GEOTECHNICAL INVESTIGATION REPORT
PROPOSED MULTI-FAMILY
RESIDENTIAL DEVEOPMENT
1346, 1350 AND 1352 W. COURT STREET
CITY OF LOS ANGELES, CALIFORNIA

Prepared for:

1300 Court Partners, LLC

9748 Topanga Canyon Boulevard
Los Angeles, California 91311

Project No. 11388.001

August 31, 2016

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A LEIGHTON GROUP COMPANY

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1300 Court Partners, LLC
9748 Topanga Canyon Boulevard
Los Angeles, California 91311

Attention: Mr. Bill McReynolds

Subject: Geotechnical Exploration Report
Proposed Multi-Family Residential Development
1346, 1350 and 1352 W. Court Street
City of Los Angeles, California

Per your request, Leighton and Associates, Inc. (Leighton) is pleased to present this geotechnical exploration report for the proposed residential development project located at 1346, 1350 and 1352 W. Court Street in the city of Los Angeles, California. The currently planned development will consist of a four-story multi-family residential apartment building over a two-story parking garage that is planned to be partially below grade with an entrance at street grade on the Douglas Street side of the property. No specific information for the building construction type or structural loading is available at the time of this report.

The purpose of this report is to present the findings of our exploration at the site and to provide preliminary geotechnical recommendations for the project. Presented herein are subsurface information obtained during our exploration and recommendations with respect to site grading, earthwork, seismic design parameters, and building foundation design. Also presented in this report are considerations for the future construction of the project.

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We appreciate the opportunity to work with you on this project. If you have any questions or if we can be of further service, please call us at your convenience.

Respectfully submitted,

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1.0 INTRODUCTION

1.1 Site Description and Proposed Development

The project site is located at 1346, 1350 and 1352 W. Court Street in the city of Los Angeles, California. The site location and immediate vicinity are shown on Figure 1, *Site Location Map*. The site is bounded by existing single-family and multi-family residential buildings to the southeast and southwest, by Court Street to the northeast, and by Douglass Street to the northwest.

The site encompasses two vacant parcels (1350 and 1352 W. Court Street) and one developed parcel (1346 W. Court Street) that contains two single-family homes and associated flatwork improvements. The vacant parcels are covered with light to moderate vegetation consisting of grasses, small shrubs and several mature trees. Topographically, the site generally slopes to the north, west and south from approximate Elevation 407 feet in the southeastern portion of the site to approximate Elevation 380 feet at street grade along Douglas Street in the western corner of the site. Locally, small near vertical slopes exist along portions of the site bounding the existing streets, on the alleyway and Court Street. Based on review of historical aerial photographs (NETR, 2016), the site appears to have been generally vacant since 1948; however, some minor miscellaneous structures may have been situated on the property at one time.

The current plan is to develop the site for a four-story multi-family residential apartment building over a two-story parking garage that is planned to be partially below grade with an entrance at street grade on the Douglas Street side of the property. The structure is expected to cover the site almost entirely. No information related to the type of building construction or structural loading is available at this time.

1.2 Purpose and Scope of Exploration

The purpose of our exploration was to evaluate the subsurface conditions beneath the site for developing preliminary geotechnical recommendations for the project as currently proposed.

The scope of this geotechnical report included the following tasks:

- *Background Review* – A background review was performed of readily available, relevant geotechnical and geological literature pertinent to the project site. References used in preparation of this report are listed in Section 7.0.

- *Field Exploration* – Our field exploration was performed on July 28, 2016, and consisted of two (2) hollow-stem auger borings (designated LB-1 and LB-2) drilled up to a maximum of 22 feet below existing ground surface (bgs). The approximate locations of the explorations are shown on Figure 2, *Boring Location Map*. Prior to the field exploration, the boring locations were marked and Underground Service Alert (USA) was notified for utility clearance. The borings as shown were located using a handheld GPS unit.

  During drilling, both bulk and relatively undisturbed drive samples were obtained from the borings for geotechnical laboratory testing. Relatively undisturbed samples were collected from the borings using the Modified California Ring sampler conducted in accordance with ASTM Test Method D 3550. The samplers were driven for a total penetration of 18 inches, unless practical refusal, using a 140-pound automatic hammer falling freely for 30 inches. 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. The samples were sealed and packaged for transportation to our laboratory. The boring logs are presented in Appendix A, *Field Exploration Logs*.

- *Laboratory Testing* – Laboratory tests were performed on representative soil samples to determine the geotechnical engineering properties of subsurface materials. The following laboratory tests were performed:

  - Moisture Density (ASTM D422);
  - Atterberg Limits (ASTM D4318);
  - Unconfined Compressive Strength (ASTM D2166);

  - Maximum Dry Density (ASTM D1557); and
  - Soluble sulfate, soluble chloride, pH and minimum resistivity (CTM 417 Part II, CTM 422, and CTM 643).

  The results of the laboratory tests are presented in Appendix B – *Laboratory Test Results.*

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

It should be noted that the recommendations in this report are subject to the limitations presented in Section 6.0. An information sheet prepared by ASFE (the Association of Engineering Firms Practicing in the Geosciences) is also included at the rear of the text. We recommend that all individuals using this report read the limitations along with the attached document.

2.0 GEOTECHNICAL FINDINGS

2.1 Geologic Setting

The project site is located along the northeastern margin of the Los Angeles basin, at the northern end of the Peninsular Ranges geomorphic province and adjacent to the Transverse Ranges geomorphic province. The Peninsular Ranges province extends approximately 900 miles southward from the Santa Monica Mountains to the tip of Baja California (Yerkes, et al., 1965). The province is characterized by elongated northwest-trending mountain ridges and sediment-floored valleys. The province includes numerous northwest trending fault zones, most of which either die out, merge with, or are terminated by faults that form the southern margin of the Transverse Ranges province. These northwest trending fault zones include the San Jacinto, Whittier-Elsinore, Palos Verdes, and Newport-Inglewood fault zones.

Approximately 65 million years ago (at the end of the Cretaceous Period) a deep, structural trough existed off the coast of southern California (Yerkes, 1972). Over time the trough was filled with sediments eroded from the surrounding highlands and mountains. About 7 million years ago the boundary between the Pacific and North American plates shifted to its present position and the geologically modern Los Angeles basin began to form. The deepest part of the Los Angeles basin contains Tertiary to Quaternary age (65 million years and younger) marine and non-marine sedimentary rocks that are about 24,000 feet thick (Yerkes, et al, 1965; Wright, 1991). During the Pleistocene epoch (the last two million years) the region was flooded as sea level rose in response to the worldwide melting of the Pleistocene glaciers.

The project site is located to the southwest of the Elysian Park Anticline, a west-northwest trending fold belt which forms a topographic high of Early Pliocene to Late Miocene-aged bedrock materials. The area is underlain by Puente Formation bedrock that is composed of deep-marine clastic and biogenic rocks interbedded and interfingering siltstone and fine sandstone, siliceous shale and siltstone, diatomaceous shale and siltstone, and fine- to coarse-grained, thinly laminated to thick-bedded sandstone. Alluvial materials are found in topographic lows, which drain into the Los Angeles River Valley and the greater Los Angeles Basin located to the south. The project site is located on an area that is slightly elevated from the greater alluvial basin to the south.

2.2 Subsurface Conditions

Based on our observations of bedrock outcrop and within the hollow-stem auger borings, the site is underlain by undocumented artificial fill soils over bedrock of the Puente Formation. The fill soils as encountered during our exploration, consisting of primarily of sandy clay, sandy silt, and sandy clay extended down to a depth of approximately 5 feet below existing grade. Locally thicker accumulations of undocumented fills may exist in areas not explored as a part of this investigation. Sedimentary bedrock of the Puente Formation, consisting of well bedded siltstone/claystone with interbedded sand was found beneath the shallow fill soils to the depths explored. Bedrock was observed in outcrop on the southwest corner of the site at the intersection of the Douglas and the alleyway.

The siltstone/claystone bedrock observed in the samples and outcrops showed well defined bedding. Grain size analysis and Atterberg Limits tests results indicate the bedrock material is classified as silty clay (CL) to fat clay (CH) in accordance with the Unified Soil Classification System (USCS) with medium plasticity. According to regional geologic mapping in the area (Yerkes and Campbell, 2005; and Dibblee, 1991) and our experience with similar projects in the nearby area, the bedding at the project site is anticipated to generally exhibit an east-west strike with dip angles ranging from approximately 10 to 40 degrees to the south. Bedding attitudes measured at the exposed bedrock on the southwest corner of the site indicated a northeast strike (approximately 55 to 65 degrees from north) and slightly steeper dip angles (55 to 65 degrees to the southeast). This creates an adverse bedding condition on the north side of the project site, where proposed cuts up to 20 feet will expose bedding planes dipping out of slope. A map showing the geologic units mapped in the area is included with this report as Figure 3 – *Regional Geology Map*.

A detailed description of the subsurface soils encountered in the borings is presented in the boring logs (Appendix A). Some of the engineering properties of these soils are described in the following subsections.

2.2.1 Expansive Soil

Expansive soils contain significant amounts of clay particles that swell considerably when wetted and shrink when dried. Foundations constructed on these soils are subject to uplifting forces caused by the swelling. Without proper mitigation measures, heaving and cracking of both building foundations and slabs-on-grade could result. Based on our exploration, the bedrock materials consist predominantly of siltstone and claystone. Laboratory test result of a claystone bedrock sample showed the material has a Plasticity Index of 42 and Liquid Limit of 65. Based on comparison on the test results with our projects in the vicinity, the bedrock at the site is considered expansive.

2.2.2 Soil Corrosivity

The bedrock materials were screened for corrosion potential to ferrous metals and concrete (e.g., footings, retaining walls). The corrosivity test results are included in Appendix B of this report and are summarized below.

Corrosivity Test Results

Test Parameter	**Test Results**	**General Classification of Corrosion Potential**
	Boring LB-2 at 10'	
Water-Soluble Sulfate in Soil (ppm)	1236	Moderate sulfate exposure to buried concrete
Water-Soluble Chloride in Soil (ppm)	158	Non-corrosive to embedded metals
pH	7.5	Mildly alkaline
Minimum Resistivity (saturated, ohm-cm)	380	Very Severely corrosive to buried ferrous pipes

A corrosion engineer should be consulted for possible mitigation measures, if necessary.

2.2.3 Strength Characteristics

Based on the laboratory testing results and our experience with the bedrock material, the bedrock materials should exhibit adequate shear strength to provide structural support for the planned improvements.

2.2.4 Collapse/Compressibility Potential

The bedrock materials at the site are expected to exhibit low compressibility characteristics when subject to the anticipated loading.

2.3 Groundwater

Groundwater was not encountered in our explorations up to a maximum depth of 22 feet bgs on this site. Explorations on nearby sites advanced to depths up to 51.5 feet did not encounter groundwater in March 2013. Groundwater seepage may be encountered within the bedrock joints, fractures and various sandy layers within the depth of the planned excavation. Groundwater levels and the amount of seepage will be affected by seasonal factors such as rainfall and or irrigation practices in the vicinity of the site.

3.0 GEOLOGIC/SEISMIC HAZARDS

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

3.1 Surface Fault Rupture

Our review of available in-house literature indicates that no known active faults have been mapped across the site, and the site is not located within a designated Alquist-Priolo Earthquake Fault Zone (CGS, 2014; Bryant and Bryant, 2007). Based on our review, we consider the potential for surface fault rupture at the site to be low.

The location of the closest active faults to the site was generated using the United States Geological Survey (USGS) Earthquake Hazards Program (USGS, 2008c). The closest active faults to the site are the Elysian Park blind thrust, Santa Monica, and Hollywood faults, located approximately 1.0, 3.5, and 3.6 miles, respectively, from the site. The San Andreas fault, which is the largest active fault in California, is approximately 33 miles northeast of the site. The nearby faults with surface expression in the vicinity of the site are shown on Figure 4, *Regional Fault and Historical Seismicity Map*.

3.2 Ground Shaking

The principal seismic hazard that could affect the site is ground shaking resulting from an earthquake occurring along several major active or potentially active faults in southern California. The site is expected to experience moderate to strong ground shaking resulting from the earthquake faults in the region. 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 2016, with magnitudes M4.0 or greater, were estimated using the EQSEARCH computer program (Blake, 2000). This historical seismicity search was performed for a 100-kilometer (62-mile) radius from the project site, and is included in Appendix D. The largest earthquake magnitudes found in the search was the M7.0 earthquake that occurred on December 8, 1812 approximately 40.6 miles (65.4 kilometers) from the site producing an estimated site acceleration of approximately 0.047g. A M7.0 earthquake also occurred on September 24, 1827 approximately 42.7 miles (68.7 kilometers) from the site producing an estimated site acceleration of approximately 0.044g. The largest estimated PGA found in the search was approximately 0.272g from an earthquake approximately 1.1 miles (1.7 kilometers) from the site.

Additional data publically available from the Center for Engineering Strong Motion Data (CESMD) website (http://strongmotioncenter.org/) was reviewed for stations in the vicinity of the project site. The data reviewed indicates that a site approximately 0.5 miles northeast of the project site experienced a peak ground acceleration of 0.141g from a M6.4 Northridge earthquake that occurred on January 17, 1994. A site-specific response analysis was developed using the computer program *EZ-FRISK* by Risk Engineering (v. 7.62) and the 2008 CGS Statewide Fault Model. The results of our analysis are presented in Section 4.4 Seismic Design Parameters.

3.3 Secondary Seismic Hazards

In general, seismic hazards due to ground shaking could include soil liquefaction, seismically-induced settlement, lateral spreading, seismically-induced landsliding, seiches and tsunamis. These potential secondary seismic hazards are discussed below.

3.3.1 Liquefaction Potential

Liquefaction is the loss of soil strength or stiffness due to increasing pore-water pressure during severe ground shaking. Liquefaction is associated primarily with loose (low density), saturated, fine- to medium-grained, cohesionless soils.

As shown on the State of California Seismic Hazard Zones Map for the Hollywood Quadrangle (see Figure 5, *Seismic Hazard Map*; CGS, 1999), this site is not located within an area that has been identified by the State of California as being potentially susceptible to liquefaction. Furthermore, the site is underlain by relatively shallow bedrock. Therefore, it is our opinion that the potential for liquefaction occurring at the site is low.

3.3.2 Seismically Induced Settlement

During a strong seismic event, seismically induced settlement can occur within loose to moderately dense, unsaturated granular soils, separate from liquefaction. Settlement caused by ground shaking is often non-uniformly distributed, which can result in differential settlement. Based on blow count records and the relatively shallow bedrock at the site, the seismically induced settlement under the proposed buildings is anticipated to be negligible.

3.3.3 Lateral Spreading

Lateral spreading is a phenomenon in which large blocks of intact, non-liquefied soil move downslope on a liquefied soil layer. Lateral spreading is often a regional event. For lateral spreading to occur, the liquefiable soil zone must be laterally continuous, unconstrained laterally, and free to move along sloping ground. Due to the low potential for liquefaction at the site, the potential for lateral spreading is considered very low.

3.3.4 Seismically-Induced Landslide

Although some slopes are located along the northern, western and southern boundaries of the site, these slopes are planned to be completely removed with the proposed development plan that includes a partial subterranean parking level. In addition, based on the State of California Seismic Hazard Zones Map for the Hollywood Quadrangle (see Figure 5, *Seismic Hazard Map;* CGS, 1999), the site is not located within an area that has been identified by the State of California as being potentially susceptible to seismically induced landslides. Based on these factors, the potential for seismically-induced landslides to occur at the site is considered low.

3.3.5 Earthquake-Induced Flooding

Earthquake-induced flooding can be caused by failure of dams or other water-retaining structures as a result of earthquakes. As shown on Figure 6, *Dam Inundation Map*, the site is not within a mapped inundation zone for any reservoirs. Therefore, the risk of seismically-induced flooding due to dam failure is considered very low.

3.3.6 Seiches and Tsunamis

Seiches are large waves generated in very large enclosed bodies of water or partially enclosed arms of the sea in response to ground shaking. Tsunamis are waves generated in large bodies of water by fault displacement or major ground movement. Based on the lack of such large enclosed water bodies nearby, seiche and tsunami risks are considered low to remote.

3.4 Flooding Hazards

As shown on Figure 7, *Flood Hazard Map* and according to a Federal Emergency Management Agency (FEMA) flood insurance rate map (FEMA, 2008), the site is not located within a flood zone.

3.5 Methane

Based on review of available Division of Oil, Gas, and Geothermal Resources (DOGGR) maps, the project site is located in the Los Angeles City Oil Field and four oil wells (Courtland City Lights Association Well Nos. 1 and 2, and Parker Morrell Oil Co. Well Nos. 3 & 4) are reported to be present across the site. In addition, based on review of the *Methane and Methane Buffer Zones* map published by the City of Los Angeles (2004), the site is located within a Methane Zone as shown on Figure 8 – *Methane Hazard Map*. We understand that Roux Environmental is providing oil well location and methane mitigation services for this project. Roux provided the recent geophysical survey performed to locate oil wells identified on Division of Oil and Gas (DOGGR) maps. Although four wells were identified by DOGGR, evidence was found for only one well on site.

4.0 DESIGN RECOMMENDATIONS

Geotechnical recommendations for the proposed development are presented in the following sections. Construction considerations are discussed Section 5.0 in this report.

The geotechnical consultant should review the grading plan, foundation plan and specifications as they become available to verify that the recommendations presented in this report have been incorporated.

Based on the current plan, excavation up to 20 feet are anticipated for the construction of the subterranean portion of the development. To support the excavation, a temporary shoring system consisting of soldier piles (with or without tie-back anchors) may be used during construction. Due to presence of adverse bedrock bedding dipping into the excavation of the site, the south-facing shoring wall and basement walls along the northern boundary will be subject to geologic surcharge from the bedrock. However, permissions from adjoining property owners and the City will be required for installation of tie-back anchors on their properties.

4.1 Shoring Design Recommendations

Excavations ranging from 15 to 20 feet in height are anticipated during construction of the subterranean parking at each site. Based on review of the regional geology map and a project we completed recently in the vicinity of the site, the bedrock structure includes bedding that dips (slopes) toward the general alignment of the proposed shoring wall along the northern property boundary. The bedding angles are anticipated to vary from 55 to 65 degrees for the excavations at the project site based on local measurements (approximately 44 degrees out of slope), which would create an adverse condition along the northern property boundary of the site. The bedrock includes thin seams of bedding materials that are lower in strength (i.e., along bedding) than the gross shear strength (i.e., across bedding) of the siltstone/claystone bedrock. Therefore, geologic surcharge should be included when designing the shoring system.

In addition, surcharge due to the existing buildings and vehicular traffic along the alley behind the excavation should also be considered in the shoring wall design. We recommend the shoring contractor perform a survey to document the existing conditions behind the site prior to construction.

It is the shoring contractor’s responsibility to design the system that meets the project specifications. The shoring contractor should submit the shoring plans and a testing program to the geotechnical engineer for review.

As the tie-back anchors and soldier piles are planned to be drilled into the bedrock, the potential of raveling and caving of loose soil is low, however, the shoring contactor should be prepared to use special techniques and measures, if necessary, to permit the proper installation of the soldier piles and tie-back anchors in case of caving and raveling of isolated loose soil layers or local groundwater seepage that may exist within the bedrock.

The shoring engineer should incorporate an adequate safety factor in designing the shoring system.

4.1.1 Tie-Back Anchors

All anchors should be designed in accordance with the recommendations by the Post-tensioning Institute (PTI) for prestressed rock and soil anchors (PTI, 2011) and the City of Los Angeles requirements.

For designing the anchored length of the tiebacks beyond the failure surface, a bond strength of 2,500 pounds per square foot (psf) may be assumed between the grout and the bedrock for gravity grouted tie-back anchors.

The anchored portion of the tiebacks should begin in the competent bedrock at least five feet behind the anticipated failure surface. The failure surface in the south-facing excavation along the northern property where geologic surcharge may be assumed to be a surface extended at an angle of 45 degrees from horizontal at the toe of the excavation. For other areas where geologic surcharge is not anticipated, the failure surface may be assumed to be a surface extended at an angle of 60 degrees from horizontal at the toe of the excavation.

The tiebacks should be installed at a minimum distance of four times the diameter of the anchor drill hole on center. The preferred installation angle should be between 5 and 30 degrees from horizontal. Obstacles behind the shoring may require a steeper installation angle.

During installation, each row of anchors should be proof-loaded and approved before excavation can proceed. The tie-back anchor capacity should be checked for each stage of the excavation to ensure adequate support of the system is maintained. Performance tests may also be required on selected tieback anchors. The number of anchors to be tested should be determined based on the results of the testing program.

4.1.2 Lateral Pressures

The recommended lateral earth pressure for shoring design is as follows:

Shoring Design (Level Ground Surface)		
Wall Height ranging from 15 to 20 feet		
	Free Cantilever (psf/ft)	With Tiebacks (psf)
Walls with geologic surcharge	60	38H
Walls without geologic surcharge	32	20H

A safety factor of 1.25 has been incorporated in the above recommended values. The south facing shoring wall at the southern site should be designed for geologic surcharge. The earth pressure without geologic surcharge may be used for the remaining three sides of the shoring wall in southern site and all four sides of the shoring wall in the northern site. A triangular pressure distribution may be assumed for designing free cantilever shoring. For design of braced or tie-back shoring, a trapezoidal distribution of lateral earth pressure can be used. The recommended pressure distribution behind shoring will be zero at the top and bottom of the shoring and at its maximum value between 0.2**H** and 0.8**H**, where **H** is the height of the shoring in feet.

In addition to the recommended earth pressure, the walls should be designed to resist any applicable surcharge loads behind the shoring.

4.1.3 Design of Soldier Piles

For the design of solider piles spaced at least two diameters on-center, the maximum spacing of the solider piles should be limited to 8 feet. The portion of a soldier pile that extends below the excavation may be used to provide passive resistance for the shoring system. A uniform passive pressure of 4,400 psf may be used for a soldier pile embedded in competent bedrock. To develop the full lateral value, provisions should be taken to assure firm contact between the solider piles and the undisturbed soils. The shoring engineer should not consider any passive resistance to a depth equal to one drill hole diameter of the soldier pile below the excavation line.

When using the frictional resistance between the soldier pile and the soil, it is assumed that the drilled hole of the soldier pile will be backfilled with lean-mix concrete, and there is full contact between the lean-mix concrete and the retained soil.

The vertical component of the tie-back load may be supported by the shaft friction and end bearing of the soldier pile embedded in the competent bedrock. A frictional coefficient of 0.44 may be used to calculate the frictional resistance between the soldier pile and the retained soil. For soldier piles penetrated at least 5 feet below the excavation line, a maximum end bearing pressure of 6,500 psf may be used.

4.1.4 Lagging

Lagging should be provided between the soldier piles to control sloughing. Lagging should be placed in such a manner to maintain a tight soil to lagging contact. All voids behind the lagging should be filled with compacted materials or slurry. Lagging may be installed with a maximum spacing of 1½ inches to allow drainage from behind the wall. The soldier piles should be designed for the full anticipated lateral pressure. However, the pressure on the lagging will be less due to arching in the soils. For clear spans of up to 8 feet, we recommend that the lagging be designed for a semi-circular distribution of earth pressure where the maximum pressure is 300 psf at the mid-line between soldier piles, and 0 psf at the soldier piles.

4.1.5 Monitoring

The performance of the shoring system should be monitored on a regular basis during and after installation. The monitoring should consist of surveying of the lateral and vertical locations of the tops of all the soldier piles. The survey data should be submitted to the shoring engineer and geotechnical consultant for review. It is recommended that the maximum deflection behind the shoring be limited to between one-half inch to one inch.

We recommend that the adjacent existing structures and streets be surveyed for horizontal and vertical locations. Also, a survey of existing cracks and offsets in the streets should be performed and recorded along with photographic records.

4.2 Earthwork

4.2.1 Site Preparation

Prior to construction, the site should be cleared of any vegetation, trash and/or debris. These materials should be removed from the site. Any underground obstructions onsite should be removed. Efforts should be made to locate any existing utility lines to be removed or rerouted where interfering with the proposed construction. Any resulting cavities should be properly backfilled and compacted. After the areas are cleared, the soils should be carefully observed for the removal of all unsuitable deposits. All unsuitable deposits and undocumented fill should be excavated and removed from within the development area prior to fill placement.

4.2.2 General Grading Recommendations

It is anticipated that the onsite undocumented artificial fill will be removed during site excavation and competent bedrock will be exposed at the bottom level of the subterranean parking. Unsuitable materials if encountered at the exposed subgrade should be removed until a competent subgrade surface is exposed. Overexcavation and recompaction if required to remove unsuitable subgrade materials should extend a minimum horizontal distance equal to the vertical distance between the proposed footing bottom and depth of overexcavation.

However, care should be used to avoid undermining existing improvements adjacent to the excavation.

After completion of the overexcavation and prior to fill placement or other improvements such as flatwork and hardscape, the exposed soils should be scarified to a minimum depth of six inches, moisture conditioned 2 to 4 percentage points above optimum moisture content and compacted to a minimum of 90 percent relative compaction (ASTM D1557).

The excavated onsite soils, less than 6 inches and free of any deleterious material or organic matter, can be used in required fills. Any required import material should consist of non-corrosive and relatively non-expansive soils with an Expansion Index (EI) less than 20. The imported materials should contain sufficient fines (binder material) so as to be relatively impermeable and result in a stable subgrade when compacted. All proposed import materials should be approved by the geotechnical engineer of record prior to being placed at the site.

All fill soil should be placed in thin, loose lifts, with each lift properly moisture conditioned 2 to 4 percentage above the optimum moisture content and compacted to a minimum of 90 percent relative compaction (ASTM D1557). Proper moisture conditioning of the soils is vital in reducing expansion potential and reducing the potential for post-construction heave that may result in distortion and possibly damage to new improvements. Aggregate base should be compacted to a minimum of 95 percent relative compaction (ASTM D1557).

4.2.3 Pipe Bedding

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 at least 12 inches over the top of the pipeline. The bedding material may consist of compacted free-draining sand, gravel, or crushed rock. Pipe bedding material should have a Sand Equivalent (SE) of at least 30. Flooding or jetting to densify the bedding material is not recommended due to clayey nature of the bedrock.

4.2.4 Trench Backfill

Trench excavations above pipe bedding may be backfilled with onsite soils under the observation of the geotechnical consultant. All fill soils should be placed in loose lifts, moisture conditioned as required and compacted to a minimum of 90 percent relative compaction based on ASTM Test Method D 1557. Lift thickness will be dependent on the equipment used as suggested in the latest edition of the Standard Specifications for Public Works Construction (Greenbook). The fill soils should extend to the bottom of the aggregate base for new pavement, or to finished grade in non-paved areas. Control Low Strength Material (CLSM) should be used for the last 2 feet of utility trench entering the building.

4.3 Conventional Retaining Walls and Basement Walls

4.3.1 Lateral Earth Pressures

The following parameters may be used for the design of conventional retaining walls and basement walls:

	Free Cantilever Walls (Active) psf/ft	Basement Walls (At-rest) psf/ft
Wall Height Ranging from 15 to 20 feet		
Earth Pressure with Geologic Surcharge	56	81
Earth Pressure without Geologic Surcharge	28	45
Seismic Pressure with Geologic Surcharge	41	
Seismic Pressure without Geologic Surcharge	25	

Seismic earth pressure should be applied in addition to static earth pressure for conventional retaining walls that are more than 12 feet in height and the unbalanced height portion (higher side) of the basement walls. The seismic earth pressure was calculated based on a seismic coefficient of 0.32 (i.e., ½ of two-third of PGAm). The distribution of the seismic earth pressure should be an invert triangle with the maximum pressure at the top. The point of application of the resultant seismic thrust may be assumed to act at a point located at 0.6 times the height of the retained height.

In addition to the recommended earth pressure, the walls should be designed to resist any applicable surcharge loads behind the walls.

4.3.2 Backfill

Retaining structures planned at the site should be backfilled with granular, non-expansive soil (Expansion Index less than 20).

Backfill should be compacted to at least 90 percent of the maximum dry density obtained by ASTM Test Method D 1557. Relatively light equipment should be used for backfilling behind retaining walls.

4.3.3 Drainage

All walls should be constructed with a backdrain. The backdrain should be sloped at a minimum of one percent toward an approved non-erosive outlet.

The walls should also be waterproofed or at least damp-proofed, depending upon the degree of moisture protection desired. Surface drainage should be designed to direct water away from foundations and toward approved drainage devices. Irrigation of landscaping should be controlled to maintain, as much as possible, consistent moisture content sufficient to provide healthy plant growth without overwatering.

4.4 Seismic Design Parameters

Moderate to strong ground shaking due to seismic activity is expected at the site during the life span of the project. A site-specific ground motion analysis was performed in accordance with the 2013 California Building Code (CBC) following the procedures of ASCE 7-10 Publication, Section 21.2, as presented in Appendix D.

The deterministic and probabilistic seismic hazard analysis was performed using the computer program EZ-FRISK (Risk Engineering, 2011) to estimate peak horizontal ground acceleration (PHGA) that could occur at the site, and to develop design response spectra. Various probabilistic density functions were used in this analysis to assess uncertainty inherent in these calculations with respect to magnitude, distance and ground motion. An averaging of the following next-generation attenuation relationships (NGAs) was used with equal weights to calculate site-specific PHGA and spectra:

- Abrahamson et al. (2014),
- Boore et al. (2014),
- Campbell and Bozorgnia (2014), and
- Chiou and Youngs (2014).

The design response spectrum shown on Figure D-1 is derived from a comparison of probabilistic Maximum Considered Earthquake (MCE) and the 84th percentile of the deterministic MCE. In accordance with the 2013 CBC, peak ground accelerations are estimated based on earthquake ground motion having a 2 percent probability of exceedance in 50 years (ASCE, 2010). The seismic coefficients for the General Procedure were calculated utilizing an interactive program on current United States Geological Survey (USGS) website using ASCE 7-10 reference. The site-specific seismic coefficients are presented in Table 1 below.

Categorization/Coefficients	**Code-Based (1) (2)**	**Site-Specific (2) (3)**
Site Longitude (decimal degrees) West	-118.25813	
Site Latitude (decimal degrees) North	34.06439	
Site Class	C	
Mapped Spectral Response Acceleration at 0.2s Period, S_s	2.514	-
Mapped Spectral Response Acceleration at 1s Period, S_1	0.887	-
Short Period Site Coefficient at 0.2s Period, F_a	1.0	-
Long Period Site Coefficient at 1s Period, F_v	1.3	-
Adjusted Spectral Response Acceleration at 0.2s Period, S_{MS}	2.514	2.514
Adjusted Spectral Response Acceleration at 1s Period, S_{M1}	1.153	1.153
Design Spectral Response Acceleration at 0.2s Period, S_{DS}	1.676	1.866
Design Spectral Response Acceleration at 1s Period, S_{D1}	0.768	0.902

1. All were derived from the USGS web page: http://earthquake.usgs.gov/designmaps/us/application.php
2. All coefficients in units of g (spectral acceleration)
3. See Appendix D for details of the site-specific evaluation.

Based on our borings, the building will be underlain by relatively dense siltstone and claystone of the Puente Shale formation. Therefore, in accordance with the 2013 CBC, this site should be classified as a Class C site. The results of this analysis also indicate that the Peak Ground Acceleration (PGA_M) for this site is 0.953g based on the USGS General Procedure. The summary reports are included in Appendix D.

4.5 Footing Foundations

New shallow spread footings established on bedrock may be used to support the proposed residential structures. Spread footing design recommendations are presented in the following subsections:

4.5.1 Minimum Embedment and Width

Continuous strip footings should have a minimum width of 12 inches. Isolated square pad column footings are recommended to be a minimum of 24 inches in width. The top of the footing should be at least 12 inches below lowest adjacent grade or finish floor elevation.

4.5.2 Allowable Bearing Capacity

The footings may be designed for a maximum net allowable soil bearing pressure of 6,500 pounds per square foot (psf) for isolated column footings and 6,000 psf continuous strip footings. The soil bearing pressure may be increased by one-third for transient loads such as wind and seismic forces.

4.5.3 Lateral Load Resistance

Resistance to lateral loads will be provided by a combination of friction between the soil and foundation interface and passive pressure acting against the vertical portion of the footings. For calculating allowable lateral resistance, a passive pressure of 250 psf per foot of depth to a maximum of 2,500 psf and a frictional coefficient of 0.25 may be used provided the foundations are supported within structural compacted fill as previously described. When combining frictional and passive resistance, the passive resistance should be reduced by one-third.

4.5.4 Settlement

The estimated total settlement of the structures supported on spread footings as recommended above is less than 1 inch. The differential settlement between adjacent columns is estimated to be less than ½ inch over a horizontal distance of 40 feet.

4.6 Slab-On-Grade

It is anticipated that the basement floor of both buildings will bear on compacted fill established on bedrock. From an expansive soil standpoint, we recommend the slab-on-grade be a minimum 5 inches thick with No. 4 rebar placed at the center of the slab at 16 inches on center in each direction. The structural engineer should design the actual thickness and reinforcement based on anticipated loading conditions. The slabs may be design for an average allowable bearing pressure of 1,500 psf for dead plus live loads with a maximum localized bearing pressure of 2,000 psf for column or wall loads. The allowable bearing pressure may be increased by one-third for short-term loading including wind and seismic loads.

A subdrain system consisting of a 9-inch layer of 1-inch open graded rock should be installed under the slab. The slab subdrain and the basement wall backdrain should be diverted to an approved discharge system.

Floor slabs are recommended to be underlain by a synthetic sheeting to serve as a retarder to moisture vapor transmission in areas where moisture-sensitive floor covering (such as vinyl, tile, or carpet) or equipment is planned. The sheeting is recommended to be a minimum 15 mil thick Stego® Wrap installed per manufacturer's specifications. Prior to installing the synthetic sheeting, the exposed subgrade surface should be clear of all extruding rock and gravel that could damage the sheeting. The sheeting should be evaluated for the presence of punctures or tears by the installer prior to pouring concrete. Installation of the sheeting should include proper overlap and taping of seams.

Leighton does not practice in the field of moisture vapor transmission evaluation, since this is not specifically a geotechnical issue. Therefore, we recommend that a qualified person, such as the flooring subcontractor and/or structural engineer, be consulted with to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. That person should provide recommendations for mitigation of potential adverse impact of moisture vapor transmission on various components of the structures as deemed appropriate.

Minor cracking of concrete after curing due to drying and shrinkage is normal and should be expected; however, concrete is often aggravated by a high water/cement ration, 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. Additionally, our experience indicates that the use of reinforcement in slabs and foundations can generally reduce the potential for concrete cracking.

To reduce the potential for excessive cracking, concrete slabs-on-grade should be provided with construction or weakened plane joints at frequent intervals. Joints should be laid out to form approximately square panels.

4.7 Corrosion Protection Measures

Corrosion test results are summarized in Section 2.2.2, Soil Corrosivity and presented in Appendix B. The results of the resistivity test indicate the soil is corrosive to buried ferrous metals. These test results should be presented to the underground contractor for specific mitigation measures to reduce the risks associated with soil corrosivity.

Based on soluble sulfate test results, the bedrock materials also exhibit corrosion potential for concrete. Specific recommendations for treatment of concrete exposed to varying sulfate content are provided by the American Concrete Institute (ACI, 2011).

4.8 Surface Drainage

Surface drainage should be designed to direct water away from building and toward approved drainage devices. Irrigation of landscaping should be controlled to maintain, as much as possible, consistent moisture content sufficient to provide healthy plant growth without over watering.

5.0 CONSTRUCTION CONSIDERATIONS

5.1 Temporary Excavations

All temporary excavations, including footings, utility trenches, should be performed in accordance with project plans, specifications, and all OSHA requirements. Excavations 5 feet or deeper should be laid back or shored in accordance with OSHA requirements before personnel are allowed to enter.

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 cut, unless the cut is shored appropriately.

During construction, the soil conditions should be regularly evaluated to verify that conditions are as anticipated. The contractor shall be responsible for providing the “competent person” required by OSHA standards to evaluate soil conditions. The bedrock can be classified as Type B soil. Soil types will vary, but Type C soils can be expected at shallow depths. Close coordination between the competent person and the geotechnical engineer should be maintained to facilitate construction while providing safe excavations.

5.2 Oil Well Abandonment and Methane Mitigation

Leighton should review the oil well abandonment recommendations and methane mitigation plans developed by Roux to address any potential conflicts with geotechnical recommendations.

5.3 Additional Geotechnical Services

The geotechnical recommendations presented in this report are based on subsurface conditions as interpreted from limited subsurface explorations and limited laboratory testing. Our conclusions and recommendations presented in this report should be reviewed and verified by Leighton during site construction and revised accordingly, if exposed geotechnical conditions vary from our preliminary findings and interpretations. The recommendations presented in this report are only valid if Leighton verifies the site conditions during construction.

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

- Grading and excavation of the site;
- Subgrade Preparation;
- Compaction of all fill materials;
- Utility trench backfilling and compaction;
- Footing excavation and slab-on-grade preparation;
- During installation of temporary shoring, wherever needed; and
- When any unusual conditions are encountered.

6.0 LIMITATIONS

This report was based solely on data obtained from a limited number of geotechnical explorations, and soil samples and tests. Such information is, by necessity, incomplete. The nature of many sites is such that differing soil or geologic conditions can be present within small distances and under varying climatic conditions. Changes in subsurface conditions can and do occur over time. Therefore, the findings, conclusions, and recommendations presented in this report are only valid if Leighton and Associates has the opportunity to observe subsurface conditions during grading and construction, to confirm that our preliminary data are representative for the site. Leighton and Associates, Inc. should also review the construction plans and project specifications, when available, to comment on the geotechnical aspects.

This report was prepared using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. The findings, conclusion, and recommendations included in this report are considered preliminary and are subject to verification. We do not make any warranty, either expressed or implied.

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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, clients can benefit from a lowered exposure to the subsurface problems 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 below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not 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. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project 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.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- 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, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been 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 your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

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

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment 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 from project start to project finish, so the individual can provide 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 not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed 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 full-time 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 on hand quickly 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 observation.

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 informational 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, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. 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. 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 Not 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 relate any 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 yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

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, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled 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 not 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 not building-envelope or mold specialists.***

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Figures

Project: 11388.001	Eng/Geol: LJD/JAR
Scale:1 " = 2,000 '	Date: August 2016

Base Map: ESRI ArcGIS Online 2016
Thematic Information: Leighton
Author: Leighton Geomatics (asakowicz)

SITE LOCATION MAP

1346, 1350 and 1352 W. Court Street
City of Los Angeles, California

Figure 1

Leighton

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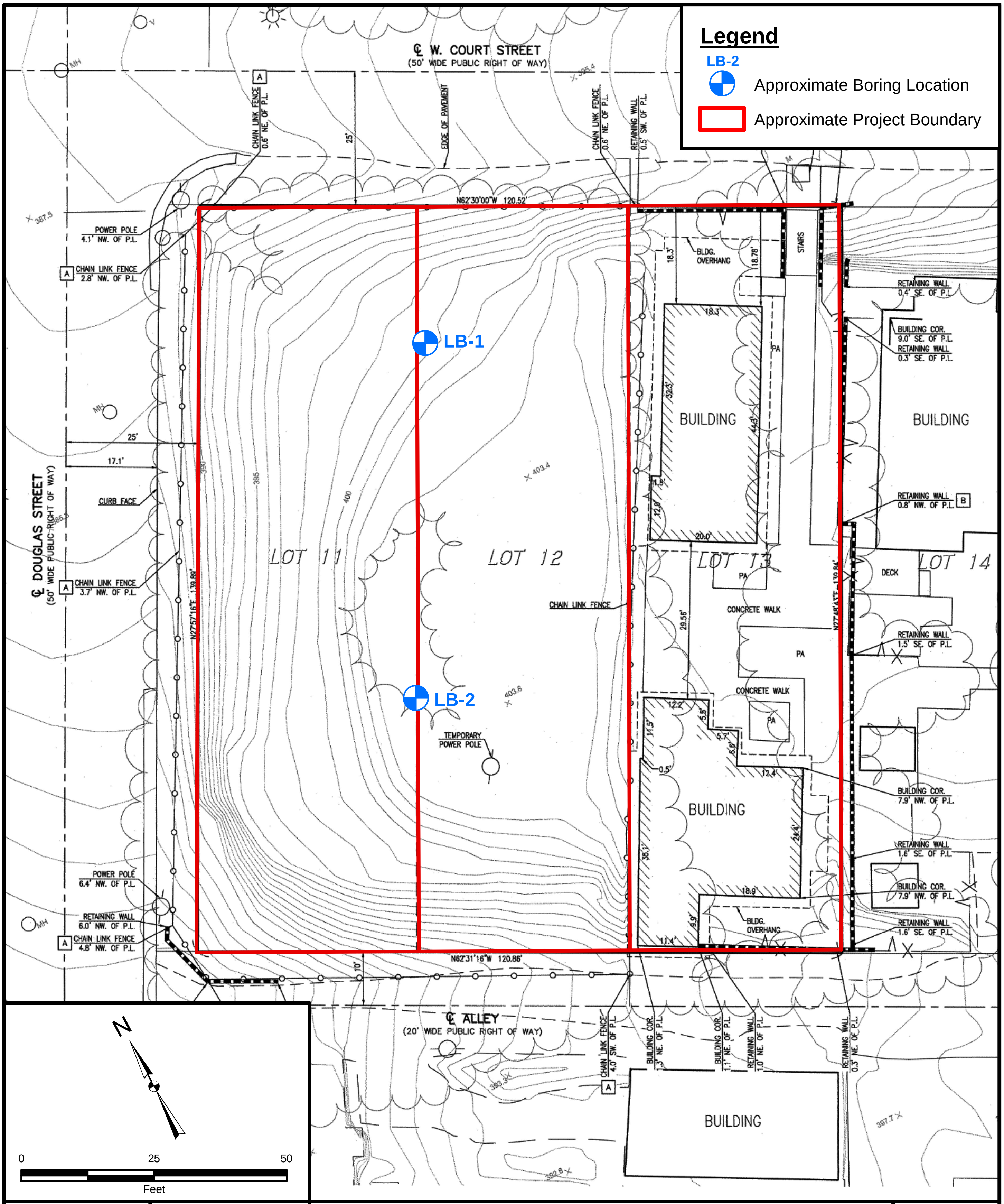

Project: 11388.001

Eng/Geol: LJD/JAR

Scale: 1 " = 25 '

Date: August 2016

Base Map: C&V Consulting, Inc. "DBCX-002 sheet #2
ALTA/ACSM Land Title Survey"
6 orchard, Suite 200 Lake Forest, CA 92630
Thematic Information: Leighton
Author: Leighton Geomatics (asakowicz)

BORING LOCATION MAP

1346, 1350 and 1352 W. Court Street
City of Los Angeles, California

Figure 2

Leighton

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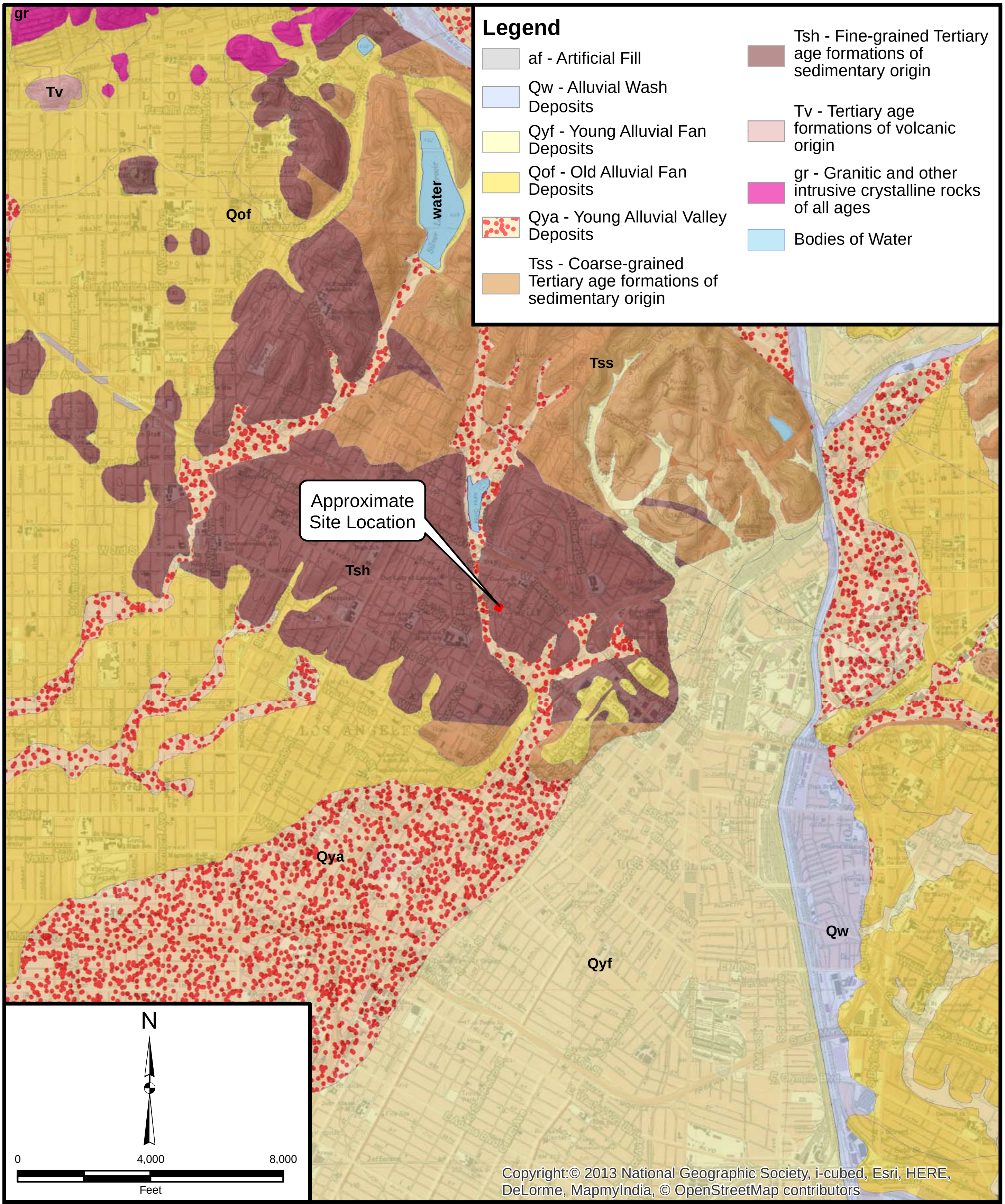

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Project: 11388.001	Eng/Geol: LJD/JAR
Scale:1 " = 4,000 '	Date: August 2016

Base Map: Preliminary Geologic Map of the Los Angeles 30' x 60' Quadrangle, Yerkes & Campbell, 2005. USGS

Thematic Information: Leighton, USGS

Author: Leighton Geomatics (asakowicz)

REGIONAL GEOLOGY MAP

1346, 1350 and 1352 W. Court Street
City of Los Angeles, California

Figure 3

Leighton

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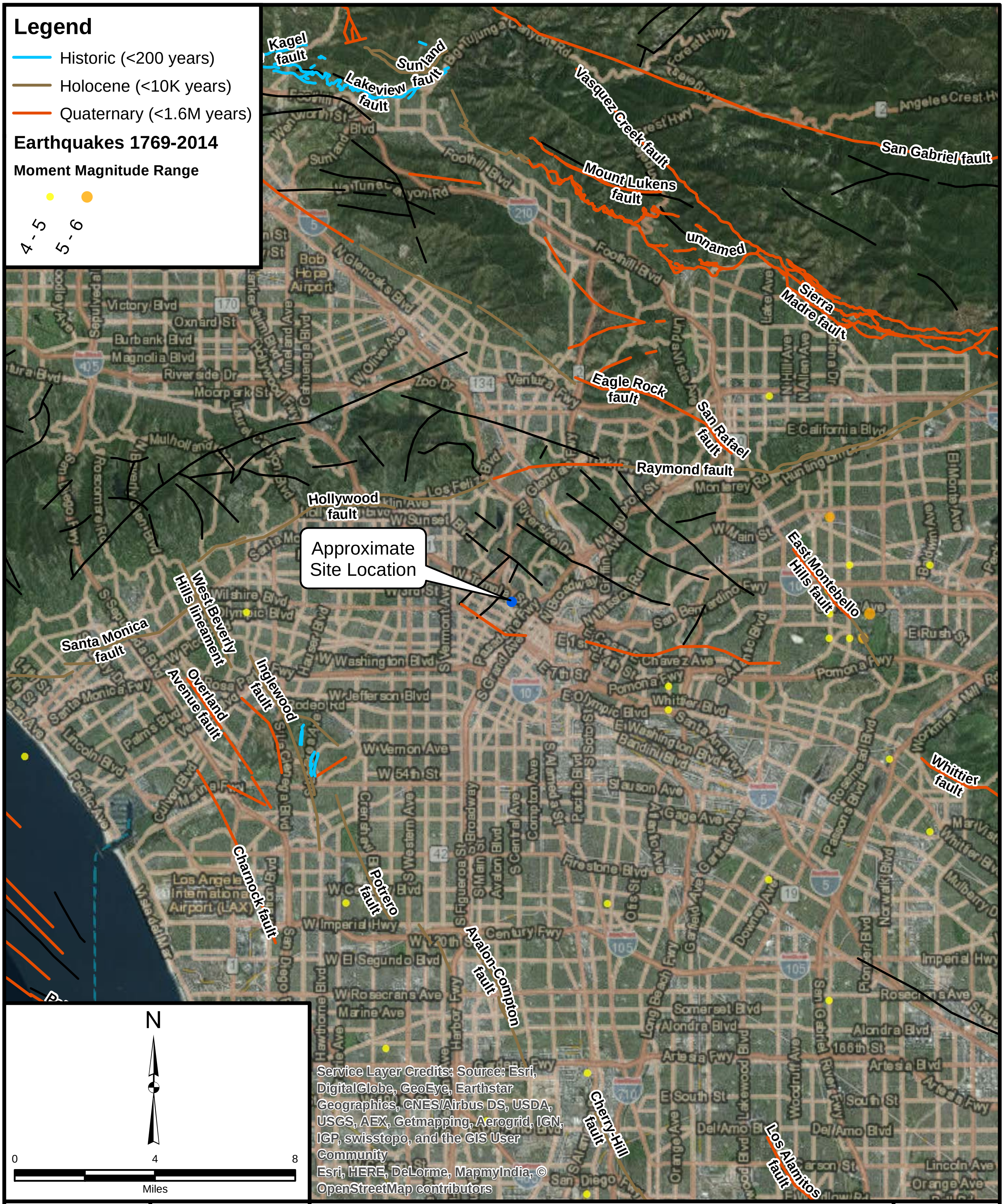

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Project: 11388.001	Eng/Geol: LJD/JAR
Scale:1 " = 4 miles	Date: August 2016

Base Map: ESRI ArcGIS Online 2016
Thematic Information: Leighton, Bryant, W. A. (compiler), 2005, Digital Database of Quaternary and Younger Faults from the Fault Activity Map of California, version 2.0: CGS USGS, SCEC.
Author: Leighton Geomatics (asakowicz)

REGIONAL FAULT AND HISTORICAL SEISMICITY MAP

1346, 1350 and 1352 W. Court Street
City of Los Angeles, California

Figure 4

Leighton

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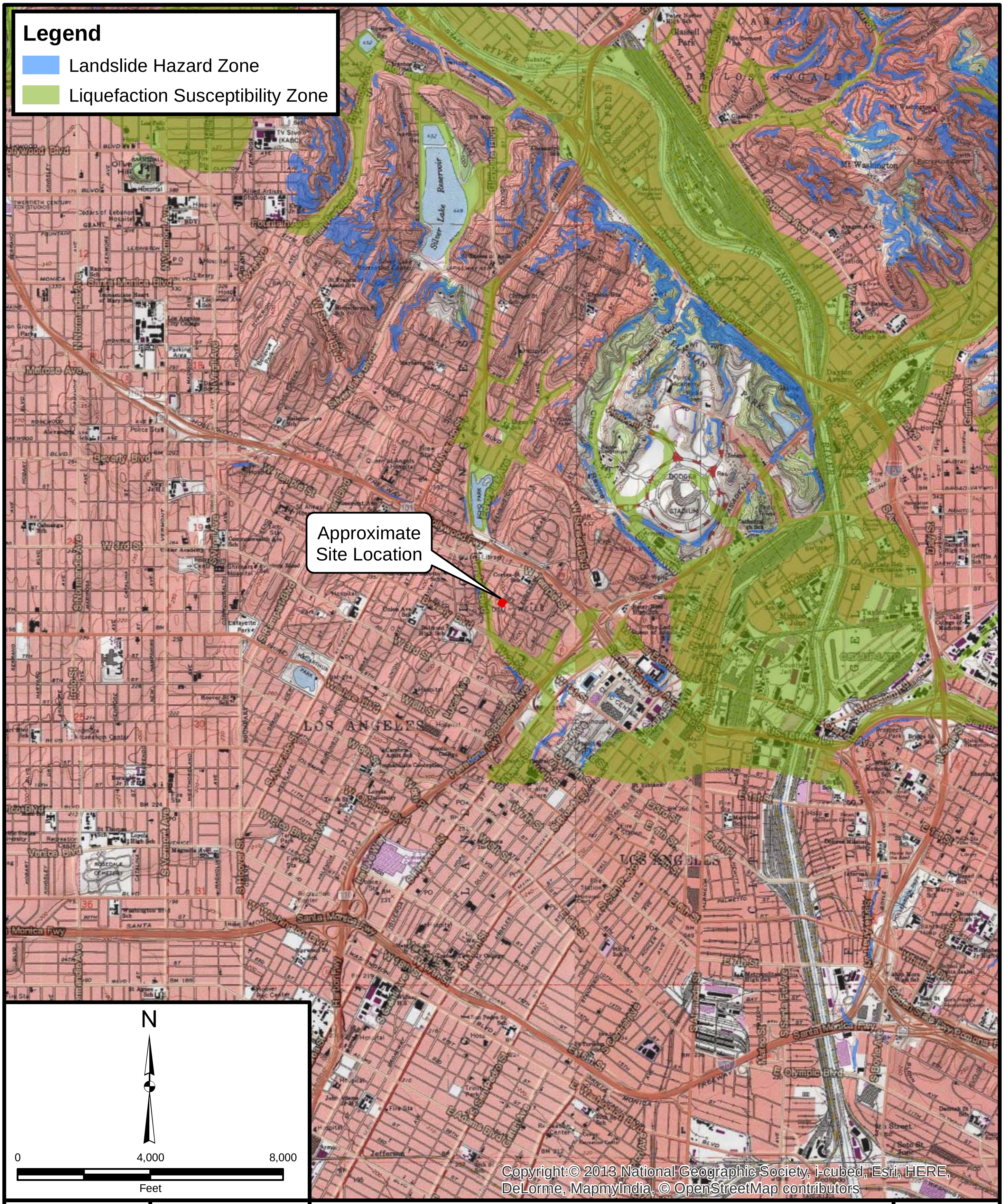

Project: 11388.001

Eng/Geol: LJD/JAR

Scale:1 " = 4,000 '

Date: August 2016

Base Map: ESRI ArcGIS Online 2016
Thematic Information: Leighton, "Seismic Hazards Zones, Los Angeles Quadrangle," 1999. California Department of Conservation
Author: Leighton Geomatics (asakowicz)

SEISMIC HAZARD MAP

1346, 1350 and 1352 W. Court Street
City of Los Angeles, California

Figure 5

Leighton

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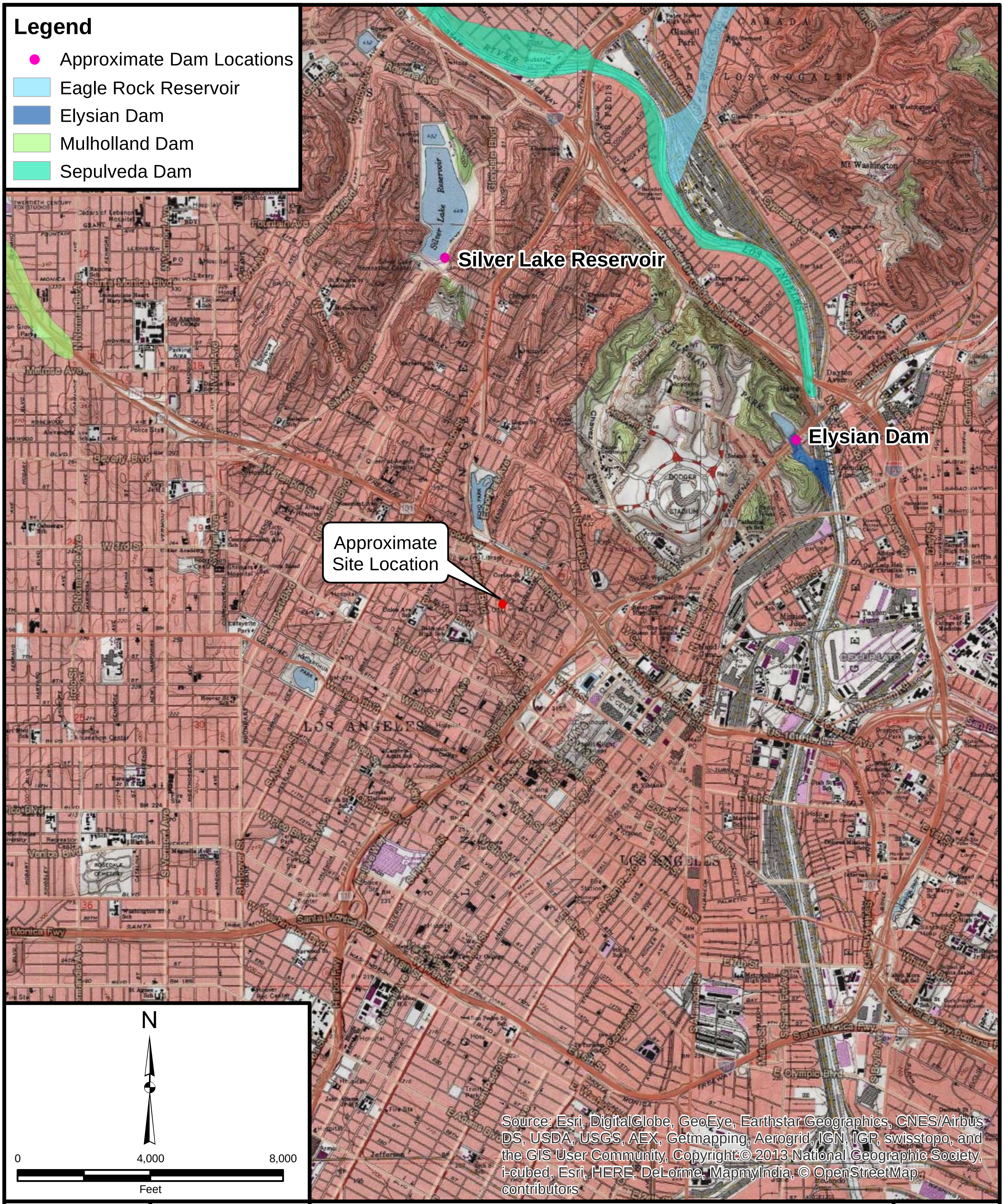

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Project: 11388.001	Eng/Geol: LJD/JAR
Scale:1 " = 4,000 '	Date: August 2016

Base Map: ESRI ArcGIS Online 2016

Thematic Information: Leighton, California Department of Emergency Services, CalOES

Author: Leighton Geomatics (asakowicz)

DAM INUNDATION MAP

1346, 1350 and 1352 W. Court Street
City of Los Angeles, California

Figure 6

Leighton

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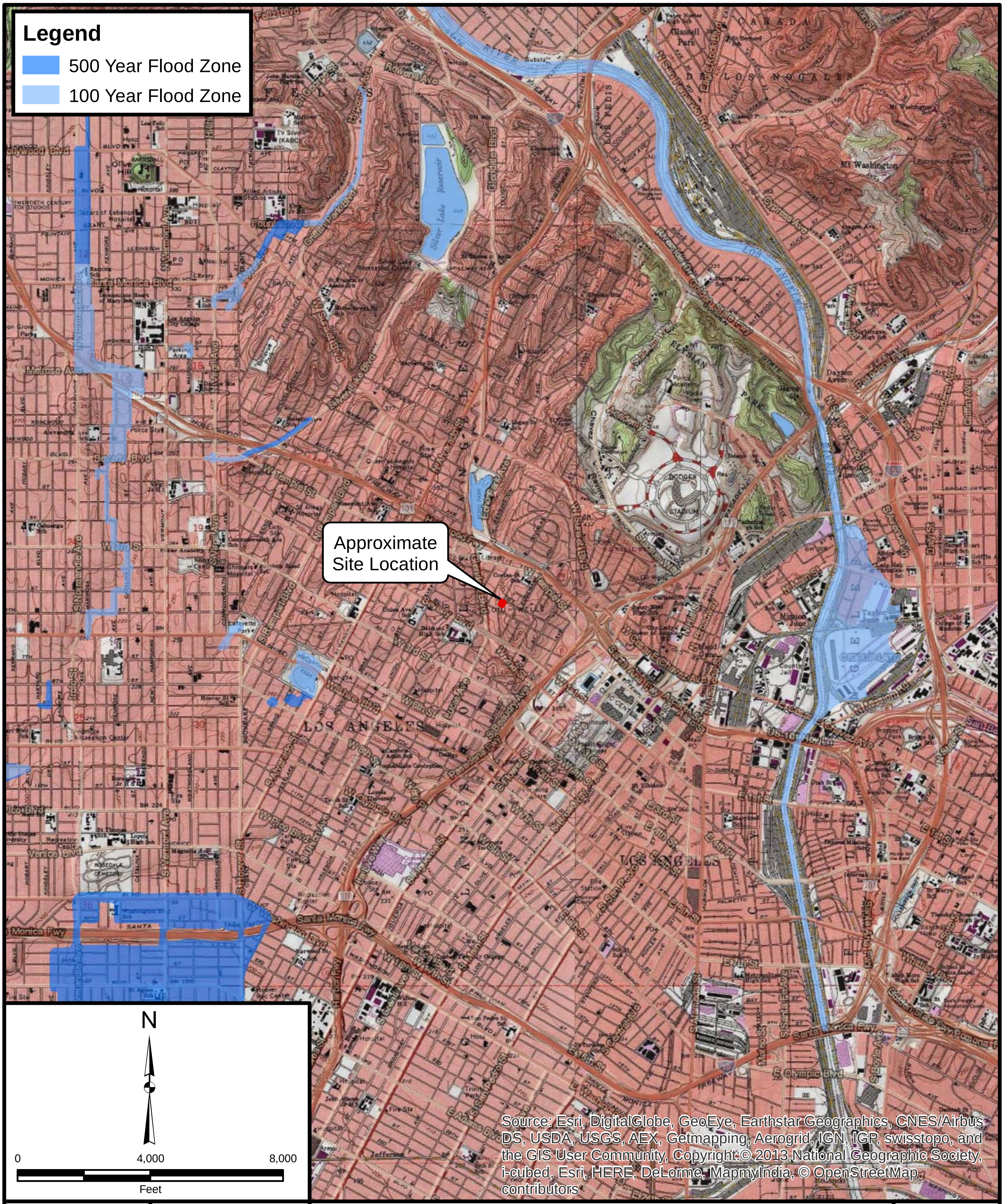

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Project: 11388.001	Eng/Geol: LJD/JAR
Scale:1 " = 4,000 '	Date: August 2016

Base Map: ESRI ArcGIS Online 2016
Thematic Information: Leighton, CA Department of Water Resources, FEMA
Author: Leighton Geomatics (asakowicz)

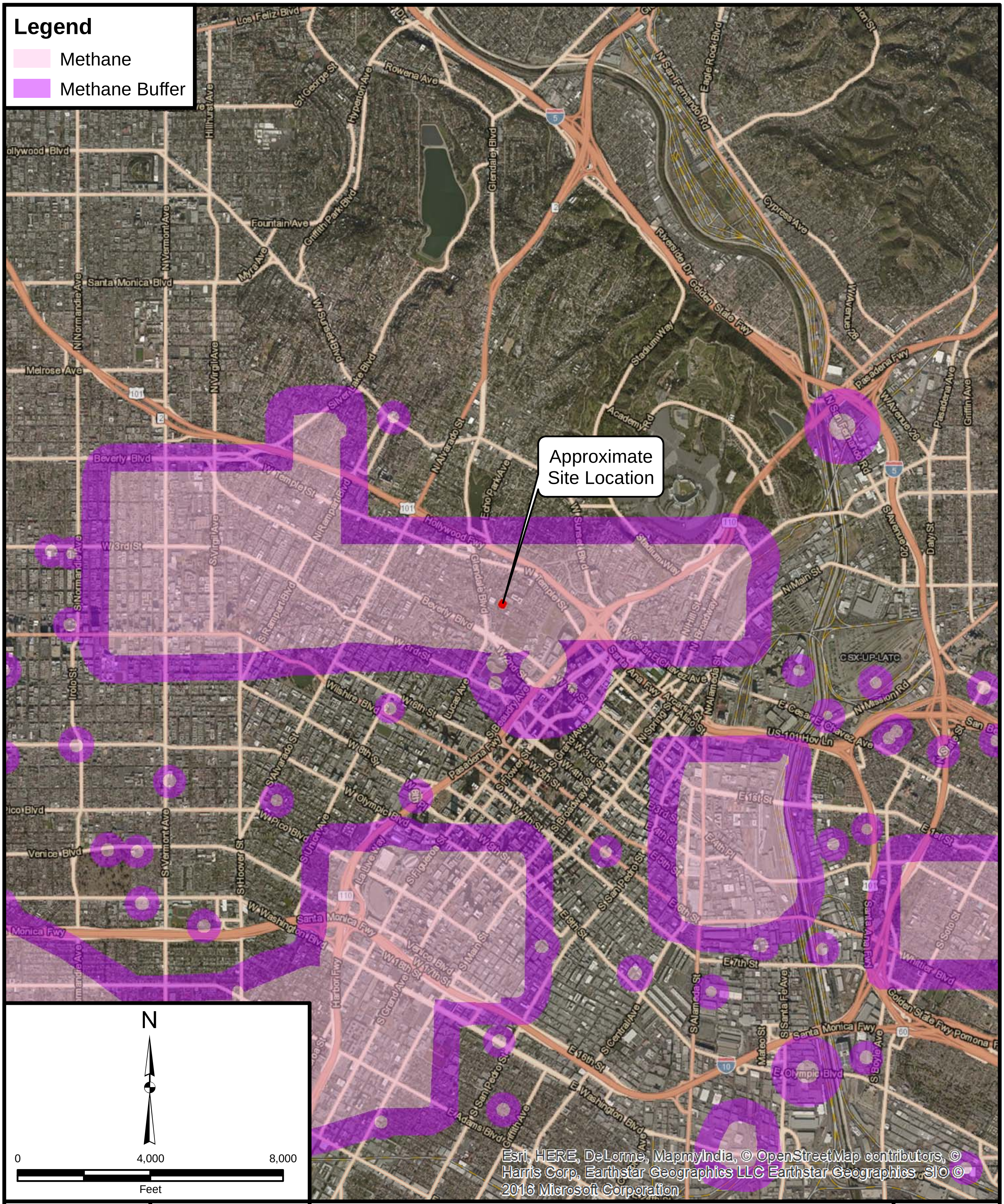
FLOOD HAZARD ZONE MAP

1346, 1350 and 1352 W. Court Street
City of Los Angeles, California

Figure 7

Leighton

Map Saved as V:\Drafting\11388\001\Maps\11388_001_F08_Flood_2016-07-13.mxd on 8/23/2016 7:13:04 AM



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Project: 11388.001

Eng/Geol: LJD/JAR

Scale:1 " = 4,000 '

Date: August 2016

Base Map: ESRI ArcGIS Online 2016
Thematic Information: Leighton
Author: Leighton Geomatics (asakowicz)

METHANE HAZARD MAP

1346, 1350 and 1352 W. Court Street
City of Los Angeles, California

Figure 8

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Map Saved as V:\Drafting\11388\001\Maps\11388_001_F09_Methane_2016-07-13.mxd on 8/23/2016 7:15:21 AM

APPENDIX A
FIELD EXPLORATION LOGS

GEOTECHNICAL BORING LOG B-1

Project No. 11388.001
Project Proposed Residential Developments-West Court Street
Drilling Co. 2R Drilling
Drilling Method Hollow Stem Auger - Autohammer
Location

Date Drilled 7-28-16
Logged By EMH
Hole Diameter "
Ground Elevation 400 ft. MSL'
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
	0			BB-1						@0': **Artificial Fill (afu)**	
										Puente Formation Bedrock (Tp)	
				R-1		20 50/4"				@2': Silty CLAYSTONE, brown to orange brown, moist, laminated, heavilty weathered	
	5			R-2		20 33 50				@5': Interbedded SILTSTONE and CLAYSTONE, orange brown and tan, thinly bedded, weathered, potentially diamtomaceous laminations	
				R-3		33 50/5"				@7': SILTSTONE, olive brown, laminated, weathered, slightly fissile, some clay and fine sand	
	10			R-4		37 50/6"				@10': Interbedded SILTSTONE and CLAYSTONE, orange brown, hard, oxidized, with minor sandy laminations	
	15			R-5		19 51/6"				@15': Increasing clay	
	20			R-6		33 50/5"				@20': Becomes more thinly laminated, oxidation staining on laminations, thin discrete claystone laminations, some fine sand	
										Total Depth: 20.7 Feet Groundwater not encountered to maximum depth explored Boring backfilled with tamped cuttings upon completion	
										Bedding attitudes, SW corner of site: N54E 60SE N65E 55SE N58E 63SE	
	25										
	30										

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

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

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

GEOTECHNICAL BORING LOG B-2

Project No. 11388.001
Project Proposed Residential Developments-West Court Street
Drilling Co. 2R Drilling
Drilling Method Hollow Stem Auger - Autohammer
Location

Date Drilled 7-28-16
Logged By EMH
Hole Diameter "
Ground Elevation 403 ft. MSL'
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						@0': **Artificial Fill (afu)**	
										Sandy SILT (ML), dark brown, hard, fine sand, with miscellaneous debris, potential cement slurry	
				R-1		9 21 31					
										Puente Formation Bedrock (Tp)	
	5			R-2		10 25 32	90	15		@5': SILTSTONE, olive brown and tan, laminated, some fine sand, heavily weathered, with diatomaceous laminations	
				R-3		12 50/6"				@7': thinly laminated, some minor clayey laminations, hard, weathered, nonfissile	
	10			R-4		20 41 50/4"	100	17		@10': Interbedded SILTSTONE and CLAYSTONE, olive brown to orange brown, thinly laminated, oxidized, with minor sandy laminations	
	15			R-5		45 50/3"	103	17		@15': SILTSTONE, orangish brown and grayish brown, hard, thinly bedded, nonfissile, some sand and clay	
	20			R-6		37 50/3"	103	20		@20': Interbedded SILTSTONE, CLAYSTONE, and SANDSTONE, olive brown, gray, and orangeish brown, hard, fine sand, nonfissile, with granitic clast	
										Total Depth: 20.8 feet Groundwater not encountered to maximum depth explored Boring backfilled with tamped cuttings upon completion	
	25										
	30										

SAMPLE TYPES:

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

TYPE OF TESTS:

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

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

APPENDIX B
LABORATORY TEST RESULTS

Sheet 1 of 1

Borehole	Depth	Liquid Limit	Plastic Limit	Plasticity Index	Maximum Size (mm)	%<#200 Sieve	Class-ification	Water Content (%)	Dry Density (pcf)	Satur-ation (%)	Void Ratio
B-2	5.0							14.6	89.8		
B-2	10.0							17.2	100.4		
B-2	15.0							17.0	102.5		
B-2	20.0							20.2	103.2		

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Summary of Laboratory Results

Project Name: Proposed Residential Developments-West Court Stre

Project Number: 11388.001

Date: 8/17/2016 11:05:13 AM

Figure No. 1

UNCONFINED COMPRESSIVE STRENGTH
of COHESIVE SOIL
ASTM D 2166

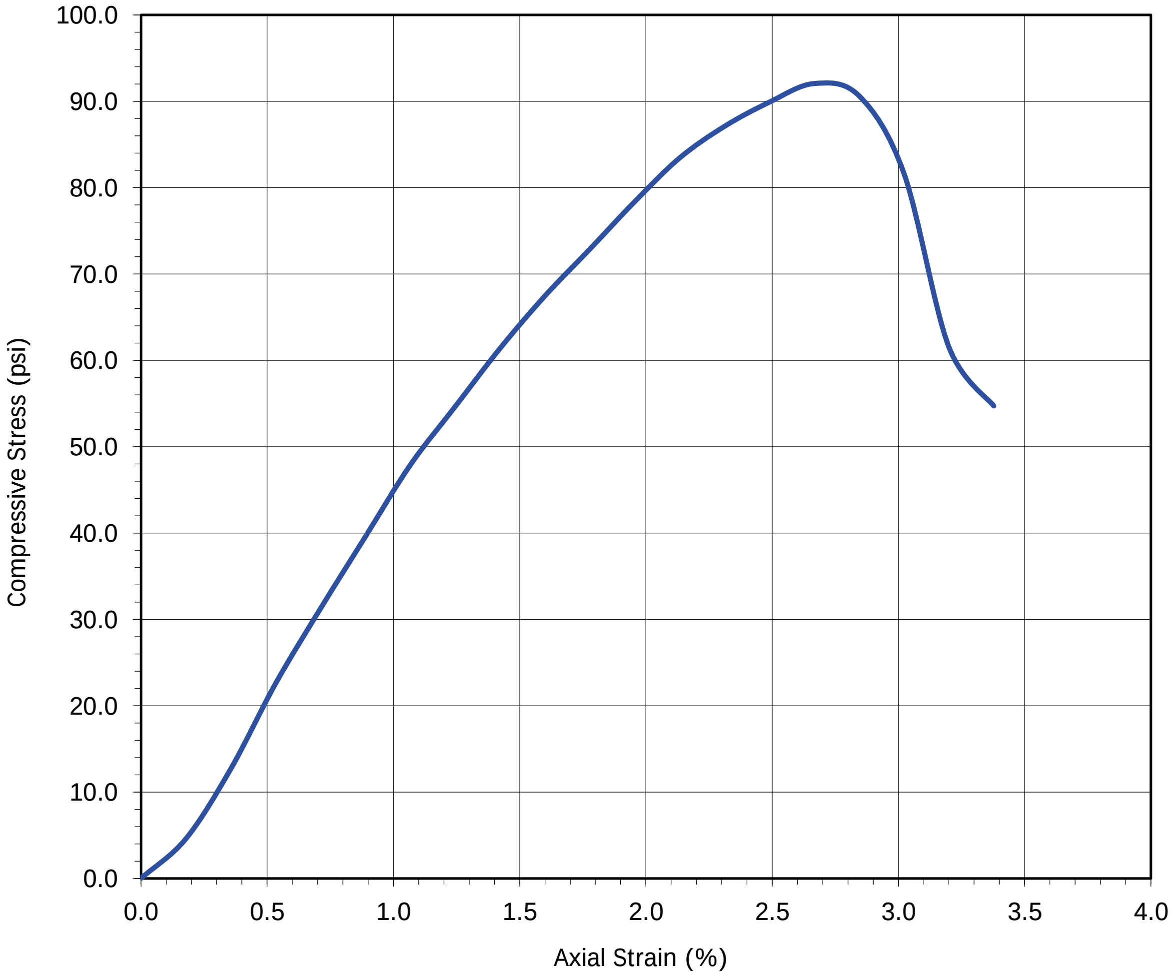
Project Name: Court Partners LLC
Tested by: A. Santos
Date: 08/16/16
Project No.: 11388.001
Checked by: J. Ward
Date: 08/17/16
Boring No.: LB-2
Sample Type: Ring
Sample No.: R6
Depth (ft): 20.0
Sample Description: Olive lean clay'stone' (CL)

Weight of Sample + Tube / Rings (g)	845.00
Weight of Tube / Rings (g)	0.00
Wet Weight of Soil + Container (g)	1144.00
Dry Weight of Soil + Container (g)	1002.40
Weight of Container (g)	300.30
Load Surcharge (lb)	2.20
Rate of Deformation (in/min)	0.045
Specific Gravity (Assumed)	2.70

Sample Measurements	
Diameter (in)	2.424
	2.423
	2.423
Area (sq.in.)	4.612
Height (in)	5.626
	5.625
	5.624

Axial Deformation (in.)	Load (lb.)	Compressive Stress (psi)	Axial Strain (%)
0.0000	0.0	0.00	0.000
0.0100	19.0	4.59	0.178
0.0200	56.5	12.68	0.356
0.0300	102.5	22.58	0.533
0.0400	143.0	31.26	0.711
0.0500	182.0	39.58	0.889
0.0600	221.0	47.88	1.067
0.0700	253.0	54.64	1.244
0.0800	285.0	61.38	1.422
0.0900	314.0	67.46	1.600
0.1000	340.0	72.87	1.778
0.1100	366.5	78.38	1.956
0.1200	391.0	83.43	2.133
0.1300	409.0	87.09	2.311
0.1400	423.0	89.89	2.489
0.1500	434.0	92.05	2.667
0.1600	428.0	90.62	2.844
0.1700	386.0	81.62	3.022
0.1800	291.0	61.53	3.200
0.1900	259.0	54.72	3.378

Axial Deformation (in.)	Load (lb.)	Compressive Stress (psi)	Axial Strain (%)



Boring No.: LB-2
Sample No.: R6
Depth (ft): 20.0
Soil Type: Ring
Sample Description: Olive lean clay'stone' (CL)

Sample Diameter (in.)	2.423
Sample Height (in.)	5.625
Initial Moisture Content (%)	20.17
Dry Density (pcf)	103.3
Specific Gravity (assumed)	2.7
Saturation (%)	86.2
Rate of Deformation (in/min)	0.0450
Height / Diameter Ratio	2.32

At Failure

Compressive Strength (psi)	**92.05**
Axial Strain (%)	2.67

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Unconfined Compressive Strength of Cohesive Soil
ASTM D 2166

Project No.: **11388.001**

Court Partners LLC

08/16/16

Leighton

TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS

Project Name: Court Partners LLC
Tested By : G. Berdy Date: 08/15/16
Project No. : 11388.001
Data Input By: J. Ward Date: 08/17/16

Boring No.	LB-2			
Sample No.	R4			
Sample Depth (ft)	10.0			
Soil Identification:	Olive brown silt'stone' (ML)			
Wet Weight of Soil + Container (g)	135.29			
Dry Weight of Soil + Container (g)	127.23			
Weight of Container (g)	56.73			
Moisture Content (%)	11.43			
Weight of Soaked Soil (g)	100.53			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	93			
Crucible No.	3			
Furnace Temperature (°C)	860			
Time In / Time Out	10:00/10:45			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	42.6789			
Wt. of Crucible (g)	42.6523			
Wt. of Residue (g) (A)	0.0266			
PPM of Sulfate (A) x 41150	1094.59			
PPM of Sulfate, Dry Weight Basis	**1236**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	30			
ml of AgNO3 Soln. Used in Titration (C)	1.6			
PPM of Chloride (C -0.2) * 100 * 30 / B	140			
PPM of Chloride, Dry Wt. Basis	**158**			

pH TEST, DOT California Test 643

pH Value	**7.45**			
Temperature °C	20.2			

SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name: Court Partners LLC

Project No. : 11388.001

Boring No.: LB-2

Sample No. : R4

Tested By : G. Berdy Date: 08/16/16

Data Input By: J. Ward Date: 08/17/16

Depth (ft.) : 10.0

Soil Identification:* Olive brown silt'stone' (ML)

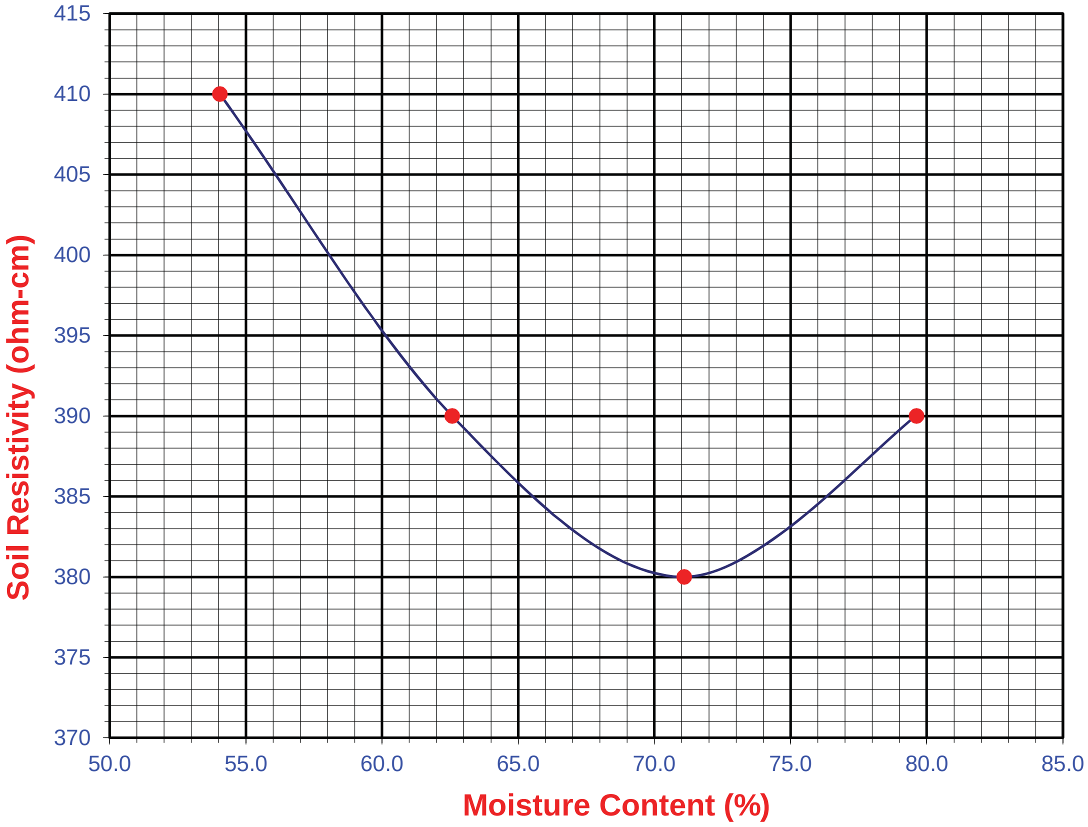
*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	50	54.05	410	410
2	60	62.58	390	390
3	70	71.10	380	380
4	80	79.62	390	390
5				

Moisture Content (%) (MCi)	11.43
Wet Wt. of Soil + Cont. (g)	135.29
Dry Wt. of Soil + Cont. (g)	127.23
Wt. of Container (g)	56.73
Container No.	
Initial Soil Wt. (g) (Wt)	130.73
Box Constant	1.000

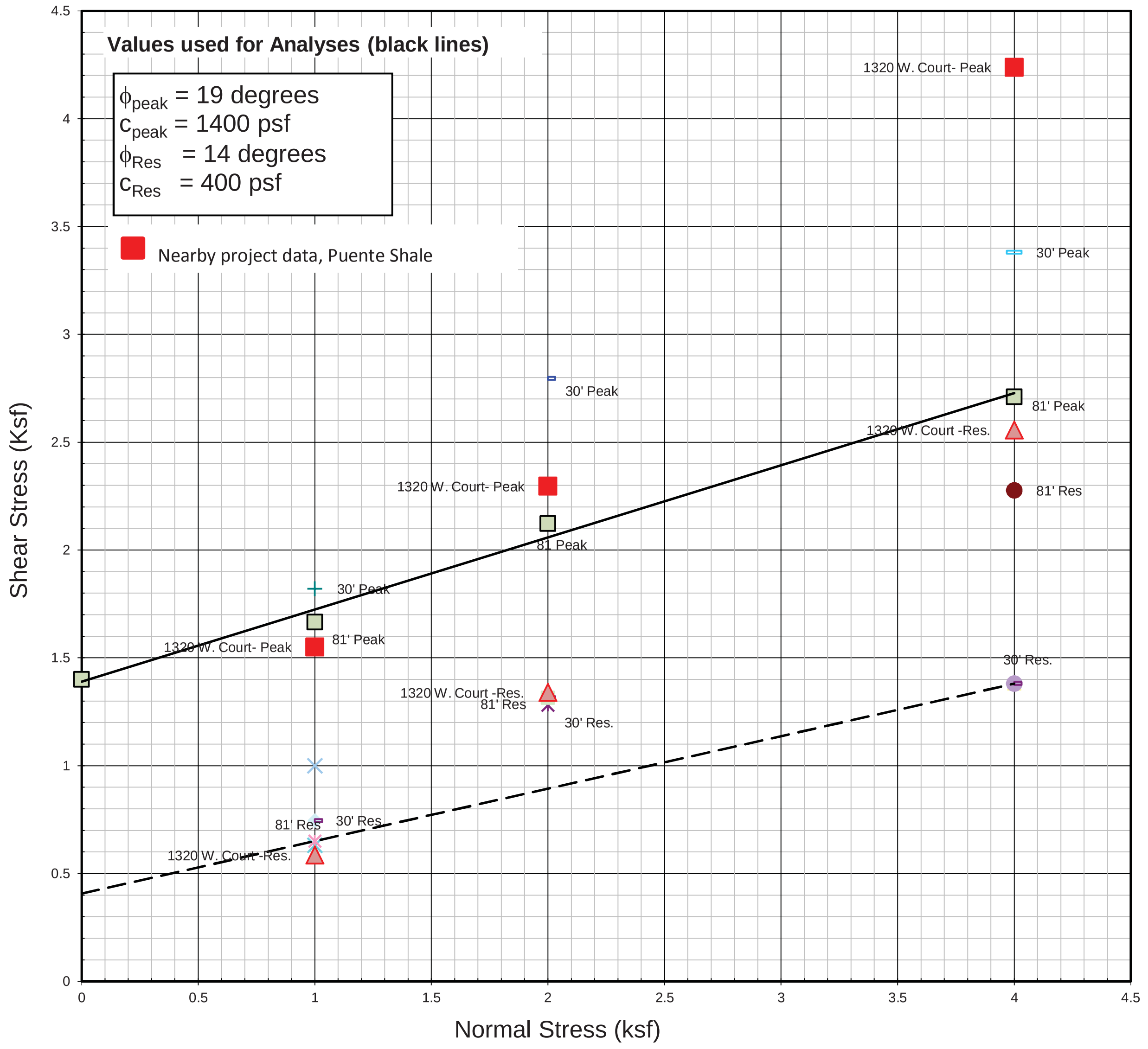
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100

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	
380	**71.1**	**1236**	**158**	**7.45**	**20.2**



Geotechnical Parameters Summary, Puente Shale (Court and Colton Street Projects) 11388.001 Court Street

Location	1346 Court Street, El.		vicinity, West Court Street, El. 440		1301 Colton Street, El.		301 Yale Street	Hollywood SHZP Report
sample depth/type	**Fill/soil, 0 to 5 ft.**	**Tpsh, 0 to 22 ft.**	**Fill/soil, 0 to 5 ft.**	**Tpsh, 5 to 51.5 ft.**	**Fill/soil, 0 to 2 ft.**	**Tpsh, 0 to 22 ft.**	**Tpsh , 15 to 30 ft.**	**Tpsh (abc)**
Classification	CH, silt clay	Siltstone, Claystone	CH, silty clay	Siltstone, Claystone		Sandstone, Claystone	Siltstone, Claystone	Siltstone, Claystone
ieve (%Gr/%Sand/%Fines)			0/8/92				0/6/94	
unit weight, pcf	103	121		111	107	122		
MC, %	14.6%	17-20%						
Atterberg Limit		PI=42, LL=65	PI=36, LL=58				PI=31, LL=55	
Expansion Index			94				76	
Strength, peak		phi = 19, C=1400		phi = 43, C=600psf			phi = 19, C=1400	
Strength, residual		phi = 14, C=400		phi = 28, C=200psf			phi = 14, C=400	phi = 19 to 26, C=300 to 364
R-value			17					
Max, pcf			112 pcf @12%OMC					
Sulfates/Chlorides (PPM)		1236/158	1428/20					
pH		7.45						
Resistivity, ohm-cm		380	400					
UC, psi		92.05		30.92		79.15	48.68	



Summary of Direct Shear Test Results
Puente Shale, vicinity of Court Street
1346 Court Street, Los Angeles, California

Project No. 11388.001
Date: August 2016

APPENDIX C
PREVIOUS GEOTECHNICAL DATA

GEOTECHNICAL BORING LOG HSA-1

Project No.		**Date Drilled**	3-19-13
Project		**Logged By**	BCP
Drilling Co.	J& H Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	~1 foot above street'
Location	Los Angeles, California	**Sampled By**	BCP

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.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0								2" Asphalt, 4" Base 0-6": Dry	
				B-1				CL	**Undocumented Artificial Fill (Afu)** @ 1-4': CLAY: brown, fine-grained sand, dry	
	5		0.0	R-1 LB-3-5'	16 31 42				**Puente Formation Bedrock (Tp)** @ 5': SILTSTONE/CLAYSTONE with interbedded sand, brown to grey, fine-grained sand, dry, hard, thinly bedded	
	10		0.0	S-1 LB-3-10'	14 35 20				@10': SILTSTONE/CLAYSTONE with interbedded sand, brown, fine-grained sand, dry, hard	
	15		0.0	R-2	5 6 16				@ 15': SILTSTONE/CLAYSTONE, light brown to grey, trace fine-grained sand, dry, very stiff	
	20		0.0	S-2	6 13 14				@ 20': SILTSTONE/CLAYSTONE, light brown to grey, trace fine-grained sand, dry, hard	
	25		0.0	R-3	12 25 29				@ 25': SILTSTONE/CLAYSTONE, grey, trace fine-grained sand, interbedded gypsum, dry, hard, thinly bedded	
	30									

SAMPLE TYPES:

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

TYPE OF TESTS:

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

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

GEOTECHNICAL BORING LOG HSA-1

Project No.
Project
Drilling Co. J& H Drilling
Drilling Method Hollow Stem Auger - 140lb - Autohammer
Location Los Angeles, California

Date Drilled 3-19-13
Logged By BCP
Hole Diameter 8"
Ground Elevation ~1 foot above street'
Sampled By BCP

Elevation Feet	Depth Feet	Graphic Log N S	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	30		0.0	S-3	13 22 40				@ 30': SILTSTONE/CLAYSTONE, light brown to grey, trace fine-grained sand, dry, very dense	
	35		0.0	R-4 LB-3-35'	18 24 50/2"				@ 35': SILTSTONE/CLAYSTONE, light brown to grey, trace interbedded fine-grained sand, dry, very dense	
	40		0.0	S-4	25 28 30				@ 40': SILTSTONE/CLAYSTONE, light brown to grey, trace fine-grained sand, dry, very dense	
	45		0.0	R-5 LB-3-45'	35 50				@ 45': SILTSTONE/CLAYSTONE with interbedded sand, light brown to grey, fine-grained sand, dry, very dense, thinnly bedded, oxidized	
	50								**Total Depth = 46 Feet** **Groundwater was not encountered at time of drilling** **No hydrocarbon staining or odor observed in boring** **Backfilled with methane probe SP-1 installed at depths of 30, 20, and 15 feet bgs on 3/19/13**	
	55									
	60									

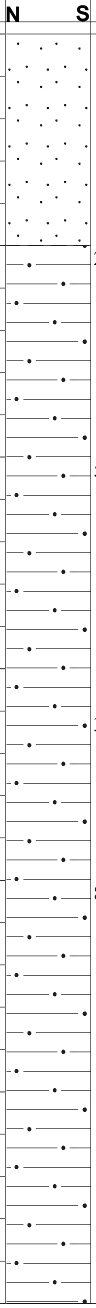
SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:

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

GEOTECHNICAL BORING LOG HSA-2

Project No.		**Date Drilled**	3-20-13
Project	[illegible]	**Logged By**	BCP
Drilling Co.	J& H Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	~15 foot above street'
Location	Los Angeles, California	**Sampled By**	BCP



Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	**SOIL DESCRIPTION** *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0							SP	**Undocumented Artificial Fill (Afu)** @Surface: poorly graded SAND with gravel, brown, dry, some clays	
	5		2.5	LB-4-5'	18 50				**Puente Formation Bedrock (Tp)** @ 5': SILTSTONE/CLAYSTONE, brown, some fine-grained sand, some fine gravel, dry, very dense, minor hydrocarbon staining and odor	
	10		5.5	LB-4-10'	12 13 19				@10': SILTSTONE/CLAYSTONE, brown, less gravel, some clay and fine-grained sand, dry, dense, hydrocarbon staining and odor	
	15									
			34.5	LB-4-16'	22 50				@ 16': SILTSTONE/CLAYSTONE, light brown to grey, some fine-grained sand, dry, very dense, hydrocarbon odor and staining	
	20		8.8	R-1 LB-4-20'	8 24 46				@ 20': SILTSTONE/CLAYSTONE with interbedded sand, light brown to grey, fine-grained sand, dry, hard, hydrocarbon staining and odor, thinly bedded, oxidized	
	25		1.8	S-1	12 24 36				@ 25': SILTSTONE/CLAYSTONE, light brown to grey, trace fine-grained sand, dry, hard, no hydrocarbon odor or staining	
	30									

SAMPLE TYPES:

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

TYPE OF TESTS:

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

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

GEOTECHNICAL BORING LOG HSA-2

Project No.		**Date Drilled**	3-20-13
Project		**Logged By**	BCP
Drilling Co.	J& H Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	~15 foot above street'
Location	Los Angeles, California	**Sampled By**	BCP

Elevation Feet	Depth Feet	Graphic Log N S	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	**SOIL DESCRIPTION** *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	30		2.5	R-2 LB-4-30'	20 23 50				@ 30': SILTSTONE/CLAYSTONE, light brown to grey, trace interbedded fine-grained sand, dry, very dense, no hydrocarbon staining or odor, thinnly bedded	
	35		5.4	S-2	13 25 30				@ 35': SILTSTONE/CLAYSTONE with interbedded sand, light brown to grey, fine-grained sand, dry, very dense, no hydrocarbon staining or odor	
	40		0.6	R-3 LB-4-40'	14 25 30				@ 40': SILTSTONE/CLAYSTONE, light brown to grey, trace interbedded fine-grained sand, dry, hard, no hydrocarbon odor or staining, thinnly bedded	
	45		0.5	S-3	15 31 50				@ 45': SILTSTONE/CLAYSTONE, light brown to grey, trace fine-grained sand, dry, hard, no hydrocarbon odor or staining	
	50		2.2	R-4	20 30 46				@ 50': SILTSTONE/CLAYSTONE, light brown to grey, trace fine-grained sand, dry, hard, no hydrocarbon odor or staining, thinnly bedded	
	55								**Total Depth = 51.5 Feet** **Groundwater was not encountered at time of drilling** **Backfilled with bentonite grout on 3/20/13**	
	60									

SAMPLE TYPES:

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

TYPE OF TESTS:

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

GEOTECHNICAL BORING LOG HSA-3

Project No.		**Date Drilled**	3-20-13
Project		**Logged By**	BCP
Drilling Co.	J& H Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	~15 foot above street'
Location	Los Angeles, California	**Sampled By**	BCP

Elevation Feet	Depth Feet	Graphic Log N S	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	**SOIL DESCRIPTION** *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0		0.0	B-1				SM	<u>Undocumented Artificial Fill (Afu)</u> @ Surface: poorly graded SAND: brown, fine to medium-grained sand with gravel, trace clay, dry, no hydrocarbon staining or odor	
	5		0.5	R-0 LB-5-5'	23 50				<u>Puente Formation Bedrock (Tp)</u> @ 5': SILTSTONE/CLAYSTONE, tan to brown, some clay, fine-grained sand, dry, very dense, no hydrocarbon odor or staining	
	10		2.1	S-1	7 8 25				@10': SILTSTONE/CLAYSTONE, grey with orange brown oxidation, some fine-grained sand with clay, dry, dense, minor hydrocarbon staining and odor	
			12.6	B-2						
	15		5.1	R-1 LB-5-15'	19 23 30				@ 15': SILTSTONE/CLAYSTONE, tan to grey, trace interbedded fine-grained sand, dry, hard, minor hydrocarbon staining and odor, thinnly bedded	
	20			S-2	9 10 35				@ 20': SILTSTONE/CLAYSTONE, tan to grey, some fine-grained sand, dry, hard, minor hydrocarbon staining and odor	
	25		2.1	R-2 LB-5-25'	25 25 30				@ 25': SILTSTONE/CLAYSTONE, light brown, trace interbedded fine-grained sand and gypsum, dry, very dense, no hydrocarbon staining or odor, thinnly bedded	
	30									

SAMPLE TYPES:

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

TYPE OF TESTS:

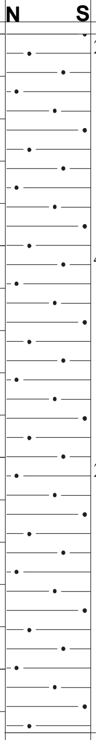
-200	% FINES PASSING
AL	ATTERBERG LIMITS
CN	CONSOLIDATION
CO	COLLAPSE
CR	CORROSION
CU	UNDRAINED TRIAXIAL
DS	DIRECT SHEAR
EI	EXPANSION INDEX
H	HYDROMETER
MD	MAXIMUM DENSITY
PP	POCKET PENETROMETER
RV	R VALUE
SA	SIEVE ANALYSIS
SE	SAND EQUIVALENT
SG	SPECIFIC GRAVITY
UC	UNCONFINED COMPRESSIVE STRENGTH

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

GEOTECHNICAL BORING LOG HSA-3

Project No.		**Date Drilled**	3-20-13
Project		**Logged By**	BCP
Drilling Co.	J& H Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	~15 foot above street'
Location	Los Angeles, California	**Sampled By**	BCP

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	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	30		2.1	S-3	10 15 19				@ 30': SILTSTONE/CLAYSTONE, light brown, fine-grained sand, dry, very dense, no hydrocarbon staining or odor	
	35		4.8	R-3 LB-5-35'	14 20 25				@ 35': SILTSTONE/CLAYSTONE, light brown, interbedded fine-grained sand, dry, very dense, no hydrocarbon staining or odor, thinnly bedded, oxidized	
	40		2.4	S-4	13 17 24				@ 40': SILTSTONE/CLAYSTONE, light brown, fine-grained sand, dry, very dense, no hydrocarbon staining or odor	
	45			R-0	4 5 6				@ 45': no recovery	
	50								**Total Depth = 46.5 Feet** **Groundwater was not encountered at time of drilling** **Backfilled with methane probe SP-3 installed at depths of 40, 30, and 25 feet bgs on 3/20/13**	
	55									
	60									

SAMPLE TYPES:

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

TYPE OF TESTS:

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

GEOTECHNICAL BORING LOG HSA-4

Project No.
Project
Drilling Co. J& H Drilling
Drilling Method Hollow Stem Auger - 140lb - Autohammer
Location Los Angeles, California

Date Drilled 3-20-13
Logged By BCP
Hole Diameter 8"
Ground Elevation ~15 foot above street'
Sampled By BCP

Elevation Feet	Depth Feet	Graphic Log N S	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0							SM	**Undocumented Artificial Fill (Afu)** @ Surface: silty SAND: brown, fine-grained sand with gravel, dry, no hydrocarbon staining or odor	
	5		0.3	LB-6-5'	9 19 20				**Puente Formation Bedrock (Tp)** @ 5': SILTSTONE/CLAYSTONE, brown, some silt, fine-grained sand, dry, dense, no hydrocarbon odor or staining	
	10		0.6	LB-6-10'	10 20 35				@10': SILTSTONE/CLAYSTONE, brown, fine-grained sand with clay, dry, dense, minor hydrocarbon staining and odor	
	15		4.4	R-1 LB-6-15'	20 26 36				@ 15': SILTSTONE/CLAYSTONE, grey, fine-grained sand with clay, dry, dense, minor hydrocarbon staining and odor	
	20		0.3	S-1	6 13 26				@ 20': SILTSTONE/CLAYSTONE, light brown, some fine-grained sand with clay, dry, dense, minor hydrocarbon staining and odor	
	25		0.6	R-2 LB-6-25'	12 16 30				@ 25': SILTSTONE/CLAYSTONE with interbedded sand, light brown to grey, fine-grained sand, dry to slightly moist, dense, no hydrocarbon staining or odor, thinnly bedded, oxidized	
	30									

SAMPLE TYPES:

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

TYPE OF TESTS:

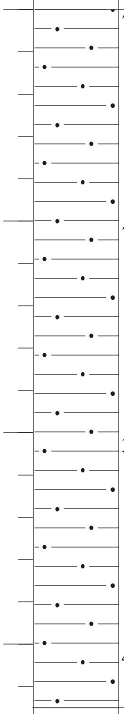
-200	% FINES PASSING
AL	ATTERBERG LIMITS
CN	CONSOLIDATION
CO	COLLAPSE
CR	CORROSION
CU	UNDRAINED TRIAXIAL
DS	DIRECT SHEAR
EI	EXPANSION INDEX
H	HYDROMETER
MD	MAXIMUM DENSITY
PP	POCKET PENETROMETER
RV	R VALUE
SA	SIEVE ANALYSIS
SE	SAND EQUIVALENT
SG	SPECIFIC GRAVITY
UC	UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG HSA-4

Project No.		**Date Drilled**	3-20-13
Project		**Logged By**	BCP
Drilling Co.	J& H Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	~15 foot above street'
Location	Los Angeles, California	**Sampled By**	BCP

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.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	30		7.0	S-2	13 15 18				@ 30': SILTSTONE/CLAYSTONE, light brown to grey, fine-grained sand, interbedded gypsum, dry, dense, minor hydrocarbon staining or odor	
	35		7.4	R-3 LB-6-35'	18 20 31				@ 35': SILTSTONE/CLAYSTONE, grey, interbedded fine-grained sand with clay, interbedded gypsum, dry, dense, minor hydrocarbon staining or odor, oxidized	
	40		3.4	S-4	16 16 24				@ 40': SILTSTONE/CLAYSTONE, grey, fine-grained sand with clay, interbedded gypsum, dry, dense, minor hydrocarbon staining or odor	
	45		4.7	R-4	18 20 19				@ 45': SILTSTONE/CLAYSTONE, grey, trace fine-grained sand, dry, hard, no hydrocarbon staining or odor	
	50								**Total Depth = 46.5 Feet** **Groundwater was not encountered at time of drilling** **Backfilled with bentonite grout on 3/20/13**	
	55									
	60									

SAMPLE TYPES:

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

TYPE OF TESTS:

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

Leighton

PARTICLE-SIZE ANALYSIS OF SOILS

ASTM D 422

Project Name:

Project No. :

Tested By : G. Bathala Date: 04/03/13

Data Input By: J. Ward Date: 04/09/13

Exploration No.: LB-3

Sample No.: B-1

Depth (feet) : 0-5

Soil Identification: Yellowish brown fat clay (CH)

% Gravel	**0**	**Soil Type**
% Sand	**8**	**CH**
% Fines	**92**	

			Moisture Content & Dry Weight of Air-Dry Soil Passing #4	Moisture Content of Oven-Dry Soil Passing #10	After Hydrometer & Wet Sieve ret. in #200 Sieve
Specific Gravity (Assumed)	2.70	Wt.of Air-Dry Soil + Cont.(g)	1248.80	121.01	
Correction for Specific Gravity	0.99	Dry Wt. of Soil + Cont. (g)	1192.03	120.29	80.22
Wt.of Air-Dry Soil + Cont. (g)	9704.40	Wt. of Container No.___ (g)	249.48	65.33	76.53
Wt. of Container	0.00	Moisture Content (%)	6.02	1.31	
Dry Wt. of Soil (g)	9704.40	Wt. of Dry Soil (g)	942.55		3.69

Coarse Sieve

U.S. Sieve	Cumulative Wt. Of Dry Soil Retained (g)	% Passing
6"	0.00	100.0
3"	0.00	100.0
1½"	0.00	100.0
3/4"	0.00	100.0
3/8"	14.50	99.9
No. 4	26.60	99.7
No. 10	3.00	99.4
Pan		

1st sample split

2nd sample split

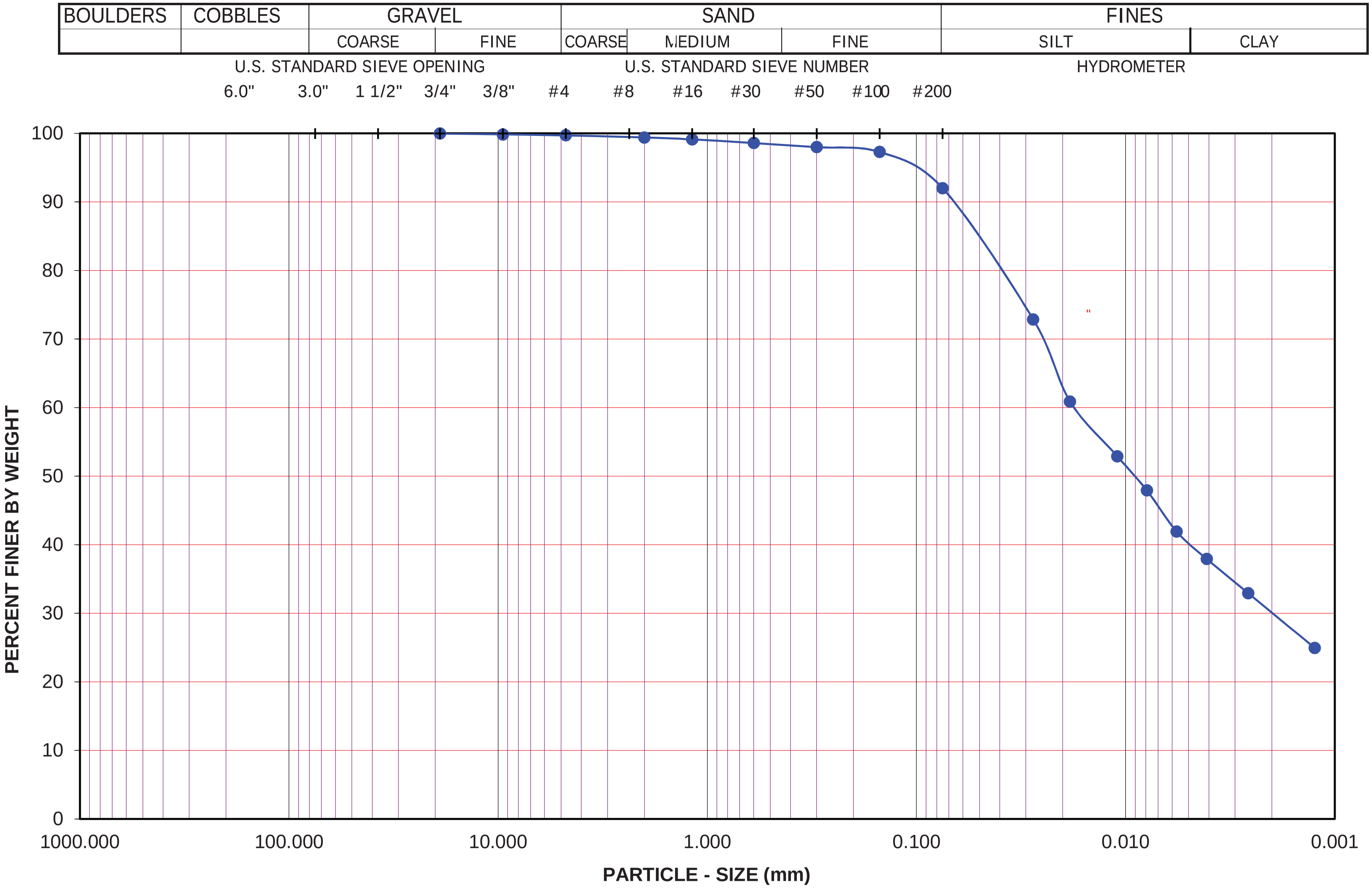
Sieve after Hydrometer & Wet Sieve

U.S. Sieve Size	Cumulative Wt. Of Dry Soil Retained (g)	% Passing	% Total Sample
No. 10	0.00	100.0	99.4
No. 16	0.14	99.7	99.1
No. 30	0.40	99.2	98.6
No. 50	0.70	98.6	98.0
No. 100	1.05	97.9	97.3
No. 200	3.67	92.6	92.0
Pan			

Hydrometer Wt. of Air-Dry Soil (g) 50.03 Wt. of Dry Soil (g) 49.38

Deflocculant 125 cc of 4% Solution

Date	Time	Elapsed Time (min)	Water Temperature (°C)	Composite Correction 152H	Actual Hydrometer Readings	% Total Sample (%)	Soil Particle Diameter (mm)
04-Apr-13	9:30	0		7.5			
	9:32	2	23.4	7.5	44.0	72.9	0.0277
	9:35	5	23.2	7.5	38.0	60.9	0.0184
	9:45	15	23.1	7.5	34.0	52.9	0.0110
	10:00	30	23.1	7.5	31.5	47.9	0.0079
	10:30	60	23.0	7.5	28.5	41.9	0.0057
	11:30	120	23.1	7.5	26.5	37.9	0.0041
	14:40	310	23.1	7.5	24.0	32.9	0.0026
05-Apr-13	9:30	1440	22.6	7.5	20.0	25.0	0.0012



Project Name:

Project No.:

Leighton

PARTICLE - SIZE
DISTRIBUTION
ASTM D 422

Exploration No.: LB-3

Depth (feet) : 0-5

Soil Identification: Yellowish brown fat clay (CH)

Sample No.: B-1

Soil Type : CH

GR:SA:FI : (%) 0 : 8 : 92

Apr-13

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name:		Tested By :	G. Berdy	Date:	04/02/13
Project No.:		Input By :	J. Ward	Date:	04/09/13
Boring No.:	LB-3	Depth (ft.)	0-5		
Sample No. :	B-1				
Soil Identification:	Yellowish brown fat clay (CH)				

Preparation Method:

☒ Moist
☐ Dry

☒ Mechanical Ram
☐ Manual Ram

Mold Volume (ft³) 0.03340

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

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3684.0	3770.0	3798.0	3784.0		
Weight of Mold (g)	1874.0	1874.0	1874.0	1874.0		
Net Weight of Soil (g)	1810.0	1896.0	1924.0	1910.0		
Wet Weight of Soil + Cont. (g)	473.40	478.30	443.20	464.70		
Dry Weight of Soil + Cont. (g)	440.10	434.10	395.30	405.90		
Weight of Container (g)	50.70	51.40	51.80	51.10		
Moisture Content (%)	8.55	11.55	13.94	16.57		
Wet Density (pcf)	119.5	125.1	127.0	126.1		
Dry Density (pcf)	110.1	112.2	111.5	108.1		

Maximum Dry Density (pcf) 112.0 **Optimum Moisture Content (%)** 12.0

PROCEDURE USED

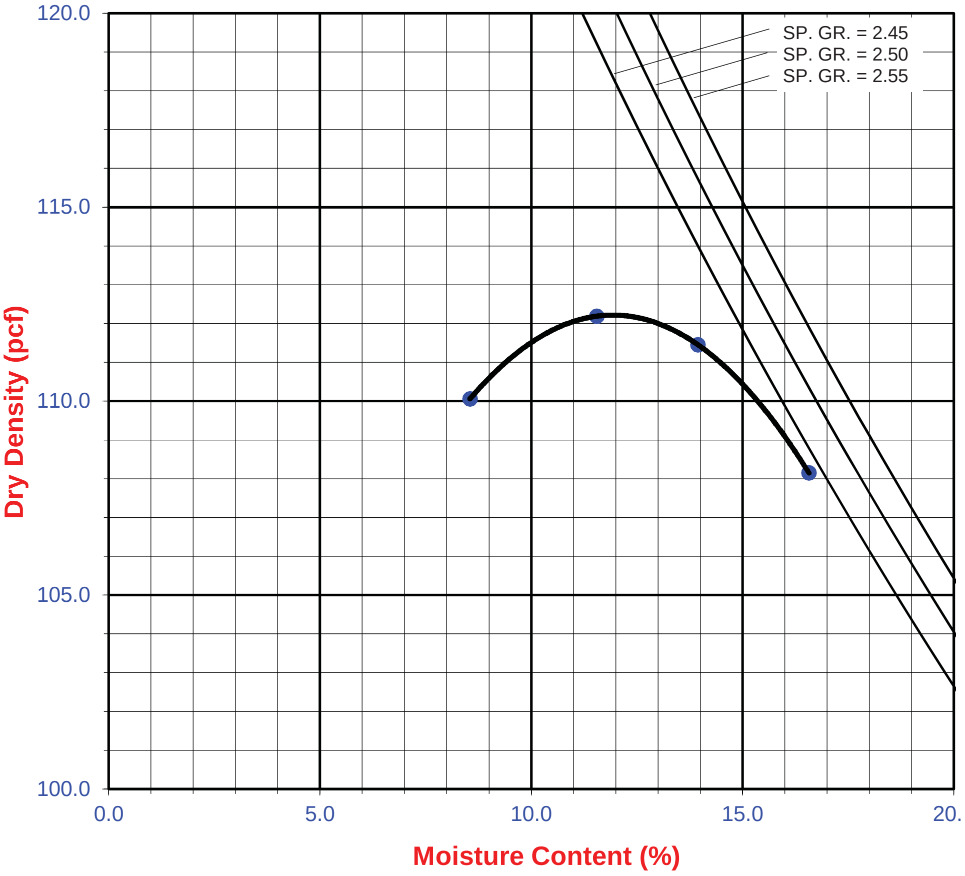
☒ **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

☐ **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:
0:8:92
GR:SA:FI

Atterberg Limits:
58:22:36
LL,PL,PI



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ATTERBERG LIMITS

ASTM D 4318

Project Name: ____ Tested By: G. Bathala Date: 04/04/13

Project No. : ____ Input By: J. Ward Date: 04/09/13

Boring No.: LB-3 Checked By: J. Ward

Sample No.: B-1 Depth (ft.) 0-5

Soil Identification: Yellowish brown fat clay (CH)

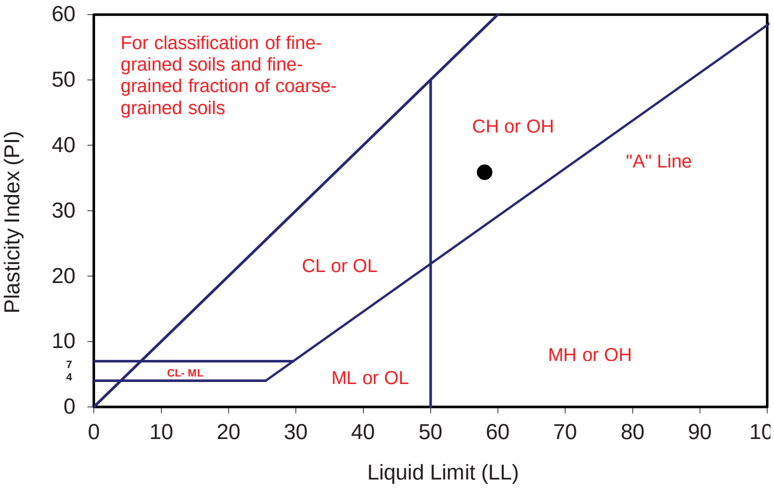
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			32	26	17	
Wet Wt. of Soil + Cont. (g)	26.27	26.17	26.92	28.82	29.26	
Dry Wt. of Soil + Cont. (g)	23.97	23.87	22.07	23.21	23.38	
Wt. of Container (g)	13.50	13.52	13.53	13.50	13.52	
Moisture Content (%) [Wn]	21.97	22.22	56.79	57.78	59.63	

Liquid Limit	**58**
Plastic Limit	**22**
Plasticity Index	**36**
Classification	**CH**

PI at "A" - Line = 0.73(LL-20) 27.74

One - Point Liquid Limit Calculation

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



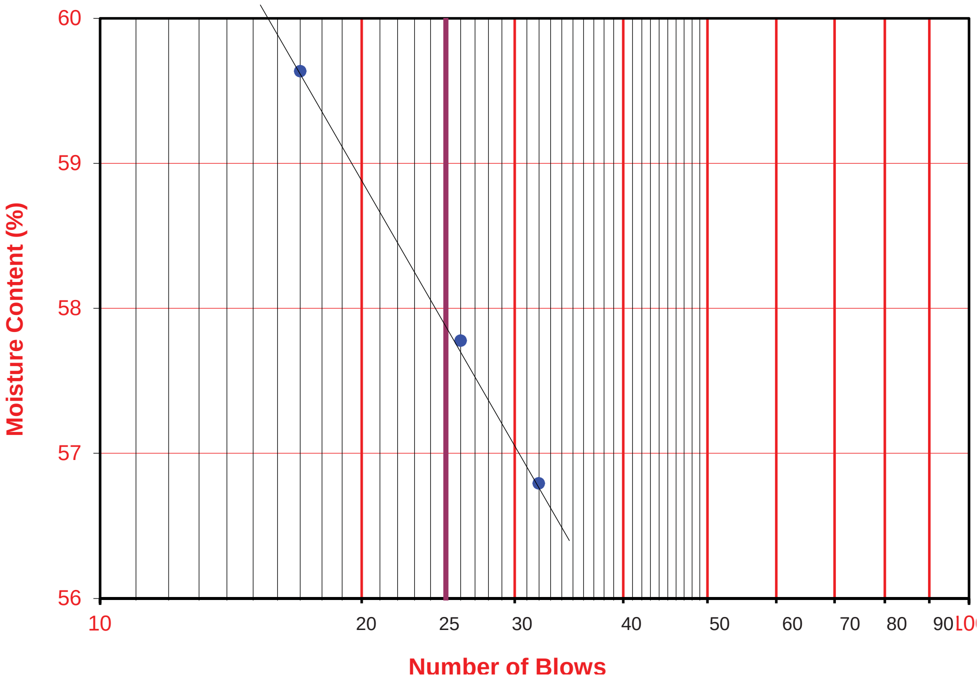
PROCEDURES USED

☐ Wet Preparation
Multipoint - Wet

☒ Dry Preparation
Multipoint - Dry

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test



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EXPANSION INDEX of SOILS

ASTM D 4829

Project Name:
Project No. :
Boring No.: LB-3
Sample No. : B-1
Soil Identification: Yellowish brown fat clay (CH)

Tested By: S. Felter Date: 04/03/13
Checked By: J. Ward Date: 04/09/13
Depth (ft.) 0-5

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	1.0935
Wt. Comp. Soil + Mold (g)	529.20	427.41
Wt. of Mold (g)	163.10	0.00
Specific Gravity (Assumed)	2.70	2.70
Container No.	O	O
Wet Wt. of Soil + Cont. (g)	723.80	590.51
Dry Wt. of Soil + Cont. (g)	634.40	483.96
Wt. of Container (g)	0.00	163.10
Moisture Content (%)	14.09	33.21
Wet Density (pcf)	110.4	117.9
Dry Density (pcf)	96.8	88.5
Void Ratio	0.742	0.905
Total Porosity	0.426	0.475
Pore Volume (cc)	88.1	107.5
Degree of Saturation (%) [S $_{meas}$]	**51.3**	99.1

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.)
04/03/13	14:07	1.0	0	0.1265
04/03/13	14:17	1.0	10	0.1265
Add Distilled Water to the Specimen				
04/03/13	15:15	1.0	58	0.1630
04/04/13	6:40	1.0	983	0.2200
04/04/13	8:11	1.0	1074	0.2200

Expansion Index (EI $_{meas}$) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	**94**

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R-VALUE TEST RESULTS

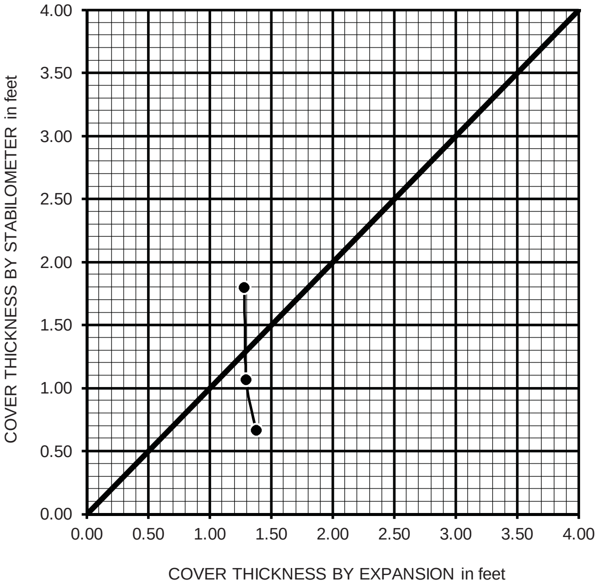
DOT CA Test 301

PROJECT NAME:
PROJECT NUMBER: 10116.001
BORING NUMBER:
DEPTH (FT.): 0-5
SAMPLE NUMBER: B-1
TECHNICIAN: S. Felter
SAMPLE DESCRIPTION: Yellowish brown CH
DATE COMPLETED: 4/4/2013

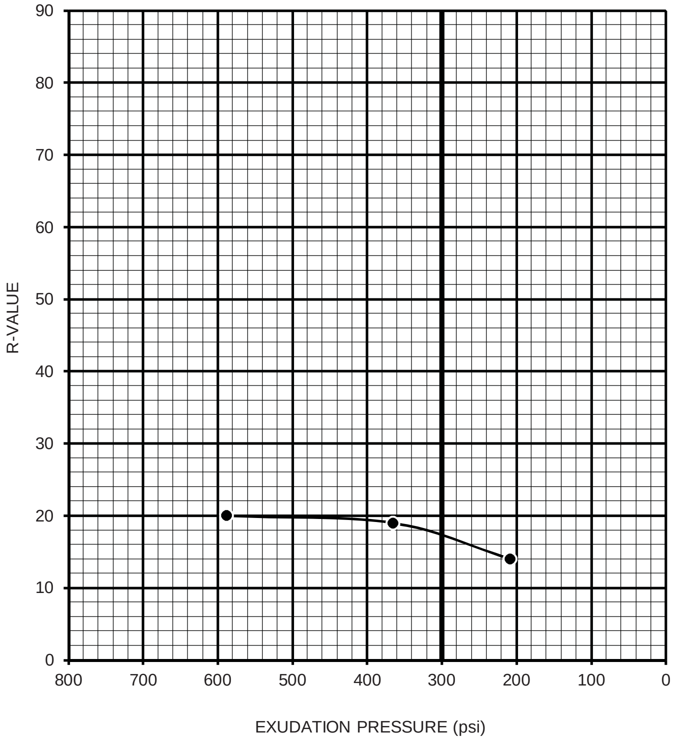
TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	21.2	22.2	22.7
HEIGHT OF SAMPLE, Inches	2.49	2.68	2.62
DRY DENSITY, pcf	107.8	104.3	104.4
COMPACTOR PRESSURE, psi	75	50	50
EXUDATION PRESSURE, psi	588	365	208
EXPANSION, Inches x 10exp-4	54	32	20
STABILITY Ph 2,000 lbs (160 psi)	120	126	132
TURNS DISPLACEMENT	3.24	3.31	3.59
R-VALUE UNCORRECTED	20	17	13
R-VALUE CORRECTED	20	19	14

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.	1.28	1.30	1.38
EXPANSION PRESSURE THICKNESS, ft.	1.80	1.07	0.67

EXPANSION PRESSURE CHART



EXUDATION PRESSURE CHART



R-VALUE BY EXPANSION: 19
R-VALUE BY EXUDATION: 17
EQUILIBRIUM R-VALUE: 17

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TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS

Project Name:

Project No. :

Tested By : GEB/ACS Date: 04/05/13

Data Input By: J. Ward Date: 04/09/13

Boring No.	LB-3			
Sample No.	R-2			
Sample Depth (ft)	15.0			
Soil Identification:	Yellowish olive ML			
Wet Weight of Soil + Container (g)	189.50			
Dry Weight of Soil + Container (g)	187.60			
Weight of Container (g)	58.26			
Moisture Content (%)	1.47			
Weight of Soaked Soil (g)	100.09			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	51			
Crucible No.	2, 31			
Furnace Temperature (°C)	840			
Time In / Time Out	10:25/11:10			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	36.2896			
Wt. of Crucible (g)	36.2554			
Wt. of Residue (g) (A)	0.0342			
PPM of Sulfate (A) x 41150	1407.33			
PPM of Sulfate, Dry Weight Basis	**1428**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	30			
ml of AgNO3 Soln. Used in Titration (C)	0.4			
PPM of Chloride (C -0.2) * 100 * 30 / B	20			
PPM of Chloride, Dry Wt. Basis	**20**			

pH TEST, DOT California Test 532/643

pH Value	**7.67**			
Temperature °C	22.1			

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SOIL RESISTIVITY TEST

DOT CA TEST 532 / 643

Project Name: ____

Project No. : ____

Boring No.: LB-3

Sample No. : R-2

Soil Identification:* Yellowish olive ML

Tested By : GEB/ACS Date: 04/09/13

Data Input By: J. Ward Date: 04/09/13

Depth (ft.) : 15.0

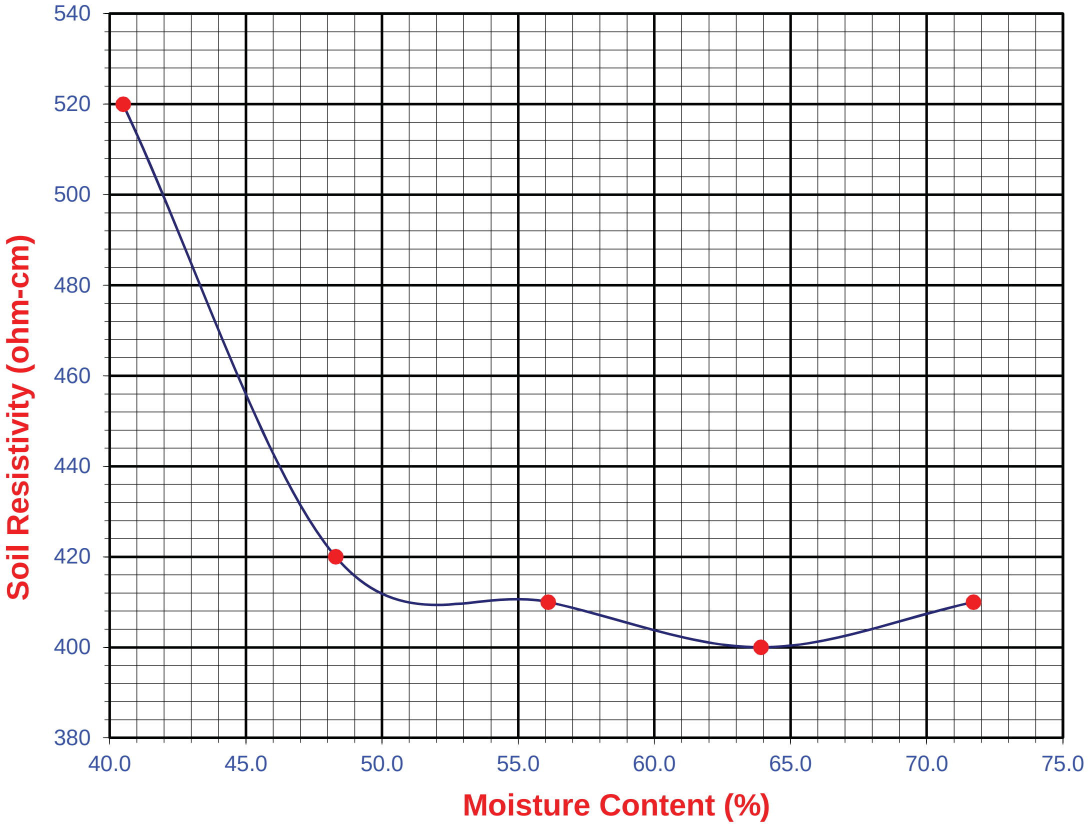
*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	50	40.50	520	520
2	60	48.30	420	420
3	70	56.11	410	410
4	80	63.91	400	400
5	90	71.72	410	410

Moisture Content (%) (MCi)	1.47
Wet Wt. of Soil + Cont. (g)	189.50
Dry Wt. of Soil + Cont. (g)	187.60
Wt. of Container (g)	58.26
Container No.	
Initial Soil Wt. (g) (Wt)	130.00
Box Constant	1.000

MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 532 / 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 532 / 643	
400	**63.9**	**1428**	**20**	**7.67**	**22.1**



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UNCONFINED COMPRESSIVE STRENGTH of COHESIVE SOIL
ASTM D 2166

Project Name:
Project No.:
Boring No.: LB-3
Sample No.: R-2
Sample Description: Yellowish olive silt (ML)

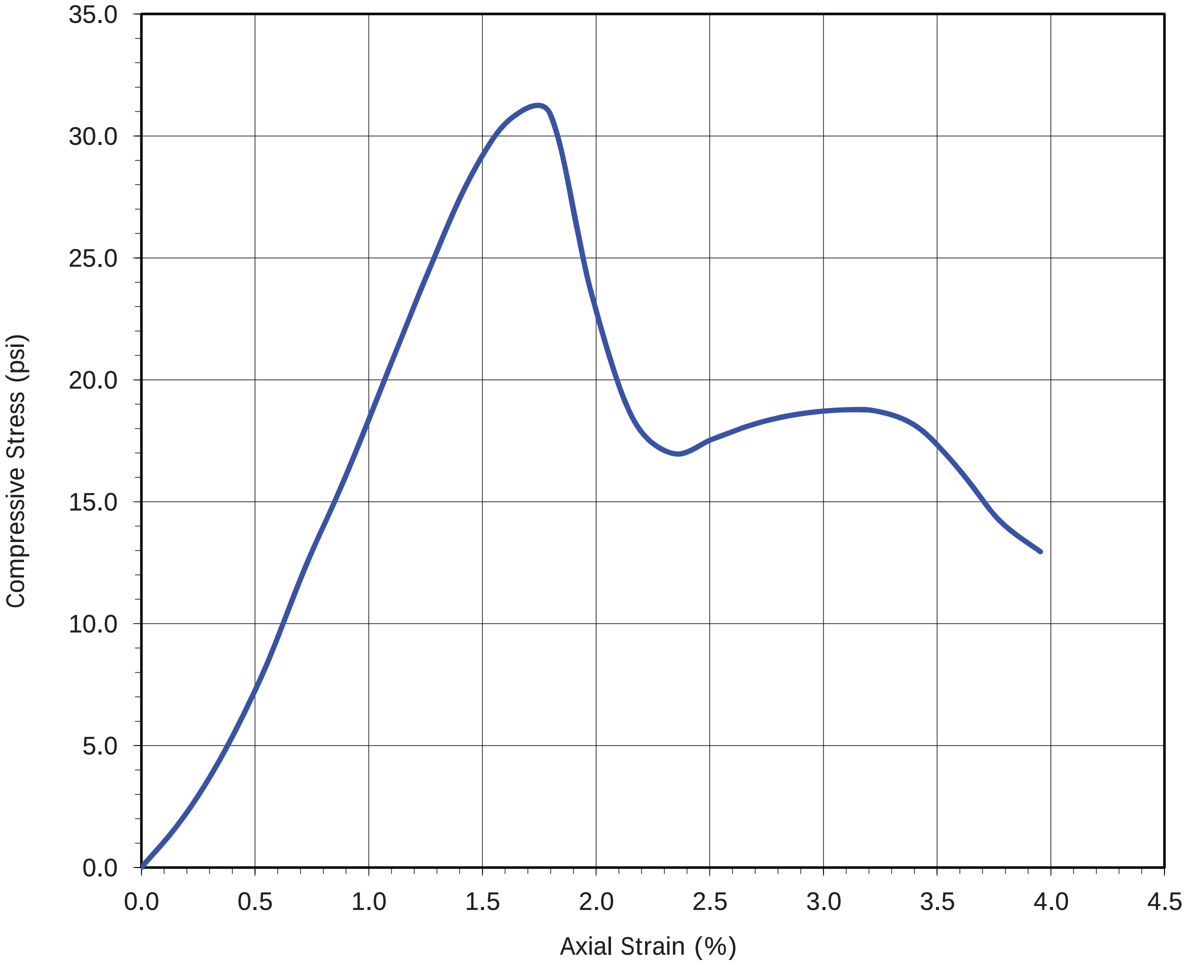
Tested by: A. Santos   Date: 04/04/13
Checked by: J. Ward   Date: 04/09/13
Sample Type: Ring
Depth (ft): 15.0

Weight of Sample + Tube / Rings (g)	798.80
Weight of Tube / Rings (g)	0.00
Wet Weight of Soil + Container (g)	906.60
Dry Weight of Soil + Container (g)	735.10
Weight of Container (g)	109.70
Load Surcharge (lb)	2.20
Rate of Deformation (in/min)	0.045
Specific Gravity (Assumed)	2.70

Sample Measurements	
Diameter (in)	2.420
	2.425
	2.425
Area (sq.in.)	4.612
Height (in)	5.562
	5.564
	5.565

Axial Deformation (in.)	Load (lb.)	Compressive Stress (psi)	Axial Strain (%)
0.0000	0.0	0.00	0.000
0.0100	7.0	1.99	0.180
0.0200	19.4	4.67	0.359
0.0300	35.2	8.07	0.539
0.0400	54.8	12.27	0.719
0.0500	72.5	16.05	0.899
0.0600	92.0	20.20	1.078
0.0700	111.5	24.34	1.258
0.0800	129.5	28.14	1.438
0.0900	141.5	30.65	1.618
0.1000	143.0	30.92	1.797
0.1100	109.0	23.63	1.977
0.1200	85.0	18.50	2.157
0.1300	78.0	16.98	2.337
0.1400	81.0	17.58	2.516
0.1500	84.0	18.19	2.696
0.1600	86.0	18.57	2.876
0.1700	87.0	18.75	3.056
0.1800	87.0	18.71	3.235
0.1900	84.0	18.05	3.415
0.2000	76.0	16.35	3.595
0.2100	66.0	14.23	3.774
0.2200	60.0	12.95	3.954

Axial Deformation (in.)	Load (lb.)	Compressive Stress (psi)	Axial Strain (%)



Boring No.:	LB-3
Sample No.:	R-2
Depth (ft):	15.0
Soil Type:	Ring
Sample Description:	Yellowish olive silt (ML)

Sample Diameter (in.)	2.423
Sample Height (in.)	5.564
Initial Moisture Content (%)	27.42
Dry Density (pcf)	93.1
Specific Gravity (assumed)	2.7
Saturation (%)	91.4
Rate of Deformation (in/min)	0.0450
Height / Diameter Ratio	2.30

At Failure

Compressive Strength (psi)	**30.92**
Axial Strain (%)	1.80

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Unconfined Compressive Strength
of Cohesive Soil
ASTM D 2166

Project I

04/04/13

Leighton

DIRECT SHEAR TEST

Consolidated Undrained

Project Name:

Project No.:

Boring No.: LB-6

Sample No.: R-1

Soil Identification: Olive silty clay'stone' (CL-ML)

Tested By: G. Bathala

Checked By: J. Ward

Sample Type: Ring

Depth (ft.): 15.0

Date: 04/03/13

Date: 04/09/13

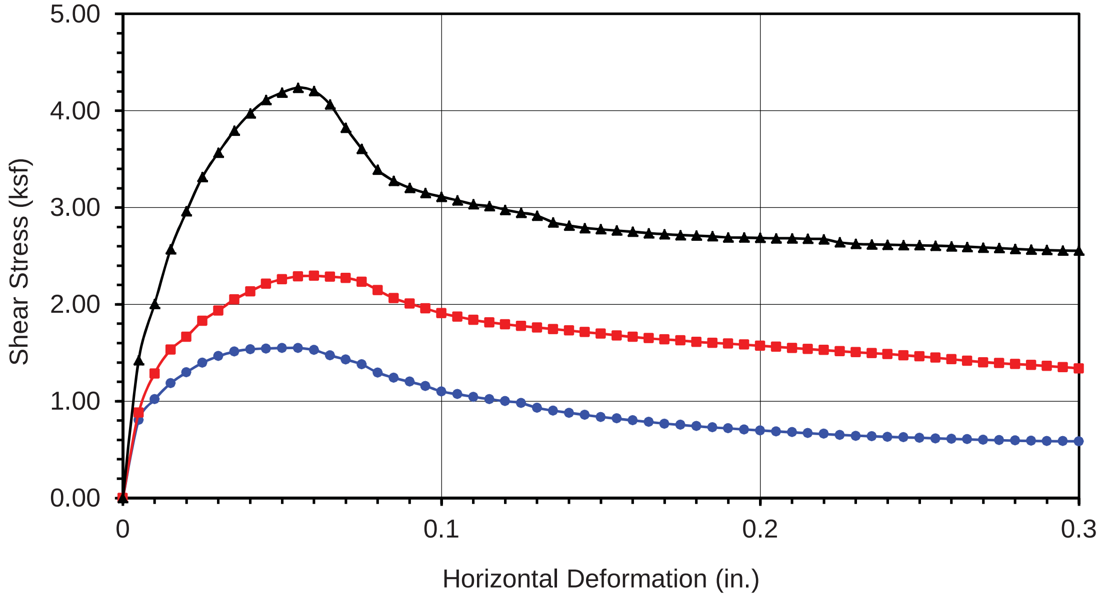
Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	176.96	182.64	186.90
Weight of Ring(gm):	42.86	43.77	43.42

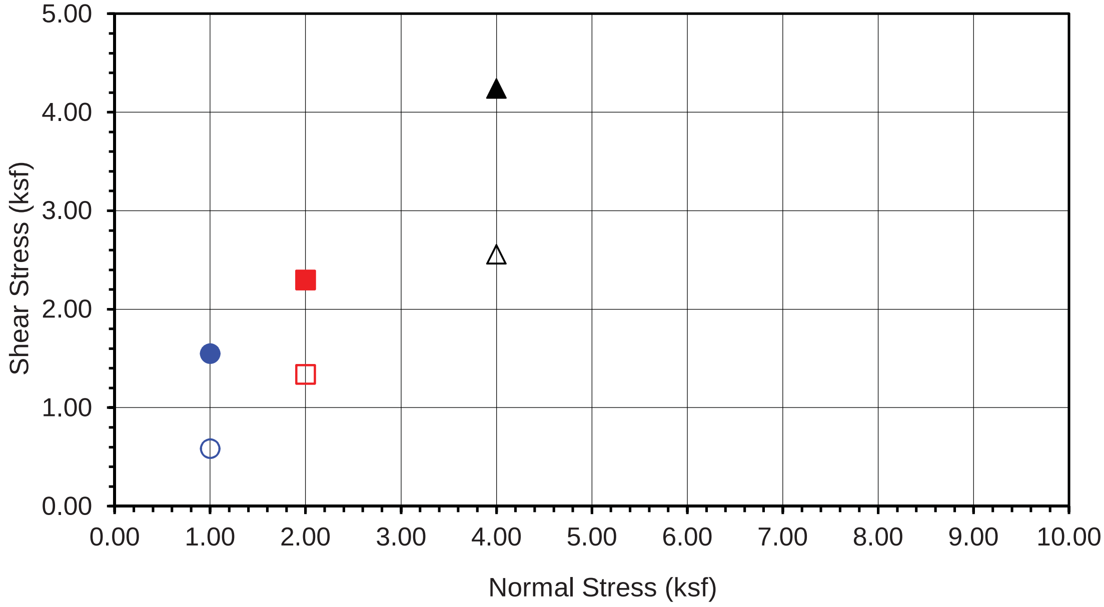
Before Shearing

Weight of Wet Sample+Cont.(gm):	209.30	209.30	209.30
Weight of Dry Sample+Cont.(gm):	168.09	168.09	168.09
Weight of Container(gm):	39.07	39.07	39.07
Vertical Rdg.(in): Initial	0.2595	0.2617	0.0000
Vertical Rdg.(in): Final	0.2599	0.2680	-0.0150

After Shearing

Weight of Wet Sample+Cont.(gm):	175.52	177.64	182.85
Weight of Dry Sample+Cont.(gm):	135.36	139.14	148.47
Weight of Container(gm):	39.29	37.31	38.44
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-6**
Sample No.	**R-1**
Depth (ft)	**15**

Sample Type:

Ring

Soil Identification:
Olive silty clay'stone' (CL-ML)

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 1.550	■ 2.295	▲ 4.238
Shear Stress @ End of Test (ksf)	○ 0.585	□ 1.339	△ 2.556
Deformation Rate (in./min.)	0.0500	0.0500	0.0500
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	31.94	31.94	31.94
Dry Density (pcf)	84.5	87.5	90.4
Saturation (%)	86.7	93.2	99.8
Soil Height Before Shearing (in.)	0.9996	0.9937	0.9850
Final Moisture Content (%)	41.8	37.8	31.2

Leighton

DIRECT SHEAR TEST RESULTS

Consolidated Undrained

Project No.:

04-13

APPENDIX D
SEISMIC DESIGN PARAMETERS AND SITE-SPECIFIC GROUND MOTION STUDY DATA

USGS Design Maps Detailed Report

ASCE 7-10 Standard (34.06439°N, 118.25813°W)

Site Class C – "Very Dense Soil and Soft Rock", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1] $S_S = 2.514$ g

From Figure 22-2 [2] $S_1 = 0.887$ g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class C, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index $PI > 20$, • Moisture content $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk–Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4–1: Site Coefficient F_a

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = C and S_S = 2.514 g, F_a = 1.000

Table 11.4–2: Site Coefficient F_v

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at 1–s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = C and S_1 = 0.887 g, F_v = 1.300

Equation (11.4–1): $S_{MS} = F_aS_S = 1.000 \times 2.514 = 2.514$ g

Equation (11.4–2): $S_{M1} = F_vS_1 = 1.300 \times 0.887 = 1.153$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

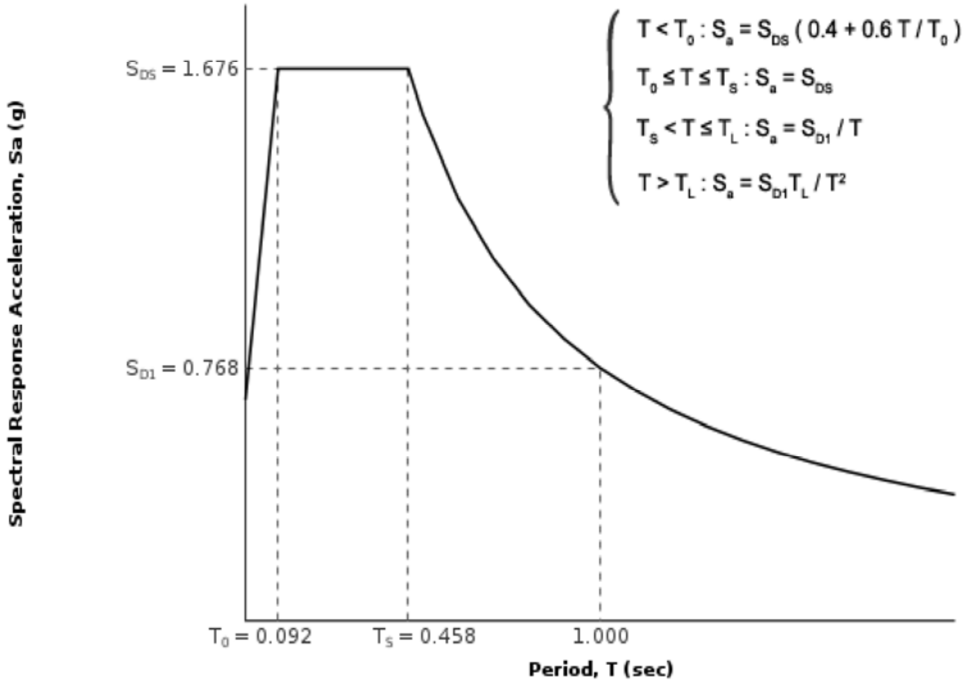
Equation (11.4–3): $S_{DS} = ⅔ S_{MS} = ⅔ \times 2.514 = 1.676$ g

Equation (11.4–4): $S_{D1} = ⅔ S_{M1} = ⅔ \times 1.153 = 0.768$ g

Section 11.4.5 — Design Response Spectrum

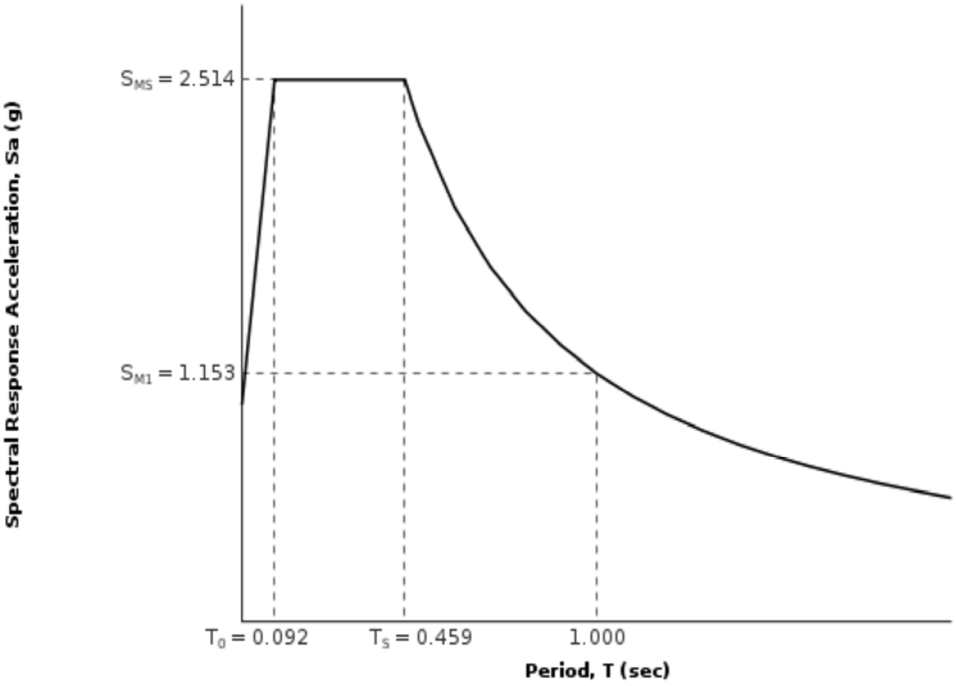
From Figure 22-12 [3] $T_L = 8$ seconds

Figure 11.4–1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4]	PGA = 0.953
Equation (11.8–1):	$PGA_M = F_{PGA} PGA = 1.000 \times 0.953 = 0.953$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = C and PGA = 0.953 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5]	C_{RS} = 0.943
From Figure 22-18 [6]	C_{R1} = 0.955

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	**RISK CATEGORY**		
	I or II	**III**	**IV**
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 1.676 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	**RISK CATEGORY**		
	I or II	**III**	**IV**
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.768 g, Seismic Design Category = D

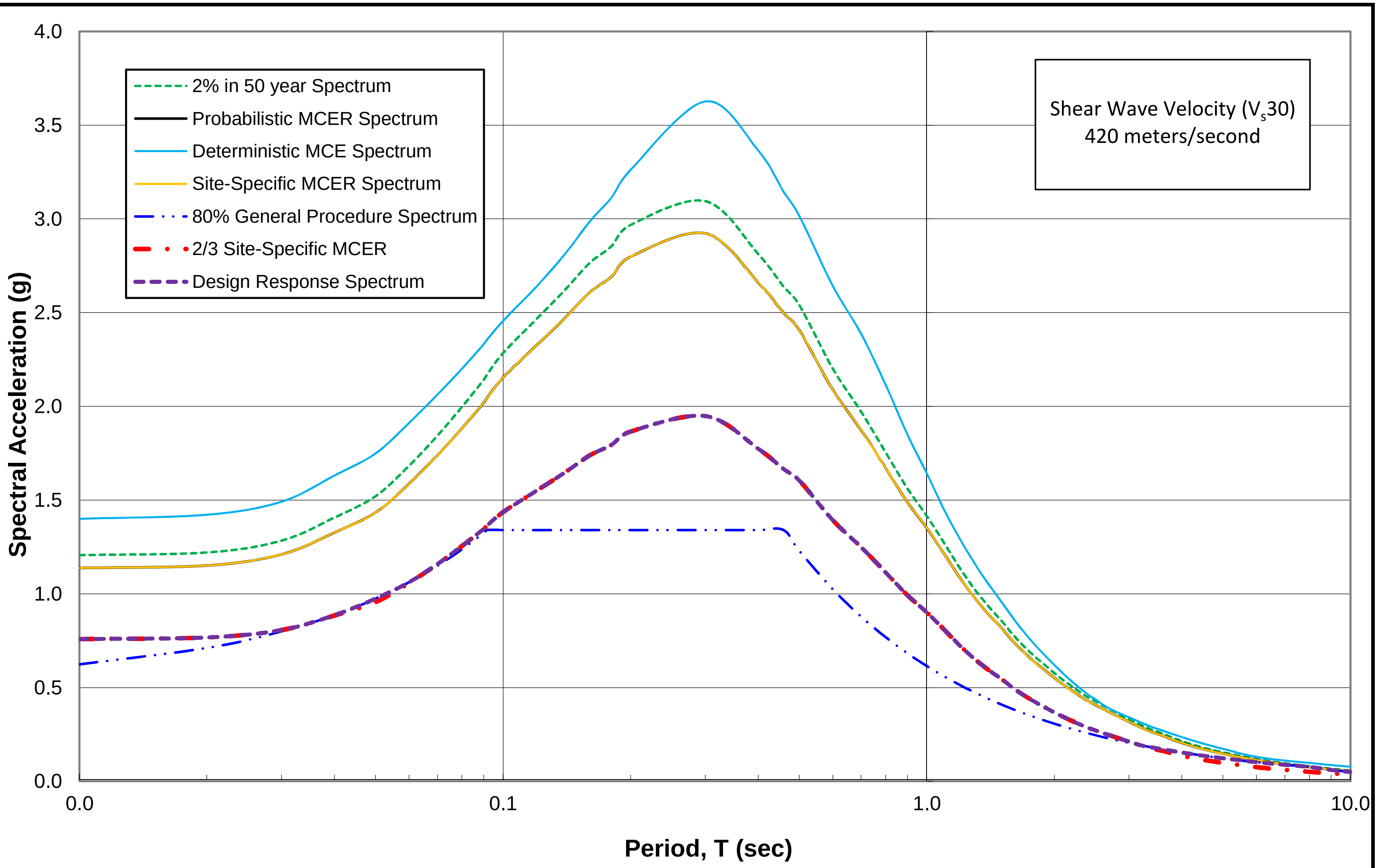
Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = E

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

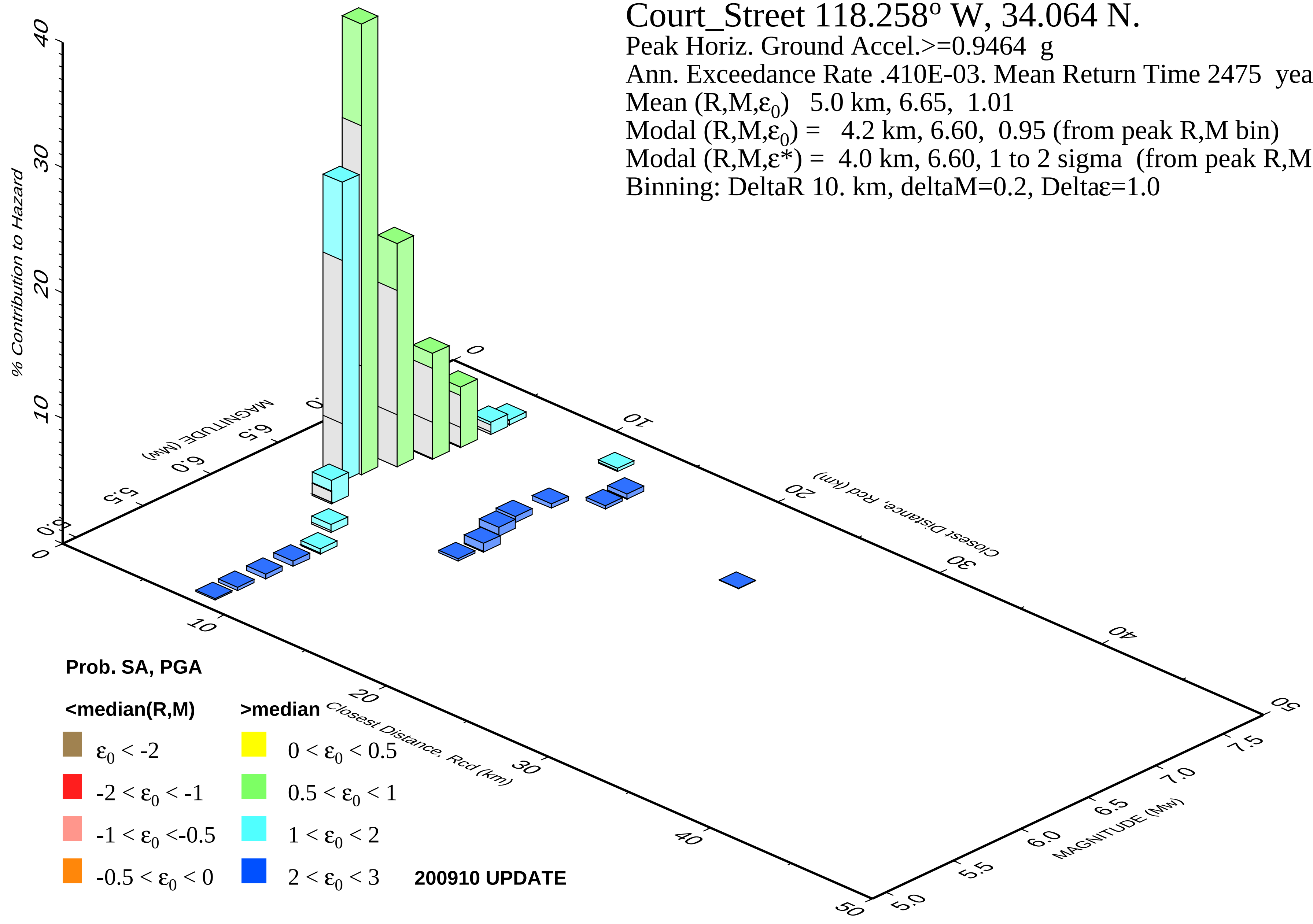


SITE-SPECIFIC RESPONSE SPECTRUM
1346, 1350 AND 1352 WEST COURT STREET
CITY OF LOS ANGELES, CALIFORNIA

Project Name: COURT STREET
Project No.: 11388.001
Designed/Checked by: SP
Date: 08/2016

Figure D-1

PSH Deaggregation on NEHRP BC rock
Court_Street 118.258° W, 34.064 N.
Peak Horiz. Ground Accel.>=0.9464 g
Ann. Exceedance Rate .410E-03. Mean Return Time 2475 years
Mean (R,M,ε_0) 5.0 km, 6.65, 1.01
Modal (R,M,ε_0) = 4.2 km, 6.60, 0.95 (from peak R,M bin)
Modal (R,M,ε^*) = 4.0 km, 6.60, 1 to 2 sigma (from peak R,M,ε bin)
Binning: DeltaR 10. km, deltaM=0.2, Deltaε=1.0



GMT 2016 Jul 13 15:13:37 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on rock with average vs= 760. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

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                           *************************
                           *                       *
                           *    E Q S E A R C H    *
                           *                       *
                           *     Version 3.00      *
                           *                       *
                           *************************

                                 ESTIMATION OF
                            PEAK ACCELERATION FROM
                        CALIFORNIA EARTHQUAKE CATALOGS


JOB NUMBER: 11388.001
                                                     DATE: 07-13-2016

JOB NAME: Court Street

EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT


MAGNITUDE RANGE:
   MINIMUM MAGNITUDE:  4.00
   MAXIMUM MAGNITUDE:  9.00

SITE COORDINATES:
   SITE LATITUDE:  34.0644
   SITE LONGITUDE:  118.2581

SEARCH DATES:
           START DATE:   1800
           END DATE:   2016

SEARCH RADIUS:
           62.0 mi
           99.8 km

ATTENUATION RELATION:  25) Campbell & Bozorgnia (1997 Rev.) - Soft Rock
   UNCERTAINTY (M=Median, S=Sigma): M       Number of Sigmas:  0.0
   ASSUMED SOURCE TYPE:  DS [SS=Strike-slip, DS=Reverse-slip, BT=Blind-thrust]
   SCOND:   0  Depth Source:  A
   Basement Depth:  5.00 km     Campbell SSR:  1     Campbell SHR:  0
   COMPUTE PEAK HORIZONTAL ACCELERATION

MINIMUM DEPTH VALUE (km):  3.0
```

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                     -------------------------
                     EARTHQUAKE SEARCH RESULTS
                     -------------------------

Page  1
-------------------------------------------------------------------------------
    |       |        |          |  TIME   |     |     | SITE  |SITE|  APPROX.
FILE|  LAT. |  LONG. |   DATE   |  (UTC)  |DEPTH|QUAKE|  ACC. | MM |  DISTANCE
CODE| NORTH |  WEST  |          | H M Sec | (km)| MAG.|   g   |INT.|  mi  [km]
----+-------+--------+----------+---------+-----+-----+-------+----+------------
MGI |34.0800|118.2600|07/16/1920|18 8 0.0|  0.0| 5.00| 0.272 | IX |  1.1(  1.7)
MGI |34.1000|118.3000|07/16/1920|2022 0.0|  0.0| 4.60| 0.156 |VIII|  3.4(  5.5)
MGI |34.1000|118.3000|07/16/1920|2127 0.0|  0.0| 4.60| 0.156 |VIII|  3.4(  5.5)
MGI |34.1000|118.3000|07/16/1920|2130 0.0|  0.0| 4.60| 0.156 |VIII|  3.4(  5.5)
MGI |34.1000|118.3000|07/26/1920|1215 0.0|  0.0| 4.00| 0.099 | VII|  3.4(  5.5)
MGI |34.1000|118.2000|04/21/1921|1538 0.0|  0.0| 4.00| 0.088 | VII|  4.1(  6.6)
MGI |34.1000|118.2000|05/02/1916|1432 0.0|  0.0| 4.00| 0.088 | VII|  4.1(  6.6)
MGI |34.1000|118.2000|01/27/1860| 830 0.0|  0.0| 4.30| 0.111 | VII|  4.1(  6.6)
T-A |34.0000|118.2500|05/02/1856| 810 0.0|  0.0| 4.30| 0.105 | VII|  4.5(  7.2)
T-A |34.0000|118.2500|01/10/1856| 0 0 0.0|  0.0| 5.00| 0.176 |VIII|  4.5(  7.2)
T-A |34.0000|118.2500|09/23/1827| 0 0 0.0|  0.0| 5.00| 0.176 |VIII|  4.5(  7.2)
T-A |34.0000|118.2500|03/26/1860| 0 0 0.0|  0.0| 5.00| 0.176 |VIII|  4.5(  7.2)
T-A |34.0000|118.2500|05/04/1857| 6 0 0.0|  0.0| 4.30| 0.105 | VII|  4.5(  7.2)
T-A |34.0000|118.2500|03/21/1880|1425 0.0|  0.0| 4.30| 0.105 | VII|  4.5(  7.2)
T-A |34.0000|118.2500|01/17/1857| 1 0 0.0|  0.0| 4.30| 0.105 | VII|  4.5(  7.2)
MGI |34.0000|118.3000|09/03/1905| 540 0.0|  0.0| 5.30| 0.211 |VIII|  5.0(  8.1)
MGI |34.0000|118.3000|06/22/1920|2035 0.0|  0.0| 4.00| 0.075 | VII|  5.0(  8.1)
MGI |34.0000|118.3000|06/30/1920| 350 0.0|  0.0| 4.00| 0.075 | VII|  5.0(  8.1)
GSP |34.0300|118.1800|06/12/1989|165718.4| 16.0| 4.40| 0.102 | VII|  5.1(  8.1)
GSP |34.0200|118.1800|06/12/1989|172225.5| 16.0| 4.10| 0.076 | VII|  5.4(  8.7)
MGI |34.0000|118.2000|06/26/1917| 424 0.0|  0.0| 4.00| 0.069 | VI |  5.5(  8.9)
MGI |34.0000|118.2000|06/26/1917|2130 0.0|  0.0| 4.60| 0.110 | VII|  5.5(  8.9)
MGI |34.0000|118.2000|02/13/1917|13 5 0.0|  0.0| 4.60| 0.110 | VII|  5.5(  8.9)
MGI |34.0000|118.2000|06/26/1917|2115 0.0|  0.0| 4.60| 0.110 | VII|  5.5(  8.9)
MGI |34.0000|118.2000|06/26/1917|2120 0.0|  0.0| 4.60| 0.110 | VII|  5.5(  8.9)
DMG |33.9830|118.3000|02/11/1940|192410.0|  0.0| 4.00| 0.063 | VI |  6.1(  9.8)
GSP |34.0590|118.3870|09/09/2001|235918.0|  4.0| 4.20| 0.060 | VI |  7.4( 11.9)
T-A |34.1700|118.1700|03/07/1888|1554 0.0|  0.0| 4.30| 0.052 | VI |  8.9( 14.3)
PAS |34.0600|118.1000|10/01/1987|1449 5.9| 11.7| 4.70| 0.069 | VI |  9.0( 14.6)
PAS |34.0490|118.1010|10/01/1987|144541.5| 13.6| 4.70| 0.069 | VI |  9.0( 14.6)
PAS |34.1490|118.1350|12/03/1988|113826.4| 13.3| 4.90| 0.080 | VII|  9.1( 14.7)
PAS |34.0730|118.0980|10/04/1987|105938.2|  8.2| 5.30| 0.111 | VII|  9.2( 14.8)
DMG |33.9390|118.2050|01/11/1950|214135.0|  0.4| 4.10| 0.043 | VI |  9.2( 14.8)
MGI |34.0000|118.4000|02/07/1927| 429 0.0|  0.0| 4.60| 0.062 | VI |  9.3( 14.9)
MGI |34.0000|118.4000|02/22/1920|1610 0.0|  0.0| 4.60| 0.062 | VI |  9.3( 14.9)
MGI |34.0000|118.4000|01/29/1927|2324 0.0|  0.0| 4.00| 0.039 |  V |  9.3( 14.9)
MGI |34.0000|118.4000|10/01/1930| 040 0.0|  0.0| 4.60| 0.062 | VI |  9.3( 14.9)
MGI |34.1000|118.1000|07/11/1855| 415 0.0|  0.0| 6.30| 0.236 | IX |  9.4( 15.1)
PAS |34.0760|118.0900|10/01/1987|1448 3.1| 11.7| 4.10| 0.040 |  V |  9.6( 15.5)
PAS |34.0520|118.0900|10/01/1987|151231.8| 10.8| 4.70| 0.064 | VI |  9.6( 15.5)
GSP |33.9380|118.3360|05/18/2009|033936.3| 13.0| 4.70| 0.063 | VI |  9.8( 15.8)
PAS |34.0500|118.0870|10/01/1987|155953.5| 10.4| 4.00| 0.036 |  V |  9.8( 15.8)
GSP |33.9220|118.2700|10/28/2001|162745.6| 21.0| 4.00| 0.036 |  V |  9.9( 15.9)
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DMG |34.0000|118.4170|12/07/1938| 338 0.0|  0.0| 4.00| 0.035 |  V | 10.1( 16.3)
PAS |34.0610|118.0790|10/01/1987|144220.0|  9.5| 5.90| 0.159 |VIII| 10.2( 16.5)
DMG |33.9500|118.1330|10/25/1933| 7 046.0|  0.0| 4.30| 0.041 |  V | 10.7( 17.1)
GSP |33.9920|118.0820|03/16/2010|110400.2| 18.0| 4.40| 0.041 |  V | 11.2( 18.1)
MGI |33.9000|118.2000|10/08/1927|1914 0.0|  0.0| 4.60| 0.045 | VI | 11.8( 19.0)
PAS |34.0770|118.0470|02/11/1988|152555.7| 12.5| 4.70| 0.047 | VI | 12.1( 19.5)
DMG |33.8830|118.3170|03/11/1933|1457 0.0|  0.0| 4.90| 0.050 | VI | 13.0( 20.9)
GSG |34.0958|118.4912|06/02/2014|023643.9|  4.4| 4.16| 0.026 |  V | 13.5( 21.7)
DMG |33.9670|118.0500|01/30/1941| 13446.9|  0.0| 4.10| 0.025 |  V | 13.7( 22.0)
DMG |33.8670|118.2170|06/19/1944| 0 333.0|  0.0| 4.50| 0.033 |  V | 13.8( 22.3)
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                        EARTHQUAKE SEARCH RESULTS
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    |       |        |          |  TIME  |     |     | SITE  |SITE|  APPROX.
FILE|  LAT. |  LONG. |   DATE   |  (UTC) |DEPTH|QUAKE|  ACC. | MM |  DISTANCE
CODE| NORTH |  WEST  |          | H M Sec| (km)| MAG.|   g   |INT.|  mi  [km]
----+-------+--------+----------+--------+-----+-----+-------+----+------------
DMG |33.8670|118.2170|06/19/1944| 3 6 7.0|  0.0| 4.40| 0.031 |  V | 13.8( 22.3)
GSG |34.1347|118.4858|03/17/2014|132536.9|  9.9| 4.39| 0.030 |  V | 13.9( 22.4)
DMG |33.8670|118.2000|11/13/1933|2128 0.0|  0.0| 4.00| 0.022 | IV | 14.0( 22.6)
DMG |33.9000|118.1000|07/08/1929|1646 6.7| 13.0| 4.70| 0.037 |  V | 14.5( 23.4)
DMG |34.0000|118.5000|11/08/1914|1140 0.0|  0.0| 4.50| 0.031 |  V | 14.5( 23.4)
MGI |34.0000|118.5000|11/19/1918|2018 0.0|  0.0| 5.00| 0.046 | VI | 14.5( 23.4)
DMG |34.0000|118.5000|08/04/1927|1224 0.0|  0.0| 5.00| 0.046 | VI | 14.5( 23.4)
DMG |34.0000|118.5000|06/22/1920| 248 0.0|  0.0| 4.90| 0.043 | VI | 14.5( 23.4)
MGI |34.0000|118.5000|06/23/1920|1220 0.0|  0.0| 4.00| 0.021 | IV | 14.5( 23.4)
MGI |34.0000|118.5000|03/08/1918|1230 0.0|  0.0| 4.00| 0.021 | IV | 14.5( 23.4)
DMG |34.0000|118.5000|03/06/1918|1820 0.0|  0.0| 4.00| 0.021 | IV | 14.5( 23.4)
DMG |33.8500|118.2670|03/11/1933|1425 0.0|  0.0| 5.00| 0.045 | VI | 14.8( 23.8)
DMG |33.8500|118.2670|03/11/1933| 629 0.0|  0.0| 4.40| 0.028 |  V | 14.8( 23.8)
DMG |33.9030|118.4310|11/29/1938|192115.8| 10.0| 4.00| 0.020 | IV | 14.9( 24.0)
MGI |34.1000|118.0000|01/27/1930|2026 0.0|  0.0| 4.60| 0.032 |  V | 15.0( 24.1)
MGI |34.0000|118.0000|05/05/1929| 735 0.0|  0.0| 4.00| 0.019 | IV | 15.4( 24.8)
MGI |34.0000|118.0000|05/05/1929| 1 7 0.0|  0.0| 4.60| 0.031 |  V | 15.4( 24.8)
MGI |34.0000|118.0000|12/25/1903|1745 0.0|  0.0| 5.00| 0.042 | VI | 15.4( 24.8)
DMG |33.9960|117.9750|06/15/1967| 458 5.5| 10.0| 4.10| 0.018 | IV | 16.9( 27.1)
GSP |34.2310|118.4750|03/20/1994|212012.3| 13.0| 5.30| 0.047 | VI | 16.9( 27.2)
DMG |33.8170|118.2170|10/22/1941| 65718.5|  0.0| 4.90| 0.033 |  V | 17.2( 27.7)
GSP |34.2840|118.4040|01/14/2001|022614.1|  8.0| 4.30| 0.020 | IV | 17.3( 27.8)
GSP |34.2450|118.4710|01/18/1994|155144.9| 12.0| 4.00| 0.016 | IV | 17.4( 28.0)
GSP |34.2930|118.3890|12/06/1994|034834.5|  9.0| 4.50| 0.024 | IV | 17.5( 28.1)
MGI |34.2000|118.0000|01/09/1921| 530 0.0|  0.0| 4.60| 0.026 |  V | 17.5( 28.1)
GSP |34.2890|118.4030|01/14/2001|025053.7|  8.0| 4.00| 0.016 | IV | 17.6( 28.3)
DMG |34.2680|118.4450|08/30/1964|225737.1| 15.4| 4.00| 0.016 | IV | 17.6( 28.4)
GSP |34.2150|118.5100|01/19/1994|140914.8| 17.0| 4.50| 0.023 | IV | 17.8( 28.6)
DMG |33.8000|118.3000|11/03/1931|16 5 0.0|  0.0| 4.00| 0.015 | IV | 18.4( 29.6)
MGI |33.8000|118.3000|12/31/1928|1045 0.0|  0.0| 4.00| 0.015 | IV | 18.4( 29.6)
GSP |34.3120|118.3930|05/25/1994|125657.1|  7.0| 4.40| 0.020 | IV | 18.7( 30.2)
GSP |34.3110|118.3980|06/15/1994|055948.6|  7.0| 4.20| 0.017 | IV | 18.8( 30.3)
GSB |34.2990|118.4280|01/23/1994|085508.7|  6.0| 4.20| 0.017 | IV | 18.9( 30.4)
GSP |34.2130|118.5370|01/17/1994|123055.4| 18.0| 6.70| 0.119 | VII| 19.0( 30.5)
DMG |34.3350|118.3310|02/09/1971|155820.7| 14.2| 4.80| 0.026 |  V | 19.1( 30.8)
GSP |34.2990|118.4390|02/03/1994|162335.4|  8.0| 4.20| 0.016 | IV | 19.2( 30.9)
GSP |34.2870|118.4660|01/19/1994|071406.2| 11.0| 4.00| 0.014 | III| 19.4( 31.2)
DMG |34.3390|118.3320|02/09/1971|141612.9| 11.1| 4.10| 0.015 | IV | 19.4( 31.3)
DMG |33.7830|118.2500|11/14/1941| 84136.3|  0.0| 5.40| 0.042 | VI | 19.4( 31.3)
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GSP |34.2920|118.4660|01/19/1994|144635.2|  6.0| 4.00| 0.013 | III| 19.7( 31.7)
GSP |34.2970|118.4580|01/21/1994|185344.6|  7.0| 4.30| 0.017 | IV | 19.7( 31.7)
DMG |33.7830|118.2000|12/27/1939|192849.0|  0.0| 4.70| 0.023 | IV | 19.7( 31.7)
GSP |34.3010|118.4520|01/21/1994|185244.2|  7.0| 4.30| 0.017 | IV | 19.7( 31.8)
DMG |34.2960|118.4640|03/30/1971| 85443.3|  2.6| 4.10| 0.014 | IV | 19.8( 31.9)
GSP |34.2500|117.9900|06/28/1991|170055.5|  9.0| 4.30| 0.016 | IV | 20.0( 32.1)
GSP |34.2910|118.4760|02/06/1994|131926.9| 11.0| 4.10| 0.014 | IV | 20.0( 32.2)
GSP |34.2620|118.0020|06/28/1991|144354.5| 11.0| 5.40| 0.040 |  V | 20.0( 32.2)
GSB |34.3000|118.4660|01/21/1994|183915.3| 10.0| 4.70| 0.022 | IV | 20.1( 32.4)
DMG |34.3080|118.4540|02/09/1971|144346.7|  6.2| 5.20| 0.033 |  V | 20.2( 32.5)
GSP |34.3110|118.4560|01/17/1994|193534.3|  2.0| 4.00| 0.013 | III| 20.4( 32.9)
GSP |34.3040|118.4730|01/17/1994|150703.2|  2.0| 4.20| 0.015 | IV | 20.6( 33.1)
DMG |34.3610|118.3060|02/09/1971|141021.5|  5.0| 4.70| 0.022 | IV | 20.7( 33.2)
DMG |33.7830|118.1330|01/13/1940| 749 7.0|  0.0| 4.00| 0.012 | III| 20.7( 33.3)



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                              EARTHQUAKE SEARCH RESULTS
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    |       |        |          |  TIME  |     |     | SITE  |SITE|  APPROX.
FILE|  LAT. |  LONG. |   DATE   |  (UTC) |DEPTH|QUAKE|  ACC. | MM |  DISTANCE
CODE| NORTH |  WEST  |          | H M Sec| (km)| MAG.|   g   |INT.|  mi  [km]
----+-------+--------+----------+--------+-----+-----+-------+----+------------
DMG |33.7830|118.1330|10/02/1933| 91017.6|  0.0| 5.40| 0.038 |  V | 20.7( 33.3)
DMG |33.7830|118.1330|11/20/1933|1032 0.0|  0.0| 4.00| 0.012 | III| 20.7( 33.3)
GSP |34.3170|118.4550|01/17/1994|132644.7|  2.0| 4.70| 0.021 | IV | 20.7( 33.4)
GSP |34.2610|118.5340|01/17/1994|123939.8| 14.0| 4.50| 0.018 | IV | 20.8( 33.5)
GSB |34.3100|118.4740|01/21/1994|184228.8|  7.0| 4.20| 0.014 | IV | 21.0( 33.7)
GSP |34.2540|118.5450|01/17/1994|130627.9|  0.0| 4.60| 0.019 | IV | 21.0( 33.8)
DMG |33.7590|118.2530|08/31/1938| 31814.2| 10.0| 4.50| 0.018 | IV | 21.1( 33.9)
GSP |34.3310|118.4420|01/17/1994|141430.3|  1.0| 4.50| 0.018 | IV | 21.2( 34.1)
DMG |34.2860|118.5150|03/31/1971|145222.5|  2.1| 4.60| 0.019 | IV | 21.2( 34.1)
DMG |34.3680|118.3140|04/25/1971|1448 6.5| -2.0| 4.00| 0.012 | III| 21.2( 34.1)
GSP |34.2280|118.5730|01/17/1994|175608.2| 19.0| 4.60| 0.019 | IV | 21.2( 34.2)
DMG |34.3700|118.3020|02/10/1971| 31212.0|  0.8| 4.00| 0.012 | III| 21.2( 34.2)
DMG |34.2730|118.5320|06/21/1971|16 1 8.5|  4.1| 4.00| 0.012 | III| 21.3( 34.2)
DMG |33.7830|118.4170|11/01/1940| 725 3.0|  0.0| 4.00| 0.012 | III| 21.5( 34.5)
DMG |33.7830|118.4170|11/02/1940| 25826.0|  0.0| 4.00| 0.012 | III| 21.5( 34.5)
DMG |33.7830|118.4170|10/14/1940|205111.0|  0.0| 4.00| 0.012 | III| 21.5( 34.5)
DMG |33.7830|118.4170|10/12/1940| 024 0.0|  0.0| 4.00| 0.012 | III| 21.5( 34.5)
GSG |33.9325|117.9172|03/29/2014|040942.3|  4.8| 5.10| 0.028 |  V | 21.5( 34.6)
DMG |34.2840|118.5280|04/02/1971| 54025.0|  3.0| 4.00| 0.011 | III| 21.6( 34.8)
DMG |34.3570|118.4060|02/09/1971|141950.2| 11.8| 4.00| 0.011 | III| 21.9( 35.2)
DMG |33.7670|118.1170|11/04/1939|2141 0.0|  0.0| 4.00| 0.011 | III| 22.1( 35.5)
GSG |33.9613|117.8923|03/29/2014|213245.9|  9.4| 4.14| 0.012 | III| 22.1( 35.6)
DMG |33.7500|118.1830|08/04/1933| 41748.0|  0.0| 4.00| 0.011 | III| 22.1( 35.6)
DMG |33.7500|118.1670|05/16/1933|205855.0|  0.0| 4.00| 0.011 | III| 22.3( 35.9)
PAS |33.9650|117.8860|01/01/1976|172012.9|  6.2| 4.20| 0.013 | III| 22.4( 36.0)
DMG |34.2000|117.9000|08/28/1889| 215 0.0|  0.0| 5.50| 0.036 |  V | 22.5( 36.2)
DMG |34.2000|117.9000|07/13/1935|105416.5|  0.0| 4.70| 0.019 | IV | 22.5( 36.2)
GSP |34.2180|118.6070|01/18/1994|113509.9| 12.0| 4.20| 0.013 | III| 22.6( 36.3)
GSP |34.2740|118.5630|01/27/1994|171958.8| 14.0| 4.60| 0.017 | IV | 22.6( 36.4)
GSP |34.3390|118.4750|09/01/2011|204708.0|  7.0| 4.20| 0.013 | III| 22.6( 36.4)
GSG |34.3340|118.4840|01/17/1994|223152.1| 10.0| 4.20| 0.013 | III| 22.6( 36.4)
DMG |33.9500|118.6320|08/31/1930| 04036.0|  0.0| 5.20| 0.028 |  V | 22.8( 36.7)
DMG |33.7500|118.1330|03/11/1933|11 4 0.0|  0.0| 4.60| 0.017 | IV | 22.9( 36.8)
DMG |34.2650|118.5770|04/15/1971|111432.0|  4.2| 4.20| 0.012 | III| 22.9( 36.8)
DMG |34.3530|118.4560|03/07/1971| 13340.5|  3.3| 4.50| 0.016 | IV | 22.9( 36.9)
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MGI |33.8000|118.5000|06/18/1915|15 5 0.0|  0.0| 4.00| 0.010 | III| 22.9( 36.9)
GSP |34.2690|118.5760|01/17/1994|125546.8| 16.0| 4.10| 0.011 | III| 23.0( 37.0)
DMG |34.3870|118.3640|02/09/1971|143917.8| -1.6| 4.00| 0.010 | III| 23.1( 37.1)
DMG |33.7670|118.4500|10/11/1940| 55712.3|  0.0| 4.70| 0.018 | IV | 23.3( 37.5)
PAS |33.9190|118.6270|01/19/1989| 65328.8| 11.9| 5.00| 0.023 | IV | 23.4( 37.6)
DMG |33.8000|118.0000|10/21/1913| 938 0.0|  0.0| 4.00| 0.010 | III| 23.5( 37.8)
DMG |34.3560|118.4740|03/25/1971|2254 9.9|  4.6| 4.20| 0.012 | III| 23.6( 38.0)
DMG |34.3960|118.3660|02/10/1971|173855.1|  6.2| 4.20| 0.012 | III| 23.7( 38.1)
GSP |34.3570|118.4800|02/25/1994|125912.6|  1.0| 4.10| 0.011 | III| 23.8( 38.4)
DMG |33.7500|118.0830|03/14/1933|1219 0.0|  0.0| 4.50| 0.015 | IV | 23.9( 38.5)
DMG |33.7500|118.0830|03/12/1933| 027 0.0|  0.0| 4.40| 0.014 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 227 0.0|  0.0| 4.60| 0.016 | IV | 23.9( 38.5)
DMG |33.7500|118.0830|03/17/1933|1651 0.0|  0.0| 4.10| 0.011 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/13/1933|1532 0.0|  0.0| 4.10| 0.011 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933|1147 0.0|  0.0| 4.40| 0.014 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/12/1933| 740 0.0|  0.0| 4.20| 0.012 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/21/1933| 326 0.0|  0.0| 4.10| 0.011 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/23/1933| 840 0.0|  0.0| 4.10| 0.011 | III| 23.9( 38.5)



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                                   EARTHQUAKE SEARCH RESULTS
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    |       |        |          |  TIME  |     |     | SITE  |SITE|  APPROX.
FILE|  LAT. |  LONG. |   DATE   |  (UTC) |DEPTH|QUAKE|  ACC. | MM |  DISTANCE
CODE| NORTH |  WEST  |          | H M Sec| (km)| MAG.|   g   |INT.|  mi  [km]
----+-------+--------+----------+--------+-----+-----+-------+----+------------
DMG |33.7500|118.0830|03/23/1933|1831 0.0|  0.0| 4.10| 0.011 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 513 0.0|  0.0| 4.70| 0.017 | IV | 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 635 0.0|  0.0| 4.20| 0.012 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 439 0.0|  0.0| 4.90| 0.020 | IV | 23.9( 38.5)
DMG |33.7500|118.0830|03/12/1933|15 2 0.0|  0.0| 4.20| 0.012 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 216 0.0|  0.0| 4.80| 0.019 | IV | 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933|1045 0.0|  0.0| 4.00| 0.010 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 832 0.0|  0.0| 4.20| 0.012 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 2 5 0.0|  0.0| 4.30| 0.012 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 210 0.0|  0.0| 4.60| 0.016 | IV | 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933|2232 0.0|  0.0| 4.10| 0.011 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/18/1933|2052 0.0|  0.0| 4.20| 0.012 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933|23 5 0.0|  0.0| 4.20| 0.012 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/13/1933| 617 0.0|  0.0| 4.00| 0.010 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 436 0.0|  0.0| 4.60| 0.016 | IV | 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933|1357 0.0|  0.0| 4.00| 0.010 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/12/1933| 546 0.0|  0.0| 4.40| 0.014 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/12/1933| 6 1 0.0|  0.0| 4.20| 0.012 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 910 0.0|  0.0| 5.10| 0.024 | IV | 23.9( 38.5)
DMG |33.7500|118.0830|03/14/1933|2242 0.0|  0.0| 4.10| 0.011 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933|1547 0.0|  0.0| 4.00| 0.010 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 258 0.0|  0.0| 4.00| 0.010 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/12/1933|1651 0.0|  0.0| 4.00| 0.010 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/12/1933|1738 0.0|  0.0| 4.50| 0.015 | IV | 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 611 0.0|  0.0| 4.40| 0.014 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933|1129 0.0|  0.0| 4.00| 0.010 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/12/1933|2354 0.0|  0.0| 4.50| 0.015 | IV | 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933|2240 0.0|  0.0| 4.40| 0.014 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933|1138 0.0|  0.0| 4.00| 0.010 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 211 0.0|  0.0| 4.40| 0.014 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/19/1933|2123 0.0|  0.0| 4.20| 0.012 | III| 23.9( 38.5)
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DMG |33.7500|118.0830|03/12/1933| 448 0.0|  0.0| 4.00| 0.010 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/13/1933|1929 0.0|  0.0| 4.20| 0.012 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/14/1933| 036 0.0|  0.0| 4.20| 0.012 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 837 0.0|  0.0| 4.00| 0.010 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 252 0.0|  0.0| 4.00| 0.010 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/30/1933|1225 0.0|  0.0| 4.40| 0.014 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/15/1933| 2 8 0.0|  0.0| 4.10| 0.011 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/15/1933| 432 0.0|  0.0| 4.10| 0.011 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 553 0.0|  0.0| 4.00| 0.010 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933|1956 0.0|  0.0| 4.20| 0.012 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933|2231 0.0|  0.0| 4.40| 0.014 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 618 0.0|  0.0| 4.20| 0.012 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 2 9 0.0|  0.0| 5.00| 0.022 | IV | 23.9( 38.5)
DMG |33.7500|118.0830|03/16/1933|1530 0.0|  0.0| 4.10| 0.011 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 339 0.0|  0.0| 4.00| 0.010 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 347 0.0|  0.0| 4.10| 0.011 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/13/1933|131828.0|  0.0| 5.30| 0.028 |  V | 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 8 8 0.0|  0.0| 4.50| 0.015 | IV | 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 440 0.0|  0.0| 4.70| 0.017 | IV | 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 759 0.0|  0.0| 4.10| 0.011 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/25/1933|1346 0.0|  0.0| 4.10| 0.011 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 515 0.0|  0.0| 4.00| 0.010 | III| 23.9( 38.5)



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                        EARTHQUAKE SEARCH RESULTS
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    |       |        |          |  TIME  |     |     | SITE  |SITE|  APPROX.
FILE|  LAT. |  LONG. |   DATE   |  (UTC) |DEPTH|QUAKE|  ACC. | MM |  DISTANCE
CODE| NORTH |  WEST  |          | H M Sec| (km)| MAG.|   g   |INT.|  mi  [km]
----+-------+--------+----------+--------+-----+-----+-------+----+------------
DMG |33.7500|118.0830|03/31/1933|1049 0.0|  0.0| 4.10| 0.011 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 926 0.0|  0.0| 4.10| 0.011 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 259 0.0|  0.0| 4.60| 0.016 | IV | 23.9( 38.5)
DMG |33.7500|118.0830|04/02/1933|1536 0.0|  0.0| 4.00| 0.010 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/16/1933|1456 0.0|  0.0| 4.00| 0.010 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/12/1933|2128 0.0|  0.0| 4.10| 0.011 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 323 0.0|  0.0| 5.00| 0.022 | IV | 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933|11 0 0.0|  0.0| 4.00| 0.010 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 336 0.0|  0.0| 4.00| 0.010 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/13/1933| 432 0.0|  0.0| 4.70| 0.017 | IV | 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 751 0.0|  0.0| 4.20| 0.012 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/12/1933| 034 0.0|  0.0| 4.00| 0.010 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 222 0.0|  0.0| 4.00| 0.010 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933|22 0 0.0|  0.0| 4.40| 0.014 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 230 0.0|  0.0| 5.10| 0.024 | IV | 23.9( 38.5)
DMG |33.7500|118.0830|03/12/1933| 616 0.0|  0.0| 4.60| 0.016 | IV | 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 555 0.0|  0.0| 4.00| 0.010 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 521 0.0|  0.0| 4.40| 0.014 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933|1653 0.0|  0.0| 4.80| 0.019 | IV | 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933|1944 0.0|  0.0| 4.00| 0.010 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/20/1933|1358 0.0|  0.0| 4.10| 0.011 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/12/1933|1825 0.0|  0.0| 4.10| 0.011 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/16/1933|1529 0.0|  0.0| 4.20| 0.012 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933|1025 0.0|  0.0| 4.00| 0.010 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 3 5 0.0|  0.0| 4.20| 0.012 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/13/1933| 343 0.0|  0.0| 4.10| 0.011 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933|1141 0.0|  0.0| 4.20| 0.012 | III| 23.9( 38.5)
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DMG |33.7500|118.0830|03/11/1933| 524 0.0|  0.0| 4.20| 0.012 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/15/1933| 540 0.0|  0.0| 4.20| 0.012 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 257 0.0|  0.0| 4.20| 0.012 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 311 0.0|  0.0| 4.20| 0.012 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/12/1933| 835 0.0|  0.0| 4.20| 0.012 | III| 23.9( 38.5)
DMG |33.7500|118.0830|04/02/1933| 8 0 0.0|  0.0| 4.00| 0.010 | III| 23.9( 38.5)
DMG |33.7500|118.0830|04/01/1933| 642 0.0|  0.0| 4.20| 0.012 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 2 4 0.0|  0.0| 4.90| 0.020 | IV | 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 911 0.0|  0.0| 4.40| 0.014 | III| 23.9( 38.5)
DMG |33.7500|118.0830|03/11/1933| 3 9 0.0|  0.0| 4.40| 0.014 | III| 23.9( 38.5)
GSB |34.3010|118.5650|01/17/1994|204602.4|  9.0| 5.20| 0.026 |  V | 24.0( 38.6)
DMG |33.7700|118.4800|04/24/1931|182754.8|  0.0| 4.40| 0.013 | III| 24.0( 38.6)
DMG |34.4110|118.3290|02/10/1971| 5 636.0|  4.7| 4.30| 0.012 | III| 24.3( 39.1)
DMG |34.3610|118.4870|02/10/1971|143526.7|  4.4| 4.20| 0.011 | III| 24.3( 39.1)
GSB |34.3190|118.5580|01/18/1994|132444.1|  1.0| 4.50| 0.014 | IV | 24.5( 39.5)
DMG |34.3920|118.4270|02/21/1971| 71511.7|  7.2| 4.50| 0.014 | IV | 24.6( 39.6)
DMG |33.7330|118.1000|03/11/1933|1447 0.0|  0.0| 4.40| 0.013 | III| 24.6( 39.6)
DMG |33.7330|118.1000|03/11/1933|1350 0.0|  0.0| 4.40| 0.013 | III| 24.6( 39.6)
DMG |33.7330|118.1000|03/11/1933|15 9 0.0|  0.0| 4.40| 0.013 | III| 24.6( 39.6)
PAS |34.3800|118.4590|08/12/1977| 21926.1|  9.5| 4.50| 0.014 | IV | 24.6( 39.6)
GSP |34.3050|118.5790|01/29/1994|112036.0|  1.0| 5.10| 0.023 | IV | 24.7( 39.8)
DMG |34.3840|118.4550|02/10/1971|113134.6|  6.0| 4.20| 0.011 | III| 24.8( 39.8)
DMG |34.3990|118.4190|02/10/1971|134953.7|  9.7| 4.30| 0.012 | III| 24.9( 40.0)
GSP |34.2780|118.6110|01/29/1994|121656.4|  2.0| 4.30| 0.012 | III| 25.0( 40.2)
DMG |34.3970|118.4390|02/21/1971| 55052.6|  6.9| 4.70| 0.016 | IV | 25.2( 40.5)
PAS |33.9330|118.6690|10/17/1979|205237.3|  5.5| 4.20| 0.011 | III| 25.2( 40.6)




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                              EARTHQUAKE SEARCH RESULTS
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    |       |        |          |  TIME  |     |     | SITE  |SITE|  APPROX.
FILE|  LAT. |  LONG. |   DATE   | (UTC)  |DEPTH|QUAKE|  ACC. | MM |  DISTANCE
CODE| NORTH |  WEST  |          | H M Sec| (km)| MAG.|   g   |INT.|  mi  [km]
----+-------+--------+----------+--------+-----+-----+-------+----+------------

DMG |34.4110|118.4010|02/09/1971|14 434.0|  8.0| 4.20| 0.011 | III| 25.3( 40.7)
DMG |34.4110|118.4010|02/09/1971|14 154.0|  8.0| 4.20| 0.011 | III| 25.3( 40.7)
DMG |34.4110|118.4010|02/09/1971|141028.0|  8.0| 5.30| 0.026 |  V | 25.3( 40.7)
DMG |34.4110|118.4010|02/09/1971|14 140.0|  8.0| 4.10| 0.010 | III| 25.3( 40.7)
DMG |34.4110|118.4010|02/09/1971|14 346.0|  8.0| 4.10| 0.010 | III| 25.3( 40.7)
DMG |34.4110|118.4010|02/09/1971|14 1 8.0|  8.0| 5.80| 0.039 |  V | 25.3( 40.7)
DMG |34.4110|118.4010|02/09/1971|14 439.0|  8.0| 4.10| 0.010 | III| 25.3( 40.7)
DMG |34.4110|118.4010|02/09/1971|14 041.8|  8.4| 6.40| 0.062 | VI | 25.3( 40.7)
DMG |34.4110|118.4010|02/09/1971|14 133.0|  8.0| 4.20| 0.011 | III| 25.3( 40.7)
DMG |34.4110|118.4010|02/09/1971|14 444.0|  8.0| 4.10| 0.010 | III| 25.3( 40.7)
DMG |34.4110|118.4010|02/09/1971|14 325.0|  8.0| 4.40| 0.012 | III| 25.3( 40.7)
DMG |34.4110|118.4010|02/09/1971|14 730.0|  8.0| 4.00| 0.009 | III| 25.3( 40.7)
DMG |34.4110|118.4010|02/09/1971|14 710.0|  8.0| 4.00| 0.009 | III| 25.3( 40.7)
DMG |34.4110|118.4010|02/09/1971|14 231.0|  8.0| 4.70| 0.016 | IV | 25.3( 40.7)
DMG |34.4110|118.4010|02/09/1971|14 8 4.0|  8.0| 4.00| 0.009 | III| 25.3( 40.7)
DMG |34.4110|118.4010|02/09/1971|14 4 7.0|  8.0| 4.10| 0.010 | III| 25.3( 40.7)
DMG |34.4110|118.4010|02/09/1971|14 541.0|  8.0| 4.10| 0.010 | III| 25.3( 40.7)
DMG |34.4110|118.4010|02/09/1971|14 230.0|  8.0| 4.30| 0.011 | III| 25.3( 40.7)
DMG |34.4110|118.4010|02/09/1971|14 244.0|  8.0| 5.80| 0.039 |  V | 25.3( 40.7)
DMG |34.4110|118.4010|02/09/1971|14 150.0|  8.0| 4.50| 0.013 | III| 25.3( 40.7)
DMG |34.4110|118.4010|02/09/1971|14 446.0|  8.0| 4.20| 0.011 | III| 25.3( 40.7)
DMG |34.4110|118.4010|02/09/1971|14 853.0|  8.0| 4.60| 0.015 | IV | 25.3( 40.7)
DMG |34.4110|118.4010|02/09/1971|14 159.0|  8.0| 4.10| 0.010 | III| 25.3( 40.7)
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DMG |34.4110|118.4010|02/09/1971|14 550.0|  8.0| 4.10| 0.010 | III| 25.3( 40.7)
DMG |34.4110|118.4010|02/09/1971|14 838.0|  8.0| 4.50| 0.013 | III| 25.3( 40.7)
DMG |34.4110|118.4010|02/09/1971|14 745.0|  8.0| 4.50| 0.013 | III| 25.3( 40.7)
DMG |34.4110|118.4010|02/09/1971|14 8 7.0|  8.0| 4.20| 0.011 | III| 25.3( 40.7)
DMG |34.4110|118.4010|02/09/1971|14 2 3.0|  8.0| 4.10| 0.010 | III| 25.3( 40.7)
GSP |34.3740|118.4950|01/28/1994|200953.4|  0.0| 4.20| 0.011 | III| 25.3( 40.7)
DMG |34.3000|118.6000|04/04/1893|1940 0.0|  0.0| 6.00| 0.045 | VI | 25.4( 40.9)
PAS |33.9440|118.6810|01/01/1979|231438.9| 11.3| 5.00| 0.020 | IV | 25.6( 41.2)
GSB |34.3450|118.5520|01/24/1994|041518.8|  6.0| 4.80| 0.017 | IV | 25.6( 41.2)
GSB |34.2850|118.6240|01/17/1994|135602.4| 19.0| 4.70| 0.015 | IV | 25.9( 41.6)
DMG |34.4310|118.3690|08/14/1974|144555.2|  8.2| 4.20| 0.010 | III| 26.1( 42.0)
DMG |34.3990|118.4730|03/09/1974| 05431.9| 24.4| 4.70| 0.015 | IV | 26.2( 42.1)
DMG |33.7500|118.0000|11/16/1934|2126 0.0|  0.0| 4.00| 0.008 | III| 26.3( 42.3)
GSP |34.3000|118.6200|08/09/2007|075849.0|  4.0| 4.40| 0.012 | III| 26.3( 42.3)
DMG |34.1000|117.8000|03/31/1931|2033 0.0|  0.0| 4.00| 0.008 | III| 26.3( 42.3)
DMG |34.4260|118.4140|02/10/1971| 518 7.2|  5.8| 4.50| 0.013 | III| 26.5( 42.6)
DMG |34.4280|118.4130|04/01/1971|15 3 3.6|  8.0| 4.10| 0.009 | III| 26.6( 42.8)
DMG |34.4330|118.3980|02/09/1971|144017.4| -2.0| 4.10| 0.009 | III| 26.7( 42.9)
GSB |34.3600|118.5710|01/19/1994|044048.0|  2.0| 4.50| 0.012 | III| 27.1( 43.6)
DMG |33.7000|118.0670|03/11/1933| 85457.0|  0.0| 5.10| 0.019 | IV | 27.4( 44.2)
DMG |33.7000|118.0670|03/11/1933| 51022.0|  0.0| 5.10| 0.019 | IV | 27.4( 44.2)
DMG |33.7000|118.0670|02/08/1940|165617.0|  0.0| 4.00| 0.008 | III| 27.4( 44.2)
DMG |33.7000|118.0670|07/20/1940| 4 113.0|  0.0| 4.00| 0.008 | III| 27.4( 44.2)
MGI |33.8000|117.9000|05/22/1902| 740 0.0|  0.0| 4.30| 0.010 | III| 27.5( 44.2)
GSP |34.3790|118.5610|01/18/1994|152346.9|  7.0| 4.80| 0.015 | IV | 27.8( 44.7)
GSP |34.3790|118.5630|01/18/1994|003935.0|  7.0| 4.40| 0.011 | III| 27.8( 44.8)
GSB |34.3330|118.6230|01/18/1994|072356.0| 14.0| 4.30| 0.010 | III| 27.9( 44.9)
DMG |34.4460|118.4360|02/10/1971|185441.7|  8.1| 4.20| 0.009 | III| 28.2( 45.4)
GSP |33.9090|117.7920|06/14/2012|031715.7|  9.0| 4.00| 0.007 | II | 28.8( 46.3)
DMG |34.4570|118.4270|02/09/1971|161926.5| -1.0| 4.20| 0.009 | III| 28.8( 46.3)



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                         EARTHQUAKE SEARCH RESULTS
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    |       |        |          |  TIME   |     |     |  SITE  |SITE|   APPROX.
FILE|  LAT. |  LONG. |   DATE   |  (UTC)  |DEPTH|QUAKE|  ACC.  | MM |  DISTANCE
CODE| NORTH |  WEST  |          | H M Sec | (km)| MAG.|   g    |INT.|  mi  [km]
----+-------+--------+----------+---------+-----+-----+--------+----+------------
GSG |33.6583|118.3722|05/15/2013|200006.2|  1.2| 4.00| 0.007 | II | 28.8( 46.3)
GSG |33.6580|118.3720|05/15/2013|200006.2|  1.2| 4.00| 0.007 | II | 28.8( 46.4)
PAS |34.4630|118.4090|09/24/1977|212824.3|  5.0| 4.20| 0.009 | III| 28.8( 46.4)
GSP |33.9050|117.7920|08/08/2012|062334.1| 10.0| 4.50| 0.011 | III| 28.9( 46.4)
DMG |33.6830|118.0500|03/11/1933|1250 0.0|  0.0| 4.40| 0.010 | III| 28.9( 46.5)
DMG |33.6830|118.0500|03/11/1933| 658 3.0|  0.0| 5.50| 0.025 |  V | 28.9( 46.5)
GSP |34.3620|118.6150|03/20/1996|073759.8| 13.0| 4.10| 0.008 | II | 28.9( 46.6)
GSP |33.9040|117.7910|08/08/2012|163322.1| 10.0| 4.50| 0.011 | III| 28.9( 46.6)
DMG |34.3440|118.6360|02/09/1971|143436.1| -2.0| 4.90| 0.015 | IV | 28.9( 46.6)
GSP |33.9070|117.7880|08/29/2012|203100.3|  9.0| 4.10| 0.008 | II | 29.0( 46.7)
GSB |34.3580|118.6220|01/18/1994|040126.8|  1.0| 4.50| 0.011 | III| 29.0( 46.7)
DMG |33.6630|118.4130|01/08/1967| 738 5.3| 17.7| 4.00| 0.007 | II | 29.1( 46.8)
GSG |34.4080|118.5590|01/17/1994|200205.4|  0.0| 4.00| 0.007 | II | 29.3( 47.1)
GSP |34.3590|118.6290|01/24/1994|055024.3| 12.0| 4.30| 0.009 | III| 29.4( 47.2)
GSP |33.9170|117.7760|09/03/2002|070851.9| 12.0| 4.80| 0.014 | III| 29.4( 47.3)
GSG |33.9530|117.7610|07/29/2008|184215.7| 14.0| 5.30| 0.020 | IV | 29.5( 47.4)
GSP |34.3630|118.6270|01/24/1994|055421.1| 10.0| 4.20| 0.008 | III| 29.5( 47.4)
GSP |34.3740|118.6220|01/17/1994|155410.8| 12.0| 4.80| 0.013 | III| 29.8( 48.0)
GSP |34.3780|118.6180|01/19/1994|211144.9| 11.0| 5.10| 0.017 | IV | 29.8( 48.0)
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PAS |34.3470|118.6560|04/08/1976|152138.1| 14.5| 4.60| 0.011 | III| 29.9( 48.2)
DMG |33.6330|118.2000|11/01/1940|20 046.0|  0.0| 4.00| 0.007 | II | 30.0( 48.2)
PAS |34.0060|117.7390|02/18/1989| 717 4.8|  3.3| 4.30| 0.009 | III| 30.0( 48.2)
GSP |34.3680|118.6370|01/17/1994|194353.4| 13.0| 4.10| 0.007 | II | 30.1( 48.5)
DMG |34.3800|118.6230|10/29/1936|223536.1| 10.0| 4.00| 0.007 | II | 30.1( 48.5)
DMG |33.6300|118.2000|09/13/1929|132338.2|  0.0| 4.00| 0.007 | II | 30.2( 48.6)
GSB |34.3430|118.6660|01/17/1994|234925.4|  8.0| 4.30| 0.009 | III| 30.2( 48.6)
GSP |33.9550|117.7460|12/14/2001|120135.5| 13.0| 4.00| 0.007 | II | 30.3( 48.7)
GSP |34.3970|118.6090|07/22/1999|095724.0| 11.0| 4.00| 0.007 | II | 30.5( 49.0)
DMG |33.6800|117.9930|11/20/1961| 85334.7|  4.4| 4.00| 0.007 | II | 30.6( 49.2)
DMG |33.6710|118.0120|10/20/1961|223534.2|  5.6| 4.10| 0.007 | II | 30.6( 49.2)
GSP |34.3610|118.6570|01/29/2002|055328.9| 14.0| 4.20| 0.008 | II | 30.6( 49.3)
DMG |33.6330|118.4000|10/17/1934| 938 0.0|  0.0| 4.00| 0.007 | II | 30.9( 49.7)
GSP |34.1100|117.7200|04/17/1990|223227.2|  4.0| 4.60| 0.011 | III| 30.9( 49.8)
GSP |34.3260|118.6980|01/17/1994|233330.7|  9.0| 5.60| 0.024 |  V | 30.9( 49.8)
GSP |34.3770|118.6490|04/27/1997|110928.4| 15.0| 4.80| 0.012 | III| 31.0( 50.0)
DMG |34.1000|118.8000|05/10/1911|1340 0.0|  0.0| 4.00| 0.007 | II | 31.1( 50.0)
GSG |34.3040|118.7220|01/17/1994|221922.3| 10.0| 4.00| 0.006 | II | 31.2( 50.3)
GSP |34.1500|117.7200|03/01/1990|032303.0| 11.0| 4.70| 0.011 | III| 31.3( 50.4)
DMG |34.5190|118.1980|08/23/1952|10 9 7.1| 13.1| 5.00| 0.014 | IV | 31.6( 50.8)
GSP |34.3690|118.6720|04/26/1997|103730.7| 16.0| 5.10| 0.015 | IV | 31.6( 50.9)
PAS |34.1360|117.7090|06/26/1988|15 458.5|  7.9| 4.60| 0.010 | III| 31.8( 51.1)
DMG |33.6650|117.9790|10/20/1961|214240.7|  7.2| 4.00| 0.006 | II | 31.9( 51.3)
DMG |33.6170|118.1170|01/20/1934|2117 0.0|  0.0| 4.50| 0.009 | III| 31.9( 51.4)
GSP |34.3040|118.7370|01/19/1994|091310.9| 13.0| 4.10| 0.007 | II | 32.0( 51.4)
MGI |33.8000|117.8000|11/07/1926|1948 0.0|  0.0| 4.60| 0.010 | III| 32.0( 51.4)
MGI |33.8000|117.8000|05/19/1917| 635 0.0|  0.0| 4.00| 0.006 | II | 32.0( 51.4)
MGI |33.8000|117.8000|11/04/1926|2238 0.0|  0.0| 4.60| 0.010 | III| 32.0( 51.4)
MGI |33.8000|117.8000|11/10/1926|1723 0.0|  0.0| 4.60| 0.010 | III| 32.0( 51.4)
MGI |33.8000|117.8000|05/20/1917| 945 0.0|  0.0| 4.00| 0.006 | II | 32.0( 51.4)
MGI |33.8000|117.8000|11/09/1926|1535 0.0|  0.0| 4.60| 0.010 | III| 32.0( 51.4)
MGI |33.8000|117.8000|05/19/1917| 719 0.0|  0.0| 4.00| 0.006 | II | 32.0( 51.4)
DMG |33.6540|117.9940|10/20/1961|194950.5|  4.6| 4.30| 0.008 | II | 32.1( 51.7)
DMG |33.6320|118.4670|01/08/1967| 73730.4| 11.4| 4.00| 0.006 | II | 32.2( 51.8)




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    |       |        |          |  TIME  |     |     | SITE  |SITE|  APPROX.
FILE|  LAT. |  LONG. |   DATE   |  (UTC) |DEPTH|QUAKE|  ACC. | MM |  DISTANCE
CODE| NORTH |  WEST  |          | H M Sec| (km)| MAG.|   g   |INT.|  mi  [km]
----+-------+--------+----------+--------+-----+-----+-------+----+------------
DMG |33.6590|117.9810|10/20/1961|20 714.5|  6.1| 4.00| 0.006 | II | 32.2( 51.8)
GSP |34.1300|117.7000|03/01/1990|003457.1|  4.0| 4.00| 0.006 | II | 32.2( 51.9)
MGI |34.0000|117.7000|12/03/1929| 9 5 0.0|  0.0| 4.00| 0.006 | II | 32.2( 51.9)
GSP |34.1400|117.7000|02/28/1990|234336.6|  5.0| 5.20| 0.016 | IV | 32.3( 52.0)
GSP |34.3540|118.7040|05/01/1996|194956.4| 14.0| 4.10| 0.007 | II | 32.4( 52.1)
GSP |33.9510|117.7090|01/05/1998|181406.5| 11.0| 4.30| 0.008 | II | 32.4( 52.1)
DMG |33.8540|117.7520|10/04/1961| 22131.6|  4.3| 4.10| 0.007 | II | 32.4( 52.2)
MGI |33.7000|117.9000|07/08/1902| 945 0.0|  0.0| 4.00| 0.006 | II | 32.5( 52.2)
DMG |33.7670|117.8170|08/22/1936| 521 0.0|  0.0| 4.00| 0.006 | II | 32.6( 52.4)
GSP |34.3940|118.6690|06/26/1995|084028.9| 13.0| 5.00| 0.013 | III| 32.7( 52.6)
DMG |34.4850|118.5210|07/16/1965| 74622.4| 15.1| 4.00| 0.006 | II | 32.7( 52.6)
GSP |34.1400|117.6900|03/02/1990|172625.4|  6.0| 4.60| 0.010 | III| 32.9( 52.9)
DMG |34.1000|117.6830|01/18/1934| 214 0.0|  0.0| 4.00| 0.006 | II | 33.0( 53.1)
DMG |34.1000|117.6830|01/09/1934|1410 0.0|  0.0| 4.50| 0.009 | III| 33.0( 53.1)
GSP |34.3650|118.7080|01/19/1994|044314.5| 12.0| 4.10| 0.006 | II | 33.0( 53.1)
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GSP |34.3770|118.6980|01/18/1994|004308.9| 11.0| 5.20| 0.016 | IV | 33.1( 53.3)
DMG |33.6170|118.0330|05/21/1938| 944 0.0|  0.0| 4.00| 0.006 | II | 33.5( 53.9)
GSB |34.3790|118.7110|01/19/1994|210928.6| 14.0| 5.50| 0.019 | IV | 33.8( 54.3)
DMG |33.6170|118.0170|03/15/1933|111332.0|  0.0| 4.90| 0.012 | III| 33.8( 54.5)
DMG |33.6170|118.0170|10/02/1933|1326 1.0|  0.0| 4.00| 0.006 | II | 33.8( 54.5)
DMG |33.6170|118.0170|03/14/1933|19 150.0|  0.0| 5.10| 0.014 | IV | 33.8( 54.5)
GSP |34.5000|118.5600|07/05/1991|174157.1| 11.0| 4.10| 0.006 | II | 34.7( 55.8)
DMG |33.6000|118.0170|12/25/1935|1715 0.0|  0.0| 4.50| 0.008 | III| 34.9( 56.2)
DMG |34.4000|117.8000|02/24/1946| 6 752.0|  0.0| 4.10| 0.006 | II | 34.9( 56.2)
DMG |33.6170|117.9670|03/11/1933| 154 7.8|  0.0| 6.30| 0.034 |  V | 35.1( 56.5)
DMG |33.6000|118.0000|03/11/1933| 217 0.0|  0.0| 4.50| 0.008 | III| 35.3( 56.8)
DMG |33.6000|118.0000|03/11/1933| 231 0.0|  0.0| 4.40| 0.007 | II | 35.3( 56.8)
DMG |34.5650|118.1130|02/28/1969| 45612.4|  5.3| 4.30| 0.007 | II | 35.5( 57.2)
GSP |34.0690|118.8820|05/02/2009|011113.7| 14.0| 4.40| 0.007 | II | 35.7( 57.4)
GSP |33.8060|117.7150|03/07/2000|002028.2| 11.0| 4.00| 0.005 | II | 35.9( 57.7)
DMG |33.5430|118.3400|09/14/1963| 35116.2|  2.2| 4.20| 0.006 | II | 36.3( 58.4)
PAS |33.5380|118.2070|05/25/1982|134430.3| 13.7| 4.10| 0.005 | II | 36.5( 58.7)
DMG |33.5610|118.0580|01/15/1937|183547.0| 10.0| 4.00| 0.005 | II | 36.6( 58.9)
GSP |33.6200|117.9000|04/07/1989|200730.2| 13.0| 4.50| 0.007 | II | 36.9( 59.4)
DMG |33.5750|117.9830|03/11/1933| 518 4.0|  0.0| 5.20| 0.013 | III| 37.3( 60.0)
GSP |34.0490|118.9150|02/19/1995|212418.1| 15.0| 4.30| 0.006 | II | 37.6( 60.5)
DMG |33.5670|117.9830|04/17/1934|1833 0.0|  0.0| 4.00| 0.005 | II | 37.8( 60.8)
DMG |33.5670|117.9830|07/07/1937|1112 0.0|  0.0| 4.00| 0.005 | II | 37.8( 60.8)
DMG |33.5170|118.1000|03/22/1941| 82240.0|  0.0| 4.00| 0.005 | II | 38.9( 62.5)
DMG |34.5290|118.6440|02/07/1956| 21656.5| 16.0| 4.20| 0.005 | II | 38.9( 62.6)
DMG |33.5000|118.2500|06/18/1920|10 8 0.0|  0.0| 4.50| 0.007 | II | 39.0( 62.7)
DMG |33.9500|117.5830|04/11/1941| 12024.0|  0.0| 4.00| 0.004 |  I | 39.4( 63.5)
DMG |34.1830|117.5830|10/03/1948| 24628.0|  0.0| 4.00| 0.004 |  I | 39.4( 63.5)
PAS |33.5080|118.0710|11/20/1988| 53928.7|  6.0| 4.50| 0.007 | II | 39.9( 64.2)
PAS |34.0540|118.9640|04/13/1982|11 212.2| 16.6| 4.00| 0.004 |  I | 40.4( 65.0)
DMG |34.3700|117.6500|12/08/1812|15 0 0.0|  0.0| 7.00| 0.047 | VI | 40.6( 65.4)
DMG |34.0170|118.9670|04/16/1948|222624.0|  0.0| 4.70| 0.007 | II | 40.7( 65.5)
GSP |34.3740|117.6490|08/20/1998|234958.4|  9.0| 4.40| 0.006 | II | 40.8( 65.7)
DMG |34.4170|118.8330|06/01/1946|11 631.0|  0.0| 4.10| 0.005 | II | 40.9( 65.7)
DMG |34.3000|117.6000|07/30/1894| 512 0.0|  0.0| 6.00| 0.021 | IV | 41.0( 65.9)
DMG |34.5860|118.6130|02/07/1956| 31638.6|  2.6| 4.60| 0.007 | II | 41.3( 66.5)
DMG |34.1830|117.5480|09/01/1937|163533.5| 10.0| 4.50| 0.006 | II | 41.4( 66.6)
DMG |33.8000|117.6000|09/16/1903|1210 0.0|  0.0| 4.00| 0.004 |  I | 41.9( 67.4)




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    |       |        |          |  TIME  |     |     | SITE  |SITE|  APPROX.
FILE|  LAT. |  LONG. |   DATE   |  (UTC) |DEPTH|QUAKE|  ACC. | MM |  DISTANCE
CODE| NORTH |  WEST  |          | H M Sec| (km)| MAG.|   g   |INT.|  mi  [km]
----+-------+--------+----------+--------+-----+-----+-------+----+------------
MGI |33.8000|117.6000|04/22/1918|2115 0.0|  0.0| 5.00| 0.009 | III| 41.9( 67.4)
PAS |34.0160|118.9880|10/26/1984|172043.5| 13.3| 4.60| 0.007 | II | 41.9( 67.4)
GSP |34.3850|117.6350|10/16/2007|085344.1|  8.0| 4.20| 0.005 | II | 41.9( 67.4)
DMG |34.1670|117.5330|03/01/1948| 81213.0|  0.0| 4.70| 0.007 | II | 42.0( 67.7)
DMG |34.1270|117.5210|12/27/1938|10 928.6| 10.0| 4.00| 0.004 |  I | 42.4( 68.2)
PAS |33.4710|118.0610|02/27/1984|101815.0|  6.0| 4.00| 0.004 |  I | 42.5( 68.4)
DMG |34.3040|117.5700|05/05/1969|16 2 9.6|  8.8| 4.40| 0.005 | II | 42.6( 68.6)
MGI |34.0000|119.0000|12/14/1912| 0 0 0.0|  0.0| 5.70| 0.016 | IV | 42.7( 68.7)
DMG |34.0000|119.0000|09/24/1827| 4 0 0.0|  0.0| 7.00| 0.044 | VI | 42.7( 68.7)
DMG |34.1400|117.5150|01/01/1965| 8 418.0|  5.9| 4.40| 0.005 | II | 42.8( 68.9)
PAS |34.2110|117.5300|10/19/1979|122237.8|  4.9| 4.10| 0.004 |  I | 42.8( 68.9)
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DMG |34.2110|117.5300|09/01/1937|1348 8.2| 10.0| 4.50| 0.006 | II | 42.8( 68.9)
DMG |34.2810|117.5520|09/13/1970| 44748.6|  8.0| 4.40| 0.005 | II | 43.0( 69.2)
DMG |34.2700|117.5400|09/12/1970|143053.0|  8.0| 5.40| 0.012 | III| 43.4( 69.9)
MGI |34.0000|117.5000|12/16/1858|10 0 0.0|  0.0| 7.00| 0.042 | VI | 43.6( 70.2)
DMG |34.0000|117.5000|07/03/1908|1255 0.0|  0.0| 4.00| 0.004 |  I | 43.6( 70.2)
GSG |34.6173|118.6302|01/04/2015|031809.5|  8.8| 4.25| 0.005 | II | 43.7( 70.3)
DMG |33.5450|117.8070|10/27/1969|1316 2.3|  6.5| 4.50| 0.006 | II | 44.2( 71.2)
DMG |34.2000|117.5000|06/14/1892|1325 0.0|  0.0| 4.90| 0.008 | II | 44.3( 71.3)
DMG |34.0650|119.0350|02/21/1973|144557.3|  8.0| 5.90| 0.017 | IV | 44.4( 71.5)
DMG |34.2670|117.5180|09/12/1970|141011.2|  8.0| 4.10| 0.004 |  I | 44.5( 71.7)
DMG |34.1240|117.4800|05/15/1955|17 326.0|  7.6| 4.00| 0.004 |  I | 44.7( 71.9)
DMG |34.1160|117.4750|06/28/1960|20 048.0| 12.0| 4.10| 0.004 |  I | 44.9( 72.3)
T-A |34.4200|118.9200|03/29/1917| 8 6 0.0|  0.0| 4.30| 0.005 | II | 45.1( 72.5)
GSP |34.1390|117.4650|03/09/2008|092232.1|  3.0| 4.00| 0.004 |  I | 45.6( 73.4)
DMG |33.9900|119.0580|05/29/1955|164335.4| 17.4| 4.10| 0.004 |  I | 46.1( 74.1)
DMG |34.3000|117.5000|07/22/1899|2032 0.0|  0.0| 6.50| 0.026 |  V | 46.3( 74.4)
DMG |34.2170|117.4670|03/25/1941|234341.0|  0.0| 4.00| 0.003 |  I | 46.4( 74.7)
PAS |34.1350|117.4480|01/08/1983| 71930.4|  4.6| 4.10| 0.004 |  I | 46.6( 74.9)
GSG |34.1430|117.4425|01/15/2014|093518.9|  3.6| 4.43| 0.005 | II | 46.9( 75.5)
GSP |34.1250|117.4380|01/06/2005|143527.7|  4.0| 4.40| 0.005 | II | 47.1( 75.8)
DMG |34.1120|117.4260|03/19/1937| 12338.4| 10.0| 4.00| 0.003 |  I | 47.7( 76.7)
DMG |34.1320|117.4260|04/15/1965|20 833.3|  5.5| 4.50| 0.005 | II | 47.8( 76.9)
T-A |34.0000|117.4200|04/12/1888|1315 0.0|  0.0| 4.30| 0.004 |  I | 48.2( 77.5)
T-A |34.0000|117.4200|09/10/1920|1415 0.0|  0.0| 4.30| 0.004 |  I | 48.2( 77.5)
DMG |33.6820|117.5530|07/05/1938|18 655.7| 10.0| 4.50| 0.005 | II | 48.3( 77.7)
DMG |33.3670|118.1500|04/16/1942| 72833.0|  0.0| 4.00| 0.003 |  I | 48.5( 78.1)
DMG |33.7170|117.5170|06/19/1935|1117 0.0|  0.0| 4.00| 0.003 |  I | 48.8( 78.5)
DMG |33.7170|117.5070|08/06/1938|22 056.0| 10.0| 4.00| 0.003 |  I | 49.3( 79.3)
MGI |34.0000|117.4000|05/22/1907| 652 0.0|  0.0| 4.60| 0.005 | II | 49.3( 79.3)
PAS |34.3780|119.0350|04/03/1985| 4 449.8| 27.9| 4.00| 0.003 |  I | 49.4( 79.4)
DMG |33.7250|117.4980|01/03/1956| 02548.9| 13.7| 4.70| 0.005 | II | 49.5( 79.6)
DMG |33.6990|117.5110|05/31/1938| 83455.4| 10.0| 5.50| 0.010 | III| 49.7( 80.0)
DMG |33.7480|117.4790|06/22/1971|104119.0|  8.0| 4.20| 0.004 |  I | 49.7( 80.0)
DMG |34.2000|117.4000|07/22/1899| 046 0.0|  0.0| 5.50| 0.010 | III| 49.9( 80.3)
USG |34.1390|117.3860|02/21/1987|231530.1|  2.6| 4.07| 0.003 |  I | 50.1( 80.7)
GSP |34.1900|117.3900|12/28/1989|094108.1| 15.0| 4.50| 0.005 |  I | 50.4( 81.0)
DMG |34.4830|118.9830|09/04/1942| 63433.0|  0.0| 4.50| 0.005 |  I | 50.5( 81.2)
DMG |34.4830|118.9830|09/03/1942|14 6 1.0|  0.0| 4.50| 0.005 |  I | 50.5( 81.2)
DMG |33.7330|117.4670|10/26/1954|162226.0|  0.0| 4.10| 0.003 |  I | 50.8( 81.7)
GSP |33.7330|117.4660|09/02/2007|172914.0|  2.0| 4.70| 0.005 | II | 50.8( 81.8)
DMG |33.8330|117.4000|06/05/1940| 82727.0|  0.0| 4.00| 0.003 |  I | 51.7( 83.2)
DMG |33.9330|117.3670|10/24/1943| 02921.0|  0.0| 4.00| 0.003 |  I | 51.8( 83.4)




                              -------------------------
                              EARTHQUAKE SEARCH RESULTS
                              -------------------------

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-------------------------------------------------------------------------------
    |       |        |          |  TIME  |     |     |  SITE |SITE|  APPROX.
FILE|  LAT. |  LONG. |   DATE   |  (UTC) |DEPTH|QUAKE|  ACC. | MM |  DISTANCE
CODE| NORTH |  WEST  |          | H M Sec| (km)| MAG.|   g   |INT.|  mi  [km]
----+-------+--------+----------+--------+-----+-----+-------+----+------------
DMG |34.0330|117.3500|04/18/1940|184343.9|  0.0| 4.40| 0.004 |  I | 52.0( 83.7)
DMG |34.6000|118.9000|05/18/1940| 91512.0|  0.0| 4.00| 0.003 |  I | 52.0( 83.7)
DMG |34.1180|117.3410|09/22/1951| 82239.1| 11.9| 4.30| 0.004 |  I | 52.6( 84.6)
GSP |33.6920|119.0580|05/30/2012|051400.8| 16.0| 4.00| 0.003 |  I | 52.6( 84.6)
DMG |34.1270|117.3380|02/23/1936|222042.7| 10.0| 4.50| 0.004 |  I | 52.8( 84.9)
DMG |34.1400|117.3390|02/26/1936| 93327.6| 10.0| 4.00| 0.003 |  I | 52.8( 85.0)
DMG |34.6670|118.8330|01/24/1950|215659.0|  0.0| 4.00| 0.003 |  I | 53.0( 85.2)
```

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PAS |33.6300|119.0200|10/23/1981|172816.9| 12.0| 4.60| 0.005 |  I | 53.0( 85.3)
PAS |34.5410|118.9890|06/12/1984| 02752.4| 11.7| 4.10| 0.003 |  I | 53.1( 85.5)
PAS |33.9060|119.1660|05/23/1978| 91650.8|  6.0| 4.00| 0.003 |  I | 53.1( 85.5)
GSP |34.1680|117.3370|06/28/1997|214525.1|  9.0| 4.20| 0.003 |  I | 53.1( 85.5)
DMG |34.0330|117.3170|09/03/1935| 647 0.0|  0.0| 4.50| 0.004 |  I | 53.9( 86.7)
T-A |34.1700|117.3200|12/02/1859|2210 0.0|  0.0| 4.30| 0.003 |  I | 54.1( 87.1)
PAS |33.6370|119.0560|10/23/1981|191552.5|  6.3| 4.60| 0.004 |  I | 54.4( 87.6)
GSP |34.1070|117.3040|01/09/2009|034946.3| 14.0| 4.50| 0.004 |  I | 54.6( 87.9)
MGI |34.2000|119.2000|06/16/1914|1052 0.0|  0.0| 4.60| 0.004 |  I | 54.6( 87.9)
MGI |34.1000|117.3000|07/15/1905|2041 0.0|  0.0| 5.30| 0.008 | II | 54.8( 88.3)
MGI |34.1000|117.3000|12/27/1901|11 0 0.0|  0.0| 4.60| 0.004 |  I | 54.8( 88.3)
MGI |34.1000|117.3000|11/22/1911| 257 0.0|  0.0| 4.00| 0.003 |  I | 54.8( 88.3)
DMG |34.1000|117.3000|02/16/1931|1327 0.0|  0.0| 4.00| 0.003 |  I | 54.8( 88.3)
DMG |34.1180|119.2200|03/18/1957|185628.0| 13.8| 4.70| 0.005 | II | 55.1( 88.7)
DMG |33.7000|117.4000|05/15/1910|1547 0.0|  0.0| 6.00| 0.013 | III| 55.2( 88.9)
DMG |33.7000|117.4000|05/13/1910| 620 0.0|  0.0| 5.00| 0.006 | II | 55.2( 88.9)
DMG |33.7000|117.4000|04/11/1910| 757 0.0|  0.0| 5.00| 0.006 | II | 55.2( 88.9)
MGI |34.2000|117.3000|04/13/1913|1045 0.0|  0.0| 4.00| 0.003 |  - | 55.5( 89.4)
PAS |33.6710|119.1110|09/04/1981|155050.3|  5.0| 5.30| 0.007 | II | 55.9( 90.0)
DMG |34.0000|117.2830|11/07/1939|1852 8.4|  0.0| 4.70| 0.004 |  I | 56.0( 90.1)
DMG |33.9960|117.2700|02/17/1952|123658.3| 16.0| 4.50| 0.004 |  I | 56.7( 91.3)
GSP |34.0470|117.2550|02/21/2000|134943.1| 15.0| 4.50| 0.004 |  I | 57.4( 92.4)
DMG |34.5000|119.1170|11/17/1954|23 351.0|  0.0| 4.40| 0.003 |  I | 57.5( 92.5)
T-A |34.0800|117.2500|10/07/1869| 0 0 0.0|  0.0| 4.30| 0.003 |  I | 57.7( 92.8)
DMG |34.0000|117.2500|07/23/1923| 73026.0|  0.0| 6.25| 0.015 | IV | 57.8( 93.1)
DMG |34.0000|117.2500|11/01/1932| 445 0.0|  0.0| 4.00| 0.002 |  - | 57.8( 93.1)
PAS |34.0230|117.2450|10/02/1985|234412.4| 15.2| 4.80| 0.005 | II | 58.0( 93.4)
DMG |33.6040|119.1050|03/25/1956| 332 2.3|  8.2| 4.20| 0.003 |  I | 58.0( 93.4)
GSP |33.5150|119.0330|08/24/2010|054216.9| 16.0| 4.00| 0.002 |  - | 58.4( 94.1)
GSP |34.4400|119.1830|05/08/2009|202714.0|  7.0| 4.10| 0.003 |  - | 58.8( 94.6)
GSP |34.0240|117.2300|03/11/1998|121851.8| 14.0| 4.50| 0.004 |  I | 58.9( 94.8)
DMG |34.0430|117.2280|04/03/1939| 25044.7| 10.0| 4.00| 0.002 |  - | 58.9( 94.9)
T-A |34.8300|118.7500|11/27/1852| 0 0 0.0|  0.0| 7.00| 0.026 |  V | 59.8( 96.3)
DMG |34.6830|119.0000|04/06/1943|223624.0|  0.0| 4.00| 0.002 |  - | 60.1( 96.7)
DMG |34.7170|118.9670|06/11/1935|1810 0.0|  0.0| 4.00| 0.002 |  - | 60.5( 97.4)
MGI |34.1000|117.2000|04/23/1923|2113 0.0|  0.0| 4.00| 0.002 |  - | 60.6( 97.4)
DMG |34.6170|119.0830|02/26/1950| 0 622.0|  0.0| 4.70| 0.004 |  I | 60.6( 97.4)
DMG |34.7000|119.0000|10/23/1916| 254 0.0|  0.0| 5.50| 0.007 | II | 60.9( 98.1)
DMG |33.9000|117.2000|12/19/1880| 0 0 0.0|  0.0| 6.00| 0.011 | III| 61.6( 99.2)
MGI |34.3000|119.3000|05/15/1927|1120 0.0|  0.0| 4.00| 0.002 |  - | 61.7( 99.3)
MGI |34.3000|119.3000|05/01/1904|1830 0.0|  0.0| 4.60| 0.004 |  I | 61.7( 99.3)
MGI |34.3000|119.3000|09/28/1926|1749 0.0|  0.0| 4.00| 0.002 |  - | 61.7( 99.3)
DMG |34.7840|118.9020|07/27/1972| 03117.4|  8.0| 4.40| 0.003 |  I | 61.7( 99.4)
GSP |34.0050|117.1800|02/13/2010|213906.6|  8.0| 4.10| 0.002 |  - | 61.8( 99.5)

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-END OF SEARCH-   528 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH:   1800  TO  2016

LENGTH OF SEARCH TIME:   217  years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 1.1 MILES (1.7 km) AWAY.
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LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 7.0

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.272 g

COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:
  a-value=  3.589
  b-value=  0.808
  beta-value=  1.860


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TABLE OF MAGNITUDES AND EXCEEDANCES:
------------------------------------

  Earthquake | Number of Times | Cumulative
   Magnitude |    Exceeded     | No. / Year
  -----------+-----------------+------------
     4.0     |       528       |   2.44444
     4.5     |       200       |   0.92593
     5.0     |        70       |   0.32407
     5.5     |        26       |   0.12037
     6.0     |        14       |   0.06481
     6.5     |         6       |   0.02778
     7.0     |         4       |   0.01852
```

APPENDIX E
SUBSURFACE SURVEYS
GEOPHYSICAL SURVEY

Subsurface Surveys & Associates, Inc.
2075 Corte Del Nogal, Suite W   Carlsbad, CA 92011
Phone: (760) 476-0492   Fax: (760) 476-0493

Roux Associates, Inc.   May 24, 2016
5150 Pacific Coast Hwy, Suite 450
Long Beach, CA 90804

Attn: Paige Farrell   Re: Geophysical Survey Summary Report
W. Court Street – Oil Well Search

This report covers the results of a geophysical survey performed at 1350 W. Court Street, in Los Angeles, California. The purpose of the survey was to search for and map the location of suspected oil wells beneath the property. A secondary objective was to locate and mark out pipe and utility lines for future drilling and excavation work.

The fieldwork was performed on May 13, 2016. An aerial photo is provided on Figure 1 that shows the location and limits of the survey.

Survey Design and Field Procedures

To provide a systematic uniform search, a rectangular survey grid was established using a 5-foot line spacing. The grid origin (0, 0) is located at the southeast corner of the site. Data was recorded in a SE to NW direction.

A Geonics EM-61 metal detector and a Gem GSM-19 magnetometer were the primary instruments used for the survey. Thick vegetation and steep slopes prevented use of the EM-61 over the entire grid. Rather, it was used in a free traversing mode across clear areas and to confirm targets found with the magnetometer.

Instruments used for the utility pipeline mark out are as follows: Ridgid line tracer, Fischer M-scope, and Schonstedt magnetic gradiometer. Data from the utility instruments are normally not recorded and stored for future processing. Instead, the display meters are monitored continuously during the traverses to detect subsurface anomalies. The location of pipes and utility lines are marked on the ground and then transferred to site plans or aerial photos.

Equipment Description and Survey Fundamentals

The Geonics EM-61 instrument is a high resolution, time-domain device for detecting buried metal objects. It consists of a powerful transmitter that generates a pulsed primary magnetic field when its coils are energized, which induces eddy currents in nearby objects. The decay of the eddy currents, following the input pulse, is measured by the receiver coils. By making the measurements at a relatively long time interval (measured in milliseconds) after termination of the primary pulse, the response is nearly independent of the electrical conductivity of the ground. Thus, the instrument is a super-sensitive metal detector. Due to its unique coil arrangement, the response curve is a single well-defined positive peak directly over a buried conductive object. This facilitates quick and accurate location of targets.

The GSM-19 is a portable high sensitivity magnetometer made by GEM Systems. This instrument provides measurements of the Earth's total magnetic field with a resolution of 0.10 nT and 0.2 nT accuracy over its entire range. A built in GPS receiver is part of the recording console. A notch 60 Hz filter helps to reduce effects from overhead and underground power lines. Spatial variations in the field as recorded along profiles lines or over a grid, are primarily the result of changes in the magnetic mineral content of the rock or soil. Iron objects and cultural features such as fences, buried pipelines and well casing will also produce a magnetic variation or "anomaly".

The Schonstedt magnetic gradiometer has two flux gate magnetic sensors that are passed closely to and over the ground. When not in close proximity to a magnetic object, the instrument emits a sound signal at a low frequency. When the instrument passes over a buried iron or ferrous metal object that produces a significant magnetic gradient, the frequency of the emitted sound increases. The frequency is a function of the gradient between the two sensors.

The Ridgid utility locator is used to passively detect energized high voltage electric lines and electrical conduit (50-60 Hz), VLF signals (14-22 kHz), as well as to actively trace other utilities. Where risers are present, the utility locator transmitter can be connected directly to the object to send a signal (9.8-82 kHz) along the conductor, pipe, conduit, etc. In the absence of a riser, the transmitter can be used to impress an input signal on the utility by induction. In either case, the receiver unit is tuned to the input signal, and is used to actively trace the pipe's surface projection.

Summary of Results

The Magnetic Contour Map displayed on Figures 2 shows the location of two suspected oil well casings. They are marked by a circular purple symbol. The amplitude and lateral extent of an oil well anomaly is primarily a function of casing length, thickness, and diameter. Because of contour interpolation and possible geometric effects from the induced fields, the well locations are thought to be accurate to about ± 2 feet.

The possible well location posted near grid coordinates (x=100, y=40) is considered uncertain because full access was limited by a large tree and waste high vegetation. Based on the measurements around this area, the suspected casing appears to be beneath the tree. The well target near (x=55, y=5) is open and accessible, and was detected with the EM-61. This indicates the top of casing is relatively shallow (i.e. 0-6 feet depth).

A prominent mag and EM anomaly was detected near grid coordinate (x=55, y=35) and is highlighted on Figure 2. The source is thought to be a fairly large metal object, about 3-5 feet in length. The SE half of this feature is located under bushes, but the NW side is clear (see photo on Figure 4).

The south side of the grid, near the power pole and metal fence, is littered with metal debris at the ground surface and along the south facing slope.

No active utility lines were detected across the top of the property. The electric control box next to the power pole is not functional. Water and electric were found at the NW corner of the grid, but do not extend onto the property.

Geophysical Survey Limitations

It should be understood that limitations inherent in geophysical instruments and/or surveying techniques exist at all sites, and nearly all sites exhibit conditions under which instruments and survey methodology may not perform optimally. Consequently, the detection of buried objects in all circumstances **cannot be guaranteed**. Such limitations are numerous and include, but are not limited to, rebar-reinforced ground cover, abrupt changes in ground cover type, above-ground obstacles preventing full traverses or traverses in one direction only, above-ground conductive objects interfering with instrument signal, nearby powerlines or EM transmitters, highly conductive background soil conditions, limited GPR penetration, non-metallic targets, shallower or larger objects shielding deeper or smaller targets, tracing signal jumping from one line to another, and inaccessible risers, cleanouts, valve boxes, and manholes. If one or more geophysical instruments is rendered ineffective and cannot be utilized, the quality of the survey can be somewhat degraded.

Subsurface Surveys reports may include maps and site plans. While they are an accurate general representation of the survey area and our findings, they are not of engineering quality (i.e., measured and mapped by a licensed land surveyor).

Subsurface Surveys and Associates makes no guarantee either expressed or implied regarding the accuracy of the findings and interpretations present. And, in no event will Subsurface Surveys and Associates be liable for any direct, indirect, special, incidental, or consequential damages resulting from interpretations and opinions presented herewith.

All data acquired during this survey is considered confidential and is available for review by your staff at any time. We appreciate the opportunity to participate in this project.

PAWalen Please call if there are any questions.

Phillip A. Walen
Senior Geophysicist
CA Registration No. GP917

Survey Location Map

1350 W. Court Street -- Los Angeles, CA

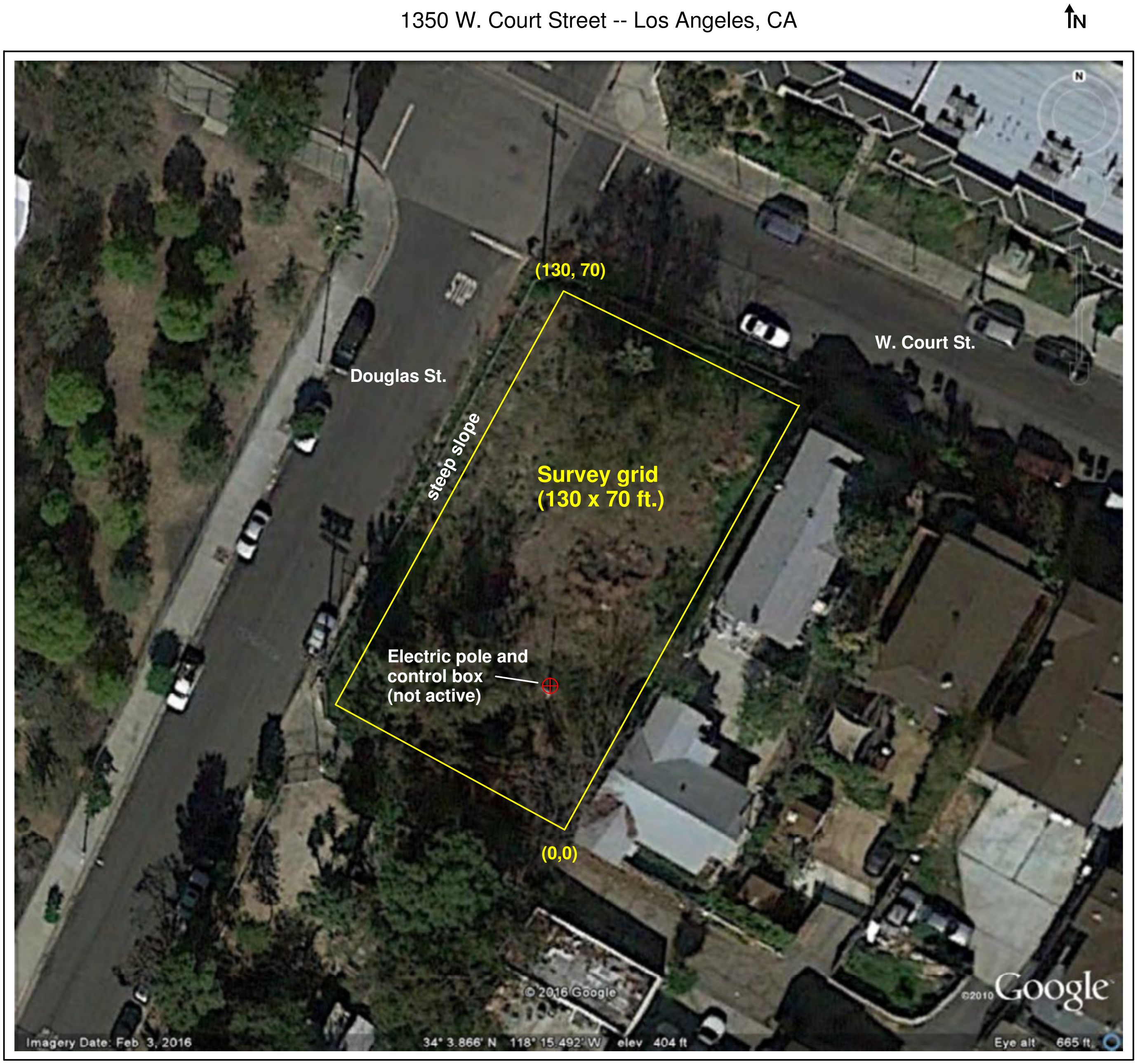
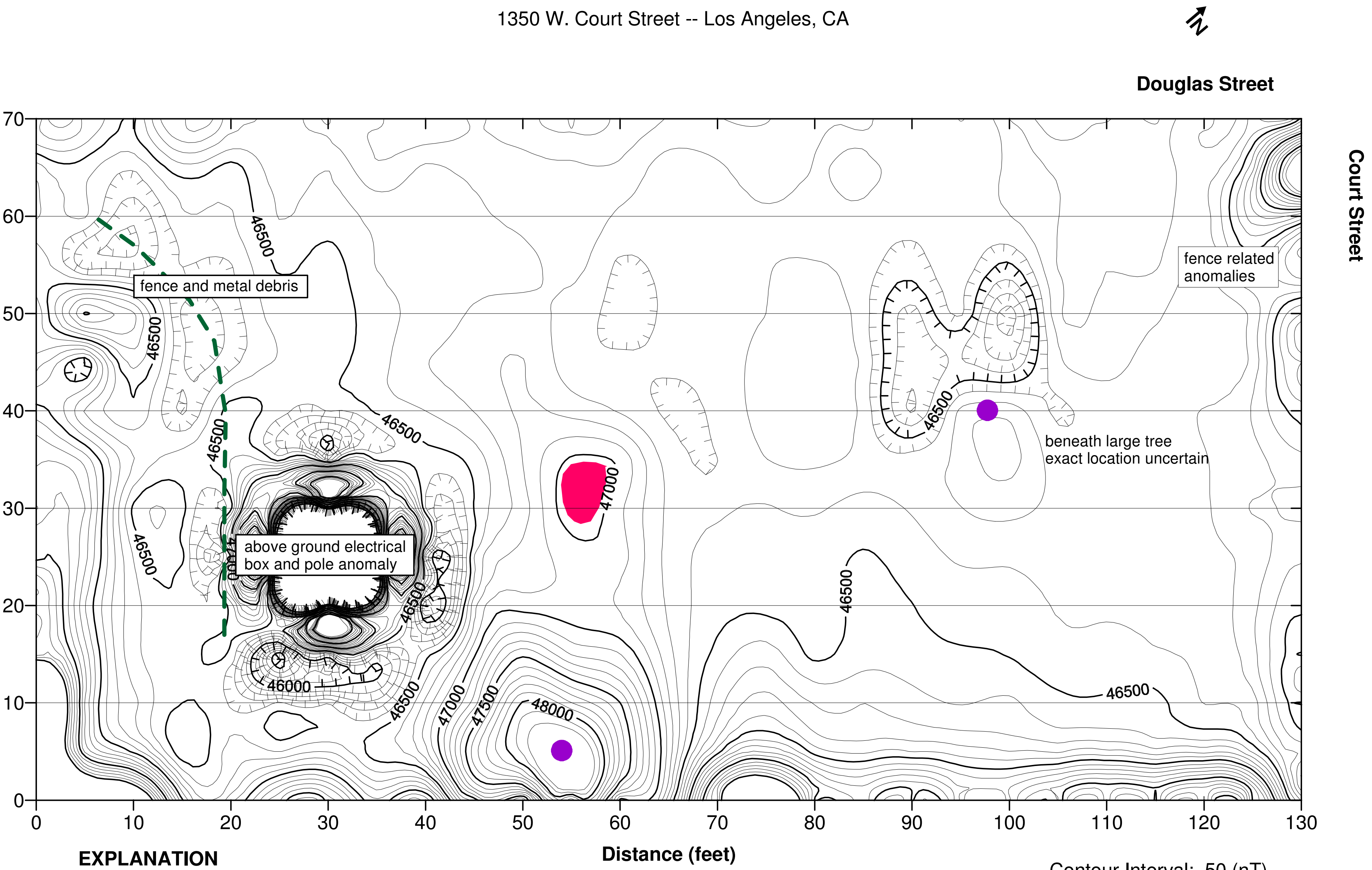


Figure 1

Magnetic Contour Map

1350 W. Court Street -- Los Angeles, CA



EXPLANATION

● Suspected oil well

🟥 Buried metal object
Depth (0-3 ft) approx.

– – – Metal fence

Contour Interval: 50 (nT)

Figure 2

Survey Photographs

Figure 3

Survey Photograph

Buried metal object detected with magnetics and EM (see Figure 2).

Figure 4

Leighton and Associates, Inc.
A LEIGHTON GROUP COMPANY

August 24, 2018

(Revised September 18, 2018)

Project No. 11388.005

Court Street, LLC
c/o DB Companies
9748 Topanga Canyon Boulevard
Los Angeles, California 91311

Attention: Mrs. Ellen Gola

Subject: Addendum to Geotechnical Exploration Report
Retaining Wall and Alley Grading
1346, 1350 and 1352 W. Court Street
City of Los Angeles, California

Reference: Leighton and Associates, Inc., 2016, *Geotechnical Exploration Report Proposed Multi-Family Residential Development 1346, 1350 and 1352 W. Court Street, City of Los Angeles, California,* Project No. 11388.001, dated August 31, 2016.

INTRODUCTION

Per your request, Leighton and Associates, Inc. (Leighton) is pleased to present this addendum letter to our original geotechnical report dated August 31, 2016 for the proposed multi-family residential development located at 1346, 1350, and 1352 West Court Street in the City of Los Angeles, California. We understand in order to accommodate site grades, the alley located directly south of the project site will need to be cut and regraded. In addition, construction of a new retaining wall located along the southern edge of the alleyway will be required. The currently planned development will consist of a four-story multi-family residential apartment building over a partial two-level subterranean parking garage with an entrance at street grade on the Douglas Street side of the property.

All recommendations provided in the above referenced geotechnical report remains valid except where modified herein. No additional field exploration was performed in association with this addendum.

PROJECT UNDERSTANDING

Based on correspondence with design staff as well as review of the undated *Court Street and Douglas Street (S/E Corner) (Voluntary Improvement) Plans,* prepared by C & V Consulting, Inc., we understand that grading involving cuts on the order of 6 to 8 feet are proposed in the alley south of the project site. In addition, a concrete masonry unit (cmu) retaining wall with a maximum height of 4 feet 8 inches is currently proposed along the southern edge of the alley (south of the project site) to accommodate design grade for the development. We understand the retaining wall is currently designed for full hydrostatic pressure and also includes weep holes and a mirafi drain.

SUBSURFACE CONDITIONS

Our original geotechnical exploration within previous project limits encountered approximately 5 feet of artificial fill soils overlying sedimentary bedrock of the Puente Formation. Bedding attitudes as measured at the bedrock outcrop in the southwest corner of the project site indicate a northeast strike of 55 to 65 degrees with a dip angle of 55 to 65 degrees to the southeast.

The bedding, as measured in our original exploration creates a favorable bedding condition for cuts proposed along the southern edge of the alley where exposed bedding planes will be dipping into slope.

RETAINING WALL RECOMMENDATIONS

The following soil parameters (Leighton 2016) may be used for the design of retaining walls with level backfill:

	Free Cantilever Walls (Active) psf/ft	Restrained Walls (At-rest) psf/ft
Retained Height up to 20 feet		
Earth Pressure with Geologic Surcharge	56	81
Earth Pressure without Geologic Surcharge	28	45
Seismic Pressure with Geologic Surcharge	41	
Seismic Pressure without Geologic Surcharge	25	

Seismic earth pressure should be applied in addition to static earth pressure for conventional retaining walls that are more than 6 feet in height and the unbalanced height portion (higher side) of the basement walls.

In addition to the recommended earth pressure, the walls should be designed to resist any applicable surcharge loads behind the walls.

The retaining wall foundation is anticipated to bear on bedrock.

Retaining Wall Foundation

New shallow spread footings established on bedrock may be used to support the proposed retaining wall

Wall foundation should have a minimum width of 12 inches. The top of the footing should be at least 12 inches below lowest adjacent grade.

The footings may be designed for a maximum net allowable soil bearing pressure of 6,000 pounds per square foot (psf). The soil bearing pressure may be increased by one-third for transient loads such as wind and seismic forces.

Resistance to lateral loads will be provided by a combination of friction between the soil and foundation interface and passive pressure acting against the vertical portion of the footings. For calculating allowable lateral resistance, a passive pressure of 250 psf per foot of depth to a maximum of 2,500 psf and a frictional coefficient of 0.25 may be used provided the foundations are supported within structural compacted fill. When combining frictional and passive resistance, the passive resistance should be reduced by one-third.

The estimated total settlement of the structures supported on spread footings as recommended above is less than 1 inch. The differential settlement between adjacent columns is estimated to be less than ½ inch over a horizontal distance of 40 feet.

Backfill

Backfill for retaining structures planned at the site should be non-expansive soil (Expansion Index less than 20).

Backfill should be compacted to at least 90 percent of the maximum dry density obtained by ASTM Test Method D 1557. Relatively light equipment should be used for backfilling behind retaining walls.

Drainage

We understand that site geometry will not allow for construction of a backdrain system to divert water to a designated discharge location. In lieu of a backdrain, a wall drain system consisting of weep hole perforations penetrating the face of the wall and mirafi drain at the back of the wall are planned. The retaining wall should be designed to withstand full hydrostatic pressure in the event that the proposed wall drain fails to function as planned.

In addition, the wall should be waterproofed or at least damp-proofed, depending upon the degree of moisture protection desired.

GRADING RECOMMENDATIONS

The exact thickness of undocumented fill beneath the alley is currently unknown. Although the planned cuts on the order of 6 to 8 feet as proposed should remove the undocumented fill and expose competent bedrock, unsuitable materials if encountered at the exposed subgrade should be removed until a competent bedrock subgrade is exposed. Overexcavation and recompaction if required to remove unsuitable subgrade materials should extend a minimum horizontal distance equal to the vertical distance between the proposed footing bottom and depth of overexcavation. However, care should be used to avoid undermining existing improvements adjacent to the excavation.

After completion of the overexcavation and prior to fill placement or other improvements such as flatwork and hardscape, the exposed soils should be scarified to a minimum depth of six inches, moisture conditioned 2 to 4 percentage points above optimum moisture content and compacted to a minimum of 90 percent relative compaction (ASTM D1557).

The excavated onsite soils, less than 6 inches and free of any deleterious material or organic matter, can be used in required fills. Any required import material should consist of non-corrosive and relatively non-expansive soils with an Expansion Index (EI) less than 20. The imported materials should contain sufficient fines (binder material) so as to be relatively impermeable and result in a stable subgrade when compacted. All proposed import materials should be approved by the geotechnical engineer of record prior to being placed at the site.

All fill soil should be placed in thin, loose lifts, with each lift properly moisture conditioned 2 to 4 percentage above the optimum moisture content and compacted to a minimum of 90 percent relative compaction (ASTM D1557). Proper moisture conditioning of the soils is vital in reducing expansion potential and reducing the potential for post-construction heave that may result in distortion and possibly damage to new improvements. Aggregate base should be compacted to a minimum of 95 percent relative compaction (ASTM D1557).

Temporary Excavations

All temporary excavations, including footings, utility trenches, should be performed in accordance with project plans, specifications, and all OSHA requirements. Excavations 5 feet or deeper should be laid back or shored in accordance with OSHA requirements before personnel are allowed to enter.

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 cut, unless the cut is shored appropriately.

During construction, the soil conditions should be regularly evaluated to verify that conditions are as anticipated. The contractor shall be responsible for providing the “competent person” required by OSHA standards to evaluate soil conditions. The bedrock can be classified as Type B soil. Soil types will vary, but Type C soils can be expected at shallow depths. Close coordination between the competent person and the geotechnical engineer should be maintained to facilitate construction while providing safe excavations.

Should an excavation depth exceed 5 feet in total depth in the area of the proposed retaining wall and alley earthwork, the “ABC” slot cut method may be used immediately adjacent to property lines or other existing improvements during excavation. The maximum width and height of the slots should not exceed eight feet. Final configuration of the slot cut should be determined based on the exposed soil conditions. Slot cut grading along the property lines will likely result in a zone of unimproved soils along the property line. Depending on the site development layout, recommendations for other alternatives to safely grade along the property lines can be provided upon request.

We appreciate the opportunity to work with you on this project. If you have any questions or if we can be of further service, please contact us at your convenience.

Respectfully submitted,

LEIGHTON AND ASSOCIATES, INC.

PROFESSIONAL GEOLOGIST
ERIC M. HOLLIDAY
No. 9219
STATE OF CALIFORNIA

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REGISTERED PROFESSIONAL ENGINEER
VINCENT PING-KUEN IP
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Exp. 3/2020
GEOTECHNICAL
STATE OF CALIFORNIA

Vincent P. Ip, PE, GE 2522
Senior Principal Engineer
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EMH/JAR/VIP/gv

Distribution: (1) Addressee