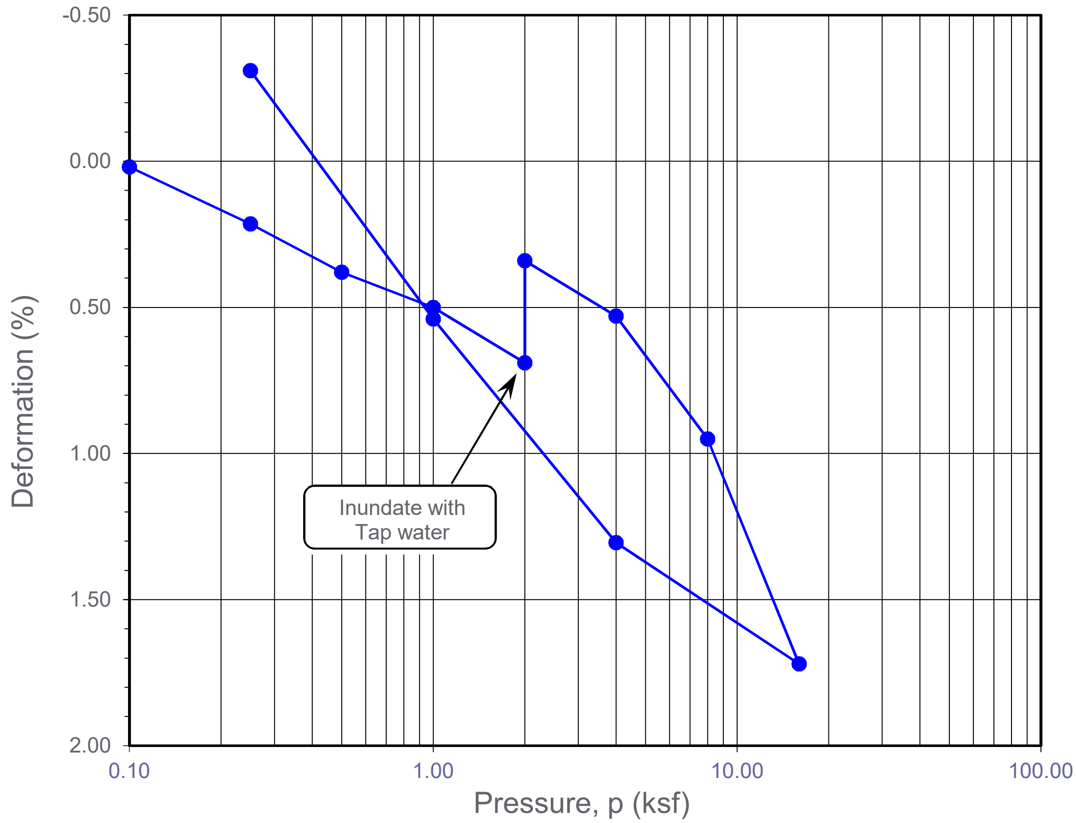
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deforma-tion (%)
0.10	0.1313	0.9998	0.00	0.02	0.428	0.02
0.25	0.1337	0.9975	0.04	0.26	0.426	0.22
0.50	0.1358	0.9953	0.09	0.47	0.423	0.38
1.00	0.1380	0.9931	0.19	0.69	0.421	0.50
2.00	0.1410	0.9901	0.30	0.99	0.419	0.69
2.00	0.1375	0.9936	0.30	0.64	0.424	0.34
4.00	0.1405	0.9906	0.41	0.94	0.421	0.53
8.00	0.1461	0.9850	0.55	1.50	0.415	0.95
16.00	0.1555	0.9756	0.72	2.44	0.404	1.72
4.00	0.1498	0.9814	0.56	1.87	0.410	1.31
1.00	0.1409	0.9902	0.44	0.98	0.421	0.54
0.25	0.1313	0.9998	0.33	0.02	0.433	-0.31

Time Readings @ 4.0 ksf

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)
9/27/21	11:25:00	0.0	0.0	0.1375
9/27/21	11:25:06	0.1	0.3	0.1391
9/27/21	11:25:15	0.2	0.5	0.1392
9/27/21	11:25:30	0.5	0.7	0.1393
9/27/21	11:26:00	1.0	1.0	0.1395
9/27/21	11:27:00	2.0	1.4	0.1396
9/27/21	11:29:00	4.0	2.0	0.1397
9/27/21	11:33:00	8.0	2.8	0.1398
9/27/21	11:40:00	15.0	3.9	0.1399
9/27/21	11:55:00	30.0	5.5	0.1400
9/27/21	12:25:00	60.0	7.7	0.1400
9/27/21	13:25:00	120.0	11.0	0.1402
9/27/21	15:25:00	240.0	15.5	0.1403
9/27/21	19:15:00	470.0	21.7	0.1404
9/28/21	11:25:00	1440.0	37.9	0.1405

Time Readings @ 4.0 ksf





Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-2	**R-2**	**7.5**	**12.9**	**15.3**	**118.4**	**117.5**	**0.429**	**0.433**	**82**	**95**

Soil Identification: Brown lean clay (CL)

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 11428.036

McKinley ES

10-21

TESTS for SULFATE CONTENT CHLORIDE CONTENT and pH of SOILS

Project Name: McKinley ES | Tested By : O. Figueroa | Date: 09/23/21

Project No. : 11428.036 | Checked By: A. Santos | Date: 10/13/21

Boring No.	LB-2			
Sample No.	BB-1			
Sample Depth (ft)	0-5			
Soil Identification:	Brown (CL)			
Wet Weight of Soil + Container (g)	0.00			
Dry Weight of Soil + Container (g)	0.00			
Weight of Container (g)	1.00			
Moisture Content (%)	0.00			
Weight of Soaked Soil (g)	100.30			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	303			
Crucible No.	8			
Furnace Temperature (°C)	860			
Time In / Time Out	9:45/10:30			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	20.3962			
Wt. of Crucible (g)	20.3919			
Wt. of Residue (g) (A)	0.0043			
PPM of Sulfate (A) x 41150	176.95			
PPM of Sulfate, Dry Weight Basis	**177**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	30			
ml of AgNO3 Soln. Used in Titration (C)	1.0			
PPM of Chloride (C -0.2) * 100 * 30 / B	80			
PPM of Chloride, Dry Wt. Basis	**80**			

pH TEST, DOT California Test 643

pH Value	**8.47**			
Temperature °C	20.7			

SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name:	McKinley ES	Tested By :	A. Willoughby	Date:	09/27/21
Project No. :	11428.036	Checked By:	A. Santos	Date:	10/13/21
Boring No.:	LB-2	Depth (ft.) :	0-5		
Sample No. :	BB-1				

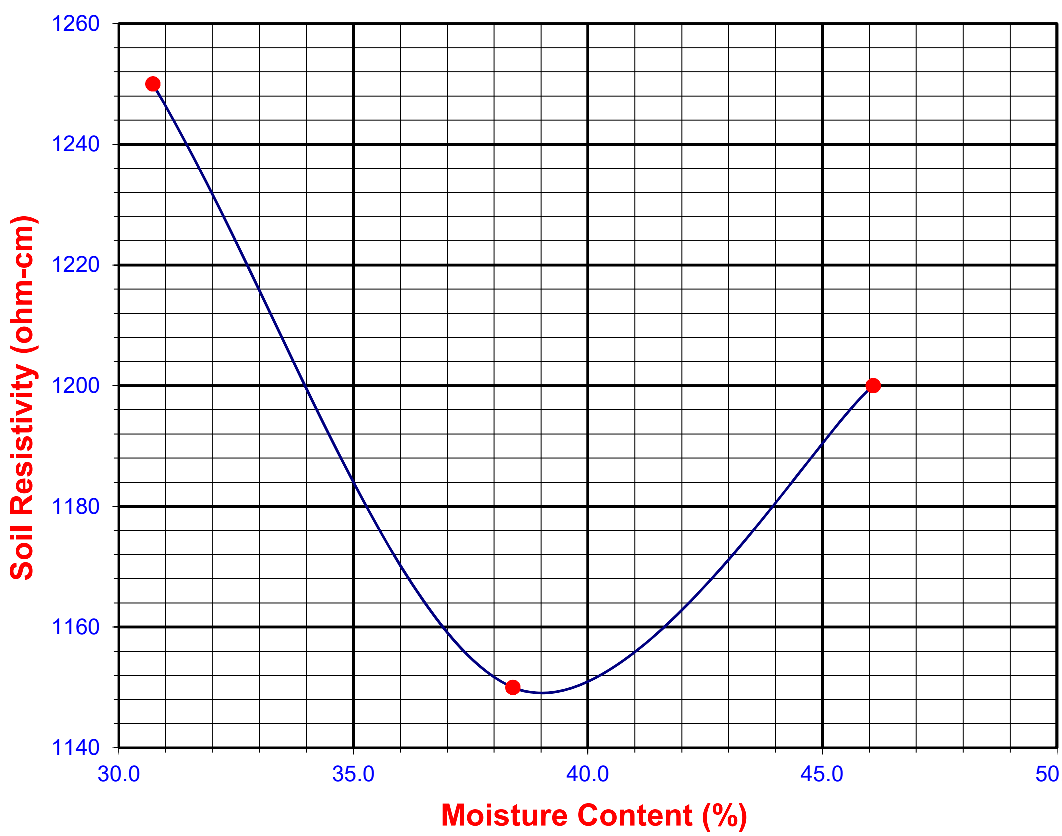
Soil Identification:* Brown (CL)

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

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	40	30.72	1250	1250
2	50	38.40	1150	1150
3	60	46.08	1200	1200
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)	130.20
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	
1149	**39.0**	**177**	**80**	**8.47**	**20.7**

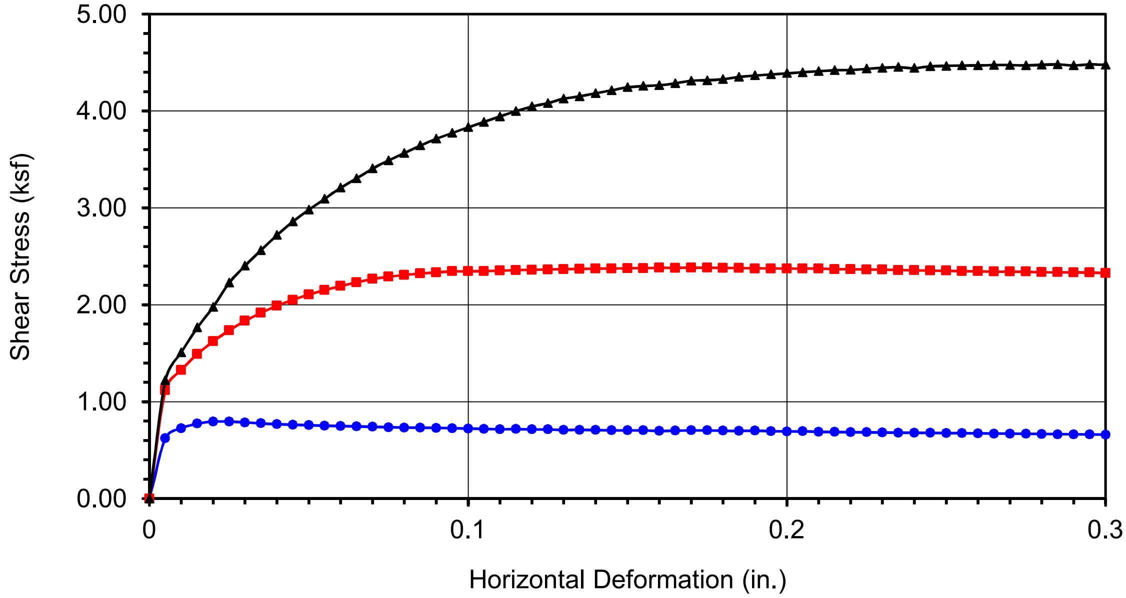


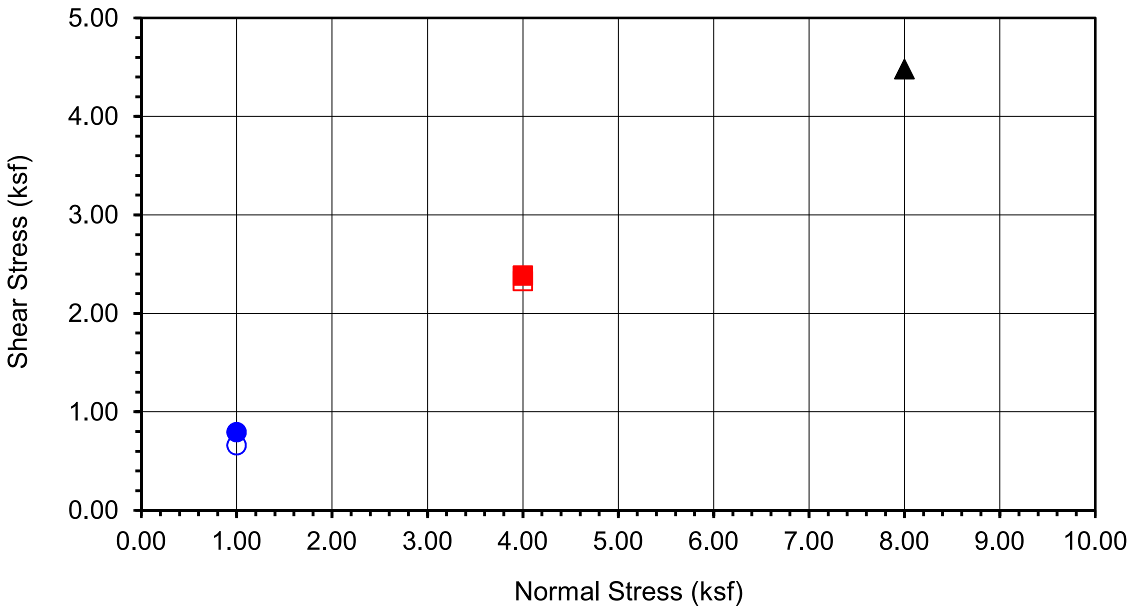
DIRECT SHEAR TEST

Consolidated Drained - ASTM D 3080

Project Name:	McKinley ES	Tested By:	G. Bathala	Date:	10/04/21
Project No.:	11428.036	Checked By:	A. Santos	Date:	10/14/21
Boring No.:	LB-2	Sample Type:	90% Remold		
Sample No.:	BB-1	Depth (ft.):	0-5		
Soil Identification:	Brown lean clay (CL)				

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	192.51	193.12	193.84
Weight of Ring(gm):	44.90	45.38	45.61
Before Shearing			
Weight of Wet Sample+Cont.(gm):	211.85	211.85	211.85
Weight of Dry Sample+Cont.(gm):	194.83	194.83	194.83
Weight of Container(gm):	64.01	64.01	64.01
Vertical Rdg.(in): Initial	0.2624	0.2650	0.0000
Vertical Rdg.(in): Final	0.2581	0.2819	-0.0535
After Shearing			
Weight of Wet Sample+Cont.(gm):	221.38	219.37	208.46
Weight of Dry Sample+Cont.(gm):	195.83	196.02	186.25
Weight of Container(gm):	65.87	66.85	56.65
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-2**
Sample No.	**BB-1**
Depth (ft)	**0-5**

Sample Type:

90% Remold

Soil Identification:

Brown lean clay (CL)

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 0.795	■ 2.383	▲ 4.480
Shear Stress @ End of Test (ksf)	○ 0.660	□ 2.326	△ 4.477
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 (%)	13.01	13.01	13.01
Dry Density (pcf)	108.6	108.7	109.1
Saturation (%)	63.7	63.8	64.4
Soil Height Before Shearing (in.)	1.0043	0.9831	0.9465
Final Moisture Content (%)	19.7	18.1	17.1

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DIRECT SHEAR TEST RESULTS

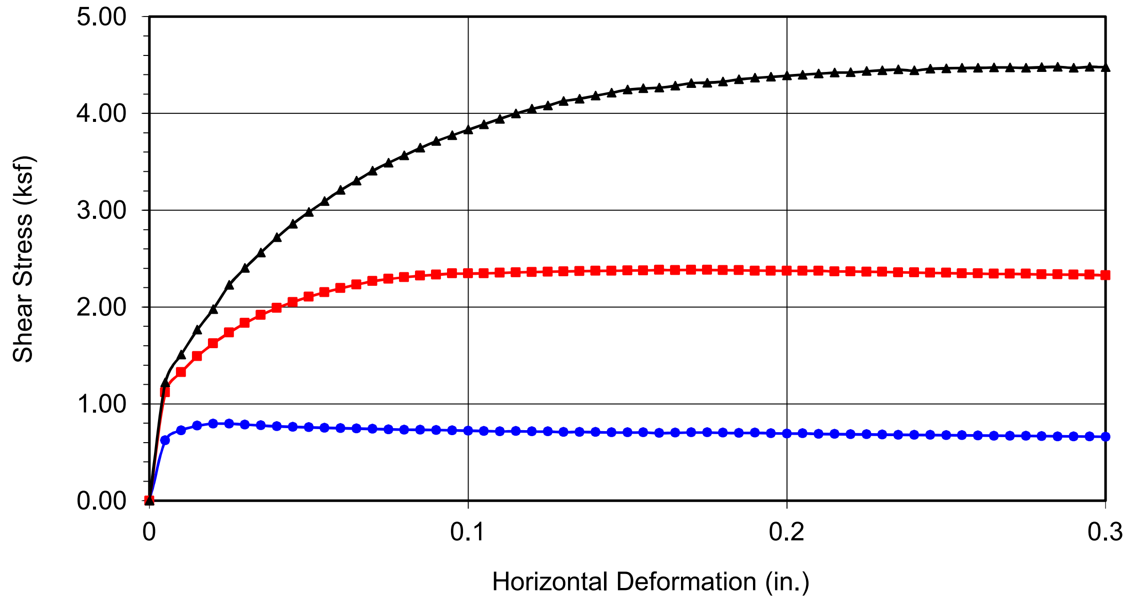
Consolidated Drained - ASTM D 3080

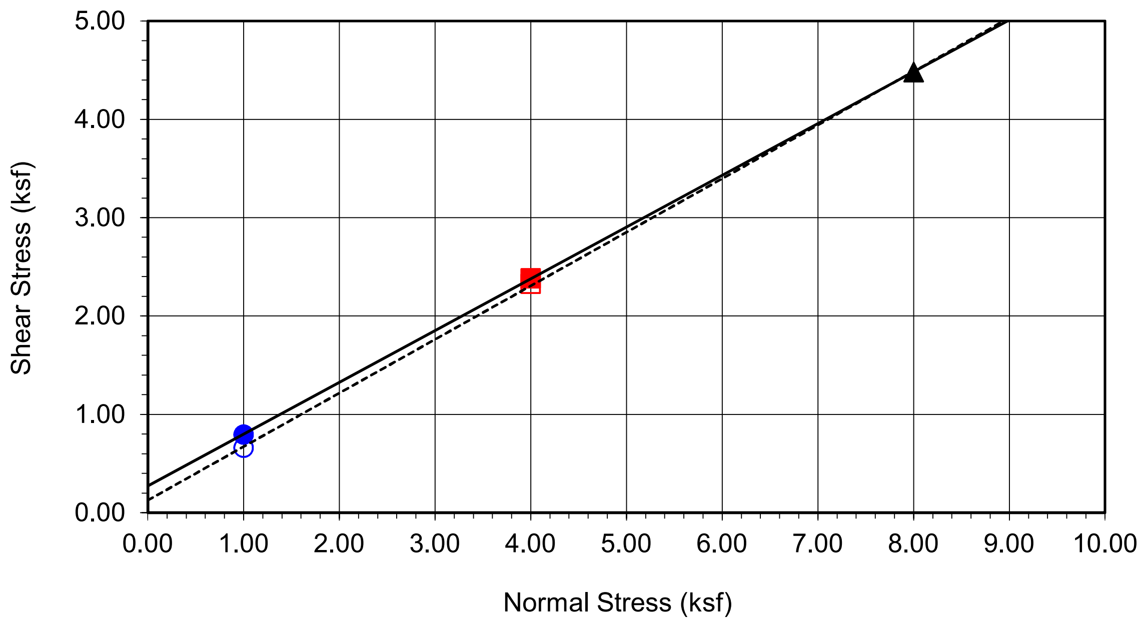
Project No.: 11428.036

McKinley ES

10-21

DS LB-2, BB-1 @ 0-5





Boring No.	**LB-2**
Sample No.	**BB-1**
Depth (ft)	**0-5**
Sample Type:	90% Remold
Soil Identification: Brown lean clay (CL)	

Strength Parameters

	C (psf)	ϕ (°)
Peak	272	28
Ultimate	127	29

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 0.795	■ 2.383	▲ 4.480
Shear Stress @ End of Test (ksf)	○ 0.660	□ 2.326	△ 4.477
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 (%)	13.01	13.01	13.01
Dry Density (pcf)	108.6	108.7	109.1
Saturation (%)	63.7	63.8	64.4
Soil Height Before Shearing (in.)	1.0043	0.9831	0.9465
Final Moisture Content (%)	19.7	18.1	17.1

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DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 11428.036

McKinley ES

10-21

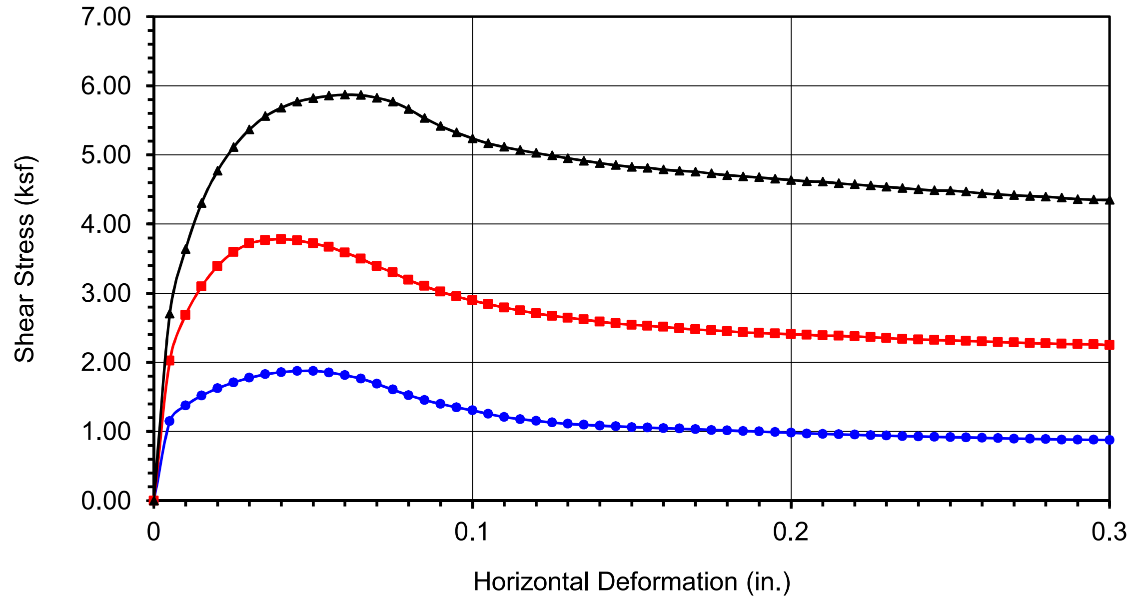
DIRECT SHEAR TEST

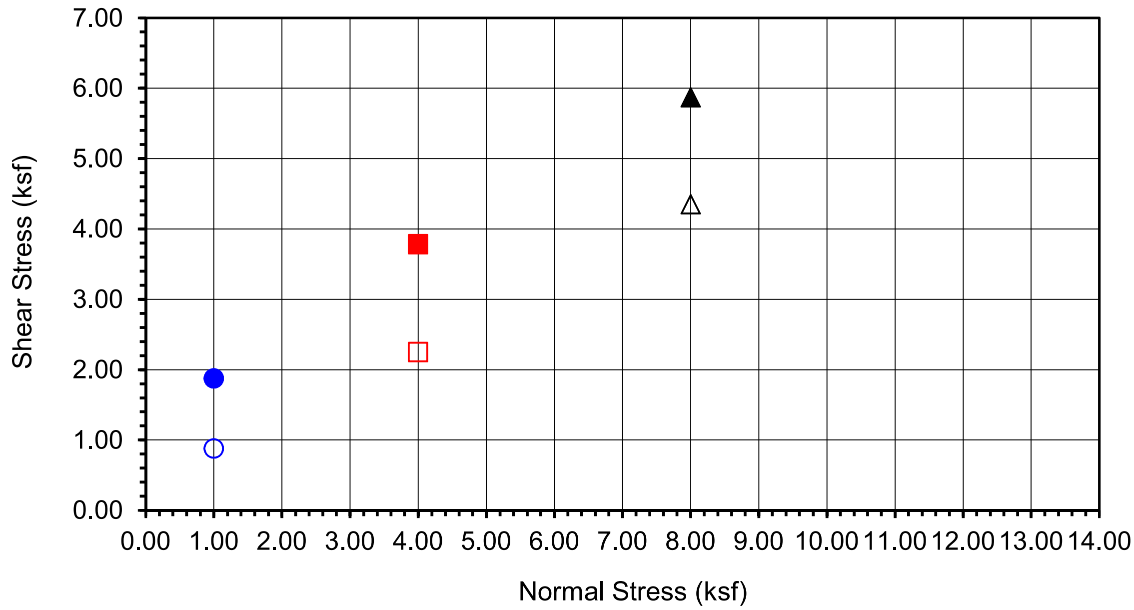
Consolidated Drained - ASTM D 3080

Project Name:	McKinley ES	Tested By:	G. Bathala	Date:	10/05/21
Project No.:	11428.036	Checked By:	A. Santos	Date:	10/15/21
Boring No.:	LB-2	Sample Type:	Ring		
Sample No.:	R-1	Depth (ft.):	5.0		

Soil Identification: Light olive brown lean clay (CL)

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	204.31	209.44	206.42
Weight of Ring(gm):	43.41	45.98	42.83
Before Shearing			
Weight of Wet Sample+Cont.(gm):	217.70	217.70	217.70
Weight of Dry Sample+Cont.(gm):	196.76	196.76	196.76
Weight of Container(gm):	57.24	57.24	57.24
Vertical Rdg.(in): Initial	0.0000	0.2623	0.2639
Vertical Rdg.(in): Final	0.0133	0.2683	0.2786
After Shearing			
Weight of Wet Sample+Cont.(gm):	214.82	203.36	230.46
Weight of Dry Sample+Cont.(gm):	188.91	179.07	206.96
Weight of Container(gm):	51.14	38.50	65.86
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-2**
Sample No.	**R-1**
Depth (ft)	**5**
Sample Type: Ring	
Soil Identification: Light olive brown lean clay (CL)	

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 1.877	■ 3.782	▲ 5.869
Shear Stress @ End of Test (ksf)	○ 0.877	□ 2.248	△ 4.348
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 (%)	15.01	15.01	15.01
Dry Density (pcf)	116.4	118.2	118.3
Saturation (%)	90.3	95.1	95.4
Soil Height Before Shearing (in.)	1.0133	0.9940	0.9853
Final Moisture Content (%)	18.8	17.3	16.7

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DIRECT SHEAR TEST RESULTS

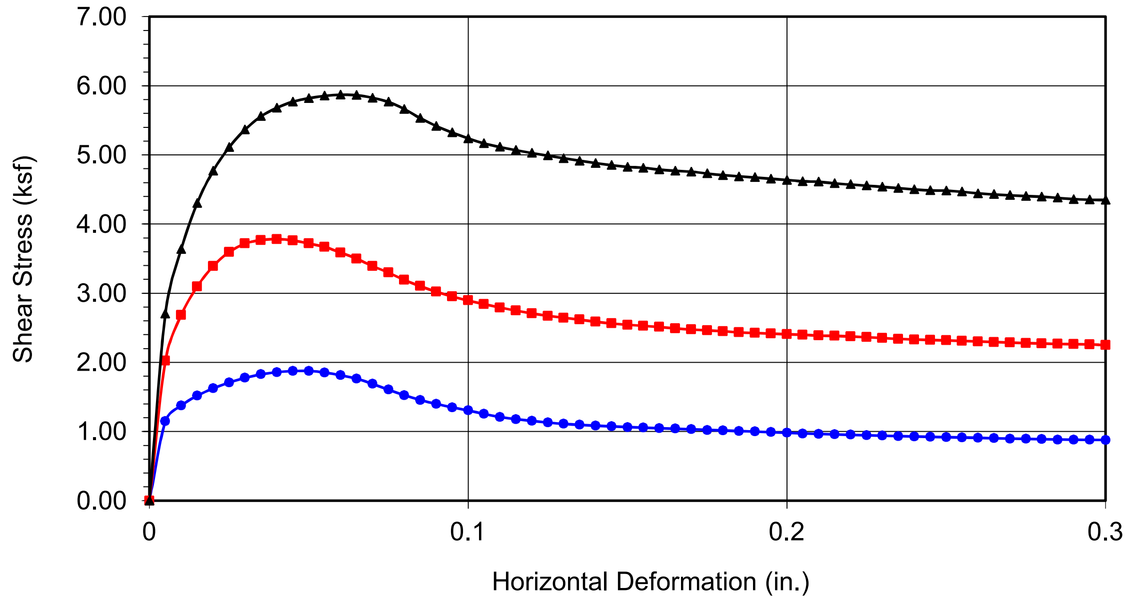
Consolidated Drained - ASTM D 3080

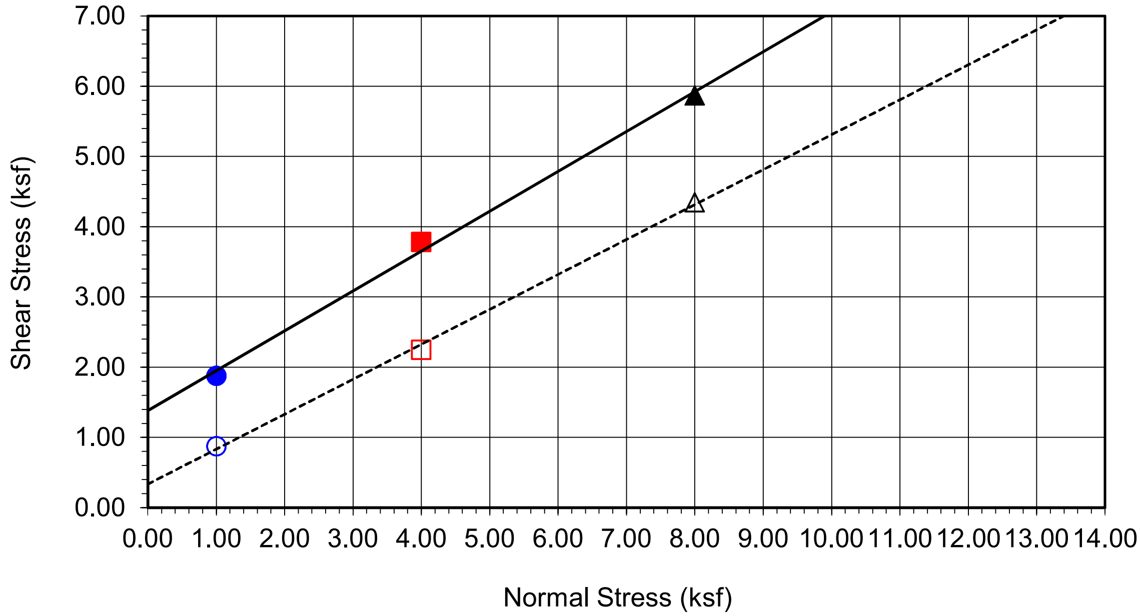
Project No.: 11428.036

McKinley ES

10-21

DS LB-2, R-1 @ 5





Boring No.	**LB-2**
Sample No.	**R-1**
Depth (ft)	**5**
Sample Type:	Ring
Soil Identification: Light olive brown lean clay (CL)	

Strength Parameters

	C (psf)	ϕ (°)
Peak	1383	30
Ultimate	335	26

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 1.877	■ 3.782	▲ 5.869
Shear Stress @ End of Test (ksf)	○ 0.877	□ 2.248	△ 4.348
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 (%)	15.01	15.01	15.01
Dry Density (pcf)	116.4	118.2	118.3
Saturation (%)	90.3	95.1	95.4
Soil Height Before Shearing (in.)	1.0133	0.9940	0.9853
Final Moisture Content (%)	18.8	17.3	16.7

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DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 11428.036

McKinley ES

10-21

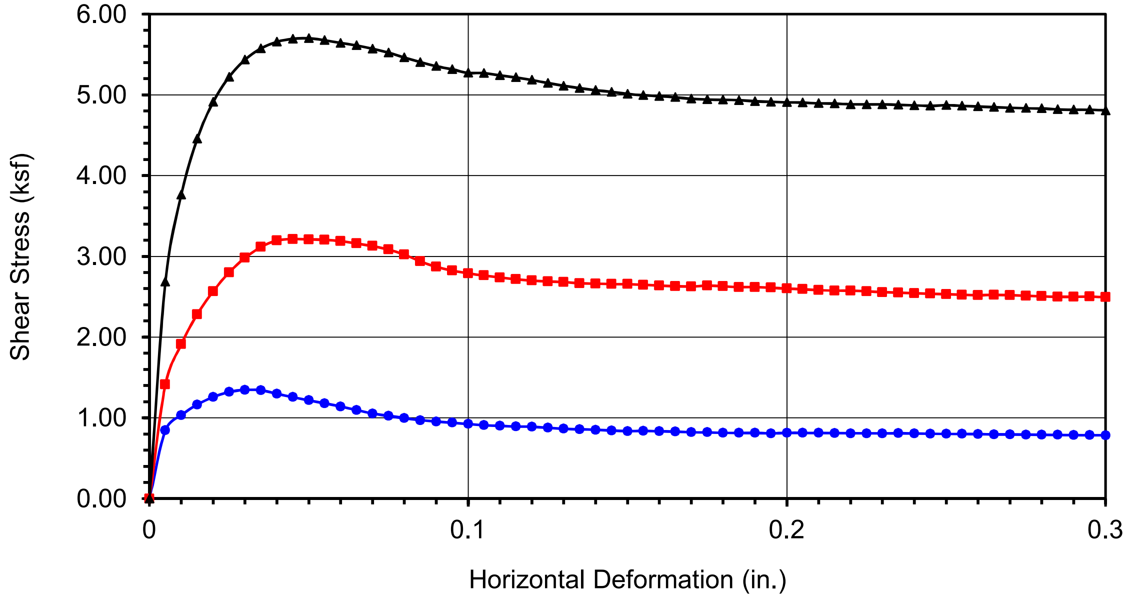
DS LB-2, R-1 @ 5

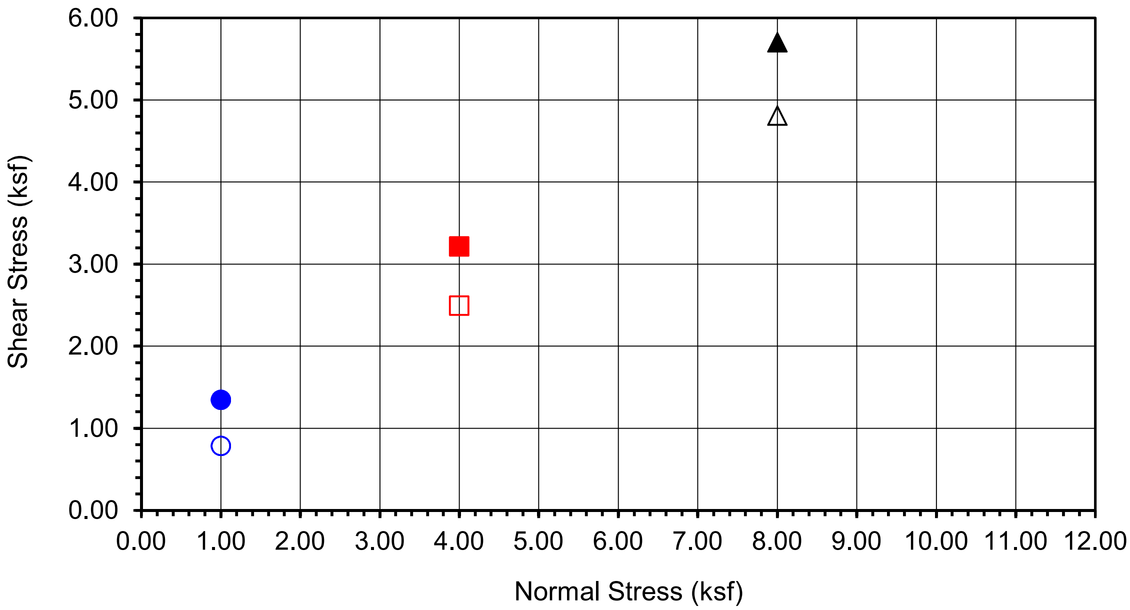
DIRECT SHEAR TEST

Consolidated Drained - ASTM D 3080

Project Name:	McKinley ES	Tested By:	G. Bathala	Date:	09/27/21
Project No.:	11428.036	Checked By:	A. Santos	Date:	10/15/21
Boring No.:	LB-2	Sample Type:	Ring		
Sample No.:	R-2	Depth (ft.):	7.5		
Soil Identification:	Brown lean clay (CL)				

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	201.38	206.70	207.07
Weight of Ring(gm):	42.35	45.30	44.45
Before Shearing			
Weight of Wet Sample+Cont.(gm):	178.96	178.96	178.96
Weight of Dry Sample+Cont.(gm):	164.62	164.62	164.62
Weight of Container(gm):	53.69	53.69	53.69
Vertical Rdg.(in): Initial	0.0000	0.2422	0.2412
Vertical Rdg.(in): Final	-0.0005	0.2563	0.2539
After Shearing			
Weight of Wet Sample+Cont.(gm):	216.56	217.54	227.71
Weight of Dry Sample+Cont.(gm):	193.49	196.11	206.18
Weight of Container(gm):	55.15	53.69	64.02
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-2**
Sample No.	**R-2**
Depth (ft)	**7.5**
Sample Type: Ring	
Soil Identification: Brown lean clay (CL)	

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 1.346	■ 3.213	▲ 5.700
Shear Stress @ End of Test (ksf)	○ 0.783	□ 2.493	△ 4.807
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 (%)	12.93	12.93	12.93
Dry Density (pcf)	117.1	118.9	119.8
Saturation (%)	79.5	83.5	85.7
Soil Height Before Shearing (in.)	0.9995	0.9859	0.9873
Final Moisture Content (%)	16.7	15.0	15.1

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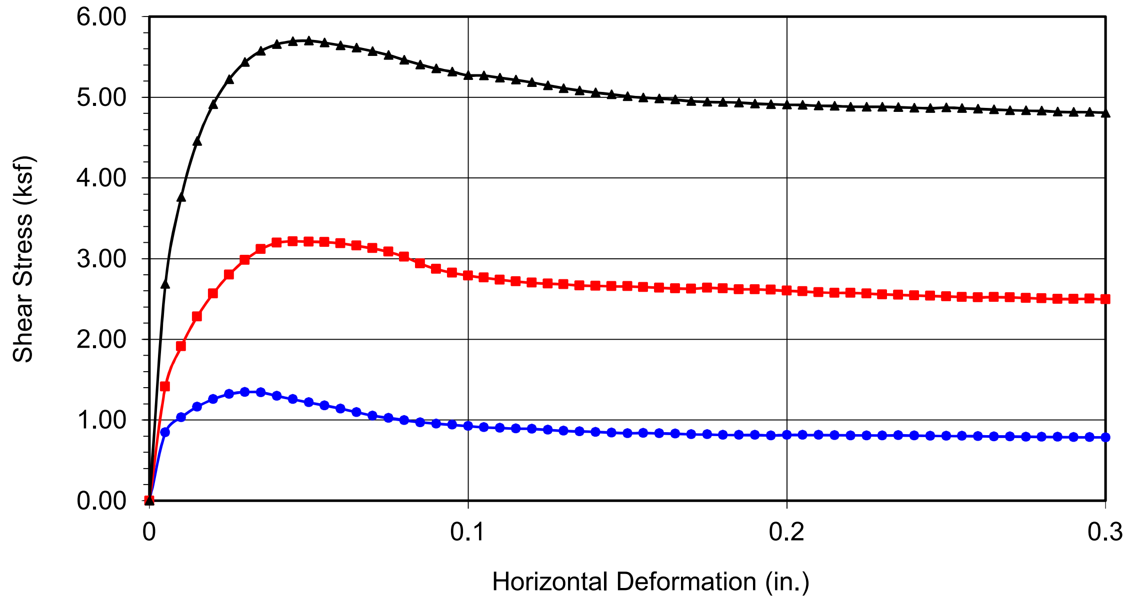
DIRECT SHEAR TEST RESULTS

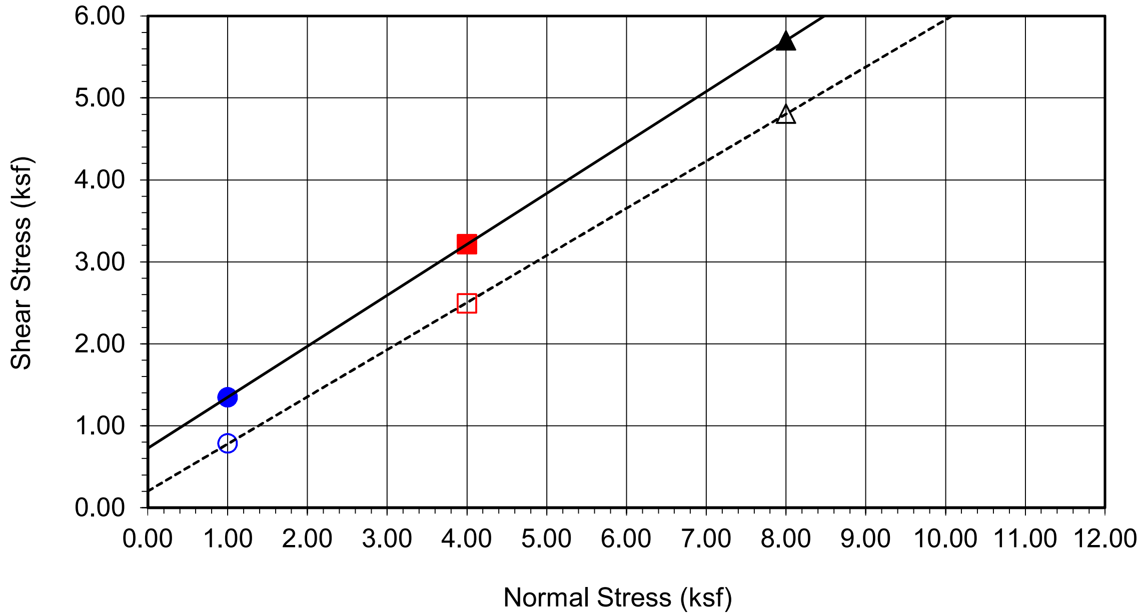
Consolidated Drained - ASTM D 3080

Project No.: 11428.036

McKinley ES

09-21





Boring No.	**LB-2**
Sample No.	**R-2**
Depth (ft)	**7.5**
Sample Type:	Ring

Soil Identification:
Brown lean clay (CL)

Strength Parameters

	C (psf)	ϕ (°)
Peak	724	32
Ultimate	202	30

Normal Stress (kip/ft²)	1.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 1.346	■ 3.213	▲ 5.700
Shear Stress @ End of Test (ksf)	○ 0.783	□ 2.493	△ 4.807
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 (%)	12.93	12.93	12.93
Dry Density (pcf)	117.1	118.9	119.8
Saturation (%)	79.5	83.5	85.7
Soil Height Before Shearing (in.)	0.9995	0.9859	0.9873
Final Moisture Content (%)	16.7	15.0	15.1

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DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 11428.036

McKinley ES

09-21

DS LB-2, R-2 @ 7.5

EXPANSION INDEX of SOILS
ASTM D 4829

Leighton

Project Name:	McKinley ES	Tested By:	J. Gonzalez	Date:	09/27/21
Project No.:	11428.036	Checked By:	A. Santos	Date:	10/14/21
Boring No.:	LB-2	Depth (ft.):	0-5		
Sample No.:	BB-1				
Soil Identification:	Brown lean clay (CL)				

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	1.0410
Wt. Comp. Soil + Mold (g)	559.10	429.90
Wt. of Mold (g)	163.30	0.00
Specific Gravity (Assumed)	2.70	2.70
Container No.	O	O
Wet Wt. of Soil + Cont. (g)	787.90	593.20
Dry Wt. of Soil + Cont. (g)	713.10	521.49
Wt. of Container (g)	0.00	163.30
Moisture Content (%)	10.49	20.02
Wet Density (pcf)	119.4	124.6
Dry Density (pcf)	108.1	103.8
Void Ratio	0.560	0.624
Total Porosity	0.359	0.384
Pore Volume (cc)	74.3	82.8
Degree of Saturation (%) [S_{meas}]	**50.6**	86.6

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.)
09/27/21	15:45	1.0	0	0.6180
09/27/21	15:55	1.0	10	0.6170
Add Distilled Water to the Specimen				
09/27/21	16:45	1.0	50	0.6250
09/28/21	7:30	1.0	935	0.6590
09/28/21	11:30	1.0	1175	0.6590

Expansion Index (EI_{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	**42**

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name:	McKinley ES	Tested By:	J. Gonzalez	Date:	09/22/21
Project No.:	11428.036	Checked By:	A. Santos	Date:	09/23/21
Boring No.:	LB-2	Depth (ft.):	0-5		
Sample No.:	BB-1				
Soil Identification:	Brown lean clay (CL)				

Preparation Method: [X] Moist [ ] Dry

[X] Mechanical Ram [ ] Manual Ram

Mold Volume (ft³) 0.03330

Ram Weight = 10 lb.; Drop = 18 in.

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3840	3910	3865			
Weight of Mold (g)	1850	1850	1850			
Net Weight of Soil (g)	1990	2060	2015			
Wet Weight of Soil + Cont. (g)	469.4	480.6	478.0			
Dry Weight of Soil + Cont. (g)	426.8	429.1	417.4			
Weight of Container (g)	38.8	38.0	40.0			
Moisture Content (%)	10.98	13.17	16.06			
Wet Density (pcf)	131.7	136.4	133.4			
Dry Density (pcf)	118.7	120.5	114.9			

Maximum Dry Density (pcf) **120.6** **Optimum Moisture Content (%)** **12.8**

PROCEDURE USED

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

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

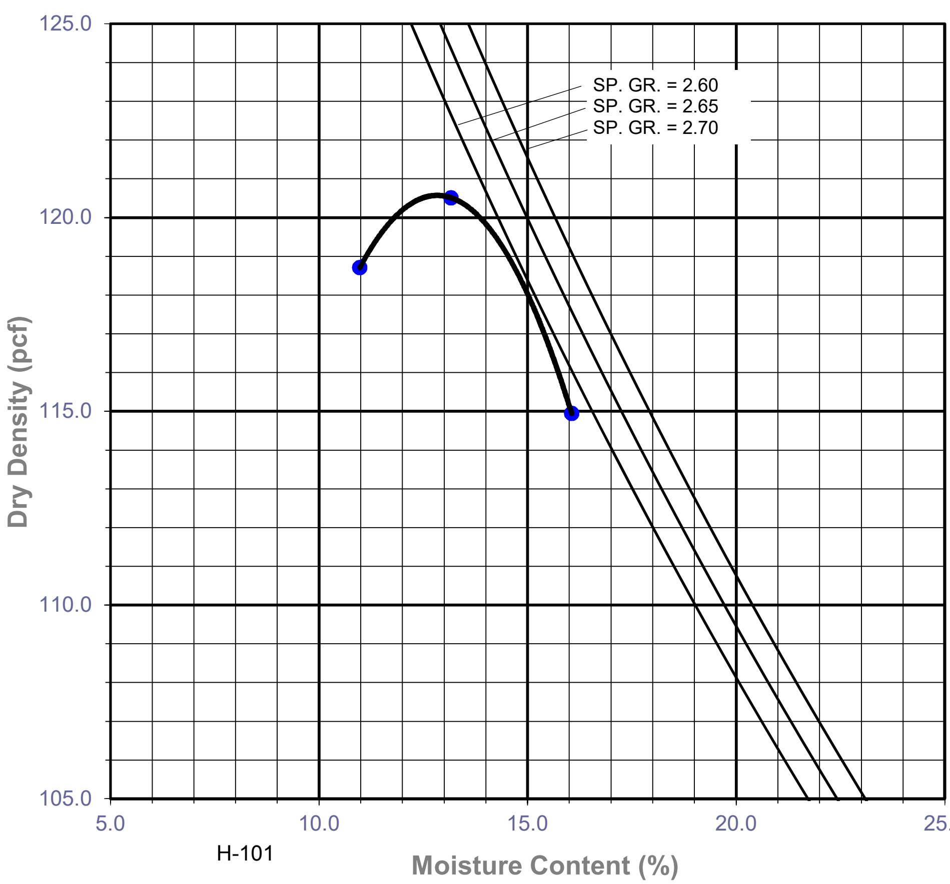
[ ] **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



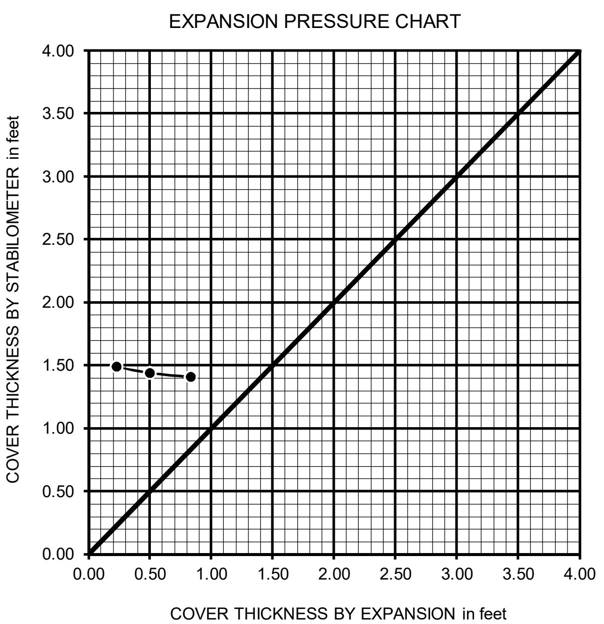
H-101

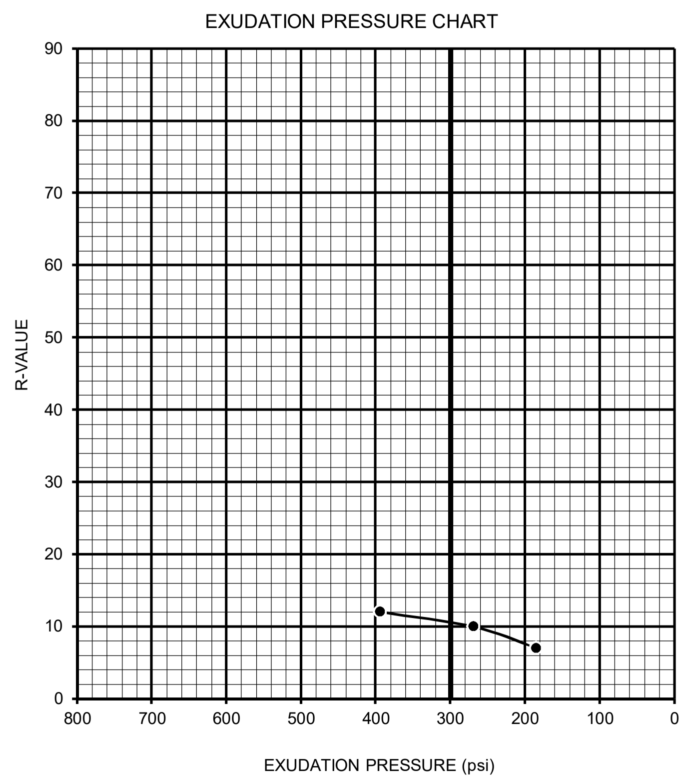
R-VALUE TEST RESULTS
DOT CA Test 301

PROJECT NAME:	McKinley ES	PROJECT NUMBER:	11428.036
BORING NUMBER:	LB-2	DEPTH (FT.):	0-5
SAMPLE NUMBER:	BB-1	TECHNICIAN:	O. Figueroa
SAMPLE DESCRIPTION:	Brown lean clay (CL)	DATE COMPLETED:	10/1/2021

TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	18.5	20.0	20.9
HEIGHT OF SAMPLE, Inches	2.53	2.53	2.54
DRY DENSITY, pcf	112.5	108.4	108.3
COMPACTOR PRESSURE, psi	80	60	50
EXUDATION PRESSURE, psi	394	269	185
EXPANSION, Inches x 10exp-4	25	15	7
STABILITY Ph 2,000 lbs (160 psi)	130	135	142
TURNS DISPLACEMENT	4.10	4.20	4.40
R-VALUE UNCORRECTED	12	10	7
R-VALUE CORRECTED	12	10	7

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.	1.41	1.44	1.49
EXPANSION PRESSURE THICKNESS, ft.	0.83	0.50	0.23





R-VALUE BY EXPANSION:	13
R-VALUE BY EXUDATION:	10
EQUILIBRIUM R-VALUE:	10

APPENDIX C

Seismicity Data

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

Project Name: McKinley ES
Date: November 2021
Project Location: 2401 Santa Monica Boulevard
Latitude: 34.0324°
Project Number: 11428.036
Longitude: -118.4768°
Site Class: D
Shear Wave Velocity: 357 m/sec
Return Period: 2475 years (2% probability of exceedance in 50 years)
Percent Damping: 5%

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

S_s	1.955	**S_{MS}**	**2.305**	T_0	0.179
S_1	0.698	**S_{M1}**	**1.531**	T_s	0.893
F_a	1	**S_{DS}**	**1.537**	T_L	8
F_v	2.5	**S_{D1}**	**1.020**	**PGA_M**	**0.930**
C_{RS}	0.908	C_{R1}	0.904		

	Sec. 21.2.1.1 Probabilistic				Sec. 21.2.2 Deterministic			Sec. 11.4.6 General Procedure	Sec. 21.3 Design Response Spectrum				Risk Targeted Spectrum
Period (sec)	Spectral Acceleration (g)	Seismic Risk Coefficients	Maximum Response Coefficients	MCE_R Response Spectrum (g)	Spectral Acceleration (g)	Maximum Response Coefficients	MCE_R Response Spectrum (g)	Design Response Spectral Acceleration (g)	Lower Limit of General Procedure - 80% of S_a (g)	MCE_R - S_{aM} (g)	2/3 * S_{aM} (g)	Design Response Spectrum (g)	1.5 * Design Response Spectrum (g)
0.01	0.930	0.908	1.19	1.005	1.196	1.19	1.424	0.565	0.452	1.005	0.670	0.670	1.005
0.02	0.938	0.908	1.19	1.014	1.220	1.19	1.451	0.609	0.487	1.014	0.676	0.676	1.014
0.03	0.987	0.908	1.19	1.066	1.240	1.19	1.476	0.653	0.522	1.066	0.711	0.711	1.066
0.05	1.161	0.908	1.19	1.254	1.408	1.19	1.676	0.740	0.592	1.254	0.836	0.836	1.254
0.075	1.469	0.908	1.19	1.587	1.693	1.19	2.014	0.850	0.680	1.587	1.058	1.058	1.587
0.1	1.724	0.908	1.19	1.863	1.967	1.19	2.341	0.959	0.768	1.863	1.242	1.242	1.863
0.15	2.015	0.908	1.20	2.195	2.307	1.20	2.768	1.178	0.943	2.195	1.463	1.463	2.195
0.2	2.163	0.908	1.21	2.376	2.599	1.21	3.145	1.303	1.043	2.376	1.584	1.584	2.376
0.25	2.256	0.908	1.22	2.498	2.789	1.22	3.403	1.303	1.043	2.498	1.665	1.665	2.498
0.3	2.314	0.908	1.22	2.562	2.987	1.22	3.645	1.303	1.043	2.562	1.708	1.708	2.562
0.4	2.224	0.907	1.23	2.481	3.031	1.23	3.729	1.303	1.043	2.481	1.654	1.654	2.481
0.5	2.080	0.907	1.23	2.319	2.830	1.23	3.481	1.303	1.043	2.319	1.546	1.546	2.319
0.75	1.674	0.905	1.24	1.879	2.284	1.24	2.832	1.303	1.043	1.879	1.253	1.253	1.879
1	1.356	0.904	1.24	1.520	1.743	1.24	2.162	1.163	0.931	1.520	1.013	1.013	1.520
1.5	0.910	0.904	1.24	1.020	1.142	1.24	1.416	0.776	0.620	1.020	0.680	0.680	1.020
2	0.671	0.904	1.24	0.752	0.819	1.24	1.015	0.582	0.465	0.752	0.501	0.501	0.752
3	0.427	0.904	1.25	0.482	0.461	1.25	0.576	0.388	0.310	0.482	0.321	0.321	0.482
4	0.298	0.904	1.26	0.340	0.294	1.26	0.370	0.291	0.233	0.340	0.227	0.233	0.349
5	0.224	0.904	1.26	0.255	0.207	1.26	0.261	0.233	0.186	0.255	0.170	0.186	0.279
7.5	0.127	0.904	1.28	0.146	0.103	1.28	0.132	0.155	0.124	0.132	0.088	0.124	0.186
10	0.076	0.904	1.29	0.089	0.057	1.29	0.074	0.093	0.074	0.074	0.049	0.074	0.112

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

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

Input

Edition

Dynamic: Conterminous U.S. 2014 (update…

Spectral Period

Peak Ground Acceleration

Latitude

Decimal degrees

34.0324

Time Horizon

Return period in years

2475

Longitude

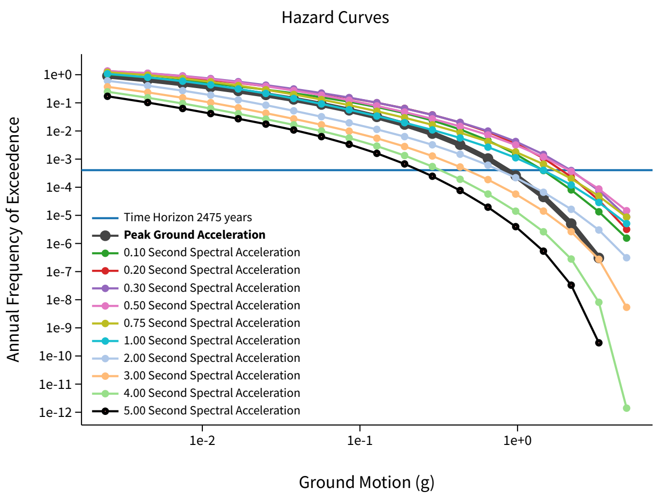
Decimal degrees, negative values for western longitudes

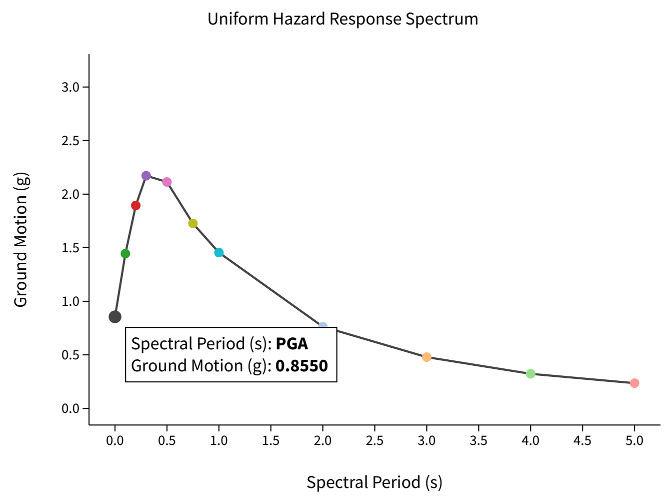
-118.4768

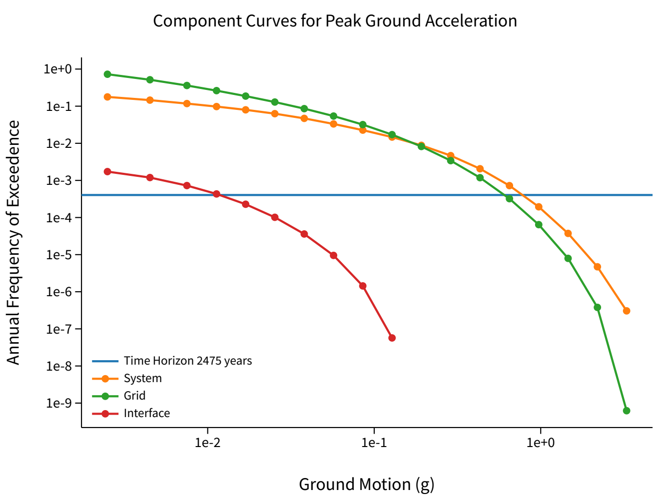
Site Class

259 m/s (Site class D)

Hazard Curve





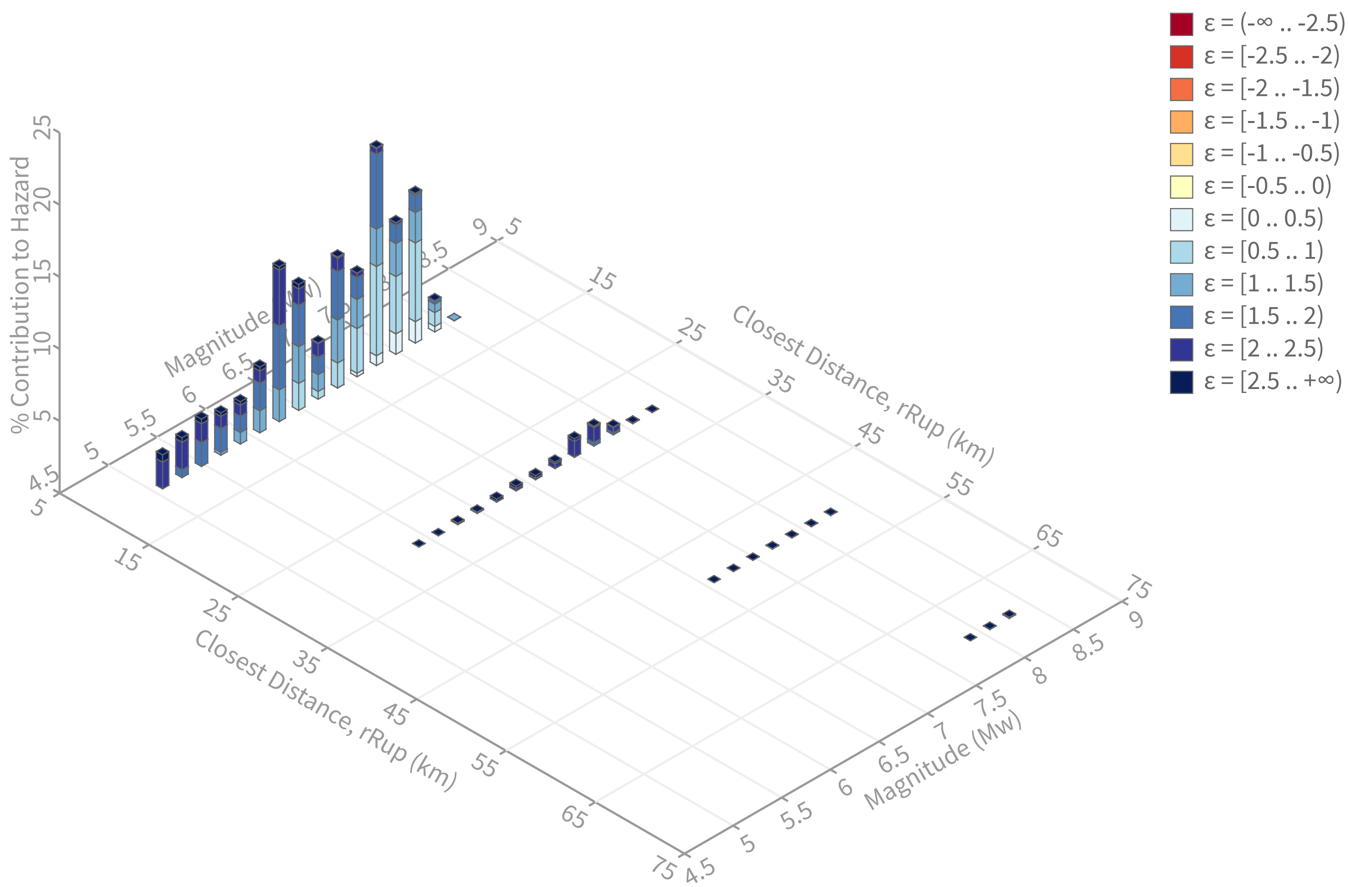


View Raw Data

^ Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.85497633 g

Recovered targets

Return period: 2966.7331 yrs
Exceedance rate: 0.0003370711 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.04 %

Mean (over all sources)

m: 6.82
r: 8.32 km
ε_0: 1.47 σ

Mode (largest m-r bin)

m: 7.31
r: 8.02 km
ε_0: 1.19 σ
Contribution: 15.1 %

Mode (largest m-r-ε_0 bin)

m: 7.32
r: 6.69 km
ε_0: 0.76 σ
Contribution: 6.19 %

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							37.36
Santa Monica alt 1 [0]		1.91	7.16	0.86	118.479°W	34.038°N	344.27	12.42
Compton [4]		10.06	7.39	0.90	118.586°W	33.981°N	240.42	5.71
Palos Verdes [15]		10.21	7.02	1.74	118.557°W	33.970°N	226.93	4.94
Newport-Inglewood alt 1 [8]		8.31	6.66	1.79	118.389°W	34.044°N	81.20	4.35
Malibu Coast alt 1 [0]		4.50	6.33	1.46	118.525°W	34.031°N	267.39	1.79
San Pedro Escarpment [1]		8.79	7.60	0.81	118.655°W	33.915°N	231.67	1.10
Compton [3]		10.53	7.35	0.99	118.533°W	33.925°N	203.50	1.09
UC33brAvg_FM32	System							37.05
Santa Monica alt 2 [2]		2.12	7.15	0.92	118.467°W	34.046°N	30.18	8.77
Malibu Coast alt 2 [0]		4.72	7.47	0.92	118.525°W	34.033°N	270.84	4.93
Hollywood [2]		7.82	6.97	1.56	118.422°W	34.084°N	41.06	4.84
Palos Verdes [15]		10.21	7.02	1.79	118.557°W	33.970°N	226.93	4.58
Newport-Inglewood alt 2 [8]		8.27	6.75	1.73	118.390°W	34.043°N	81.39	3.54
Compton [4]		10.06	7.47	0.88	118.586°W	33.981°N	240.42	2.87
Compton [3]		10.53	7.26	0.99	118.533°W	33.925°N	203.50	1.62
UC33brAvg_FM31 (opt)	Grid							13.26
PointSourceFinite: -118.477, 34.073		6.61	5.75	1.71	118.477°W	34.073°N	0.00	2.90
PointSourceFinite: -118.477, 34.073		6.61	5.75	1.71	118.477°W	34.073°N	0.00	2.90
PointSourceFinite: -118.477, 34.100		8.35	5.88	1.92	118.477°W	34.100°N	0.00	2.15
PointSourceFinite: -118.477, 34.100		8.35	5.88	1.92	118.477°W	34.100°N	0.00	2.15
UC33brAvg_FM32 (opt)	Grid							12.33
PointSourceFinite: -118.477, 34.073		6.59	5.77	1.70	118.477°W	34.073°N	0.00	2.49
PointSourceFinite: -118.477, 34.073		6.59	5.77	1.70	118.477°W	34.073°N	0.00	2.49
PointSourceFinite: -118.477, 34.100		8.35	5.88	1.92	118.477°W	34.100°N	0.00	2.02
PointSourceFinite: -118.477, 34.100		8.35	5.88	1.92	118.477°W	34.100°N	0.00	2.02

McKinley ES

Latitude, Longitude: 34.0324, -118.4768

Date	11/8/2021, 4:18:35 PM
Design Code Reference Document	ASCE7-16
Risk Category	IV
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.955	MCE_R ground motion. (for 0.2 second period)
S_1	0.698	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.955	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.303	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	0.834	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.918	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.955	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.154	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.443	Factored deterministic acceleration value. (0.2 second)
S1RT	0.698	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.773	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.824	Factored deterministic acceleration value. (1.0 second)
PGAd	0.988	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.908	Mapped value of the risk coefficient at short periods
C_{R1}	0.904	Mapped value of the risk coefficient at a period of 1 s

DISCLAIMER

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OpenSHA PSHA Output

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

DATASET #1

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

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

Site Param List:

Longitude = -118.4768; Latitude = 34.0324; Vs30 = 357.0; Vs30 Type = Measured; Depth 2.5 km/sec = 5.05; Depth 1.0 km/sec = 700.0

IML/Prob Param List:

Map Type = IML@Prob; Probability = 0.02

Forecast Param List:

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

TimeSpan Param List:

Duration = 50.0

Maximum Distance = 200.0; Pt Src Dist Corr = None

X, Y Data:
0.01 0.9303186
0.02 0.93801564
0.03 0.98700035
0.05 1.160825
0.075 1.4690263
0.1 1.7241318
0.15 2.0145957
0.2 2.162522
0.25 2.2556472
0.3 2.3136613
0.4 2.2238133

0.5	2.079747
0.75	1.6741652
1.0	1.356126
1.5	0.91027033
2.0	0.67070514
3.0	0.42654446
4.0	0.29842028
5.0	0.22367641
7.5	0.12650016
10.0	0.07634516

Compton (3)

PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

WEIGHTED AVERAGE of 2014 NGA WEST-2 GMPEs

Last updated: 04 14 15

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

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

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

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

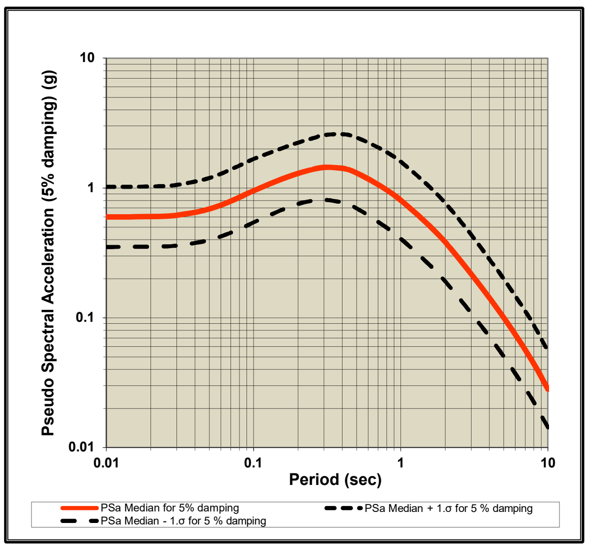
GMPEs	ASK14	BSSA14	CB14	CY14	I14
Weight	0.25	0.25	0.25	0.25	0

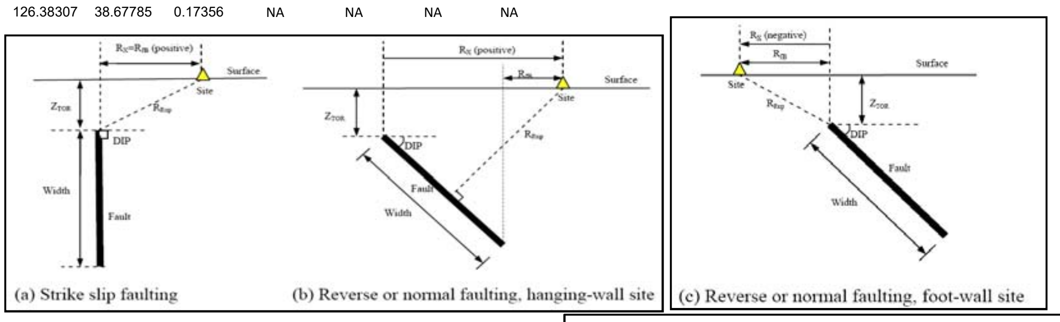
# of std. dev.	1	
Damping ratio (%)	5	*Modification factors are calculated in Sheet DSF*

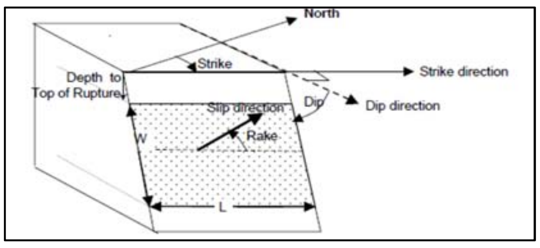
ASK14 Abrahamson & Silva & Kamai 2014 NGA West-2 Model
BSSA14 Boore & Stewart & Seyhan & Atkinson 2014 NGA West-2 Model
CB14 Campbell & Bozorgnia 2014 NGA West-2 Model
CY14 Chiou & Youngs 2014 NGA West-2 Model
I14 Idriss 2014 NGA West-2 Model

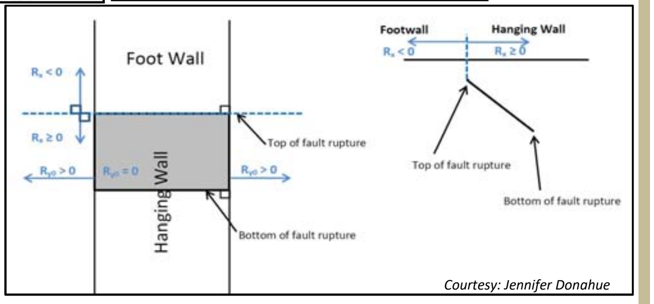
RotD50 Horizontal Component of PGA, PGV and *IMs*

GMP	*T* (s)	Baseline: 5% Damping: PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	User defined: 5% Damping: PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	0.01	0.59816	1.01975	0.35087	0.00148	0.59816	1.01975	0.35087	0.00148
	0.02	0.60203	1.02697	0.35292	0.00598	0.60203	1.02697	0.35292	0.00598
	0.03	0.61649	1.05638	0.35978	0.01377	0.61588	1.05533	0.35942	0.01376
	0.05	0.68769	1.19308	0.39638	0.04268	0.68769	1.19308	0.39638	0.04268
	0.075	0.81686	1.43507	0.46497	0.11406	0.81932	1.43938	0.46637	0.11440
	0.1	0.94927	1.66680	0.54063	0.23564	0.95212	1.67180	0.54225	0.23635
	0.15	1.14904	1.98606	0.66478	0.64178	1.15134	1.99003	0.66611	0.64306
	0.2	1.29307	2.22721	0.75073	1.28395	1.29566	2.23167	0.75223	1.28652
	0.25	1.38052	2.39909	0.79440	2.14185	1.38466	2.40628	0.79678	2.14827
	0.3	1.43617	2.54844	0.80936	3.20860	1.43905	2.55354	0.81098	3.21502
	0.4	1.41508	2.59481	0.77171	5.62038	1.41649	2.59741	0.77248	5.62601
	0.5	1.30628	2.44722	0.69727	8.10667	1.30759	2.44967	0.69797	8.11478
	0.75	1.01320	1.97203	0.52057	14.14765	1.01320	1.97203	0.52057	14.14765
	1	0.80420	1.59142	0.40639	19.96312	0.80420	1.59142	0.40639	19.96312
	1.5	0.53445	1.06864	0.26729	29.85100	0.53499	1.06971	0.26756	29.88085
	2	0.38404	0.77026	0.19147	38.13307	0.38327	0.76872	0.19109	38.05681
	3	0.21897	0.43959	0.10908	48.92148	0.21875	0.43915	0.10897	48.87256
	4	0.14217	0.28260	0.07153	56.46835	0.14203	0.28232	0.07146	56.41188
	5	0.10062	0.20042	0.05052	62.44326	0.10032	0.19981	0.05036	62.25593
	7.5	0.05037	0.10005	0.02536	70.32847	0.05022	0.09975	0.02528	70.11748
	10	0.02824	0.05557	0.01435	70.10062	0.02813	0.05535	0.01429	69.82022
PGA (g)	0	0.59527	1.01405	0.34944	0.00148	0.59527	1.01405	0.34944	0.00148
PGV (cm/s)	-1	69.91584	126.38307	38.67785	0.17356	NA	NA	NA	NA









Courtesy: Jennifer Donahue

Input variables	Errors and warnings
M_w: 7.45	
R_{RUP} (km): 10.53	
R_{JB} (km): 2.47	
R_X (km): 12.96	
Ry0 (km): 999	*If unknown use 999*
V_{S30} (m/sec): 357	
U (BSSA13): 0	*1: Unspecified fault mech.*
F_{RV}: 1	*1: reverse fault*
F_{NM}: 0	*1: normal fault*
F_{HW}: 1	*1: hanging wall side*
Dip (deg): 20	
Z_{TOR} (km): 5.2	*If unknown use 999*
Z_{HYP} (km): 999	*If unknown use 999*
$Z_{1.0}$ (km): 0.7	*If unknown use 999*
$Z_{2.5}$ (km): 5.05	*If unknown use 999*
W (km): 27.37	*If unknown use 999*
Vs30Flag: measured	*Choose options for V_{s30} from the list*
F_{AS}: no	*Aftershock effect is not applicable.*
Region: California	*Choose region from the list*

Calculated Variables/Flags

ΔDPP: 0	*Always 0 for median calcs.*
PGA_r (g): 0.435	
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 illustation
R_{y0} = The horizontal distance off the end of the rupture measured parallel to strike (km)
V_{S30} = The average shear-wave velocity (m/s) over a subsurface depth of 30 m
U = Unspecified-mechanism factor: 1 for unspecified; 0 otherwise
F_{RV} = Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
F_{NM} = Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal
F_{HW} = Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise
Dip = Average dip of rupture plane (degrees)
Z_{TOR} = Depth to top of coseismic rupture (km)
Z_{HYP} = Hypocentral depth from the earthquake
$Z_{1.0}$ = Depth to Vs=1 km/sec
$Z_{2.5}$ = Depth to Vs=2.5 km/sec
W = Fault rupture width (km)
$V_{s30flag}$ = 1 for measured, 0 for inferred Vs30
F_{AS} = 0 for mainshock; 1 for aftershock
Region = Specific regions considered in the models, Click on Region to see codes
ΔDPP = Directivity term, direct point parameter; uses 0 for median predictions
PGA_r (g) = Peak ground acceleration on rock (g), this specific cell is updated in the cell for BSSA14 and CB14, for others it is taken account for in the macros
Z_{BOT} (km) = The depth to the bottom of the seismogenic crust
Z_{BOR} (km) = The depth to the bottom of the rupture plane
SS = 1 for strike slip, automatically updated in the cell

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

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

DEFAULTs	USER defined	ASK14	BSSA14	CB14	CY14	I14
W (km)	27.37			42.251		
$Z_{1.0}$ (km)	0.700	0.700			0.406	
$\delta Z_{1.0}$ (km)	0.294		0.294			
$Z_{2.5}$ (V_{S30}=1100)(km)	5.050			0.398		
$Z_{2.5}$ (V_{S30})(km)	5.050			1.440		
Z_{hyp} (km)	999.00			5.000		
Z_{tor} (km)	5.20			0.549	0.549	
Z_{BOR} (km)	-			15.000		

ACKNOWLEDGEMENTS

Nick Gregor, Bechtel
Silvia Mazzoni, Consultant

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

PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

WEIGHTED AVERAGE of 2014 NGA WEST-2 GMPEs

Last updated: 04 14 15

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

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

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

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

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

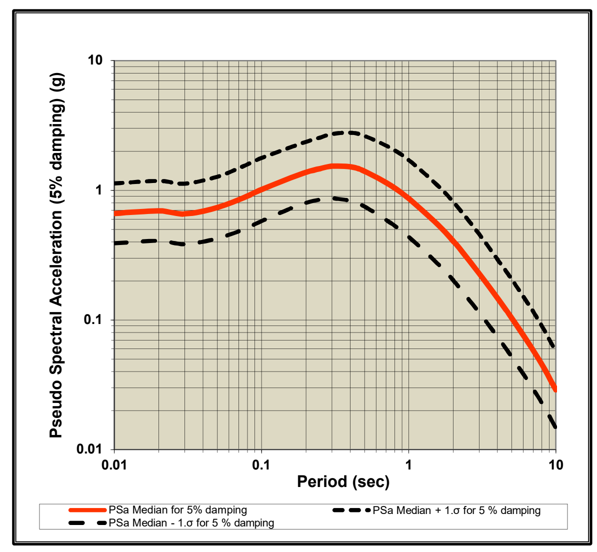
# of std. dev.	1	
Damping ratio (%)	5	*Modification factors are calculated in Sheet DSF*

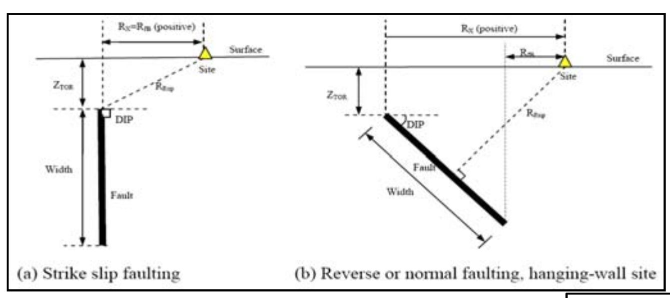
ASK14 Abrahamson & Silva & Kamai 2014 NGA West-2 Model
BSSA14 Boore & Stewart & Seyhan & Atkinson 2014 NGA West-2 Model
CB14 Campbell & Bozorgnia 2014 NGA West-2 Model
CY14 Chiou & Youngs 2014 NGA West-2 Model
I14 Idriss 2014 NGA West-2 Model

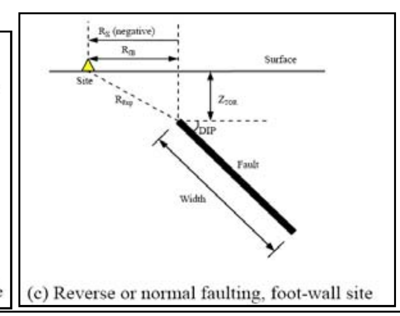
RotD50 Horizontal Component of PGA, PGV and *IMs*

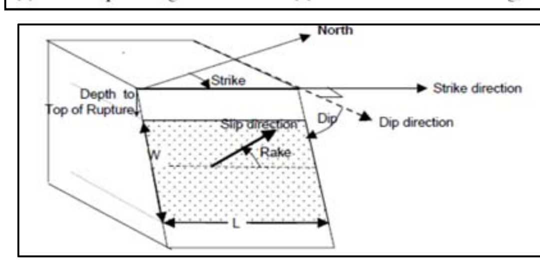
Input variables	Errors and warnings
M_w: 7.45	
R_{RUP} (km): 10.06	
R_{JB} (km): 0	
R_X (km): 11.57	
Ry0 (km): 999	*If unknown use 999*
V_{S30} (m/sec): 357	
U (BSSA13): 0	*1: Unspecified fault mech.*
F_{RV}: 1	*1: reverse fault*
F_{NM}: 0	*1: normal fault*
F_{HW}: 1	*1: hanging wall side*
Dip (deg): 20	
Z_{TOR} (km): 5.2	*If unknown use 999*
Z_{HYP} (km): 999	*If unknown use 999*
$Z_{1.0}$ (km): 0.7	*If unknown use 999*
$Z_{2.5}$ (km): 5.05	*If unknown use 999*
W (km): 27.37	*If unknown use 999*
Vs30Flag: measured	*Choose options for V_{s30} from the list*
F_{AS}: no	*Aftershock effect is not applicable.*
Region: California	*Choose region from the list*

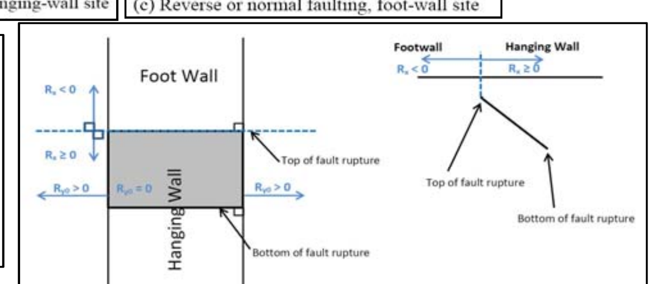
Baseline: 5% Damping						User defined: 5% Damping			
GMP	T (s)	PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	0.01	0.66346	1.12889	0.38993	0.00165	0.66346	1.12889	0.38993	0.00165
	0.02	0.69359	1.18082	0.40740	0.00689	0.69359	1.18082	0.40740	0.00689
	0.03	0.65800	1.12519	0.38479	0.01470	0.65734	1.12407	0.38440	0.01469
	0.05	0.73471	1.27220	0.42431	0.04560	0.73471	1.27220	0.42431	0.04560
	0.075	0.87127	1.52791	0.49683	0.12166	0.87388	1.53249	0.49832	0.12202
	0.1	1.01068	1.77135	0.57667	0.25089	1.01371	1.77666	0.57840	0.25164
	0.15	1.21772	2.10030	0.70602	0.68014	1.22016	2.10450	0.70743	0.68150
	0.2	1.37235	2.35842	0.79856	1.36267	1.37509	2.36314	0.80015	1.36539
	0.25	1.46494	2.54026	0.84481	2.27282	1.46933	2.54788	0.84735	2.27964
	0.3	1.53037	2.71027	0.86413	3.41904	1.53343	2.71569	0.86586	3.42588
	0.4	1.51614	2.77668	0.82785	6.02178	1.51765	2.77946	0.82868	6.02780
	0.5	1.40199	2.62419	0.74902	8.70061	1.40339	2.62681	0.74977	8.70931
	0.75	1.09299	2.12642	0.56180	15.26180	1.09299	2.12642	0.56180	15.26180
	1	0.86440	1.71013	0.43692	21.45765	0.86354	1.70842	0.43648	21.43619
	1.5	0.57132	1.14223	0.28576	31.91027	0.57189	1.14338	0.28605	31.94218
	2	0.40816	0.81861	0.20351	40.52816	0.40734	0.81698	0.20310	40.44711
	3	0.22968	0.46108	0.11441	51.31390	0.22945	0.46062	0.11430	51.26258
	4	0.14778	0.29374	0.07435	58.69539	0.14763	0.29345	0.07427	58.63670
	5	0.10385	0.20686	0.05214	64.44991	0.10354	0.20624	0.05198	64.25656
	7.5	0.05173	0.10275	0.02604	72.22839	0.05157	0.10244	0.02596	72.01170
	10	0.02900	0.05706	0.01474	71.98056	0.02888	0.05683	0.01468	71.69264
PGA (g)	0	0.63486	1.07939	0.37340	0.00158	0.63486	1.07939	0.37340	0.00158
PGV (cm/s)	-1	74.10190	133.90833	41.00635	0.18395	NA	NA	NA	NA











Courtesy: Jennifer Donahue

Calculated Variables/Flags

ΔDPP: 0	*Always 0 for median calcs.*
PGA_r (g): 0.471	
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 illustation
- **R_{y0}** = The horizontal distance off the end of the rupture measured parallel to strike (km)
- **V_{S30}** = The average shear-wave velocity (m/s) over a subsurface depth of 30 m
- **U** = Unspecified-mechanism factor: 1 for unspecified; 0 otherwise
- **F_{RV}** = Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
- **F_{NM}** = Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal
- **F_{HW}** = Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise
- **Dip** = Average dip of rupture plane (degrees)
- **Z_{TOR}** = Depth to top of coseismic rupture (km)
- **Z_{HYP}** = Hypocentral depth from the earthquake
- **$Z_{1.0}$** = Depth to Vs=1 km/sec
- **$Z_{2.5}$** = Depth to Vs=2.5 km/sec
- **W** = Fault rupture width (km)
- **$V_{s30flag}$** = 1 for measured, 0 for inferred Vs30
- **F_{AS}** = 0 for mainshock; 1 for aftershock
- **Region** = Specific regions considered in the models, Click on Region to see codes
- **ΔDPP** = Directivity term, direct point parameter; uses 0 for median predictions
- **PGA_r (g)** = Peak ground acceleration on rock (g), this specific cell is updated in the cell for BSSA14 and CB14, for others it is taken account for in the macros
- **Z_{BOT} (km)** = The depth to the bottom of the seismogenic crust
- **Z_{BOR} (km)** = The depth to the bottom of the rupture plane
- **SS** = 1 for strike slip, automatically updated in the cell

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

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

DEFAULTs	USER defined	ASK14	BSSA14	CB14	CY14	I14
W (km)	27.37			42.251		
$Z_{1.0}$ (km)	0.700	0.700			0.406	
$\delta Z_{1.0}$ (km)	0.294		0.294			
$Z_{2.5}$ (V_{S30}=1100)(km)	5.050			0.398		
$Z_{2.5}$ (V_{S30})(km)	5.050			1.440		
Z_{hyp} (km)	999.00			5.000		
Z_{tor} (km)	5.20			0.549	0.549	
Z_{BOR} (km)	-			15.000		

ACKNOWLEDGEMENTS

Nick Gregor, Bechtel
Silvia Mazzoni, Consultant

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

Hollywood (2)

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

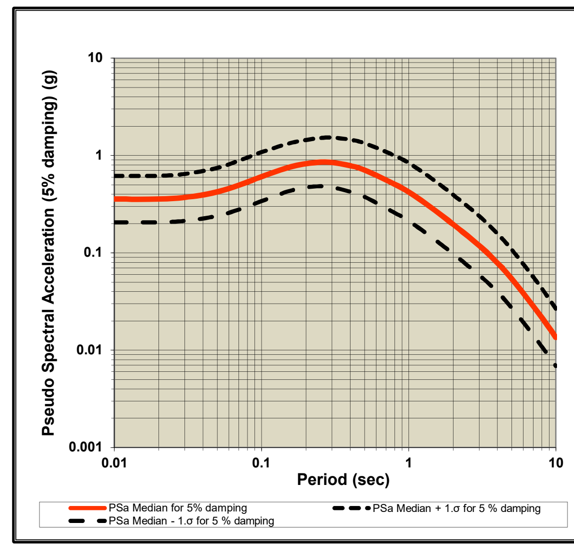
GMPEs	ASK14	BSSA14	CB14	CY14	I14
Weight	0.25	0.25	0.25	0.25	0

# 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

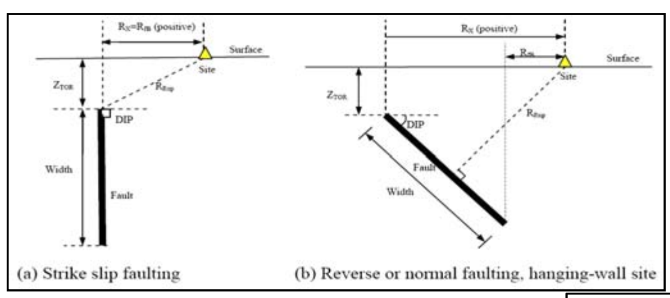
RotD50 Horizontal Component of PGA, PGV and *IMs*

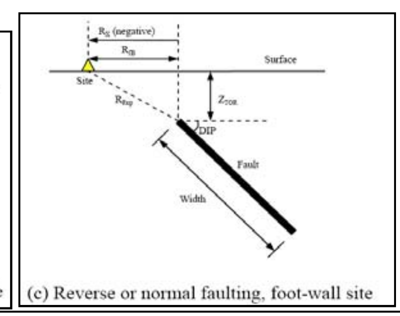
GMP	T (s)	Baseline: 5% Damping				User defined: 5% Damping			
		PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	0.01	0.35626	0.61680	0.20578	0.00088	0.35626	0.61680	0.20578	0.00088
	0.02	0.35719	0.61896	0.20612	0.00355	0.35719	0.61896	0.20612	0.00355
	0.03	0.37115	0.64657	0.21305	0.00829	0.37115	0.64657	0.21305	0.00829
	0.05	0.42207	0.74445	0.23930	0.02619	0.42207	0.74445	0.23930	0.02619
	0.075	0.51368	0.91641	0.28793	0.07173	0.51471	0.91825	0.28851	0.07187
	0.1	0.60441	1.07791	0.33891	0.15004	0.60622	1.08114	0.33992	0.15049
	0.15	0.74832	1.31631	0.42542	0.41796	0.74982	1.31894	0.42627	0.41880
	0.2	0.82245	1.44310	0.46873	0.81665	0.82327	1.44454	0.46919	0.81746
	0.25	0.85234	1.50781	0.48181	1.32238	0.85404	1.51083	0.48277	1.32503
	0.3	0.84930	1.53001	0.47144	1.89744	0.85015	1.53154	0.47191	1.89934
	0.4	0.78527	1.45213	0.42465	3.11891	0.78605	1.45358	0.42507	3.12203
	0.5	0.70964	1.33859	0.37621	4.40396	0.71035	1.33993	0.37658	4.40837
	0.75	0.53313	1.04266	0.27260	7.44423	0.53313	1.04266	0.27260	7.44423
	1	0.42231	0.83896	0.21258	10.48340	0.42231	0.83896	0.21258	10.48340
	1.5	0.27455	0.55067	0.13689	15.33470	0.27483	0.55122	0.13702	15.35003
	2	0.19676	0.39569	0.09784	19.53692	0.19636	0.39490	0.09764	19.49784
	3	0.11951	0.24048	0.05940	26.70103	0.11939	0.24024	0.05934	26.67432
	4	0.07905	0.15748	0.03968	31.39584	0.07897	0.15732	0.03964	31.36445
	5	0.05446	0.10872	0.02728	33.79889	0.05424	0.10829	0.02717	33.66369
	7.5	0.02486	0.04949	0.01249	34.70982	0.02481	0.04939	0.01246	34.64040
	10	0.01357	0.02676	0.00688	33.67960	0.01351	0.02665	0.00685	33.54488
PGA (g)	0	0.35294	0.61058	0.20402	0.00088	0.35294	0.61058	0.20402	0.00088
PGV (cm/s)	-1	41.10465	74.39518	22.71105	0.10204	NA	NA	NA	NA

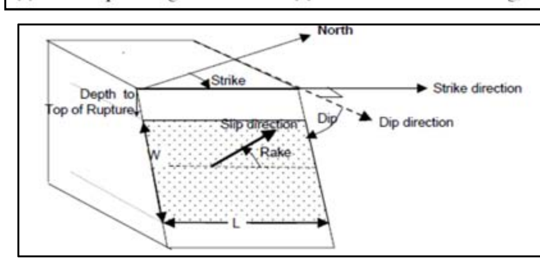


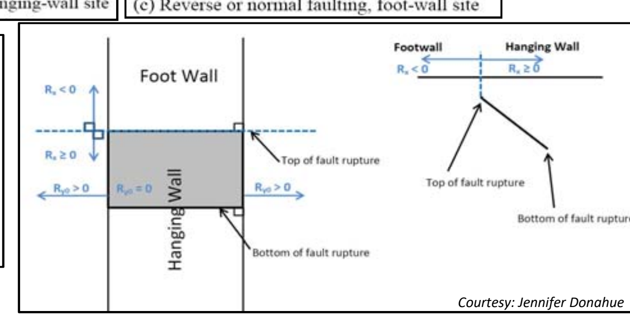
Input variables | **Errors and warnings**

M_w	6.7	
R_{RUP} (km)	7.82	
R_{JB} (km)	7.62	
R_X (km)	-2.8	
Ry0 (km)	999	If unknown use 999
V_{S30} (m/sec)	357	
U (BSSA13)	0	1: Unspecified fault mech.
F_{RV}	0	1: reverse fault
F_{NM}	0	1: normal fault
F_{HW}	0	1: hanging wall side
Dip (deg)	70	
Z_{TOR} (km)	999	If unknown use 999
Z_{HYP} (km)	999	If unknown use 999
$Z_{1.0}$ (km)	0.7	If unknown use 999
$Z_{2.5}$ (km)	5.05	If unknown use 999
W (km)	16.57	If unknown use 999
Vs30Flag	measured	Choose options for V_{S30} from the list
F_{AS}	no	Aftershock effect is not applicable.
Region	California	Choose region from the list









Calculated Variables/Flags

ΔDPP	0	Always 0 for median calcs.
PGA_r (g)	0.264	
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)	16.57			15.430		
$Z_{1.0}$ (km)	0.700	0.700			0.406	
$\delta Z_{1.0}$ (km)	0.294		0.294			
$Z_{2.5}$ (V_{S30}=1100)(km)	5.050			0.398		
$Z_{2.5}$ (V_{S30})(km)	5.050			1.440		
Z_{hyp} (km)	999.00			10.237		
Z_{tor} (km)	999.00			0.501	0.501	
Z_{BOR} (km)	-			15.000		

ACKNOWLEDGEMENTS

CEA CALIFORNIA EARTHQUAKE AUTHORITY | Caltrans | PG&E

Nick Gregor, Bechtel
Silvia Mazzoni, Consultant

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

PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

WEIGHTED AVERAGE of 2014 NGA WEST-2 GMPEs

Last updated: 04 14 15

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

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

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

GMPE averaging	Geometric

Weighted average of the natural logarithm of the spectral values

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

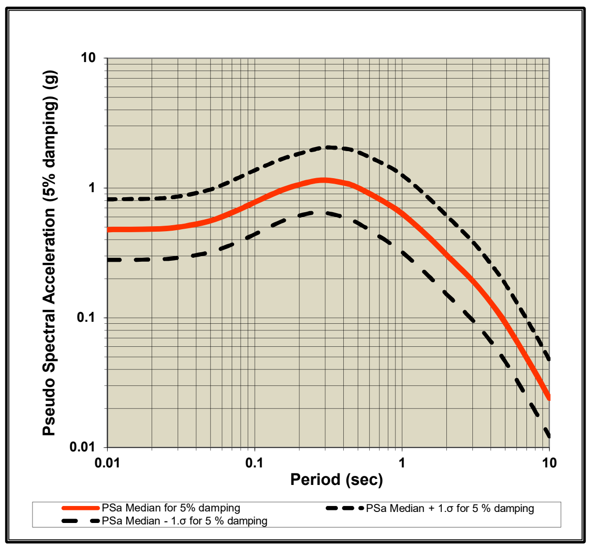
# of std. dev.	1
Damping ratio (%)	5

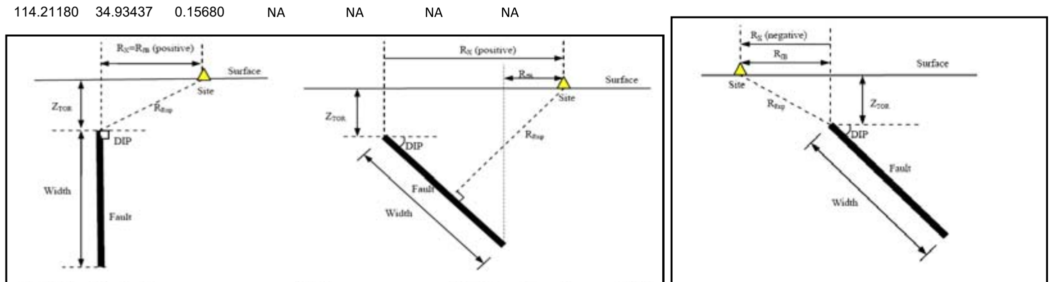
Modification factors are calculated in Sheet DSF

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

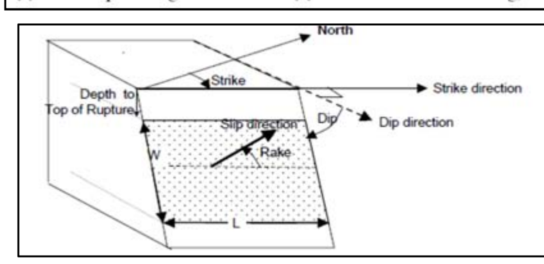
RotD50 Horizontal Component of PGA, PGV and *IMs*

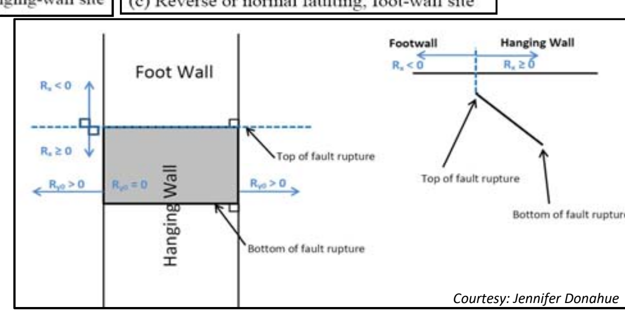
GMP	*T* (s)	Baseline: 5% Damping PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	User defined: 5% Damping PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	**0.01**	0.47738	0.81821	0.27853	0.00119	0.47738	0.81821	0.27853	0.00119
	0.02	0.48181	0.82638	0.28091	0.00478	0.48181	0.82638	0.28091	0.00478
	0.03	0.49806	0.85824	0.28904	0.01113	0.49806	0.85824	0.28904	0.01113
	0.05	0.55803	0.97326	0.31995	0.03463	0.55803	0.97326	0.31995	0.03463
	0.075	0.66625	1.17585	0.37751	0.09303	0.66758	1.17820	0.37826	0.09322
	0.1	0.77446	1.36606	0.43906	0.19225	0.77601	1.36879	0.43994	0.19263
	0.15	0.95226	1.65479	0.54798	0.53187	0.95416	1.65810	0.54908	0.53293
	0.2	1.05742	1.83232	0.61023	1.04996	1.05848	1.83416	0.61084	1.05101
	0.25	1.12307	1.96366	0.64231	1.74242	1.12756	1.97151	0.64488	1.74939
	0.3	1.14664	2.04516	0.64287	2.56174	1.14778	2.04720	0.64352	2.56430
	0.4	1.09493	2.01230	0.59577	4.34883	1.09602	2.01432	0.59637	4.35318
	0.5	1.00685	1.88949	0.53652	6.24842	1.00785	1.89138	0.53705	6.25467
	0.75	0.78548	1.53041	0.40315	10.96796	0.78548	1.53041	0.40315	10.96796
	1	0.63522	1.25797	0.32076	15.76855	0.63459	1.25671	0.32044	15.75278
	1.5	0.42278	0.84576	0.21134	23.61349	0.42320	0.84660	0.21155	23.63710
	2	0.30620	0.61437	0.15261	30.40418	0.30559	0.61314	0.15231	30.34337
	3	0.19346	0.38846	0.09635	43.22184	0.19327	0.38807	0.09625	43.17861
	4	0.13085	0.26014	0.06581	51.96952	0.13072	0.25988	0.06575	51.91755
	5	0.09232	0.18393	0.04634	57.29551	0.09195	0.18320	0.04616	57.06633
	7.5	0.04305	0.08554	0.02167	60.11434	0.04301	0.08545	0.02165	60.05422
	10	0.02407	0.04738	0.01223	59.75216	0.02397	0.04719	0.01218	59.51315
PGA (g)	**0**	0.47499	0.81348	0.27734	0.00118	0.47499	0.81348	0.27734	0.00118
PGV (cm/s)	**-1**	63.16579	114.21180	34.93437	0.15680	NA	NA	NA	NA





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





Courtesy: Jennifer Donahue

Input variables | **Errors and warnings**

- ***M_w***: 6.97
- ***R_{RUP} (km)***: 4.5
- ***R_{JB} (km)***: 4.41
- ***R_X (km)***: 0.92
- ***Ry0 (km)***: 999 — *If unknown use 999*
- ***V_{S30} (m/sec)***: 357
- ***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)***: 74
- ***Z_{TOR} (km)***: 0 — *If unknown use 999*
- ***Z_{HYP} (km)***: 999 — *If unknown use 999*
- ***$Z_{1.0}$ (km)***: 0.7 — *If unknown use 999*
- ***$Z_{2.5}$ (km)***: 5.05 — *If unknown use 999*
- ***W (km)***: 15.51 — *If unknown use 999*
- ***Vs30Flag***: measured — *Choose options for V_{S30} from the list*
- ***F_{AS}***: no — *Aftershock effect is not applicable.*
- ***Region***: California — *Choose region from the list*

Calculated Variables/Flags

- ***ΔDPP***: 0 — *Always 0 for median calcs.*
- ***PGA_r (g)***: 0.362
- ***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 illustation
R_{JB} = Closest distance to surface projection of coseismic rupture (km). See Figures a, b and c for illustation
R_X = Horizontal distance from top of rupture measured perpendicular to fault strike (km). See Figures a, b and c for illustation
R_{y0} = The horizontal distance off the end of the rupture measured parallel to strike (km)
V_{S30} = The average shear-wave velocity (m/s) over a subsurface depth of 30 m
U = Unspecified-mechanism factor: 1 for unspecified; 0 otherwise
F_{RV} = Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
F_{NM} = Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal
F_{HW} = Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise
Dip = Average dip of rupture plane (degrees)
Z_{TOR} = Depth to top of coseismic rupture (km)
Z_{HYP} = Hypocentral depth from the earthquake
$Z_{1.0}$ = Depth to Vs=1 km/sec
$Z_{2.5}$ = Depth to Vs=2.5 km/sec
W = Fault rupture width (km)
$V_{s30flag}$ = 1 for measured, 0 for inferred Vs30
F_{AS} = 0 for mainshock; 1 for aftershock
Region = Specific regions considered in the models, Click on Region to see codes
ΔDPP = Directivity term, direct point parameter; uses 0 for median predictions
PGA_r (g) = Peak ground acceleration on rock (g), this specific cell is updated in the cell for BSSA14 and CB14, for others it is taken account for in the macros
Z_{BOT} (km) = The depth to the bottom of the seismogenic crust
Z_{BOR} (km) = The depth to the bottom of the rupture plane
SS = 1 for strike slip, automatically updated in the cell

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

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

DEFAULTs	USER defined	ASK14	BSSA14	CB14	CY14	I14
W (km)	15.51			15.437		
$Z_{1.0}$ (km)	0.700	0.700			0.406	
$\delta Z_{1.0}$ (km)	0.294		0.294			
$Z_{2.5}$ (V_{S30}=1100)(km)	5.050			0.398		
$Z_{2.5}$ (V_{S30})(km)	5.050			1.440		
Z_{hyp} (km)	999.00			10.387		
Z_{tor} (km)	0.00			0.161	0.161	
Z_{BOR} (km)	-			15.000		

ACKNOWLEDGEMENTS

Nick Gregor, Bechtel
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All NGA West-2 participants are acknowledged for their constructive comments and feedback.

Malibu Coast Alt 2 (0)

PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

PEER

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

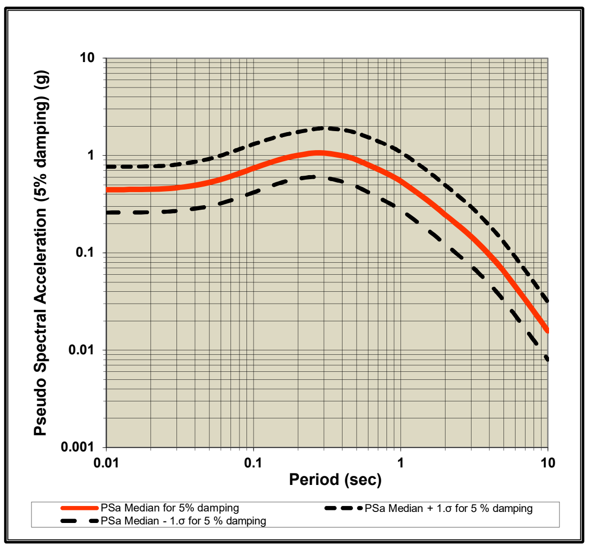
# of std. dev. | 1

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*

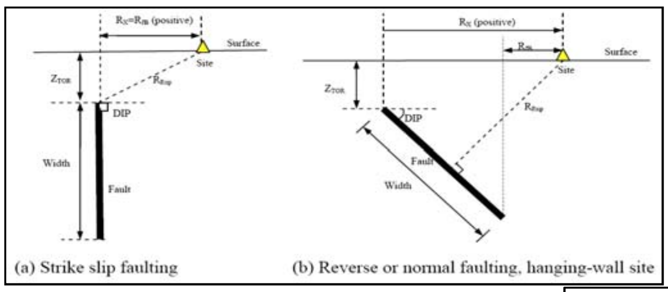
GMP	T (s)	Baseline: 5% Damping				User defined: 5% Damping			
		PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	0.01	0.44569	0.76670	0.25908	0.00111	0.44569	0.76670	0.25908	0.00111
	0.02	0.45010	0.77482	0.26146	0.00447	0.45010	0.77482	0.26146	0.00447
	0.03	0.46651	0.80676	0.26976	0.01042	0.46651	0.80676	0.26976	0.01042
	0.05	0.52669	0.92170	0.30097	0.03269	0.52669	0.92170	0.30097	0.03269
	0.075	0.63277	1.12024	0.35742	0.08836	0.63403	1.12248	0.35814	0.08853
	0.1	0.73771	1.30531	0.41693	0.18313	0.73919	1.30792	0.41776	0.18349
	0.15	0.90643	1.58065	0.51980	0.50627	0.90824	1.58381	0.52084	0.50728
	0.2	1.00020	1.74001	0.57493	0.99314	1.00120	1.74175	0.57551	0.99414
	0.25	1.05170	1.84668	0.59896	1.63170	1.05486	1.85222	0.60075	1.63659
	0.3	1.06156	1.90138	0.59268	2.37166	1.06262	1.90328	0.59327	2.37403
	0.4	0.99502	1.83528	0.53946	3.95201	0.99602	1.83712	0.54000	3.95596
	0.5	0.90290	1.70016	0.47950	5.60335	0.90381	1.70186	0.47998	5.60895
	0.75	0.68853	1.34558	0.35231	9.61412	0.68853	1.34558	0.35231	9.61412
	1	0.54610	1.08455	0.27498	13.55632	0.54556	1.08347	0.27470	13.54276
	1.5	0.35067	0.70337	0.17482	19.58584	0.35102	0.70408	0.17500	19.60543
	2	0.24782	0.49850	0.12320	24.60710	0.24732	0.49751	0.12295	24.55788
	3	0.14818	0.29829	0.07361	33.10645	0.14804	0.29799	0.07354	33.07334
	4	0.09547	0.19028	0.04790	37.91904	0.09538	0.19009	0.04785	37.88112
	5	0.06504	0.12990	0.03257	40.36312	0.06478	0.12938	0.03243	40.20167
	7.5	0.02884	0.05743	0.01448	40.26544	0.02881	0.05738	0.01446	40.22517
	10	0.01580	0.03118	0.00801	39.22647	0.01575	0.03108	0.00799	39.10879
PGA (g)	0	0.44346	0.76229	0.25799	0.00110	0.44346	0.76229	0.25799	0.00110
PGV (cm/s)	-1	53.04955	95.94480	29.33202	0.13169	NA	NA	NA	NA



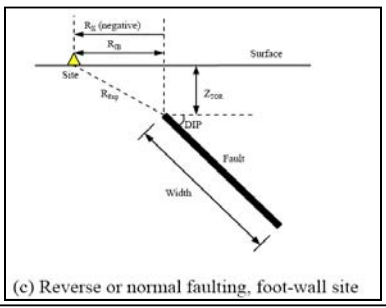
Input variables	Errors and warnings
M_w: 6.64	
R_{RUP} (km): 4.72	
R_{JB} (km): 4.41	
R_X (km): 0.65	
Ry0 (km): 999	If unknown use 999
V_{S30} (m/sec): 357	
U (BSSA13): 0	1: Unspecified fault mech.
F_{RV}: 0	1: reverse fault
F_{NM}: 0	1: normal fault
F_{HW}: 1	1: hanging wall side
Dip (deg): 75	
Z_{TOR} (km): 0	If unknown use 999
Z_{HYP} (km): 999	If unknown use 999
$Z_{1.0}$ (km): 0.7	If unknown use 999
$Z_{2.5}$ (km): 5.05	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

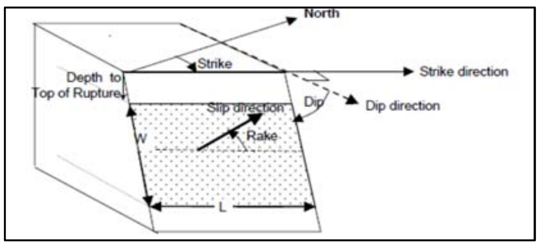
Variable	Value	Comment
ΔDPP	0	Always 0 for median calcs.
PGA_r (g)	0.340	
Z_{BOT} (km) (CB14)	15	Enter for default W calcs
SS	1	auto calculated
$V_{s30Flag}$	1	measured
F_{AS}	0	Aftershock effect is not applicable.
Region	0	California
Option for Sa value	1	Weighted average of the natural logarithm of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

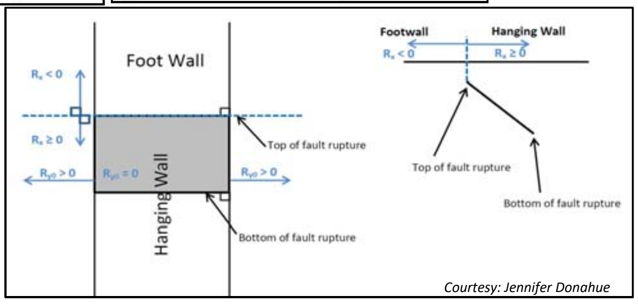


(a) Strike slip faulting (b) Reverse or normal faulting, hanging-wall site



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





Courtesy: Jennifer Donahue

Definition of Parameters

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

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

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

DEFAULTs	USER defined	ASK14	BSSA14	CB14	CY14	I14
W (km)	7.27			14.906		
$Z_{1.0}$ (km)	0.700	0.700			0.406	
$\delta Z_{1.0}$ (km)	0.294		0.294			
$Z_{2.5}$ (V_{S30}=1100)(km)	5.050			0.398		
$Z_{2.5}$ (V_{S30})(km)	5.050			1.440		
Z_{hyp} (km)	999.00			9.780		
Z_{tor} (km)	0.00			0.602	0.602	
Z_{BOR} (km)	-			15.000		

ACKNOWLEDGEMENTS

CEA CALIFORNIA EARTHQUAKE AUTHORITY · Caltrans · PG&E

Nick Gregor, Bechtel
Silvia Mazzoni, Consultant

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

Newport-Inglewood Alt 1 (8)

PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

WEIGHTED AVERAGE of 2014 NGA WEST-2 GMPEs

Last updated: 04 14 15

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

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

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

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

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

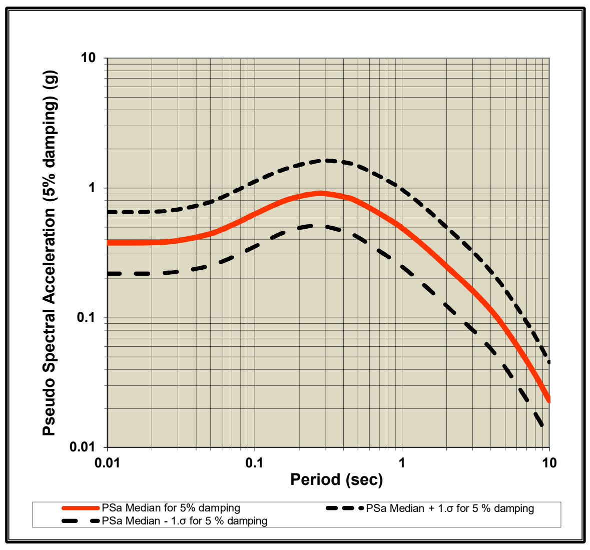
# of std. dev.	1	
Damping ratio (%)	5	*Modification factors are calculated in Sheet DSF*

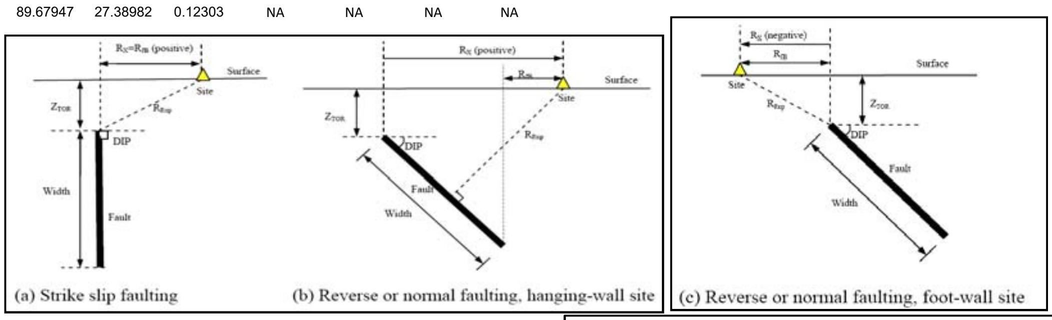
ASK14 Abrahamson & Silva & Kamai 2014 NGA West-2 Model
BSSA14 Boore & Stewart & Seyhan & Atkinson 2014 NGA West-2 Model
CB14 Campbell & Bozorgnia 2014 NGA West-2 Model
CY14 Chiou & Youngs 2014 NGA West-2 Model
I14 Idriss 2014 NGA West-2 Model

RotD50 Horizontal Component of PGA, PGV and *IMs*

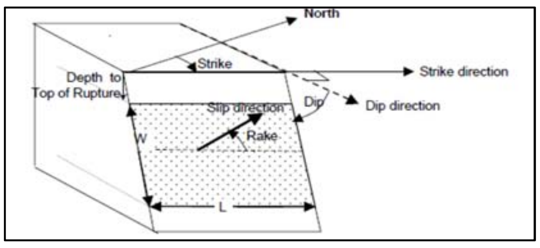
Input variables	Errors and warnings
M_w	
7.15	
R_{RUP} (km)	
8.31	
R_{JB} (km)	
8.17	
R_X (km)	
-7.67	
Ry0 (km)	*If unknown use 999*
999	
V_{S30} (m/sec)	
357	
U (BSSA13)	*1: Unspecified fault mech.*
0	
F_{RV}	*1: reverse fault*
0	
F_{NM}	*1: normal fault*
0	
F_{HW}	*1: hanging wall side*
0	
Dip (deg)	
88	
Z_{TOR} (km)	*If unknown use 999*
0	
Z_{HYP} (km)	*If unknown use 999*
999	
$Z_{1.0}$ (km)	*If unknown use 999*
0.7	
$Z_{2.5}$ (km)	*If unknown use 999*
5.05	
W (km)	*If unknown use 999*
13.51	
Vs30Flag	
measured	*Choose options for V_{S30} from the list*
F_{AS}	
no	*Aftershock effect is not applicable.*
Region	
California	*Choose region from the list*

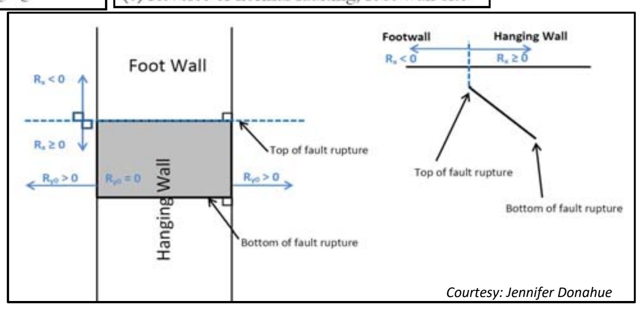
GMP	T (s)	Baseline: 5% Damping				User defined: 5% Damping			
		PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	0.01	0.37657	0.65002	0.21816	0.00093	0.37657	0.65002	0.21816	0.00093
	0.02	0.37892	0.65469	0.21931	0.00376	0.37892	0.65469	0.21931	0.00376
	0.03	0.39262	0.68199	0.22604	0.00877	0.39262	0.68199	0.22604	0.00877
	0.05	0.44329	0.77967	0.25203	0.02751	0.44329	0.77967	0.25203	0.02751
	0.075	0.53550	0.95273	0.30098	0.07477	0.53657	0.95464	0.30159	0.07492
	0.1	0.62730	1.11564	0.35272	0.15572	0.62919	1.11899	0.35378	0.15619
	0.15	0.77550	1.35988	0.44224	0.43314	0.77705	1.36260	0.44312	0.43401
	0.2	0.85734	1.49913	0.49030	0.85129	0.85905	1.50212	0.49128	0.85299
	0.25	0.89711	1.58123	0.50898	1.39185	0.89980	1.58597	0.51051	1.39603
	0.3	0.90463	1.62379	0.50397	2.02106	0.90553	1.62542	0.50448	2.02308
	0.4	0.85513	1.57629	0.46390	3.39640	0.85598	1.57787	0.46437	3.39979
	0.5	0.78592	1.47798	0.41792	4.87738	0.78671	1.47946	0.41834	4.88226
	0.75	0.60659	1.18304	0.31102	8.47005	0.60659	1.18304	0.31102	8.47005
	1	0.49147	0.97381	0.24804	12.20007	0.49098	0.97284	0.24779	12.18787
	1.5	0.33522	0.67070	0.16754	18.72291	0.33555	0.67137	0.16771	18.74163
	2	0.24875	0.49909	0.12398	24.69996	0.24826	0.49809	0.12374	24.65056
	3	0.16235	0.32594	0.08087	36.27187	0.16219	0.32561	0.08079	36.23560
	4	0.11465	0.22790	0.05768	45.53780	0.11454	0.22767	0.05762	45.49226
	5	0.08307	0.16545	0.04170	51.55047	0.08282	0.16496	0.04158	51.39582
	7.5	0.04122	0.08188	0.02075	57.55470	0.04114	0.08171	0.02071	57.43959
	10	0.02311	0.04548	0.01174	57.37064	0.02302	0.04530	0.01170	57.14116
PGA (g)	0	0.37479	0.64644	0.21729	0.00093	0.37479	0.64644	0.21729	0.00093
PGV (cm/s)	-1	49.56112	89.67947	27.38982	0.12303	NA	NA	NA	NA





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





Courtesy: Jennifer Donahue

Calculated Variables/Flags

ΔDPP	0	*Always 0 for median calcs.*
PGA_r (g)	0.285	
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 illustation
R_{JB} = Closest distance to surface projection of coseismic rupture (km). See Figures a, b and c for illustation
R_X = Horizontal distance from top of rupture measured perpendicular to fault strike (km). See Figures a, b and c for illustation
R_{y0} = The horizontal distance off the end of the rupture measured parallel to strike (km)
V_{S30} = The average shear-wave velocity (m/s) over a subsurface depth of 30 m
U = Unspecified-mechanism factor: 1 for unspecified; 0 otherwise
F_{RV} = Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
F_{NM} = Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal
F_{HW} = Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise
Dip = Average dip of rupture plane (degrees)
Z_{TOR} = Depth to top of coseismic rupture (km)
Z_{HYP} = Hypocentral depth from the earthquake
$Z_{1.0}$ = Depth to Vs=1 km/sec
$Z_{2.5}$ = Depth to Vs=2.5 km/sec
W = Fault rupture width (km)
$V_{s30flag}$ = 1 for measured, 0 for inferred Vs30
F_{AS} = 0 for mainshock; 1 for aftershock
Region = Specific regions considered in the models, Click on Region to see codes
ΔDPP = Directivity term, direct point parameter; uses 0 for median predictions
PGA_r (g) = Peak ground acceleration on rock (g), this specific cell is updated in the cell for BSSA14 and CB14, for others it is taken account for in the macros
Z_{BOT} (km) = The depth to the bottom of the seismogenic crust
Z_{BOR} (km) = The depth to the bottom of the rupture plane
SS = 1 for strike slip, automatically updated in the cell

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

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

DEFAULTs	USER defined	ASK14	BSSA14	CB14	CY14	I14
W (km)	13.51			14.970		
$Z_{1.0}$ (km)	0.700	0.700			0.406	
$\delta Z_{1.0}$ (km)	0.294		0.294			
$Z_{2.5}$ (V_{S30}=1100)(km)	5.050			0.398		
$Z_{2.5}$ (V_{S30})(km)	5.050			1.440		
Z_{hyp} (km)	999.00			10.265		
Z_{tor} (km)	0.00			0.039	0.039	
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.

Newport-Inglewood Alt 2 (8)

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*

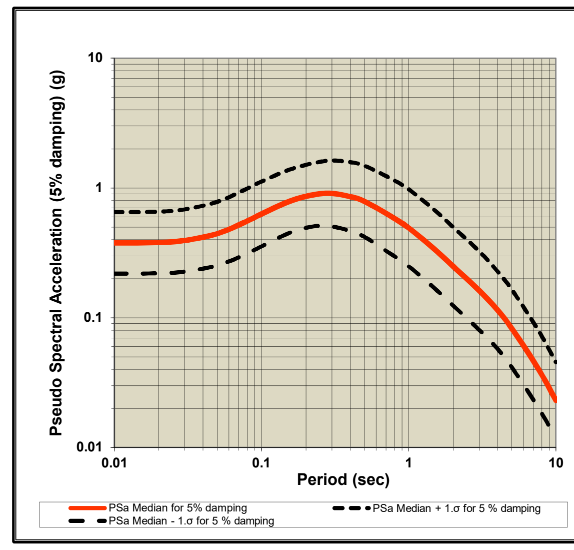
GMPEs	ASK14	BSSA14	CB14	CY14	I14
Weight	0.25	0.25	0.25	0.25	0

# 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*

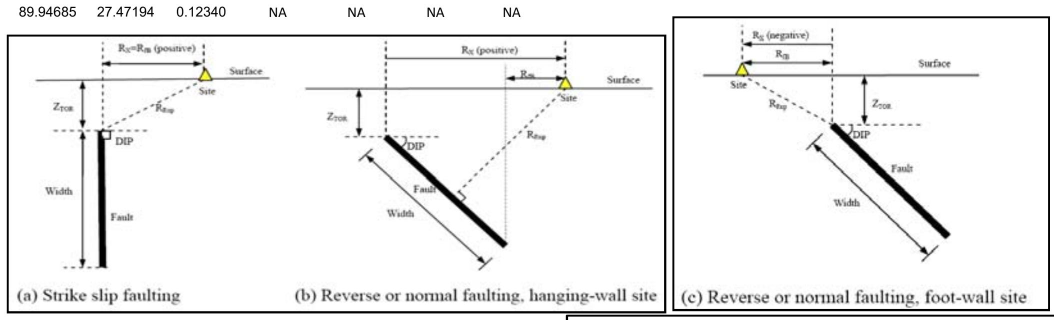
GMP	*T* (s)	Baseline: 5% Damping				User defined: 5% Damping			
		PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	**0.01**	0.37746	0.65151	0.21869	0.00094	0.37746	0.65151	0.21869	0.00094
	0.02	0.37983	0.65621	0.21985	0.00377	0.37983	0.65621	0.21985	0.00377
	0.03	0.39355	0.68355	0.22659	0.00879	0.39355	0.68355	0.22659	0.00879
	0.05	0.44430	0.78138	0.25263	0.02757	0.44430	0.78138	0.25263	0.02757
	0.075	0.53666	0.95472	0.30166	0.07494	0.53773	0.95663	0.30226	0.07508
	0.1	0.62862	1.11788	0.35349	0.15605	0.63050	1.12123	0.35455	0.15651
	0.15	0.77709	1.36255	0.44319	0.43403	0.77865	1.36527	0.44408	0.43490
	0.2	0.85915	1.50215	0.49138	0.85309	0.86086	1.50516	0.49237	0.85479
	0.25	0.89913	1.58466	0.51016	1.39499	0.90183	1.58942	0.51169	1.39917
	0.3	0.90677	1.62754	0.50520	2.02585	0.90768	1.62917	0.50571	2.02788
	0.4	0.85730	1.58023	0.46510	3.40500	0.85815	1.58181	0.46556	3.40841
	0.5	0.78798	1.48180	0.41902	4.89011	0.78876	1.48328	0.41944	4.89500
	0.75	0.60829	1.18633	0.31190	8.49371	0.60829	1.18633	0.31190	8.49371
	1	0.49289	0.97661	0.24876	12.23527	0.49239	0.97564	0.24851	12.22304
	1.5	0.33618	0.67263	0.16802	18.77679	0.33652	0.67330	0.16819	18.79557
	2	0.24946	0.50050	0.12433	24.76996	0.24896	0.49950	0.12409	24.72042
	3	0.16281	0.32686	0.08110	36.37474	0.16265	0.32653	0.08102	36.33836
	4	0.11497	0.22852	0.05784	45.66264	0.11485	0.22829	0.05778	45.61698
	5	0.08329	0.16590	0.04182	51.68872	0.08304	0.16540	0.04169	51.53365
	7.5	0.04132	0.08208	0.02080	57.69529	0.04124	0.08191	0.02076	57.57990
	10	0.02317	0.04559	0.01177	57.51298	0.02308	0.04541	0.01173	57.28293
PGA (g)	**0**	0.37567	0.64793	0.21782	0.00093	0.37567	0.64793	0.21782	0.00093
PGV (cm/s)	**-1**	49.70930	89.94685	27.47194	0.12340	NA	NA	NA	NA



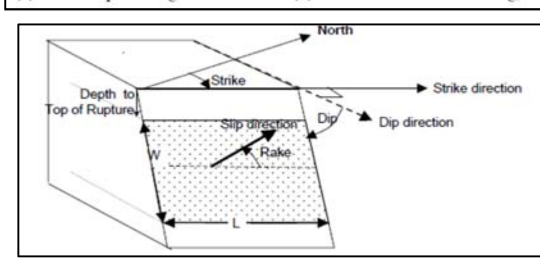
Input variables	Errors and warnings
M_w	
7.15	
R_{RUP} (km)	
8.27	
R_{JB} (km)	
8.13	
R_X (km)	
-7.33	
Ry0 (km)	*If unknown use 999*
999	
V_{S30} (m/sec)	
357	
U (BSSA13)	*1: Unspecified fault mech.*
0	
F_{RV}	*1: reverse fault*
0	
F_{NM}	*1: normal fault*
0	
F_{HW}	*1: hanging wall side*
0	
Dip (deg)	
90	
Z_{TOR} (km)	*If unknown use 999*
0	
Z_{HYP} (km)	*If unknown use 999*
999	
$Z_{1.0}$ (km)	*If unknown use 999*
0.7	
$Z_{2.5}$ (km)	*If unknown use 999*
5.05	
W (km)	*If unknown use 999*
13.59	
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*

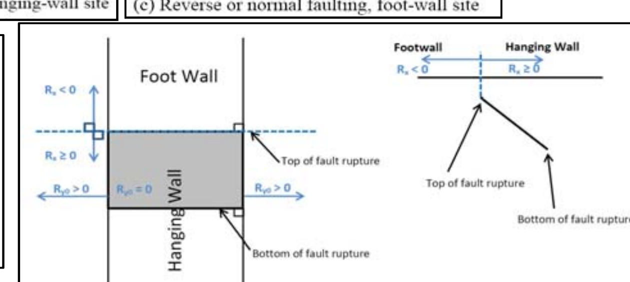
Calculated Variables/Flags

ΔDPP	*Always 0 for median calcs.*
0	
PGA_r (g)	
0.286	
Z_{BOT} (km) (CB14)	*Enter for default W calcs*
15	
SS	
1	*auto calculated*
$V_{s30Flag}$	
1	*measured*
F_{AS}	
0	*Aftershock effect is not applicable.*
Region	
0	*California*
Option for Sa value	
1	*Weighted average of the natural logarithm of the spectral values*



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





Courtesy: Jennifer Donahue

Definition of Parameters

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

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

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

DEFAULTs	USER defined	ASK14	BSSA14	CB14	CY14	I14
W (km)	13.59			14.961		
$Z_{1.0}$ (km)	0.700	0.700			0.406	
$\delta Z_{1.0}$ (km)	0.294		**0.294**			
$Z_{2.5}$ (V_{S30}=1100)(km)	5.050			0.398		
$Z_{2.5}$ (V_{S30})(km)	5.050			1.440		
Z_{hyp} (km)	999.00			**10.265**		
Z_{tor} (km)	0.00			0.039	0.039	
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.

Palos Verdes (15)

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*

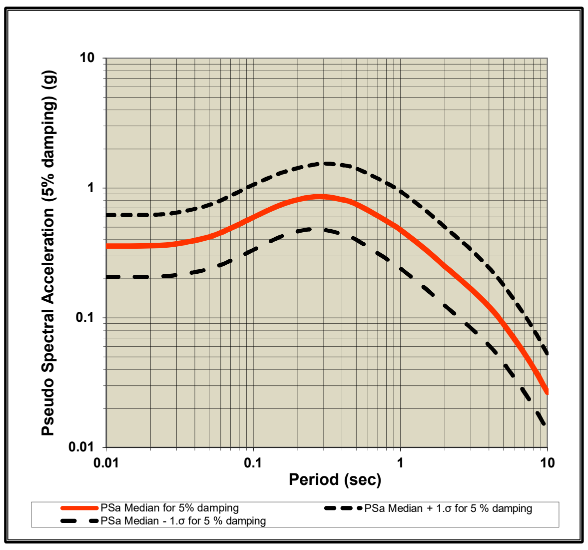
GMPEs	ASK14	BSSA14	CB14	CY14	I14
Weight	0.25	0.25	0.25	0.25	0

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

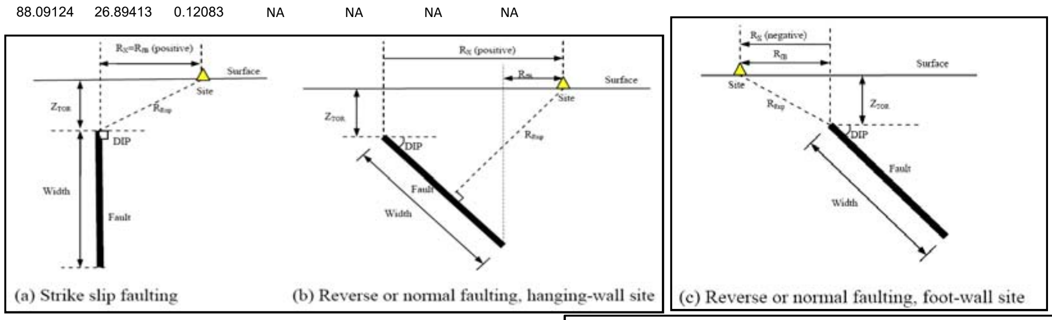
ASK14 Abrahamson & Silva & Kamai 2014 NGA West-2 Model
BSSA14 Boore & Stewart & Seyhan & Atkinson 2014 NGA West-2 Model
CB14 Campbell & Bozorgnia 2014 NGA West-2 Model
CY14 Chiou & Youngs 2014 NGA West-2 Model
I14 Idriss 2014 NGA West-2 Model

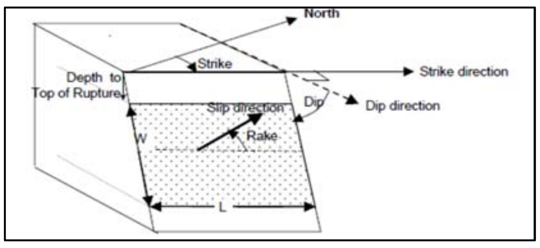
RotD50 Horizontal Component of PGA, PGV and *IMs*

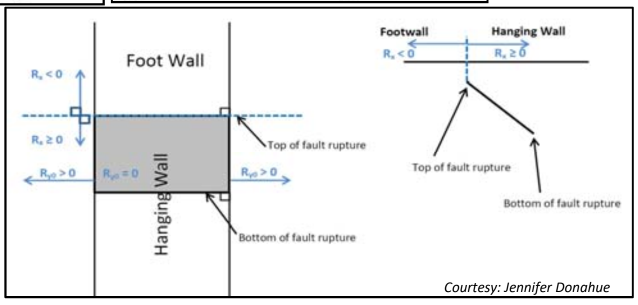
GMP	T (s)	Baseline: 5% Damping				User defined: 5% Damping			
		PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	0.01	0.35686	0.61712	0.20636	0.00089	0.35686	0.61712	0.20636	0.00089
	0.02	0.35858	0.62071	0.20715	0.00356	0.35858	0.62071	0.20715	0.00356
	0.03	0.37124	0.64619	0.21329	0.00829	0.37087	0.64554	0.21307	0.00829
	0.05	0.41879	0.73827	0.23757	0.02599	0.41879	0.73827	0.23757	0.02599
	0.075	0.50580	0.90199	0.28363	0.07063	0.50681	0.90380	0.28420	0.07077
	0.1	0.59228	1.05591	0.33222	0.14703	0.59406	1.05908	0.33322	0.14747
	0.15	0.73158	1.28612	0.41614	0.40861	0.73304	1.28869	0.41697	0.40943
	0.2	0.80942	1.41867	0.46181	0.80371	0.81104	1.42151	0.46273	0.80532
	0.25	0.84707	1.49598	0.47963	1.31421	0.84961	1.50047	0.48107	1.31815
	0.3	0.85551	1.53806	0.47586	1.91133	0.85723	1.54114	0.47681	1.91516
	0.4	0.81254	1.49915	0.44040	3.22724	0.81335	1.50065	0.44084	3.23047
	0.5	0.75067	1.41257	0.39892	4.65857	0.75142	1.41398	0.39932	4.66323
	0.75	0.58274	1.13685	0.29871	8.13695	0.58274	1.13685	0.29871	8.13695
	1	0.47511	0.94156	0.23974	11.79404	0.47511	0.94156	0.23974	11.79404
	1.5	0.33139	0.66309	0.16561	18.50908	0.33172	0.66375	0.16578	18.52758
	2	0.25003	0.50167	0.12462	24.82706	0.24953	0.50066	0.12437	24.77741
	3	0.16790	0.33708	0.08363	37.51186	0.16774	0.33674	0.08355	37.47434
	4	0.12203	0.24256	0.06139	48.46828	0.12191	0.24232	0.06133	48.41982
	5	0.09037	0.18000	0.04537	56.08305	0.09010	0.17946	0.04523	55.91480
	7.5	0.04678	0.09293	0.02355	65.32626	0.04664	0.09265	0.02348	65.13028
	10	0.02672	0.05258	0.01358	66.33266	0.02661	0.05237	0.01353	66.06733
PGA (g)	0	0.35521	0.61378	0.20556	0.00088	0.35521	0.61378	0.20556	0.00088
PGV (cm/s)	-1	48.67378	88.09124	26.89413	0.12083	NA	NA	NA	NA



Input variables	Errors and warnings
M$_w$	
7.38	
R$_{RUP}$ ***(km)***	
10.21	
R$_{JB}$ ***(km)***	
10.12	
R$_X$ ***(km)***	
9.98	
Ry0 (km)	*If unknown use 999*
999	
V$_{S30}$ ***(m/sec)***	
357	
U (BSSA13)	*1: Unspecified fault mech.*
0	
F$_{RV}$	*1: reverse fault*
0	
F$_{NM}$	*1: normal fault*
0	
F$_{HW}$	*1: hanging wall side*
1	
Dip (deg)	
90	
Z$_{TOR}$ ***(km)***	*If unknown use 999*
0	
Z$_{HYP}$ ***(km)***	*If unknown use 999*
999	
Z$_{1.0}$ ***(km)***	*If unknown use 999*
0.7	
Z$_{2.5}$ ***(km)***	*If unknown use 999*
5.05	
W (km)	*If unknown use 999*
12.24	
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	*Always 0 for median calcs.*
0	
PGA$_r$ ***(g)***	
0.270	
Z$_{BOT}$ ***(km) (CB14)***	*Enter for default W calcs*
15	
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 illustation
R$_{JB}$ = Closest distance to surface projection of coseismic rupture (km). See Figures a, b and c for illustation
R$_X$ = Horizontal distance from top of rupture measured perpendicular to fault strike (km). See Figures a, b and c for illustation
R$_{y0}$ = The horizontal distance off the end of the rupture measured parallel to strike (km)
V$_{S30}$ = The average shear-wave velocity (m/s) over a subsurface depth of 30 m
U = Unspecified-mechanism factor: 1 for unspecified; 0 otherwise
F$_{RV}$ = Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
F$_{NM}$ = Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal
F$_{HW}$ = Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise
Dip = Average dip of rupture plane (degrees)
Z$_{TOR}$ = Depth to top of coseismic rupture (km)
Z$_{HYP}$ = Hypocentral depth from the earthquake
Z$_{1.0}$ = Depth to Vs=1 km/sec
Z$_{2.5}$ = Depth to Vs=2.5 km/sec
W = Fault rupture width (km)
V$_{s30flag}$ = 1 for measured, 0 for inferred Vs30
F$_{AS}$ = 0 for mainshock; 1 for aftershock
Region = Specific regions considered in the models, Click on Region to see codes
ΔDPP = Directivity term, direct point parameter; uses 0 for median predictions
PGA$_r$ ***(g)*** = Peak ground acceleration on rock (g), this specific cell is updated in the cell for BSSA14 and CB14, for others it is taken account for in the macros
Z$_{BOT}$ ***(km)*** = The depth to the bottom of the seismogenic crust
Z$_{BOR}$ ***(km)*** = The depth to the bottom of the rupture plane
SS = 1 for strike slip, automatically updated in the cell

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

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

DEFAULTs	USER defined	ASK14	BSSA14	CB14	CY14	I14
W (km)	12.24			15.000		
$Z_{1.0}$ (km)	0.700	0.700			0.406	
$\delta Z_{1.0}$ (km)	0.294		0.294			
$Z_{2.5}$ (V_{S30}=1100)(km)	5.050			0.398		
$Z_{2.5}$ (V_{S30})(km)	5.050			1.440		
Z_{hyp} (km)	999.00			10.227		
Z_{tor} (km)	0.00			0.000	0.000	
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 Escarpment (1)

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*

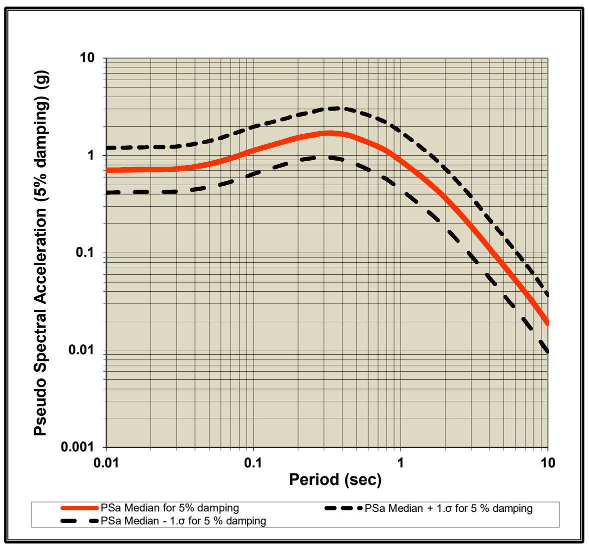
GMPEs	ASK14	BSSA14	CB14	CY14	I14
Weight	0.25	0.25	0.25	0.25	0

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

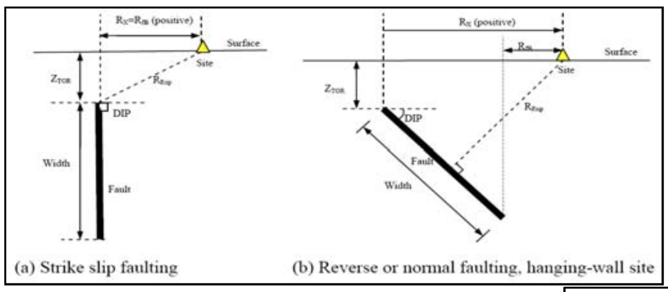
ASK14 Abrahamson & Silva & Kamai 2014 NGA West-2 Model
BSSA14 Boore & Stewart & Seyhan & Atkinson 2014 NGA West-2 Model
CB14 Campbell & Bozorgnia 2014 NGA West-2 Model
CY14 Chiou & Youngs 2014 NGA West-2 Model
I14 Idriss 2014 NGA West-2 Model

RotD50 Horizontal Component of PGA, PGV and *IMs*

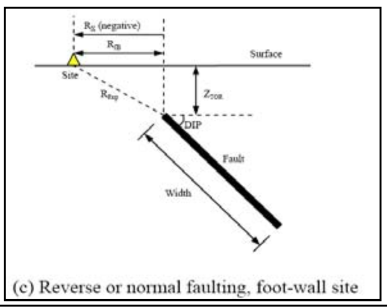
GMP	*T* (s)	Baseline: 5% Damping				User defined: 5% Damping			
		PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	0.01	0.70559	1.19623	0.41618	0.00175	0.70559	1.19623	0.41618	0.00175
	0.02	0.71902	1.21960	0.42390	0.00714	0.71902	1.21960	0.42390	0.00714
	0.03	0.72791	1.24011	0.42726	0.01626	0.72791	1.24011	0.42726	0.01626
	0.05	0.81586	1.40813	0.47270	0.05063	0.81586	1.40813	0.47270	0.05063
	0.075	0.96815	1.69284	0.55370	0.13519	0.97009	1.69622	0.55480	0.13546
	0.1	1.12593	1.96720	0.64443	0.27950	1.12931	1.97310	0.64637	0.28034
	0.15	1.34273	2.30692	0.78153	0.74996	1.34541	2.31153	0.78309	0.75146
	0.2	1.51879	2.59884	0.88760	1.50808	1.52183	2.60404	0.88937	1.51110
	0.25	1.61539	2.78928	0.93554	2.50625	1.62024	2.79765	0.93835	2.51377
	0.3	1.69352	2.98743	0.96002	3.78354	1.69521	2.99042	0.96098	3.78732
	0.4	1.65959	3.03136	0.90858	6.59154	1.66125	3.03439	0.90949	6.59813
	0.5	1.51470	2.82984	0.81075	9.40009	1.51621	2.83267	0.81157	9.40949
	0.75	1.17491	2.28396	0.60439	16.40561	1.17491	2.28396	0.60439	16.40561
	1	0.88151	1.74326	0.44575	21.88236	0.88151	1.74326	0.44575	21.88236
	1.5	0.54571	1.09090	0.27299	30.47976	0.54626	1.09199	0.27326	30.51024
	2	0.36782	0.73773	0.18339	36.52233	0.36708	0.73625	0.18302	36.44928
	3	0.18857	0.37857	0.09393	42.13002	0.18839	0.37819	0.09384	42.08789
	4	0.11097	0.22057	0.05583	44.07419	0.11086	0.22035	0.05577	44.03012
	5	0.07423	0.14786	0.03727	46.06864	0.07401	0.14742	0.03716	45.93044
	7.5	0.03454	0.06861	0.01739	48.22720	0.03447	0.06847	0.01735	48.13075
	10	0.01894	0.03727	0.00962	47.01462	0.01886	0.03712	0.00959	46.82656
PGA (g)	0	0.70015	1.18611	0.41329	0.00174	0.70015	1.18611	0.41329	0.00174
PGV (cm/s)	-1	67.77802	122.40881	37.52884	0.16825	NA	NA	NA	NA



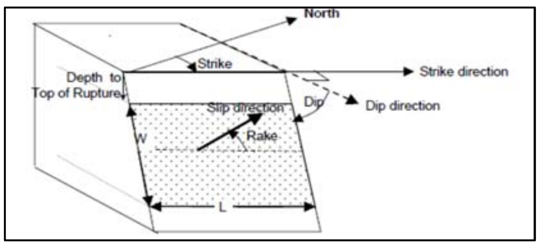
Input variables	Errors and warnings
M_w	
7	
R_{RUP} (km)	
8.71	
R_{JB} (km)	
0	
R_X (km)	
20.7	
Ry0 (km)	*If unknown use 999*
999	
V_{S30} (m/sec)	
357	
U (BSSA13)	*1: Unspecified fault mech.*
0	
F_{RV}	*1: reverse fault*
1	
F_{NM}	*1: normal fault*
0	
F_{HW}	*1: hanging wall side*
1	
Dip (deg)	
17	
Z_{TOR} (km)	*If unknown use 999*
999	
Z_{HYP} (km)	*If unknown use 999*
999	
$Z_{1.0}$ (km)	*If unknown use 999*
0.7	
$Z_{2.5}$ (km)	*If unknown use 999*
5.05	
W (km)	*If unknown use 999*
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*

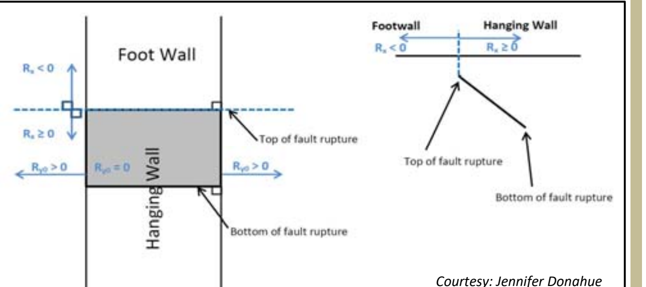


(a) Strike slip faulting (b) Reverse or normal faulting, hanging-wall site



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





Courtesy: Jennifer Donahue

Calculated Variables/Flags

ΔDPP	*Always 0 for median calcs.*
0	
PGA_r (g)	
0.446	
Z_{BOT} (km) (CB14)	*Enter for default W calcs*
15	
SS	
0	*auto calculated*
$V_{s30Flag}$	
1	*measured*
F_{AS}	
0	*Aftershock effect is not applicable.*
Region	
0	*California*
Option for Sa value	
1	*Weighted average of the natural logarithm of the spectral values*

Definition of Parameters

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

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

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

DEFAULTs	USER defined	ASK14	BSSA14	CB14	CY14	I14
W (km)	999.00			**31.253**		
$Z_{1.0}$ (km)	0.700	0.700			0.406	
$\delta Z_{1.0}$ (km)	0.294		**0.294**			
$Z_{2.5}$ (V_{S30}=1100)(km)	5.050			0.398		
$Z_{2.5}$ (V_{S30})(km)	5.050			1.440		
Z_{hyp} (km)	999.00			**5.346**		
Z_{tor} (km)	999.00			**1.672**	**1.672**	
Z_{BOR} (km)	-			10.809		

ACKNOWLEDGEMENTS

Nick Gregor, Bechtel
Silvia Mazzoni, Consultant

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

Santa Monica Alt 1 (0)

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

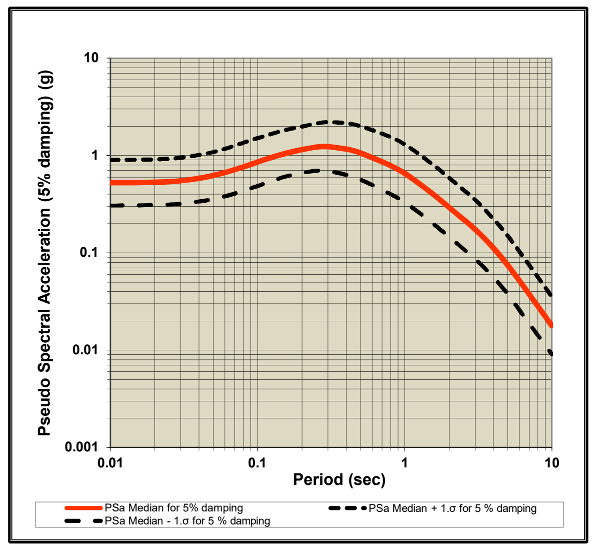
# of std. dev.	1
Damping ratio (%)	5

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*

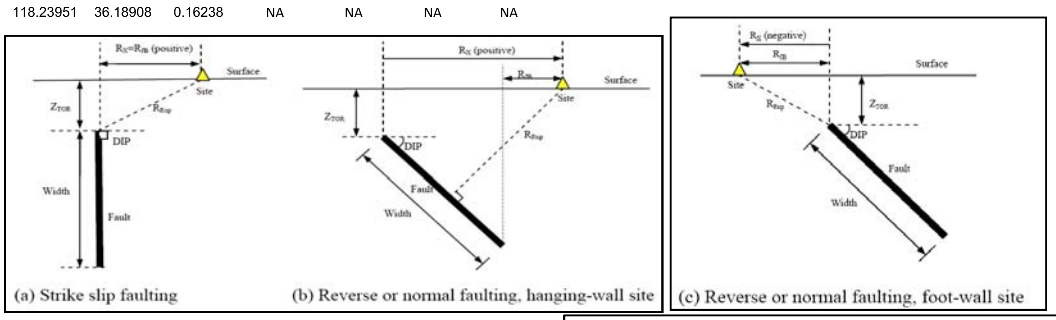
GMP	*T* (s)	Baseline: 5% Damping				User defined: 5% Damping			
		PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	**0.01**	0.52446	0.89954	0.30578	0.00130	0.52446	0.89954	0.30578	0.00130
	0.02	0.53145	0.91206	0.30967	0.00528	0.53145	0.91206	0.30967	0.00528
	0.03	0.55096	0.94966	0.31965	0.01231	0.55096	0.94966	0.31965	0.01231
	0.05	0.62128	1.08346	0.35625	0.03856	0.62128	1.08346	0.35625	0.03856
	0.075	0.73928	1.30426	0.41904	0.10323	0.74076	1.30686	0.41987	0.10343
	0.1	0.85376	1.50506	0.48431	0.21194	0.85547	1.50807	0.48528	0.21236
	0.15	1.03810	1.80260	0.59783	0.57981	1.04017	1.80621	0.59902	0.58097
	0.2	1.14332	1.98049	0.66003	1.13526	1.14446	1.98247	0.66069	1.13639
	0.25	1.21270	2.12111	0.69333	1.88148	1.21876	2.13172	0.69680	1.89088
	0.3	1.23287	2.20116	0.69054	2.75441	1.23287	2.20116	0.69054	2.75441
	0.4	1.16458	2.14416	0.63253	4.62547	1.16575	2.14630	0.63317	4.63010
	0.5	1.06165	1.99651	0.56453	6.58851	1.06271	1.99851	0.56510	6.59509
	0.75	0.82516	1.61174	0.42245	11.52197	0.82433	1.61013	0.42203	11.51045
	1	0.65693	1.30440	0.33085	16.30739	0.65561	1.30179	0.33018	16.27477
	1.5	0.41930	0.84113	0.20902	23.41922	0.41930	0.84113	0.20902	23.41922
	2	0.29350	0.59055	0.14587	29.14302	0.29291	0.58936	0.14558	29.08473
	3	0.17512	0.35264	0.08696	39.12301	0.17476	0.35194	0.08678	39.04477
	4	0.11198	0.22330	0.05616	44.47743	0.11187	0.22307	0.05610	44.43295
	5	0.07541	0.15068	0.03774	46.80060	0.07511	0.15008	0.03759	46.61340
	7.5	0.03254	0.06484	0.01633	45.43311	0.03257	0.06490	0.01635	45.47855
	10	0.01791	0.03534	0.00907	44.44782	0.01785	0.03524	0.00904	44.31448
PGA (g)	**0**	0.52172	0.89414	0.30442	0.00130	0.52172	0.89414	0.30442	0.00130
PGV (cm/s)	**-1**	65.41391	118.23951	36.18908	0.16238	NA	NA	NA	NA

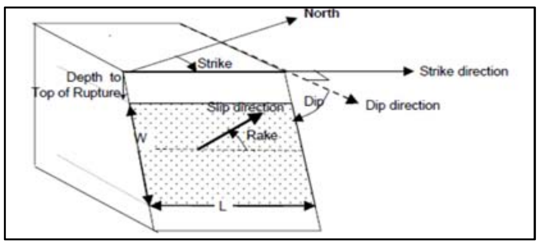


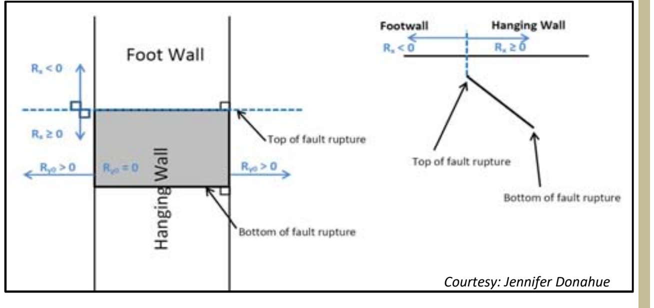
Input variables	Errors and warnings
M_w = 6.58	
R_{RUP} (km) = 1.91	
R_{JB} (km) = 0.68	
R_X (km) = -0.64	
Ry0 (km) = 999	*If unknown use 999*
V_{S30} (m/sec) = 357	
U (BSSA13) = 0	*1: Unspecified fault mech.*
F_{RV} = 0	*1: reverse fault*
F_{NM} = 0	*1: normal fault*
F_{HW} = 0	*1: hanging wall side*
Dip (deg) = 75	
Z_{TOR} (km) = 0	*If unknown use 999*
Z_{HYP} (km) = 999	*If unknown use 999*
$Z_{1.0}$ (km) = 0.7	*If unknown use 999*
$Z_{2.5}$ (km) = 5.05	*If unknown use 999*
W (km) = 16.6	*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) = 0.433	
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 illustation
R_{y0} = The horizontal distance off the end of the rupture measured parallel to strike (km)
V_{S30} = The average shear-wave velocity (m/s) over a subsurface depth of 30 m
U = Unspecified-mechanism factor: 1 for unspecified; 0 otherwise
F_{RV} = Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
F_{NM} = Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal
F_{HW} = Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise
Dip = Average dip of rupture plane (degrees)
Z_{TOR} = Depth to top of coseismic rupture (km)
Z_{HYP} = Hypocentral depth from the earthquake
$Z_{1.0}$ = Depth to Vs=1 km/sec
$Z_{2.5}$ = Depth to Vs=2.5 km/sec
W = Fault rupture width (km)
$V_{s30flag}$ = 1 for measured, 0 for inferred Vs30
F_{AS} = 0 for mainshock; 1 for aftershock
Region = Specific regions considered in the models, Click on Region to see codes
ΔDPP = Directivity term, direct point parameter; uses 0 for median predictions
PGA_r (g) = Peak ground acceleration on rock (g), this specific cell is updated in the cell for BSSA14 and CB14, for others it is taken account for in the macros
Z_{BOT} (km) = The depth to the bottom of the seismogenic crust
Z_{BOR} (km) = The depth to the bottom of the rupture plane
SS = 1 for strike slip, automatically updated in the cell

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

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

DEFAULTs	USER defined	ASK14	BSSA14	CB14	CY14	I14
W (km)	16.60			14.792		
$Z_{1.0}$ (km)	0.700	0.700			0.406	
$\delta Z_{1.0}$ (km)	0.294		0.294			
$Z_{2.5}$ (V_{S30}=1100)(km)	5.050			0.398		
$Z_{2.5}$ (V_{S30})(km)	5.050			1.440		
Z_{hyp} (km)	999.00			9.364		
Z_{tor} (km)	0.00			0.712	0.712	
Z_{BOR} (km)	-			15.000		

ACKNOWLEDGEMENTS

Nick Gregor, Bechtel
Silvia Mazzoni, Consultant

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

Santa Monica Alt 2 (2)

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

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

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

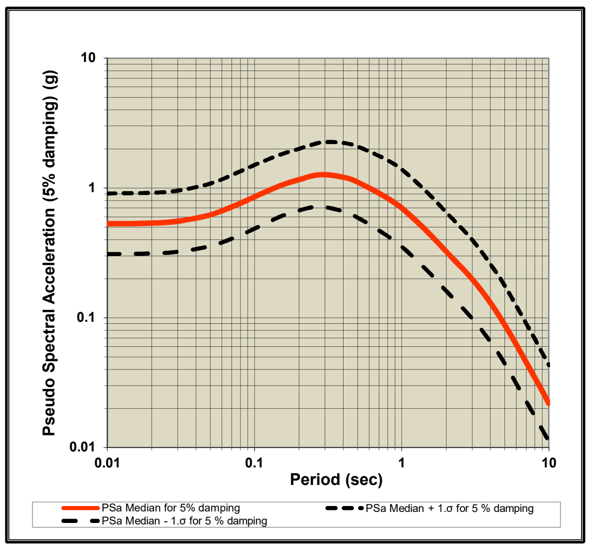
GMPEs	ASK14	BSSA14	CB14	CY14	I14
Weight	0.25	0.25	0.25	0.25	0

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

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

RotD50 Horizontal Component of PGA, PGV and *IMs*

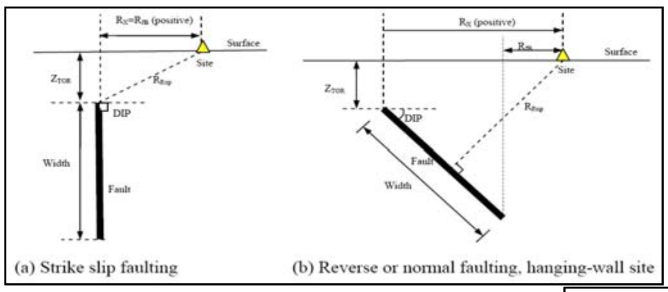
GMP	T (s)	Baseline: 5% Damping: PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	User defined: 5% Damping: PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	0.01	0.52935	0.90601	0.30928	0.00131	0.52935	0.90601	0.30928	0.00131
	0.02	0.53588	0.91774	0.31291	0.00532	0.53588	0.91774	0.31291	0.00532
	0.03	0.55421	0.95333	0.32219	0.01238	0.55421	0.95333	0.32219	0.01238
	0.05	0.62019	1.07960	0.35628	0.03849	0.62019	1.07960	0.35628	0.03849
	0.075	0.73674	1.29771	0.41827	0.10287	0.73821	1.30030	0.41910	0.10308
	0.1	0.85235	1.50025	0.48425	0.21158	0.85405	1.50325	0.48522	0.21201
	0.15	1.04139	1.80517	0.60077	0.58165	1.04347	1.80878	0.60197	0.58281
	0.2	1.15538	1.99727	0.66836	1.14723	1.15653	1.99927	0.66903	1.14838
	0.25	1.23578	2.15653	0.70815	1.91728	1.24196	2.16732	0.71169	1.92687
	0.3	1.26808	2.25875	0.71191	2.83305	1.26808	2.25875	0.71191	2.83305
	0.4	1.21021	2.22372	0.65863	4.80670	1.21142	2.22594	0.65929	4.81151
	0.5	1.11007	2.08364	0.59140	6.88904	1.11118	2.08572	0.59199	6.89593
	0.75	0.87171	1.69972	0.44707	12.17203	0.87084	1.69802	0.44662	12.15986
	1	0.70192	1.39143	0.35409	17.42423	0.70052	1.38865	0.35338	17.38938
	1.5	0.45773	0.91676	0.22854	25.56552	0.45773	0.91676	0.22854	25.56552
	2	0.32450	0.65192	0.16152	32.22115	0.32385	0.65061	0.16120	32.15671
	3	0.19957	0.40128	0.09925	44.58727	0.19917	0.40048	0.09906	44.49810
	4	0.13016	0.25914	0.06537	51.69585	0.13003	0.25888	0.06531	51.64415
	5	0.08901	0.17759	0.04462	55.23999	0.08866	0.17688	0.04444	55.01903
	7.5	0.03953	0.07864	0.01987	55.19216	0.03953	0.07864	0.01987	55.19216
	10	0.02192	0.04321	0.01112	54.41973	0.02186	0.04308	0.01109	54.25648
PGA (g)	0	0.52658	0.90057	0.30790	0.00131	0.52658	0.90057	0.30790	0.00131
PGV (cm/s)	-1	70.14587	126.77991	38.81090	0.17413	NA	NA	NA	NA

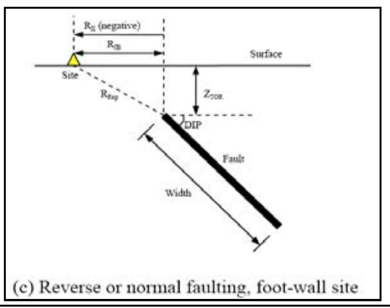


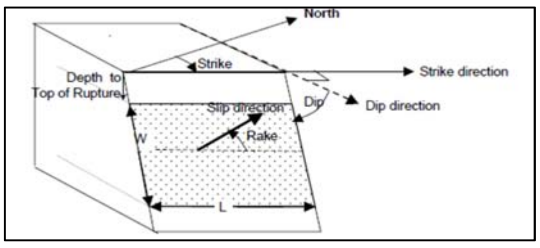
Input variables	Errors and warnings
M_w: 6.78	
R_{RUP} (km): 2.12	
R_{JB} (km): 1.78	
R_X (km): -1.76	
Ry0 (km): 999	*If unknown use 999*
V_{S30} (m/sec): 357	
U (BSSA13): 1	*1: Unspecified fault mech.*
F_{RV}: 0	*1: reverse fault*
F_{NM}: 0	*1: normal fault*
F_{HW}: 0	*1: hanging wall side*
Dip (deg): 50	
Z_{TOR} (km): 0	*If unknown use 999*
Z_{HYP} (km): 999	*If unknown use 999*
$Z_{1.0}$ (km): 0.7	*If unknown use 999*
$Z_{2.5}$ (km): 5.05	*If unknown use 999*
W (km): 13.63	*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*

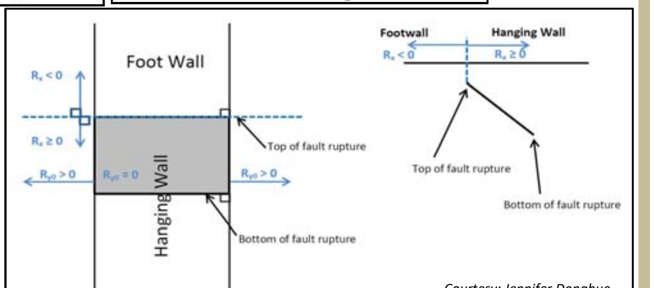
Calculated Variables/Flags

ΔDPP: 0	*Always 0 for median calcs.*
PGA_r (g): 0.408	
Z_{BOT} (km) (CB14): 15	*Enter for default W calcs*
SS: 0	*auto calculated*
$V_{s30Flag}$: 1	*measured*
F_{AS}: 0	*Aftershock effect is not applicable.*
Region: 0	*California*
Option for Sa value: 1	*Weighted average of the natural logarithm of the spectral values*









Courtesy: Jennifer Donahue

Definition of Parameters

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

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

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

DEFAULTs	USER defined	ASK14	BSSA14	CB14	CY14	I14
W (km)	13.63			19.084		
$Z_{1.0}$ (km)	0.700	0.700			0.406	
$\delta Z_{1.0}$ (km)	0.294		0.294			
$Z_{2.5}$ (V_{S30}=1100)(km)	5.050			0.398		
$Z_{2.5}$ (V_{S30})(km)	5.050			1.440		
Z_{hyp} (km)	999.00			10.607		
Z_{tor} (km)	0.00			0.380	0.380	
Z_{BOR} (km)	-			15.000		

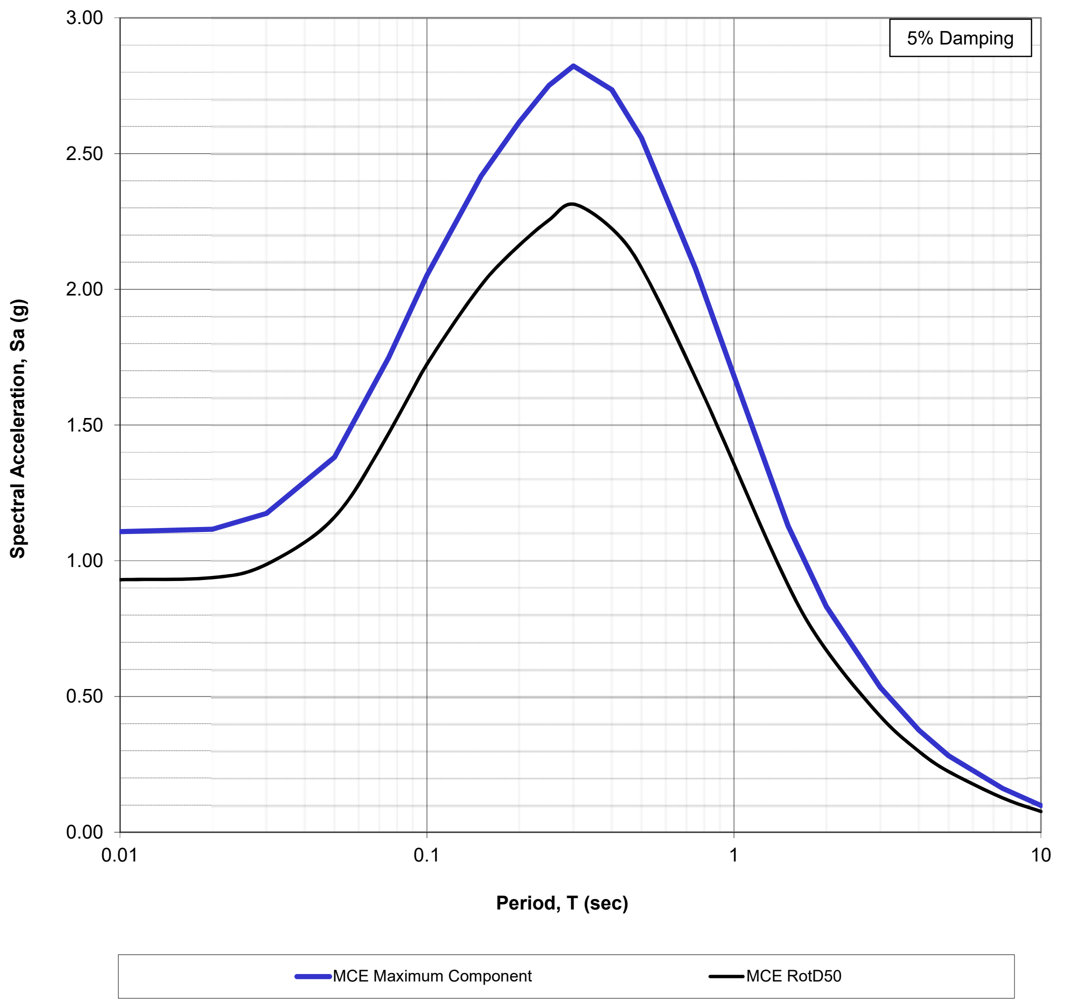
ACKNOWLEDGEMENTS

Nick Gregor, Bechtel
Silvia Mazzoni, Consultant

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

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

Project: McKinley ES
Project Number: 11428.036
Location: 2401 Santa Monica Boulevard



Period T (s)	MCE GEOMEAN Sa (g)	Maximum Component Factor	MCE MAX COMP Site-Specific Sa (g)
0.01	0.930	1.19	1.107
0.02	0.938	1.19	1.116
0.03	0.987	1.19	1.175
0.05	1.161	1.19	1.381
0.075	1.469	1.19	1.748
0.10	1.724	1.19	2.052
0.15	2.015	1.20	2.418
0.20	2.163	1.21	2.617
0.25	2.256	1.22	2.752
0.30	2.314	1.22	2.823
0.40	2.224	1.23	2.735
0.50	2.080	1.23	2.558
0.75	1.674	1.24	2.076
1.00	1.356	1.24	1.682
1.50	0.910	1.24	1.129
2.00	0.671	1.24	0.832
3.00	0.427	1.25	0.533
4.00	0.298	1.26	0.376
5.00	0.224	1.26	0.282
7.50	0.127	1.28	0.162
10.00	0.076	1.29	0.098

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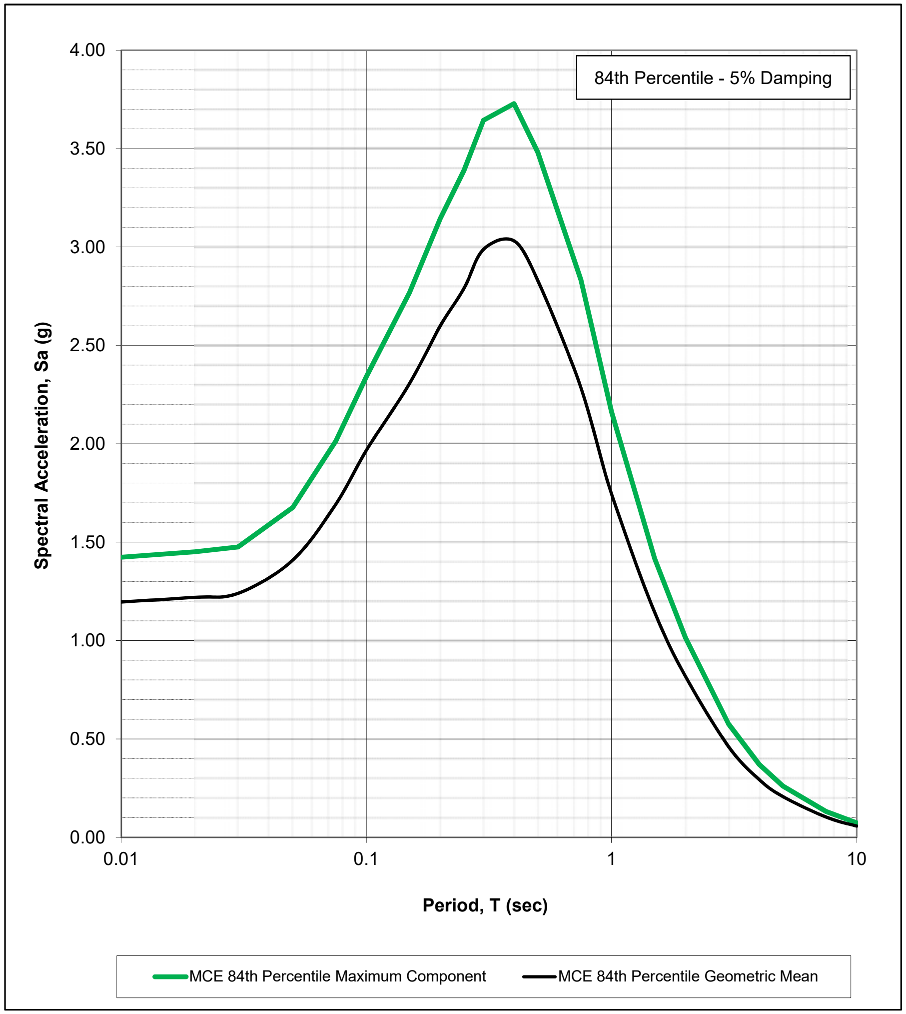
Figure C1

MCE DETERMINISTIC SPECTRA

Project: McKinley ES
Project Number: 11428.036
Location: 2401 Santa Monica Boulevard



DETERMINISTIC PGA MAGNITUDE				
MC FACTOR		DSHA - 84TH PERCENTILE		
Period T (s)	Maximum Component Factor	Period T (s)	MCE GEOMEAN Sa (g)	MCE MAX COMP Sa (g)
0.01	1.19	0.01	1.196	1.424
0.02	1.19	0.02	1.220	1.451
0.03	1.19	0.03	1.240	1.476
0.05	1.19	0.05	1.408	1.676
0.075	1.19	0.075	1.693	2.014
0.10	1.19	0.10	1.967	2.341
0.15	1.20	0.15	2.307	2.768
0.20	1.21	0.20	2.599	3.145
0.25	1.22	0.25	2.789	3.389
0.30	1.22	0.30	2.987	3.645
0.40	1.23	0.40	3.031	3.729
0.50	1.23	0.50	2.830	3.481
0.75	1.24	0.75	2.284	2.832
1.00	1.24	1.00	1.743	2.162
1.50	1.24	1.50	1.142	1.416
2.00	1.24	2.00	0.819	1.015
3.00	1.25	3.00	0.461	0.576
4.00	1.26	4.00	0.294	0.370
5.00	1.26	5.00	0.207	0.261
7.50	1.28	7.50	0.103	0.132
10.00	1.29	10.00	0.057	0.074

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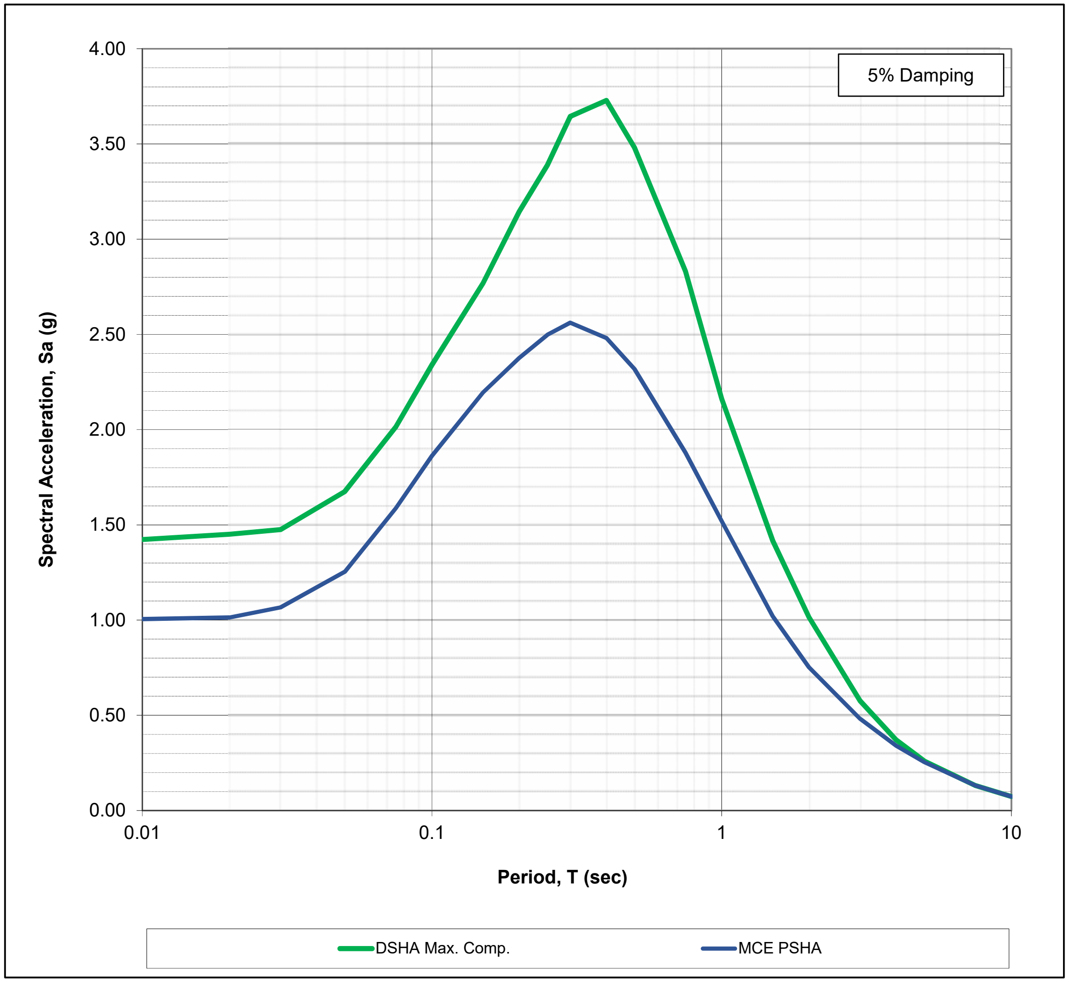
Figure C2

MCE SPECTRA COMPARISON - MAXIMUM HORIZONTAL COMPONENT

Project: McKinley ES

Project Number: 11428.036

Location: 2401 Santa Monica Boulevard



DSHA		PSHA			
Period T (s)	MAX COMP. Sa (g)	Period T (s)	MCE MAX COMP. Sa (g)	Site Risk Coefficient (Cs)	MCE_R Sa (g)
0.01	1.424	0.01	1.107	0.908	1.005
0.02	1.451	0.02	1.116	0.908	1.014
0.03	1.476	0.03	1.175	0.908	1.066
0.05	1.676	0.05	1.381	0.908	1.254
0.075	2.014	0.075	1.748	0.908	1.587
0.10	2.341	0.10	2.052	0.908	1.863
0.15	2.768	0.15	2.418	0.908	2.195
0.20	3.145	0.20	2.617	0.908	2.376
0.25	3.389	0.25	2.752	0.908	2.498
0.30	3.645	0.30	2.823	0.908	2.562
0.40	3.729	0.40	2.735	0.907	2.481
0.50	3.481	0.50	2.558	0.907	2.319
0.75	2.832	0.75	2.076	0.905	1.879
1.00	2.162	1.00	1.682	0.904	1.520
1.50	1.416	1.50	1.129	0.904	1.020
2.00	1.015	2.00	0.832	0.904	0.752
3.00	0.576	3.00	0.533	0.904	0.482
4.00	0.370	4.00	0.376	0.904	0.340
5.00	0.261	5.00	0.282	0.904	0.255
7.50	0.132	7.50	0.162	0.904	0.132
10.00	0.074	10.00	0.098	0.904	0.074

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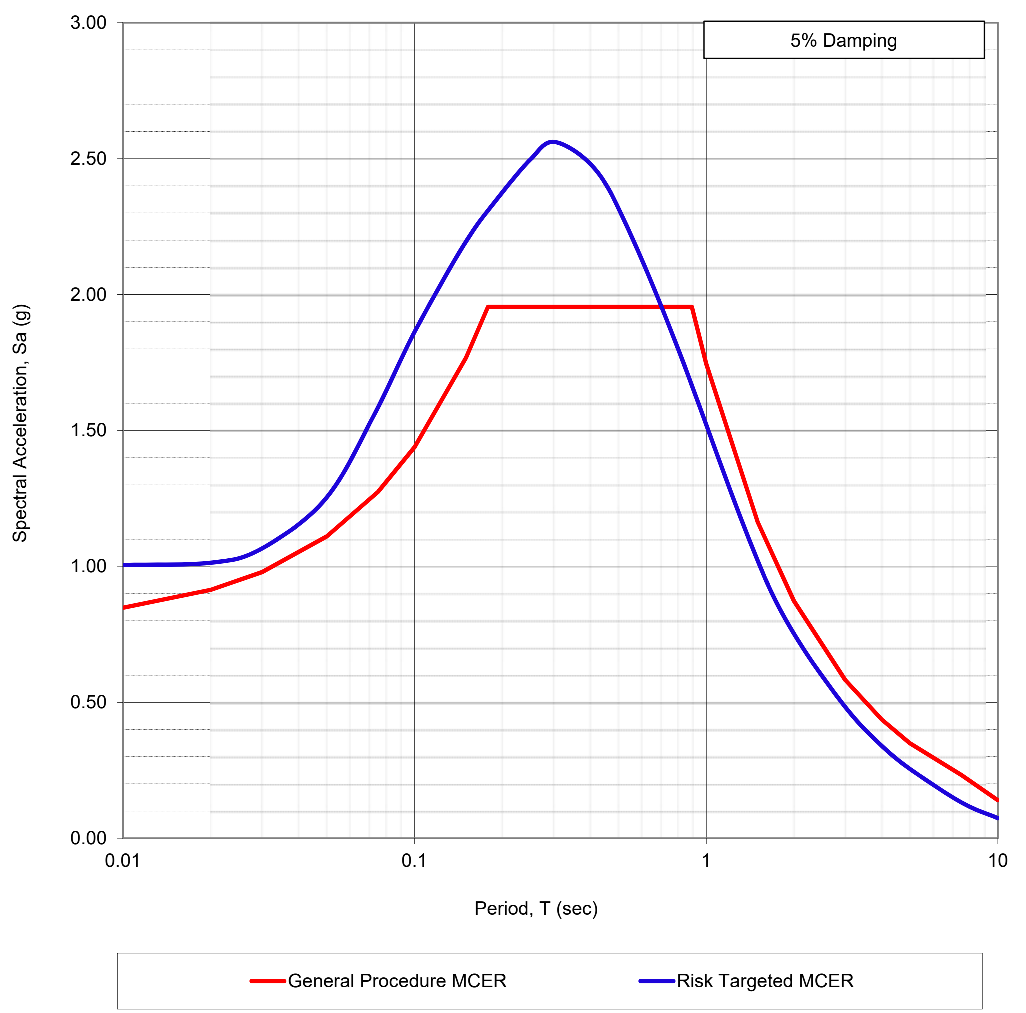
Figure C3

RISK TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) RESPONSE SPECTRUM

Project: McKinley ES

Project Number: 11428.036

Location: 2401 Santa Monica Boulevard



SITE-SPECIFIC vs. GENERAL CODE-BASED SPECTRA

Period T (s)	DETERM. MCE_R Sa (g)	PROB. MCE_R Sa (g)	**Risk TGT MCE_R Sa (g)**	General Procedure Sa (g)
0.01	1.424	1.005	**1.005**	0.848
0.02	1.451	1.014	**1.014**	0.913
0.03	1.476	1.066	**1.066**	0.979
0.05	1.676	1.254	**1.254**	1.111
0.075	2.014	1.587	**1.587**	1.275
0.10	2.341	1.863	**1.863**	1.439
0.15	2.768	2.195	**2.195**	1.768
0.20	3.145	2.376	**2.376**	1.955
0.25	3.403	2.498	**2.498**	1.955
0.30	3.645	2.562	**2.562**	1.955
0.40	3.729	2.481	**2.481**	1.955
0.50	3.481	2.319	**2.319**	1.955
0.75	2.832	1.879	**1.879**	1.955
1.00	2.162	1.520	**1.520**	1.745
1.50	1.416	1.020	**1.020**	1.163
2.00	1.015	0.752	**0.752**	0.873
3.00	0.576	0.482	**0.482**	0.582
4.00	0.370	0.340	**0.340**	0.436
5.00	0.261	0.255	**0.255**	0.349
7.50	0.132	0.146	**0.132**	0.233
10.00	0.074	0.089	**0.074**	0.140

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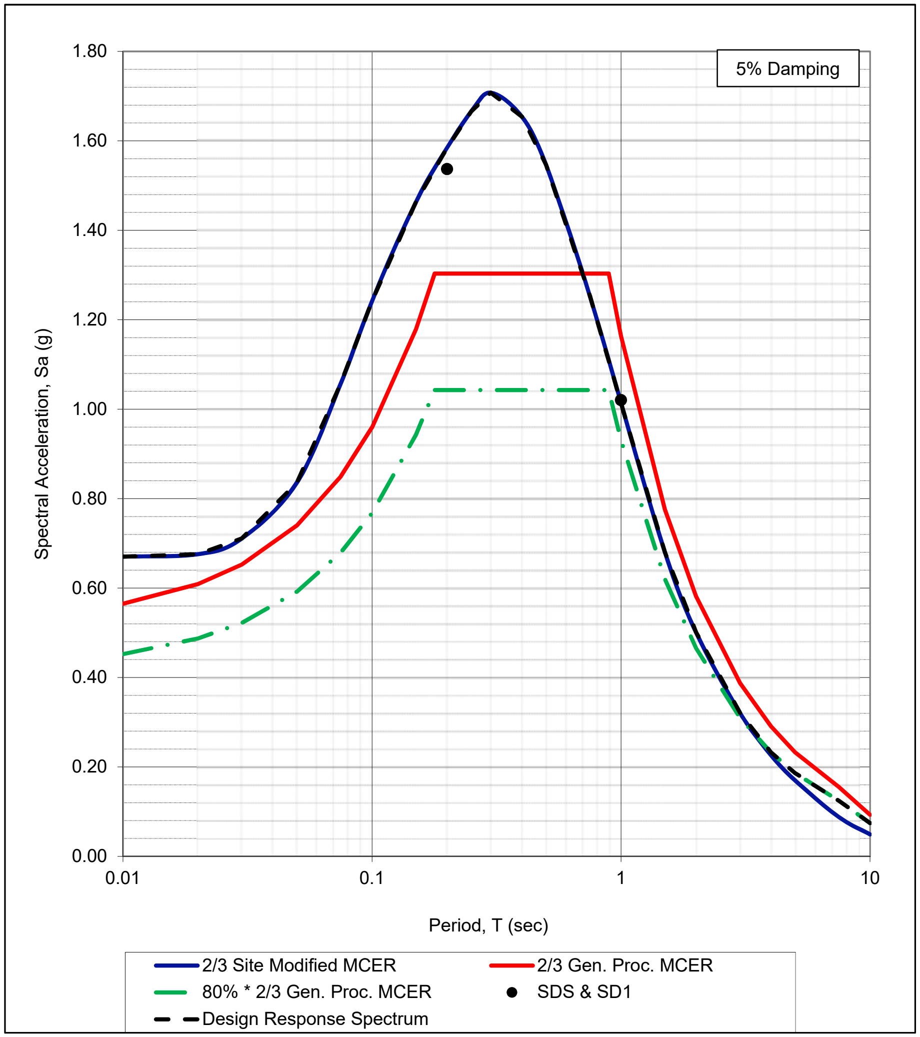
Figure C4

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

Project: McKinley ES
Project Number: 11428.036
Location: 2401 Santa Monica Boulevard



	CODE BASED GENERAL PROCEDURE SPECTRUM			RISK TGT SPECTRUM	DESIGN RESPONSE SPECTRUM
Period T (s)	GENERAL PROC. MCER CURVE Sa (g)	2/3 GENERAL PROC. MCER CURVE Sa (g)	80% * 2/3 GENERAL PROC. MCER CURVE Sa (g)	$2/3*MCE_R$ CURVE Sa (g)	MAX of 2/3 MCE_R and 80% * 2/3 GENERAL PROC. MCER Sa (g)
0.01	0.848	0.565	0.452	0.670	0.670
0.02	0.913	0.609	0.487	0.676	0.676
0.03	0.979	0.653	0.522	0.711	0.711
0.05	1.111	0.740	0.592	0.836	0.836
0.075	1.275	0.850	0.680	1.058	1.058
0.10	1.439	0.959	0.768	1.242	1.242
0.15	1.768	1.178	0.943	1.463	1.463
0.20	1.955	1.303	1.043	1.584	1.584
0.25	1.955	1.303	1.043	1.665	1.665
0.30	1.955	1.303	1.043	1.708	1.708
0.40	1.955	1.303	1.043	1.654	1.654
0.50	1.955	1.303	1.043	1.546	1.546
0.75	1.955	1.303	1.043	1.253	1.253
1.00	1.745	1.163	0.931	1.013	1.013
1.50	1.163	0.776	0.620	0.680	0.680
2.00	0.873	0.582	0.465	0.501	0.501
3.00	0.582	0.388	0.310	0.321	0.321
4.00	0.436	0.291	0.233	0.227	0.233
5.00	0.349	0.233	0.186	0.170	0.186
7.50	0.233	0.155	0.124	0.088	0.124
10.00	0.140	0.093	0.074	0.049	0.074

S_{DS} = 1.537 g
S_{D1} = 1.020 g

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

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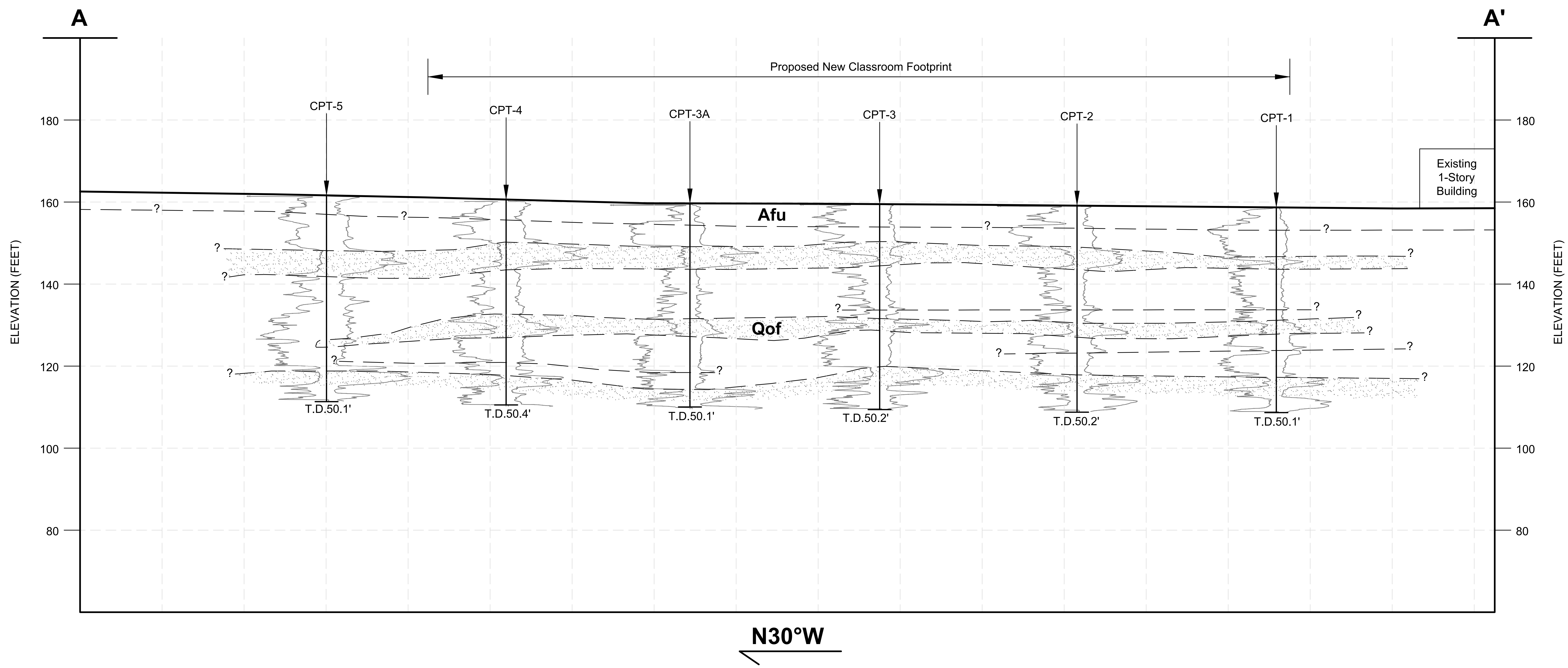
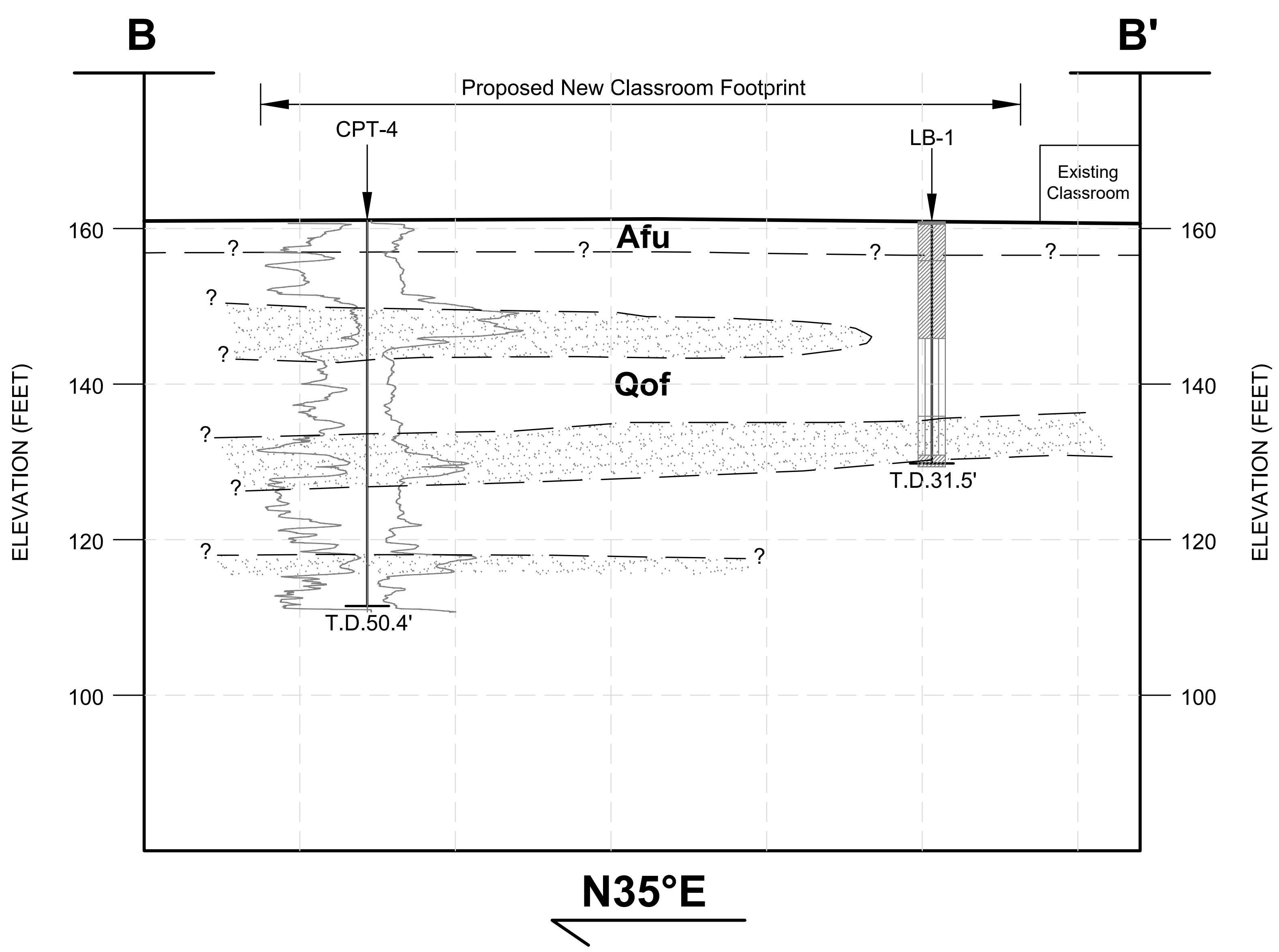
Figure C5





UNIFIED SOIL CLASSIFICATION SYSTEM (USCS) GRAPHIC

SAND (SP)

SILTY SAND (SM)

CLAYEY SAND (SC)

SAND WITH GRAVEL TO SANDY GRAVEL (SPg -GP)

CLAY (CL)

SANDY SILT (ML)

LEGEND

Afu Artificial Fill, Undocumented

Qof Older alluvial sediments in part non marine, pebble gravle, sand, silt and clay derived from Santa Monica Mountains; Consolidated, dissected and eroded

GEOTECHNICAL CROSS SECTION A-A' AND B-B' McKinley Elementary School 2401 Santa Monica Boulevard Santa Monica, California	PLATE 1
	Scale: 1"=20'
	Date: November 2021
Leighton	Proj: 11428.036
	Eng/Geol: EMH

V:\DRAFTING\11428\036\CAD\2021-10-15\11428-036_P01_CS_2021-11-18.DWG (11-18-21 4:31:40PM) Plotted by: btran