# Appendix IS-6

Geology and Soils

# Appendix IS-6.1

## Geotechnical Engineering Investigation

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Celebrating 50 YEARS OF SERVICE
1971-2021

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SCD 1811 Sacramento, LLC
633 West 5th Street, Floor 68
Los Angeles, California 90071

Attention: Fei Ye

<u>Subject</u>: Geotechnical Engineering Investigation
Proposed Office Development
1727 through 1829 East Sacramento Street, Los Angeles, California

Dear Ms. Ye:

This letter transmits the Geotechnical Engineering Investigation for the subject site prepared by Geotechnologies, Inc. This report provides geotechnical recommendations for the development of the site, including earthwork, seismic design, retaining walls, excavations, shoring and foundation design. Engineering for the proposed project should not begin until approval of the geotechnical investigation is granted by the local building official. Significant changes in the geotechnical recommendations may result due to the building department review process.

The validity of the recommendations presented herein is dependent upon review of the geotechnical aspects of the project during construction by this firm. The subsurface conditions described herein have been projected from limited subsurface exploration and laboratory testing. The exploration and testing presented in this report should in no way be construed to reflect any variations which may occur between the exploration locations, or which may result from changes in subsurface conditions.

Should you have any questions please contact this office.

Respectfully submitted,
GEOTECHNOLOGIES, INC.

REGISTERED PROFESSIONAL ENGINEER
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**TABLE OF CONTENTS**

**SECTION** **PAGE**

## TABLE OF CONTENTS

**SECTION** **PAGE**

## TABLE OF CONTENTS

**SECTION** **PAGE**

**GEOTECHNICAL ENGINEERING INVESTIGATION**
**PROPOSED OFFICE DEVELOPMENT**
**1727 – 1829 EAST SACRAMENTO STREET**
**LOS ANGELES, CALIFORNIA**

## INTRODUCTION

This report presents the results of the geotechnical engineering investigation performed on the subject site. The purpose of this investigation was to identify the distribution and engineering properties of the geologic materials underlying the site, and to provide geotechnical recommendations for the design of the proposed development.

This investigation included three exploratory excavations, collection of representative samples, laboratory testing, engineering analysis, review of published geologic data, review of available geotechnical engineering information and the preparation of this report. The exploratory excavation locations are shown on the enclosed Plot Plan. The results of the exploration and the laboratory testing are presented in the Appendix of this report.

## PROPOSED DEVELOPMENT

Information concerning the proposed development was furnished by the client. In addition, the Schematic Design Plans prepared by Perkins & Will, dated July 15, 2022, were reviewed for the preparation of this report. The proposed development consists of the construction of a 15-story office building. Levels one through six will consist of a podium, and will include parking, office and mixed-use space. Levels six through fifteen will be concentrated on the eastern portion of the structure, and will consist of office space. The enclosed Plot Plan illustrates the location and alignment of the proposed structure. The proposed structure will be built at, or near, the existing site grade.

Column loads are estimated to be up to a maximum of 2,400 kips. This load reflects the dead plus live load. Grading is expected to consist of excavations on the order of 5 to 7 feet in depth for the recommended removal and recompaction.

Any changes in the design of the project or location of any structure, as outlined in this report, should be reviewed by this office. The recommendations contained in this report should not be considered valid until reviewed and modified or reaffirmed, in writing, subsequent to such review.

## SITE CONDITIONS

The site is located at 1727 through 1829 East Sacramento Street, in the eastern downtown area of the City of Los Angeles, California. The site is approximately 1¾ acres in area, bounded by warehouse developments to the north, Wilson Street to the east, Sacramento Street to the south, and a warehouse development to the southwest. The site is shown relative to nearby topographic features in the enclosed Vicinity Map.

The site grade is relatively level, with no pronounced highs or lows. The site is currently developed with three warehouse buildings, and paved parking lots. The warehouse buildings are one story in height, and were built near the existing grade. The site is relatively level, with no pronounced highs or lows. Vegetation at the site is nonexistent. Drainage across the site appears to be by sheetflow to the city streets.

## GEOTECHNICAL EXPLORATION

### FIELD EXPLORATION

The site was explored on December 8 and 10, 2021 by excavating three borings. The borings were drilled to depths between 30 and 55 feet below the existing grade, with the aid of a truck-mounted drilling machine using 8-inch diameter hollowstem augers. Exploration could not be conducted deeper than 55 feet due to refusal from the very dense native soils. The exploration locations are shown on the Plot Plan and the geologic materials encountered are logged on Plates A-1 through A-3.

The location of exploratory excavations was determined from hardscaped features shown in the enclosed Plot Plan. The location of the exploratory excavations should be considered accurate only to the degree implied by the method used.

## Geologic Materials

Fill materials were encountered in all three exploratory borings, to depths ranging between 3 and 7 feet below the existing grade. The fill consists of silty sand, which is yellowish brown to dark brown in color, moist, medium dense and fine grained.

The fill is in turn underlain by native alluvial soils. To an approximate depth of 15 feet, the upper native alluvial soils are composed of sand, silty sand and sandy silt, which are yellowish brown to grayish brown in color, moist, medium dense, or stiff and fine to medium grained. Below a depth of 15 feet, the alluvial soils consist mainly of sands, which are yellowish brown to dark brown in color, moist, dense to very dense, and fine to coarse grained, with interlayered gravel and cobbles.

More detailed descriptions of the earth materials encountered may be obtained from individual logs of the subsurface excavations.

## Groundwater

Groundwater was not encountered during exploration, conducted to a maximum depth of 55 feet below the existing grade. According to groundwater data provided in the Seismic Hazard Zone Report of the Los Angeles 7½-Minute Quadrangle, the historically highest groundwater level for the site was on the order of 145 feet below the ground surface (CDMG, 1998, Revised 2006). A copy of the historically highest water map is enclosed herein.

Fluctuations in the level of groundwater would be expected to occur over time due to variations in rainfall, temperature, and other factors. Fluctuations also may occur in the vicinity of the site.

**Caving**

Caving could not be directly observed during exploration due to the continuously-cased design of the excavation equipment utilized. However, based on the experience of this firm, large diameter excavations, excavations that encounter granular, cohesionless soils and excavations below the groundwater table will most likely experience caving.

## SEISMIC EVALUATION

### REGIONAL GEOLOGIC SETTING

The subject site is located in the Los Angeles Basin and within the Peninsular Ranges Geomorphic Province. The Peninsular Ranges are characterized by northwest-trending blocks of mountain ridges and sediment-floored valleys. The dominant geologic structural features are northwest trending fault zones that either die out to the northwest or terminate at east-west trending reverse faults that form the southern margin of the Transverse Ranges (Yerkes, 1965).

The Los Angeles Basin is located at the northern end of the Peninsular Ranges Geomorphic Province. The basin is bounded by the east and southeast by the Santa Ana Mountains and San Joaquin Hills. It is bounded to the northwest by the Santa Monica Mountains. Over 22 million years ago the Los Angeles basin was a deep marine basin formed by tectonic forces between the North American and Pacific plates. Since that time, over 5 miles of marine and non-marine sedimentary rock as well as intrusive and extrusive igneous rocks have filled the basin. During the last 2 million years, defined by the Pleistocene and Holocene epochs, the Los Angeles basin and surrounding mountain ranges have been uplifted to form the present-day landscape. Erosion of the surrounding mountains has resulted in deposition of unconsolidated sediments in low-lying areas by rivers such as the Los Angeles River. Areas that have experienced subtle uplift have been eroded with gullies.

## REGIONAL FAULTING

Based on criteria established by the California Division of Mines and Geology (CDMG) now called California Geologic Survey (CGS), Faults may be categorized as Holocene-active, Pre-Holocene faults, and Age-undetermined faults. Holocene-active faults are those which show evidence of surface displacement within the last 11,700 years. Pre-Holocene faults are those that have not moved in the past 11,700 years. Age-undetermined faults are faults where the recency of fault movement has not been determined.

Buried thrust faults are faults without a surface expression but are a significant source of seismic activity. They are typically broadly defined based on the analysis of seismic wave recordings of hundreds of small and large earthquakes in the southern California area. Due to the buried nature of these thrust faults, their existence is usually not known until they produce an earthquake. The risk for surface rupture potential of these buried thrust faults is inferred to be low (Leighton, 1990). However, the seismic risk of these buried structures in terms of recurrence and maximum potential magnitude is not well established. Therefore, the potential for surface rupture on these surface-verging splays at magnitudes higher than 6.0 cannot be precluded.

## SEISMIC HAZARDS AND DESIGN CONSIDERATIONS

The primary geologic hazard at the site is moderate to strong ground motion (acceleration) caused by an earthquake on any of the local or regional faults. The potential for other earthquake-induced hazards was also evaluated including surface rupture, liquefaction, dynamic settlement, inundation and landsliding.

**Surface Rupture**

In 1972, the Alquist-Priolo Special Studies Zones Act (now known as the Alquist-Priolo Earthquake Fault Zoning Act) was passed into law. As revised in 2018, The Act defines "Holocene-active" Faults utilizing the same aging criteria as that used by California Geological Survey (CGS). However, established state policy has been to zone only those faults which have direct evidence of movement within the last 11,700 years. It is this recency of fault movement that the CGS considers as a characteristic for faults that have a relatively high potential for ground rupture in the future.

CGS policy is to delineate a boundary from 200 to 500 feet wide on each side of the Holocene-Active fault trace based on the location precision, the complexity, or the regional significance of the fault. If a site lies within an Earthquake Fault Zone, a geologic fault rupture investigation must be performed that demonstrates that the proposed building site is not threatened by surface displacement from the fault before development permits may be issued.

Ground rupture is defined as surface displacement which occurs along the surface trace of the causative fault during an earthquake. Based on research of available literature and results of site reconnaissance, no known Holocene-active or Pre-Holocene faults underlie the subject site. In addition, the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Based on these considerations, the potential for surface ground rupture at the subject site is considered low.

**Liquefaction**

Liquefaction is a phenomenon in which saturated silty to cohesionless soils below the groundwater table are subject to a temporary loss of strength due to the buildup of excess pore pressure during cyclic loading conditions such as those induced by an earthquake. Liquefaction-related effects include loss of bearing strength, amplified ground oscillations, lateral spreading, and flow failures.

The Seismic Hazards Map of the Los Angeles Quadrangle by the State of California (CDMG, 1999) does not classify the site as part of a Liquefiable area. This determination is based on groundwater depth records, soil type and distance to a fault capable of producing a substantial earthquake. A copy of this Seismic Hazard Zones Map is enclosed herein.

Based on the density of the soils underlying the site, the current groundwater level, and the mapped depth to the historically highest groundwater level, the soils underlying the site are not considered capable of liquefaction during the ground motion expected during the design-based earthquake.

## Dynamic Dry Settlement

Seismically-induced settlement or compaction of dry or moist, cohesionless soils can be an effect related to earthquake ground motion. Such settlements are typically most damaging when the settlements are differential in nature across the length of structures.

Some seismically-induced settlement of the proposed structures should be expected as a result of strong ground-shaking, however, due to the uniform nature of the underlying geologic materials, excessive differential settlements are not expected to occur.

## Tsunamis, Seiches and Flooding

Tsunamis are large ocean waves generated by sudden water displacement caused by a submarine earthquake, landslide, or volcanic eruption. Review of the County of Los Angeles Flood and Inundation Hazards Map (Leighton, 1990) indicates the site does not lie within mapped tsunami inundation boundaries.

Review of the County of Los Angeles Flood and Inundation Hazards Map, Leighton (1990), indicates the site lies within the potential mapped inundation boundaries of the Hansen and Sepulveda Reservoirs, should the dam retaining these reservoirs fail during a seismic event. A determination of whether a higher site elevation would remove the site from the potential inundation zones is beyond the scope of this assessment.

Review of the applicable Flood Insurance Rate Map (06037C1638G) indicates the site lies within an area of minimal flood hazard (Zone X).

### Landsliding

The probability of seismically-induced landslides occurring on the site is considered to be low due to the general lack of elevation difference across or adjacent to the site.

## CONCLUSIONS AND RECOMMENDATIONS

Based upon the exploration, laboratory testing, and research, it is the finding of Geotechnologies, Inc. that construction of the proposed office development is considered feasible from a geotechnical engineering standpoint provided the advice and recommendations presented herein are followed and implemented during construction.

During exploration fill materials were observed to extend to depths ranging between 3 and 7 feet below the existing grade. The existing fill materials are unsuitable for support of new foundations and concrete slabs-on-grade. However, the existing fill materials may be reused in the preparation of an engineered compacted fill pad.

It is recommended that the 15-story portion of the structure, which will be subject to the higher load demand, is supported on a mat foundation system. The remainder of the structure, which is not expected to exceed 6 stories in height, may be supported on a conventional foundation system. Recommendations to aid in the design of both foundation systems are provided herein.

Conventional foundations and mat foundations must bear in a newly placed engineered compacted fill pad. For creation of an engineered compacted fill pad, it is recommended that all existing fill materials and the upper native soils be properly removed and recompacted to a minimum depth of 5 feet below the existing grade, or 3 feet below the bottom of the proposed foundations, whichever is deeper. In addition, the proposed fill pad shall be over excavated horizontally beyond the edge of foundations for a minimum of 3 feet, or a distance equal to the depth of fill below the foundations, whichever is greater. Based on the maximum depth of fill observed during exploration, it is anticipated that the thickness of the compacted fill pad may be up to 7 feet in thickness.

As shown on the enclosed Plot Plan, it is anticipated that the proposed structure will extend adjacent to the southern property line. Where new foundations will be built immediately adjacent to property lines, the recommended horizontal over-excavation for the creation of a compacted fill pad will not be possible. In areas where the horizontal over-excavation is not possible, the proposed foundations should be deepened as appropriate, so the lateral capacity of the new foundation is derived from native alluvial soils and/or compacted fill.

The implementation of slot-cutting, and/or the installation of a temporary shoring system, will be required where the temporary grading and foundation excavations will undermine the property lines, adjacent foundations, and the public right of way. This condition is anticipated along the southern property line.

The validity of the conclusions and design recommendations presented herein is dependent upon review of the geotechnical aspects of the proposed construction by this firm. The subsurface conditions described herein have been projected from excavations on the site as indicated and should in no way be construed to reflect any variations which may occur between these excavations or which may result from changes in subsurface conditions. Any changes in the design, as outlined in this report, should be reviewed by this office. The recommendations contained herein should not be considered valid until reviewed and modified or reaffirmed subsequent to such review.

## SEISMIC DESIGN CONSIDERATIONS

### California Building Code Seismic Parameters

A Surface Wave Measurement has been recently performed at the site by GeoVision. The intention of this work was to measure the shear-wave velocity at the site, to obtain the corresponding site classification. This work was summarized in the Report of Surface Wave Measurement, dated June 15, 2022. A copy of this report is enclosed in the Appendix.

Based on the shear-wave velocities measured by GeoVision, the shear-wave velocity on the upper 30 meters of the site ($V_{S30}$) below the existing ground surface was determined to be 366 m/s. This $V_{S30}$ value corresponds to a site classification for seismic design of Site Class C, which corresponds to a "Very Dense Soil and Soft Rock" Profile, according to NEHRP. This information and the site coordinates were input into the OSHPD seismic utility program at https://seismicmaps.org in order to calculate ground motion parameters for the site.

| 2019 CALIFORNIA BUILDING CODE SEISMIC PARAMETERS | |
|---|---|
| Site Class | C |
| Mapped Spectral Acceleration at Short Periods ($S_S$) | 1.910g |
| Site Coefficient ($F_a$) | 1.2 |
| Maximum Considered Earthquake Spectral Response for Short Periods ($S_{MS}$) | 2.292g |
| Five-Percent Damped Design Spectral Response Acceleration at Short Periods ($S_{DS}$) | 1.528g |
| Mapped Spectral Acceleration at One-Second Period ($S_1$) | 0.680g |
| Site Coefficient ($F_v$) | 1.4 |
| Maximum Considered Earthquake Spectral Response for One-Second Period ($S_{M1}$) | 0.952g |
| Five-Percent Damped Design Spectral Response Acceleration for One-Second Period ($S_{D1}$) | 0.635g |

## EXPANSIVE SOILS

The onsite geologic materials are in the very low expansion range. The Expansion Index was found to be 4 and 5 for a representative bulk samples. Recommended reinforcing is provided in the "Foundation Design" and "Slab-On-Grade" sections of this report.

## WATER-SOLUBLE SULFATES

The Portland cement portion of concrete is subject to attack when exposed to water-soluble sulfates. Usually the two most common sources of exposure are from soil and marine environments.

The sources of natural sulfate minerals in soils include the sulfates of calcium, magnesium, sodium, and potassium. When these minerals interact and dissolve in subsurface water, a sulfate concentration is created, which will react with exposed concrete. Over time sulfate attack will destroy improperly proportioned concrete well before the end of its intended service life.

The water-soluble sulfate content of the onsite geologic materials was tested by California Test 417. The water-soluble sulfate content was determined to be less than 0.1% percentage by weight for the soils tested. Based on the most recent revision to American Concrete Institute (ACI) Standard 318, the sulfate exposure is considered to be negligible for geologic materials with less than 0.1% and Type I cement may be utilized for concrete foundations in contact with the site soils.

## METHANE ZONES

Based on review of the NavigateLA Website, developed by the City of Los Angeles, Bureau of Engineering, Department of Public Works, the subject site is located outside the limits of a City of Los Angeles Methane Zone and Methane Buffer Zone. A copy of this plan has been enclosed.

## GRADING GUIDELINES

### Site Preparation

- A thorough search should be made for possible underground utilities and/or structures. Any existing or abandoned utilities or structures located within the footprint of the proposed grading should be removed or relocated as appropriate.
- All vegetation, existing fill, and soft or disturbed geologic materials should be removed from the areas to receive controlled fill. All existing fill materials and any disturbed geologic materials resulting from grading operations shall be completely removed and properly recompacted prior to foundation excavation.
- Any vegetation or associated root system located within the footprint of the proposed structures should be removed during grading.
- Subsequent to the indicated removals, the exposed grade shall be scarified to a depth of six inches, moistened to optimum moisture content, and recompacted in excess of the minimum required comparative density.
- The excavated areas shall be observed by the geotechnical engineer prior to placing compacted fill.

### Recommended Overexcavation

The proposed building areas shall be excavated to a minimum depth of 5 feet below the existing grade, or 3 feet below the bottom of the proposed foundations, whichever is deeper. The excavation shall extend at least 3 feet beyond the edge of foundations or for a distance equal to the depth of fill below the foundations, whichever is greater. Based on the maximum depth of fill observed during exploration, it is anticipated that the thickness of the compacted fill pad may be up to 7 feet in thickness. It is very important that the positions of the proposed structures are accurately located so that the limits of the graded area are accurate and the grading operation proceeds efficiently.

**Compaction**

The City of Los Angeles Department of Building and Safety requires a minimum comparative compaction of 95 percent of the laboratory maximum density where the soils to be utilized in the fill have less than 15 percent finer than 0.005 millimeters. Fill soil having more than 15 percent finer than 0.005 millimeters may be compacted to a minimum of 90 percent of the maximum density. The majority of the native site soils would be subject to the 95 percent compaction rate.

All fill should be mechanically compacted in layers not more than 8 inches thick. It is recommended that fill materials are moisture conditioned to within 3 percent of the optimum moisture content before recompaction.

Field observation and testing shall be performed by a representative of the geotechnical engineer during grading to assist the contractor in obtaining the required degree of compaction and the proper moisture content. Where compaction is less than required, additional compactive effort shall be made with adjustment of the moisture content, as necessary, until a minimum of 95 percent compaction is obtained.

**Acceptable Materials**

The excavated onsite soils are considered satisfactory for reuse in compacted fills as long as any oversize material, debris and/or organic matter is removed. Cobbles exceeding 3 inches in dimension shall not be utilized in compacted fills. Any imported soil shall be observed and tested by the representative of the geotechnical engineer prior to use in fill areas. Imported materials should contain sufficient fines so as to be relatively impermeable and result in a stable subgrade when compacted. Any required import soil should have an expansion index less than 50. The water-soluble sulfate content of the import materials should be less than 0.1% percentage by weight.

Imported soil should be free from chemical or organic substances which could affect the proposed development. A competent professional should be retained in order to test imported materials and address environmental issues and organic substances which might affect the proposed development.

## Utility Trench Backfill

Utility trenches should be backfilled with controlled fill. The utility should be bedded with clean sands at least one foot over the crown. The remainder of the backfill may be onsite soil compacted to 95 percent of the laboratory maximum density. Utility trench backfill should be tested by representatives of this firm in accordance with the most recent revision of ASTM D-1557.

## Shrinkage

Shrinkage results when a volume of soil removed at one density is compacted to a higher density. A shrinkage factor between 5 and 15 percent should be anticipated when excavating and recompacting the existing fill and underlying native geologic materials on the site to an average comparative compaction of 95 percent.

## Weather Related Grading Considerations

When rain is forecast all fill that has been spread and awaits compaction shall be properly compacted prior to stopping work for the day or prior to stopping due to inclement weather. These fills, once compacted, shall have the surface sloped to drain to an area where water can be removed.

Temporary drainage devices should be installed to collect and transfer excess water to the street in non-erosive drainage devices. Water should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. Water should not be allowed to flow uncontrolled over any descending slope.

Work may start again, after a period of rainfall, once the site has been reviewed by a representative of this office. Any soils saturated by the rain shall be removed and aerated so that the moisture content will fall within three percent of the optimum moisture content.
Surface materials previously compacted before the rain shall be scarified, brought to the proper moisture content and recompacted prior to placing additional fill, if considered necessary by a representative of this firm.

**Geotechnical Observations and Testing During Grading**

Geotechnical observations and testing during grading are considered to be a continuation of the geotechnical investigation. It is critical that the geotechnical aspects of the project be reviewed by representatives of Geotechnologies, Inc. during the construction process. Compliance with the design concepts, specifications or recommendations during construction requires review by this firm during the course of construction. Any fill which is placed should be observed, tested, and verified if used for engineered purposes. Please advise this office at least twenty-four hours prior to any required site visit.

Proper compaction is necessary to reduce settlement of overlying improvements. Some settlement of compacted fill should be anticipated. Any utilities supported therein should be designed to accept differential settlement. Differential settlement should also be considered at the points of entry to the structure.

## FOUNDATION DESIGN

In order to limit the amount of differential settlement, it is recommended that the 15-story portion of the structure, which will be subject to the higher load demand, is supported on a mat foundation system. The remainder of the structure, which is not expected to exceed 6 stories in height, may be supported on a conventional foundation system. As an option, the entire structure may be supported on a mat foundation system. Recommendations to aid in the design of both foundation system alternatives are provided in the following sections.

Conventional foundations and mat foundations must bear in a newly placed engineered compacted fill pad. As shown on the enclosed Plot Plan, it is anticipated that the proposed structure will extend adjacent to the southern property line. Where new foundations will be built immediately adjacent to property lines, the recommended horizontal over-excavation for the creation of a compacted fill pad will not be possible. In areas where the horizontal over-excavation is not possible, the proposed foundations should be deepened as appropriate so the lateral capacity of the new foundation is derived from native alluvial soils and/or compacted fill.

### Mat Foundation for 15-Story Portion of the Structure

The mat foundation shall bear in in a newly built compacted fill pad. For design purposes, an average bearing pressure of up to 6,000 pounds per square foot, with locally higher pressures up to 11,000 pounds per square foot may be utilized in the mat foundation design.

The mat foundation may be designed utilizing a modulus of subgrade reaction of 250 pounds per cubic inch. This value is a unit value for use with a one-foot square footing. The modulus should be reduced in accordance with the following equation when used with larger foundations.

$$K = K_1 * [ (B + 1) / (2 * B) ]^2$$

where K = Reduced Subgrade Modulus
K1 = Unit Subgrade Modulus
B = Foundation Width (feet)

The bearing values indicated above are for the total of dead and frequently applied live loads, and may be increased by one third for short duration loading, which includes the effects of wind or seismic forces. Since the recommended bearing value is a net value, the weight of concrete in the foundations may be taken as 50 pounds per cubic foot and the weight of the soil backfill may be neglected when determining the downward load on the foundations.

**Conventional Foundations for 6-story Podium**

Continuous foundations may be designed for a bearing capacity of 2,500 pounds per square foot, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended compacted fill pad.

Column foundations may be designed for a bearing capacity of 3,000 pounds per square foot, and should be a minimum of 24 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended compacted fill pad.

The bearing capacity increase for each additional foot of width is 200 pounds per square foot. The bearing capacity increase for each additional foot of depth is 500 pounds per square foot. The maximum recommended bearing capacity is 5,000 pounds per square foot.

The bearing capacities indicated above are for the total of dead and frequently applied live loads, and may be increased by one third for short duration loading, which includes the effects of wind or seismic forces.

**Miscellaneous Foundations**

Conventional foundations for structures such as privacy walls or trash enclosures, which will not be rigidly connected to the proposed structure, may be deepened to bear in undisturbed alluvium, or they may bear in properly compacted fill. Continuous footings may be designed for a bearing capacity of 2,000 pounds per square foot, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended material. No bearing capacity increases are recommended.

Since the recommended bearing capacity is a net value, the weight of concrete in the foundations may be taken as 50 pounds per cubic foot and the weight of the soil backfill may be neglected when determining the downward load on the foundations.

**Lateral Design**

Resistance to lateral loading may be provided by friction acting at the base of foundations and by passive geologic pressure. An allowable coefficient of friction of 0.35 may be used with the dead load forces.

Passive geologic pressure for the sides of foundations poured against undisturbed or recompacted soil may be computed as an equivalent fluid having a density of 250 pounds per cubic foot with a maximum earth pressure of 2,000 pounds per square foot.

When combining passive and friction for lateral resistance, the passive component should be reduced by one third. A one-third increase in the passive value may be used for wind or seismic loads.

**Foundation Reinforcement**

All continuous foundations should be reinforced with a minimum of four #4 steel bars. Two should be placed near the top of the foundation, and two should be placed near the bottom.

**Foundation Settlement**

Settlement of the foundation system is expected to occur on initial application of loading. It is anticipated that total settlement on the order of 1½-inches will occur below the more heavily loaded central core portions of the mat foundation proposed beneath the 15-story structure. Settlement on the edges of the mat foundation is not expected to exceed ¾-inch.

The maximum settlement of a conventional column foundation below the 6-story portion of the structure is not expected to exceed 1-inch. Differential settlement between these conventional foundations is not expected to exceed ½-inch.

The differential settlement between the conventional foundations supporting the 6-story portion of the structure, and the edge of the mat foundation supporting the 15-story portion of the structure, is not expected to exceed ½-inch.

**Foundation Observations**

It is critical that all foundation excavations are observed by a representative of this firm to verify penetration into the recommended bearing materials. The observation should be performed prior to the placement of reinforcement. Foundations should be deepened to extend into satisfactory geologic materials, if necessary.

Foundation excavations should be cleaned of all loose soils prior to placing steel and concrete. Any required foundation backfill should be mechanically compacted, flooding is not permitted.

## RETAINING WALL DESIGN

The proposed structure will be built near the existing site grade. Therefore, the only retaining walls anticipated for the project would consist of miscellaneous retaining walls for elevator pits, planters, and potential vehicular ramps. Retaining walls may be designed as indicated below, depending on whether the walls will be restrained or cantilevered. Retaining wall foundations may be designed in accordance with the provisions of the "Foundation Design" section of this report.

Additional pressure should be added for a surcharge condition due to vehicular traffic or adjacent structures. A following section of this report provides recommendations to aid in determining the surcharge loads from these existing structures. For traffic surcharge, the upper 10 feet of any retaining wall adjacent to streets, driveways or parking areas should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot traffic surcharge. If the traffic is more than 10 feet from the retaining walls, the traffic surcharge may be neglected.

**Miscellaneous Cantilever Retaining Walls**

Retaining walls supporting a level backslope may be designed utilizing a triangular distribution of pressure. Cantilever retaining walls may be designed utilizing the following table:

| HEIGHT OF WALL (feet) | EQUIVALENT FLUID PRESSURE (pounds per cubic foot) |
|---|---|
| Up to 12 | 31 |

The lateral earth pressures recommended assume that a permanent drainage system will be installed so that external water pressure will not be developed against the walls. Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures.

**Restrained Retaining Walls**

Restrained retaining walls up to 12 feet in height and supporting a level back slope may be designed to resist a triangular distribution of earth pressure. It is recommended the walls be designed to resist the greater of the at-rest pressure, or the active pressure plus the seismic pressure, as discussed in the “Dynamic (Seismic) Earth Pressure” section below.

| RESTRAINED WALLS | | |
|---|---|---|
| | **AT-REST EARTH PRESSURE** | **ACTIVE EARTH PRESSURE *(To be Combined with Dynamic Seismic Earth Pressure)** |
| Height of Wall (Feet) | Triangular Distribution of Pressure (Pounds per Cubic Foot) | Triangular Distribution of Pressure (Pounds per Cubic Foot) |
| Up to 12 feet | 60 | 31* |

The lateral earth pressure recommended above for retaining walls assumes that a permanent drainage system will be installed so that external water pressure will not be developed against the walls. Also, where necessary, the retaining walls should be designed to accommodate any surcharge pressures that may be imposed by adjacent traffic and existing structures.

**Dynamic (Seismic) Earth Pressure**

Retaining walls exceeding 6 feet in height shall be designed to resist the additional earth pressure caused by seismic ground shaking. A triangular pressure distribution should be utilized for the additional seismic loads, with an equivalent fluid pressure of 24 pounds per cubic foot. When using the load combination equations from the building code, the seismic earth pressure should be combined with the lateral active earth pressure for analyses of restrained walls under seismic loading condition. The dynamic earth pressure may be omitted where the retaining wall is 6 feet in height or less.

**Surcharge from Adjacent Structures**

The following surcharge equation provided in the LADBS Information Bulletin Document No. P/BC 2020-83, may be utilized to determine the surcharge loads on basement walls and shoring system for existing structures located within the 1:1 (h:v) surcharge influence zone of the excavation and basement.

Resultant lateral force: $R = (0.3*P*h^2)/(x^2+h^2)$

Location of lateral resultant: $d = x*[(x^2/h^2+1)*\tan^{-1}(h/x)-(x/h)]$

where:

| | | |
|---|---|---|
| R | = | resultant lateral force measured in pounds per foot of wall width. |
| P | = | resultant surcharge loads of continuous or isolated footings measured in pounds per foot of length parallel to the wall. |
| x | = | distance of resultant load from back face of wall measured in feet. |
| h | = | depth below point of application of surcharge loading to bottom of wall footing measured in feet. |
| d | = | depth of lateral resultant below point of application of surcharge loading measure in feet. |
| $\tan^{-1}(h/x)$ | = | the angle in radians whose tangent is equal to h/x. |

The structural engineer and shoring engineer may use this equation to determine the surcharge loads based on the loading of the adjacent structures located within the surcharge influence zone.

**Retaining Wall Drainage**

All retaining walls shall be provided with a subdrain system in order to minimize the potential for future hydrostatic pressure buildup behind the proposed retaining walls. Subdrains may consist of four-inch diameter perforated pipes, placed with perforations facing down. The pipe shall be encased in at least one-foot of gravel around the pipe. The gravel shall be wrapped in filter fabric. The gravel may consist of three-quarter inch to one inch crushed rocks.

As an alternative to the standard perforated subdrain pipe and gravel drainage system, the use of gravel pockets and weepholes is an acceptable drainage method. Weepholes shall be a minimum of 2 inches in diameter, placed at 8 feet on center along the base of the wall. Gravel pockets shall be a minimum of 1 cubic foot in dimension, and may consist of three-quarter inch to one inch crushed rocks, wrapped in filter fabric. A collector pipe shall be installed to direct collected waters to a sump.

Certain types of subdrain pipe are not acceptable to the various municipal agencies, it is recommended that prior to purchasing subdrainage pipe, the type and brand is cleared with the proper municipal agencies. Subdrainage pipes should outlet to an acceptable location. Some municipalities do not allow the use of flat-drainage products, such as Miradrain. The use of such a product should be researched with the building official. The City of Los Angeles only allows the use of flat drainage products when in conjunction with a conventional perforated subdrain pipe and gravel, or gravel pockets and weepholes.

The lateral earth pressures recommended above for retaining walls assume that a permanent drainage system will be installed so that external water pressure will not be developed against the walls. If a drainage system is not provided, the walls should be designed to resist an external hydrostatic pressure due to water in addition to the lateral earth pressure. In any event, it is recommended that retaining walls be waterproofed.

**Sump Pump Design**

The purpose of the recommended retaining wall backdrainage system is to relieve hydrostatic pressure. Groundwater was not encountered during exploration, conducted to a depth of 55 feet below grade. Therefore, the only water which could affect the proposed retaining walls would be irrigation water and precipitation. Additionally, the proposed site grading is such that all drainage is directed to the street and the structure has been designed with adequate non-erosive drainage devices.

Based on these considerations the retaining wall backdrainage system is not expected to experience an appreciable flow of water, and in particular, no groundwater will affect it. However, for the purposes of design, a flow of 5 gallons per minute may be assumed.

**Waterproofing**

Moisture effecting retaining walls is one of the most common post construction complaints. Poorly applied or omitted waterproofing can lead to efflorescence or standing water inside the building. Efflorescence is a process in which a powdery substance is produced on the surface of the concrete by the evaporation of water. The white powder usually consists of soluble salts such as gypsum, calcite, or common salt. Efflorescence is common to retaining walls and does not affect their strength or integrity.

It is recommended that retaining walls be waterproofed. Waterproofing design and inspection of its installation is not the responsibility of the geotechnical engineer. A qualified waterproofing consultant should be retained in order to recommend a product or method which would provide protection to below grade walls.

**Retaining Wall Backfill**

Any required backfill should be mechanically compacted in layers not more than 8 inches thick, to at least 95 percent relative compaction, obtainable by the most recent revision of ASTM D 1557 method of compaction. Flooding should not be permitted. Compaction within 5 feet, measured horizontally, behind a retaining structure should be achieved by use of light weight, hand operated compaction equipment.

Proper compaction of the backfill will be necessary to reduce settlement of overlying walks and paving. Some settlement of required backfill should be anticipated, and any utilities supported therein should be designed to accept differential settlement.

**TEMPORARY EXCAVATIONS**

Excavations in the order of 5 to 7 feet in depth are anticipated for the recommended removal and recompaction. The excavations are expected to expose fill and dense native soils, which are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic or structures. Surcharged and unsurcharged vertical excavations may be performed to a maximum height of 7 feet with the aid of slot-cuts, as recommended in the following section. Temporary shoring will be required for vertical excavations exceeding a height of 7 feet.

Where sufficient space is available, temporary unsurcharged embankments could be cut at a uniform 1:1 slope gradient to a maximum depth of 20 feet. A uniform sloped excavation does not have a vertical component.

Where sloped embankments are utilized, the tops of the slopes should be barricaded to prevent vehicles and storage loads near the top of slope within a horizontal distance equal to the depth of the excavation. If the temporary construction embankments are to be maintained during the rainy season, berms are strongly recommended along the tops of the slopes to prevent runoff water from entering the excavation and eroding the slope faces. Water should not be allowed to pond on top of the excavation nor to flow towards it.

It is critical that the soils exposed in the cut slopes are observed by a representative of Geotechnologies, Inc. during excavation so that modifications of the slopes can be made if variations in the geologic material conditions occur. Many building officials require that temporary excavations should be made during the continuous observations of the geotechnical engineer. All excavations should be stabilized within 30 days of initial excavation.

**Slot Cutting**

Where a property line, a neighboring structure, the public right of way, or traffic will surcharge a temporary excavation, the slot cutting method may be utilized to maintain a stable excavation. The slot cutting method may also be utilized for the deepening of foundations. The height of the excavation is limited to 7 feet. The “A-B-C” slot-cutting procedure is recommended.

The slot cutting method employs the earth as a buttress and allows the earth excavation to proceed in phases. The initial excavation consists of excavating the “A” slots. Alternate “A” slots of 8 feet may be worked. The remaining earth buttresses (“B” and “C” slots) should be 8 feet in width for a combined intervening length of 16 feet. The “A” slots should be properly backfilled, before the “B” slots are excavated. The height of the slots shall not exceed 7 feet in height. Calculations indicating that slots 8 feet in width will be stable for the maximum recommended height of 7 feet, including a surcharge load from adjacent foundations and vehicular traffic, have been included in the appendix of this report.

## SHORING DESIGN

Conventional shoring may also be utilized to stabilize grading or foundation excavations. The following information on the design and installation of the shoring is as complete as possible at this time. It is suggested that Geotechnologies, Inc. review the final shoring plans and specifications prior to bidding or negotiating with a shoring contractor.

One method of shoring would consist of steel soldier piles, placed in drilled holes and backfilled with concrete. Based on the anticipated excavation depth, it is anticipated that the soldier piles will be designed for a cantilever condition.

**Soldier Piles**

Drilled cast-in-place soldier piles should be placed no closer than 2 diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the earth materials. For design purposes, an allowable passive value for the earth materials below the bottom plane of excavation may be assumed to be 600 pounds per square foot per foot, up to a maximum of 6,000 pounds per square foot. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed earth materials.

The frictional resistance between the soldier piles and retained earth material may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.4 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 500 pounds per square foot. The minimum depth of embedment for shoring piles is 5 feet below the bottom of the footing excavation or 5 feet below the bottom of excavated plane whichever is deeper.

The proposed shoring pile excavations are not anticipated to encounter water. Caving may be experienced while drilling within the granular native soils. If caving is experienced, it will be necessary to utilize casing to maintain open pile shafts. If casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet. Large sized materials should also be anticipated during drilling (i.e. cobbles).

**Lagging**

Soldier piles and anchors should be designed for the full anticipated pressures. Due to arching in the geologic materials, the pressure on the lagging will be less. It is recommended that the lagging should be designed for the full design pressure but is limited to a maximum of 400 pounds per square foot. It is recommended that a representative of this firm observe the installation of lagging to insure uniform support of the excavated embankment.

**Lateral Pressures**

Cantilevered shoring supporting a level backslope may be designed utilizing a triangular distribution of pressure as indicated in the following table:

| HEIGHT OF SHORING "H" (feet) | EQUIVALENT FLUID PRESSURE (pounds per cubic foot) |
|---|---|
| Up to 12 | 28 |

Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination. Additional active pressure should be applied where the shoring will be surcharged by adjacent traffic or structures.

**Deflection**

It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized that some deflection will occur. It is recommended that shoring deflection be limited to ½ inch at the top of the shored embankment where a structure is within a 1:1 plane projected up from the base of the excavation. A maximum deflection of 1-inch has been allowed, provided there are no structures within a 1:1 plane drawn upward from the base of the excavation. If greater deflection occurs during construction, additional bracing may be necessary to minimize settlement of adjacent buildings and utilities in adjacent street and alleys. If desired to reduce the deflection, a greater active pressure could be used in the shoring design.

**Monitoring**

Because of the depth of the excavation, some means of monitoring the performance of the shoring system is suggested. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all soldier piles and the lateral movement along the entire lengths of selected soldier piles. Also, some means of periodically checking the load on selected anchors will be necessary, where applicable.

Some movement of the shored embankments should be anticipated as a result of the relatively deep excavation. It is recommended that photographs of the existing buildings on the adjacent properties be made during construction to record any movements for use in the event of a dispute.

**Shoring Observations**

It is critical that the installation of shoring is observed by a representative of Geotechnologies, Inc. Many building officials require that shoring installation should be performed during continuous observation of a representative of the geotechnical engineer. The observations insure that the recommendations of the geotechnical report are implemented and so that modifications of the recommendations can be made if variations in the geologic material or groundwater conditions warrant. The observations will allow for a report to be prepared on the installation of shoring for the use of the local building official, where necessary.

## SLABS ON GRADE

**Concrete Slabs-on Grade**

Concrete floor slabs-on-grade and outdoor concrete flatwork should be a minimum of 4 inches in thickness. Slabs-on-grade should be cast over undisturbed alluvial soils or properly controlled fill materials. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted 95 percent of the maximum dry density.

## Design of Slabs That Receive Moisture-Sensitive Floor Coverings

Geotechnologies, Inc. does not practice in the field of moisture vapor transmission evaluation and mitigation. Therefore, it is recommended that a qualified consultant be engaged to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. The qualified consultant should provide recommendations for mitigation of potential adverse impacts of moisture vapor transmission on various components of the structure.

Where dampness would be objectionable, it is recommended that the floor slabs should be waterproofed. A qualified waterproofing consultant should be retained in order to recommend a product or method which would provide protection for concrete slabs-on-grade.

All concrete slabs-on-grade should be supported on vapor retarder. The design of the slab and the installation of the vapor retarder should comply with the most recent revisions of ASTM E 1643 and ASTM E 1745. The vapor retarder should comply with ASTM E 1745 Class A requirements.

Where a vapor retarder is used, a low-slump concrete should be used to minimize possible curling of the slabs. The barrier can be covered with a layer of trimmable, compactible, granular fill, where it is thought to be beneficial. See ACI 302.2R-32, Chapter 7 for information on the placement of vapor retarders and the use of a fill layer.

## Concrete Crack Control

The recommendations presented in this report are intended to reduce the potential for cracking of concrete slabs-on-grade due to settlement. However even where these recommendations have been implemented, foundations, stucco walls and concrete slabs-on-grade may display some cracking due to minor soil movement and/or concrete shrinkage. The occurrence of concrete cracking may be reduced and/or controlled by limiting the slump of the concrete used, proper concrete placement and curing, and by placement of crack control joints at reasonable intervals, in particular, where re-entrant slab corners occur.

For standard control of concrete cracking, a maximum crack control joint spacing of 15 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended. The crack control joints should be installed as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. Construction joints should be designed by a structural engineer.

Complete removal of the existing fill soils beneath outdoor flatwork such as walkways or patio areas, is not required, however, due to the rigid nature of concrete, some cracking, a shorter design life and increased maintenance costs should be anticipated. In order to provide uniform support beneath the flatwork it is recommended that a minimum of 12 inches of the exposed subgrade beneath the flatwork be scarified and recompacted to 95 percent relative compaction.

**Slab Reinforcing**

Concrete slabs-on-grade should be reinforced with a minimum of #4 steel bars on 16-inch centers each way. Outdoor flatwork should be reinforced with a minimum of #3 steel bars on 24-inch centers each way.

## PAVEMENTS

Prior to placing paving, the existing grade should be scarified to a depth of 12 inches, moistened as required to obtain optimum moisture content, and recompacted to 95 relative compaction, as determined by the most recent revision of ASTM D 1557. The client should be aware that removal of all existing fill in the area of new paving is not required, however, pavement constructed in this manner will most likely have a shorter design life and increased maintenance costs. The following pavement sections are recommended:

| Service | Asphalt Pavement Thickness Inches | Base Course Inches |
|---|---|---|
| Passenger Cars | 3 | 4 |
| Moderate Truck | 4 | 6 |
| Heavy Truck | 5 | 8 |

Aggregate base should be compacted to a minimum of 95 percent of the most recent revision of ASTM D 1557 laboratory maximum dry density. Base materials should conform to Sections 200-2.2 or 200-2.4 of the “Standard Specifications for Public Works Construction”, (Green Book), latest edition.

Concrete paving may also be utilized for the project. For concrete paving sections to be subject to passenger cars and medium truck traffic, concrete paving shall be a minimum of 6 inches in thickness, and shall be underlain by 4 inches of aggregate base. For heavy truck traffic, concrete paving shall be a minimum of 7½ inches in thickness, and shall be underlain by 4 inches of aggregate base. For standard crack control maximum expansion joint spacing of 15 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended. Concrete paving should be reinforced with a minimum of #3 steel bars on 24-inch centers each way.

The performance of pavement is highly dependent upon providing positive surface drainage away from the edges. Ponding of water on or adjacent to pavement can result in saturation of the subgrade materials and subsequent pavement distress.

## SITE DRAINAGE

Proper surface drainage is critical to the future performance of the project. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the designed engineering properties. Proper site drainage should be maintained at all times.

All site drainage, with the exception of any required to disposed of onsite by stormwater regulations, should be collected and transferred to the street in non-erosive drainage devices. The proposed structure should be provided with roof drainage. Discharge from downspouts, roof drains and scuppers should not be permitted on unprotected soils within five feet of the building perimeter. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. Drainage should not be allowed to flow uncontrolled over any descending slope. Planters which are located within a distance equal to the depth of a retaining wall should be sealed to prevent moisture adversely affecting the wall. Planters which are located within five feet of a foundation should be sealed to prevent moisture affecting the earth materials supporting the foundation.

## STORMWATER DISPOSAL

### Introduction

Recently regulatory agencies have been requiring the disposal of a certain amount of stormwater generated on a site by infiltration into the site soils. Increasing the moisture content of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the designed engineering properties. This means that any overlying structure, including buildings, pavements and concrete flatwork, could sustain damage due to saturation of the subgrade soils. Structures serviced by subterranean levels could be adversely impacted by stormwater disposal by increasing the design fluid pressures on retaining walls and causing leaks in the walls. Proper site drainage is critical to the performance of any structure in the built environment.

### Percolation Testing

Percolation testing was conducted in Boring B2, which was drilled to a depth of 55 feet below the existing grade. At the completion of drilling, a 2-inch diameter casing was placed within the center of the borehole for the purpose of conducting percolation testing. The casing consisted of a slotted PVC pipe within the lower 20 feet of the borehole, and solid PVC pipe to the top of the borehole. A sand pack consisting of #3 Monterey Sand was poured into the annular space around the slotted portion of the casing. A 1-foot thick, hydrated bentonite seal was placed over the sand and drill cuttings were placed to the ground surface.

Prior to testing, the borehole was filled with water for the purpose of pre-soaking for 2 hours. After presoaking, the borehole was refilled with water, and the rate of drop in the water level was measured. The percolation test readings were recorded a minimum of 8 times, or until a stabilized rate of drop was obtained, whichever occurred first. The percolation testing was performed within the native alluvial soils encountered between depths of 35 and 55 feet.

Based on results of the percolation testing, a percolation rate of 90 inches per hour may be assign to the native soils encountered at depths between 35 and 55 feet below the existing grade. No safety factors or reduction factors have been applied to this percolation rate. The civil engineer must apply the required factors of safety to the percolation rate provided herein.

At the completion of the percolation testing, the PVC casing was completely removed from the percolation testing borings, and the resulting hole was backfilled with on-site soils to the ground surface.

## **The Proposed System**

It anticipated that the proposed infiltration system will consist of a MaxWell dry well system, to be installed within a driveway located to the northeast of the proposed structure. The proposed drywell location is shown in the enclosed Plot Plan. Preliminarily, it is anticipated that the dry well will be designed to infiltrate between depths of 20 and 58 feet below the ground surface.

The proposed stormwater infiltration system, and its general location, is acceptable to this firm provided that the recommendations presented herein are implemented. This office shall review the final drainage plan and infiltration system details prior to construction, to evaluate whether the intent of the recommendations presented herein are satisfied.

**Recommendations**

Based on the results of the exploration, testing and research, it is the finding of this firm that on-site stormwater infiltration is feasible for the site. A suitable stormwater infiltration system may consist of a drywell system. The potential stormwater infiltration system is not expected to impact the proposed development, or existing neighboring development, provided the advice and recommendations presented herein are implemented during design and construction.

If possible, it is recommended that the potential infiltration drywell system be installed outside the footprint of the proposed structure. However, if necessary, the drywell may also be installed within the footprint of the proposed structure. In the event that the drywell system has to be installed within the structure's footprint, it is recommended that it is installed within the 5-story podium, centered in between surrounding conventional foundations.

It is recommended that the edge of any potential drywell system shall maintain a minimum horizontal setback of 15 feet away from private property lines. If allowed by the municipality, the drywell may be installed immediately adjacent to the public right-of-way.

It is recommended that stormwater infiltration occurs in the native alluvial soils located at, or deeper, than 20 feet below the existing site grade. Soils located within the upper 20 feet should not become wet or saturated as a result of a drywell. It is anticipated that a settling chamber will be installed within this upper soil layer; therefore, the seams and bottom of the settling chamber should be adequately sealed to prevent infiltration at this zone. Depending on the final location, the settling chamber of the drywell may be surcharged by proposed adjacent foundations, in which case the chamber should be designed to withstand this additional surcharge load. The final location of the proposed drywells shall be reviewed and approved by this office prior to construction.

State regulations require that the bottom of infiltration units maintain a minimum vertical distance of 10 feet above the groundwater level. Groundwater was not encountered at the site during exploration, conducted to a depth of 55 feet below grade. The historically-highest groundwater level for the site is reported at a depth of 145 feet. Therefore, it is recommended that the drywell system does not extend deeper than 135 feet below the existing grade.

The subject site is not located in an area considered susceptible to liquefaction. The proposed stormwater infiltration system will not be located in hillside area, and no slopes are nearby. The onsite soils are in the very low expansion range, and are not susceptible to significant hydroconsolidation.

It is recommended that the design team, including the structural engineer, waterproofing consultant, plumbing engineer, environmental engineer and landscape architect be consulted in regard to the design and construction of infiltration systems. The design and construction of stormwater infiltration systems is not the responsibility of the geotechnical engineer. However, based on the experience of this firm, it is recommended that several aspects of the use of such facilities should be considered by the design and construction team:

- All infiltration devices should be provided with overflow protection. Once the device is full of water, additional water flowing to the device should be diverted to another acceptable disposal area or disposed offsite in an acceptable manner.

- All connections associated with stormwater infiltration devices should be sealed and water-tight. Water leaking into the subgrade soils can lead to loss of strength, piping, erosion, settlement and/or expansion of the effected earth materials.

- Excavations proposed for the installation of stormwater facilities should comply with the “Temporary Excavations” sections of this report as well as CalOSHA Regulations where applicable.

- Cobbles will be encountered during drilling of the drywell. Also, caving may be experienced during drilling. Where caving occurs, it will be necessary to utilize casing to maintain an open shaft.

## DESIGN REVIEW

Engineering of the proposed project should not begin until approval of the geotechnical report by the Building Official is obtained in writing. Significant changes in the geotechnical recommendations may result during the building department review process.
It is recommended that the geotechnical aspects of the project be reviewed by this firm during the design process. This review provides assistance to the design team by providing specific recommendations for particular cases, as well as review of the proposed construction to evaluate whether the intent of the recommendations presented herein are satisfied.

## CONSTRUCTION MONITORING

Geotechnical observations and testing during construction are considered to be a continuation of the geotechnical investigation. It is critical that this firm review the geotechnical aspects of the project during the construction process. Compliance with the design concepts, specifications or recommendations during construction requires review by this firm during the course of construction. All foundations should be observed by a representative of this firm prior to placing concrete or steel. Any fill which is placed should be observed, tested, and verified if used for engineered purposes. Please advise Geotechnologies, Inc. at least twenty-four hours prior to any required site visit.

If conditions encountered during construction appear to differ from those disclosed herein, notify Geotechnologies, Inc. immediately so the need for modifications may be considered in a timely manner.

It is the responsibility of the contractor to ensure that all excavations and trenches are properly sloped or shored. All temporary excavations should be cut and maintained in accordance with applicable OSHA rules and regulations.

**EXCAVATION CHARACTERISTICS**

The exploration performed for this investigation is limited to the geotechnical excavations described. Direct exploration of the entire site would not be economically feasible. The owner, design team and contractor must understand that differing excavation and drilling conditions may be encountered based on boulders, gravel, oversize materials, groundwater and many other conditions. Fill materials, especially when they were placed without benefit of modern grading codes, regularly contain materials which could impede efficient grading and drilling. Southern California sedimentary bedrock is known to contain variable layers which reflect differences in depositional environment. Such layers may include abundant gravel, cobbles and boulders. Similarly bedrock can contain concretions. Concretions are typically lenticular and follow the bedding. They are formed by mineral deposits. Concretions can be very hard. Excavation and drilling in these areas may require full size equipment and coring capability. The contractor should be familiar with the site and the geologic materials in the vicinity.

**CLOSURE AND LIMITATIONS**

The purpose of this report is to aid in the design and completion of the described project. Implementation of the advice presented in this report is intended to reduce certain risks associated with construction projects. The professional opinions and geotechnical advice contained in this report are sought because of special skill in engineering and geology and were prepared in accordance with generally accepted geotechnical engineering practice. Geotechnologies, Inc. has a duty to exercise the ordinary skill and competence of members of the engineering profession. Those who hire Geotechnologies, Inc. are not justified in expecting infallibility, but can expect reasonable professional care and competence.

The recommendations of this report pertain only to the site investigated and are based upon the assumption that the geologic conditions do not deviate from those disclosed in the investigation. If any variations are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geotechnologies, Inc. should be notified so that supplemental recommendations can be prepared.

This report is issued with the understanding that it is the responsibility of the owner, or the owner's representatives, to ensure that the information and recommendations contained herein are brought to the attention of the project architect and engineer and are incorporated into the plans. The owner is also responsible to see that the contractor and subcontractors carry out the geotechnical recommendations during construction.

The findings of this report are valid as of the date of this report. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside control of this firm. Therefore, this report is subject to review and should not be relied upon after a period of three years.

Geotechnical observations and testing during construction is considered to be a continuation of the geotechnical investigation. It is, therefore, most prudent to employ the consultant performing the initial investigative work to provide observation and testing services during construction. This practice enables the project to flow smoothly from the planning stages through to completion.

Should another geotechnical firm be selected to provide the testing and observation services during construction, that firm should prepare a letter indicating their assumption of the responsibilities of geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for review. The letter should acknowledge the concurrence of the new geotechnical engineer with the recommendations presented in this report.

## EXCLUSIONS

Geotechnologies, Inc. does not practice in the fields of methane gas, radon gas, environmental engineering, waterproofing, dewatering organic substances or the presence of corrosive soils or wetlands which could affect the proposed development including mold and toxic mold. Nothing in this report is intended to address these issues and/or their potential effect on the proposed development. A competent professional consultant should be retained in order to address environmental issues, waterproofing, organic substances and wetlands which might affect the proposed development.

## GEOTECHNICAL TESTING

### Classification and Sampling

The soil is continuously logged by a representative of this firm and classified by visual examination in accordance with the Unified Soil Classification system. The field classification is verified in the laboratory, also in accordance with the Unified Soil Classification System. Laboratory classification may include visual examination, Atterberg Limit Tests and grain size distribution. The final classification is shown on the excavation logs.

Samples of the geologic materials encountered in the exploratory excavations were collected and transported to the laboratory. Undisturbed samples of soil are obtained at frequent intervals. Unless noted on the excavation logs as an SPT sample, samples acquired while utilizing a hollow-stem auger drill rig are obtained by driving a thin-walled, California Modified Sampler with successive 30-inch drops of a 140-pound automatic-trip hammer. The soil is retained in brass rings of 2.50 inches outside diameter and 1.00 inch in height. The central portion of the samples are stored in close fitting, waterproof containers for transportation to the laboratory. Samples noted on the excavation logs as SPT samples are obtained in general accordance with the most recent revision of ASTM D 1586. Samples are retained for 30 days after the date of the geotechnical report.

**Moisture and Density Relationships**

The field moisture content and dry unit weight are determined for each of the undisturbed soil samples, and the moisture content is determined for SPT samples in general accordance with the most recent revision of ASTM D 4959 or ASTM D 4643. This information is useful in providing a gross picture of the soil consistency between exploration locations and any local variations. The dry unit weight is determined in pounds per cubic foot and shown on the “Excavation Logs”, A-Plates. The field moisture content is determined as a percentage of the dry unit weight.

**Direct Shear Testing**

Shear tests are performed in general accordance with the most recent revision of ASTM D 3080 with a strain controlled, direct shear machine manufactured by Soil Test, Inc. or a Direct Shear Apparatus manufactured by GeoMatic, Inc. The rate of deformation is approximately 0.025 inches per minute. Each sample is sheared under varying confining pressures in order to determine the Mohr-Coulomb shear strength parameters of the cohesion intercept and the angle of internal friction. Samples are generally tested in an artificially saturated condition. Depending upon the sample location and future site conditions, samples may be tested at field moisture content. The results are plotted on the "Shear Test Diagram," B-Plates.

The most recent revision of ASTM 3080 limits the particle size to 10 percent of the diameter of the direct shear test specimen. The sheared sample is inspected by the laboratory technician running the test. The inspection is performed by splitting the sample along the sheared plane and observing the soils exposed on both sides. Where oversize particles are observed in the shear plane, the results are discarded and the test run again with a fresh sample.

**Consolidation Testing**

Settlement predictions of the soil's behavior under load are made on the basis of the consolidation tests in general accordance with the most recent revision of ASTM D 2435. The consolidation apparatus is designed to receive a single one-inch high ring. Loads are applied in several increments in a geometric progression, and the resulting deformations are recorded at selected time intervals. Porous stones are placed in contact with the top and bottom of each specimen to permit addition and release of pore fluid. Samples are generally tested at increased moisture content to determine the effects of water on the bearing soil. The normal pressure at which the water is added is noted on the drawing. Results are plotted on the "Consolidation Test," C-Plates.

**Expansion Index Testing**

The expansion tests performed on the remolded samples are in accordance with the Expansion Index testing procedures, as described in the most recent revision of ASTM D 4829. The soil sample is compacted into a metal ring at a saturation degree of 50 percent. The ring sample is then placed in a consolidometer, under a vertical confining pressure of 1 lbf/square inch and inundated with distilled water. The deformation of the specimen is recorded for a period of 24 hour or until the rate of deformation becomes less than 0.0002 inches/hour, whichever occurs first. The expansion index, EI, is determined by dividing the difference between final and initial height of the ring sample by the initial height, and multiplied by 1,000. Results are presented in Plate D of this report.

**Laboratory Compaction Characteristics**

The maximum dry unit weight and optimum moisture content of a soil are determined in general accordance with the most recent revision of ASTM D 1557. A soil at a selected moisture content is placed in five layers into a mold of given dimensions, with each layer compacted by 25 blows of a 10 pound hammer dropped from a distance of 18 inches subjecting the soil to a total compactive effort of about 56,000 pounds per cubic foot. The resulting dry unit weight is determined. The procedure is repeated for a sufficient number of moisture contents to establish a relationship between the dry unit weight and the water content of the soil. The data when plotted represent a curvilinear relationship known as the compaction curve. The values of optimum moisture content and modified maximum dry unit weight are determined from the compaction curve. Results are presented in Plate D of this report.

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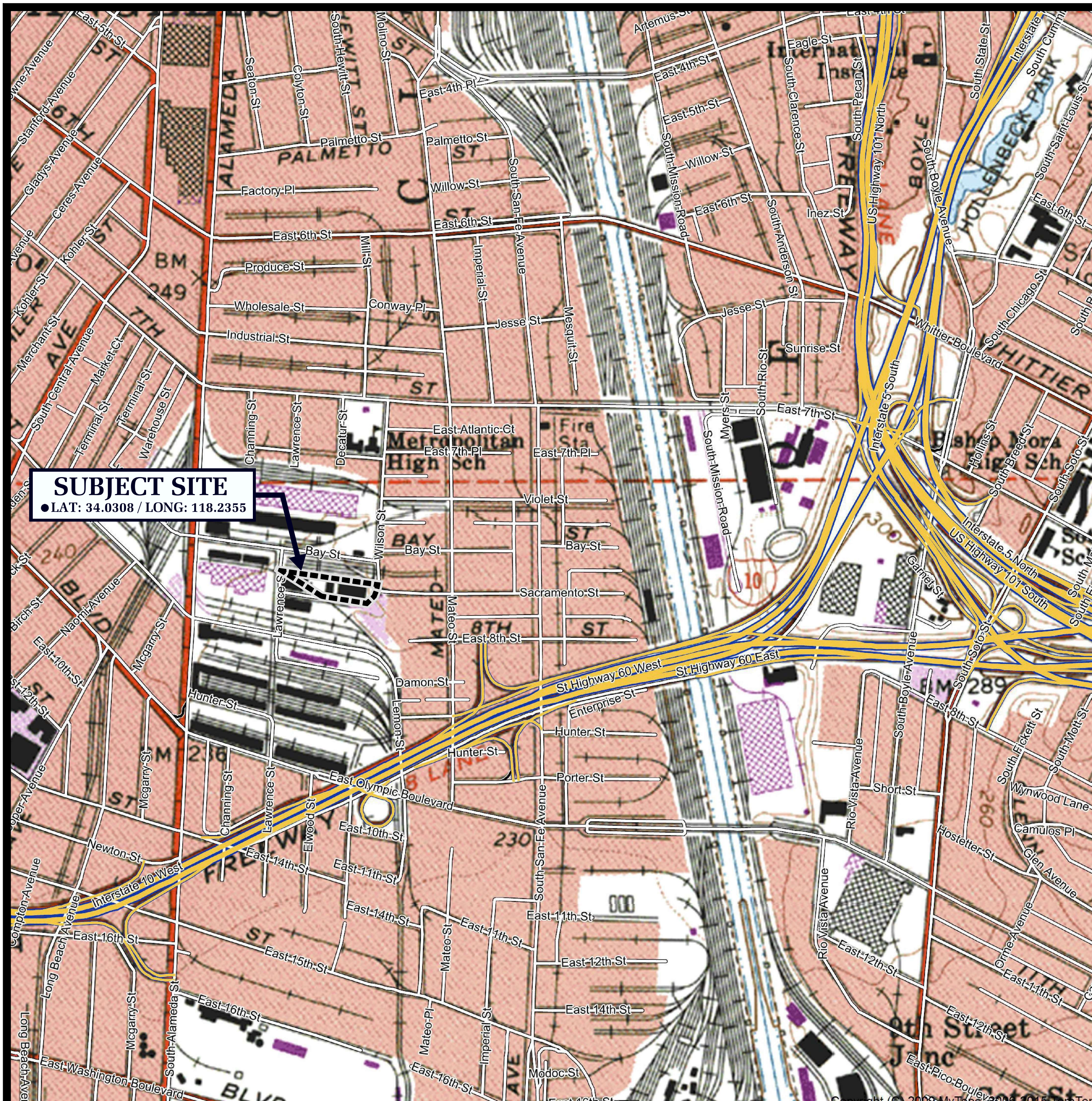



SCALE 1:12000

0.0 0.1 0.2 0.3 0.4 0.5 0.6
MILES

0 1000 2000 3000
FFFT

N

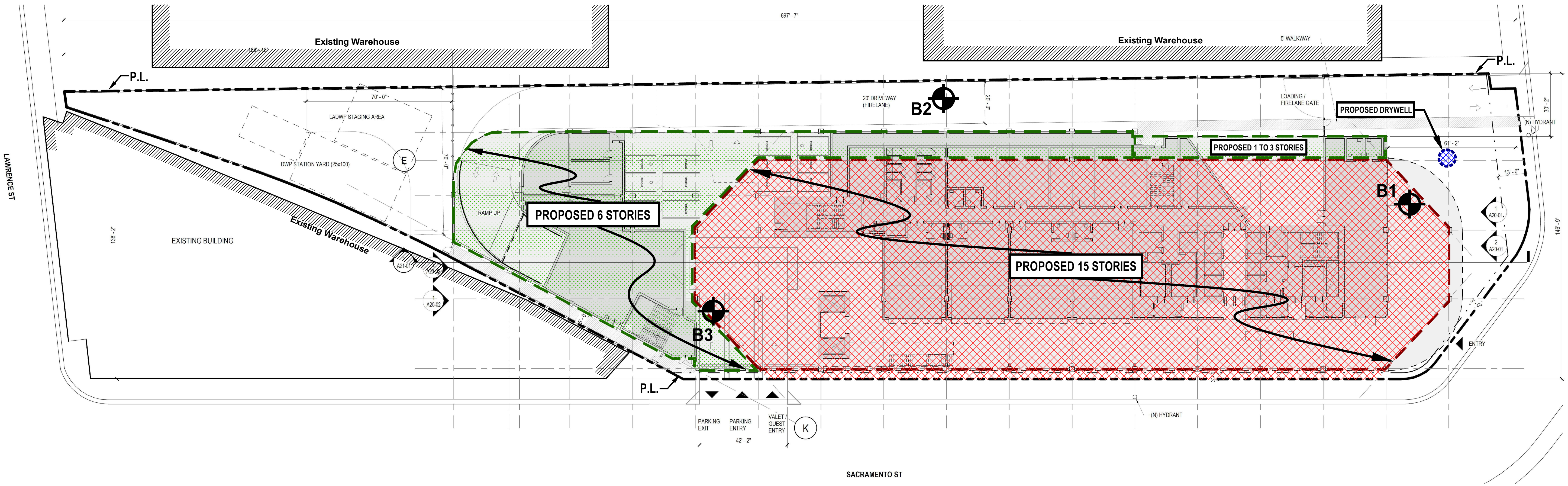
**REFERENCE:** U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES,
LOS ANGELES, CA QUADRANGLE

# VICINITY MAP

**Geotechnologies, Inc.**
Consulting Geotechnical Engineers

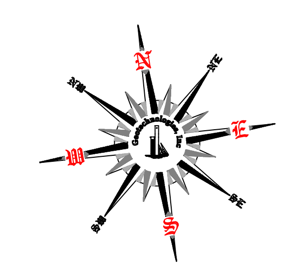
SCD 1811 SACRAMENTO, LLC

FILE NO. 21971




## LEGEND

B3 — LOCATION & NUMBER OF BORING



SCALE IN FEET

0 30 50 100

REFERENCE: ARCHITECTURAL SITE PLAN BY PERKINS & WILL NOT DATED

**Geotechnologies, Inc.**
*Consulting Geotechnical Engineers*

# PLOT PLAN

SCD 1811 SACRAMENTO, LLC

File No.: 21971

Date: August 2022


**LEGEND**

Qg: Surficial Sediments - alluvial sand and clay of valley areas
Qa: Surficial Sediments - alluvium: unconsolidated floodplain deposits of silt, sand and gravel
Qoa: Older Surficial Sediments - remnants of older weakly consolidated alluvial deposits of gravel, sand & silt

Folds - arrow on axial trace of fold indicates direction of plunge
Fault - dashed where indefinite or inferred, dotted where concealed, queried where existence is doubtful

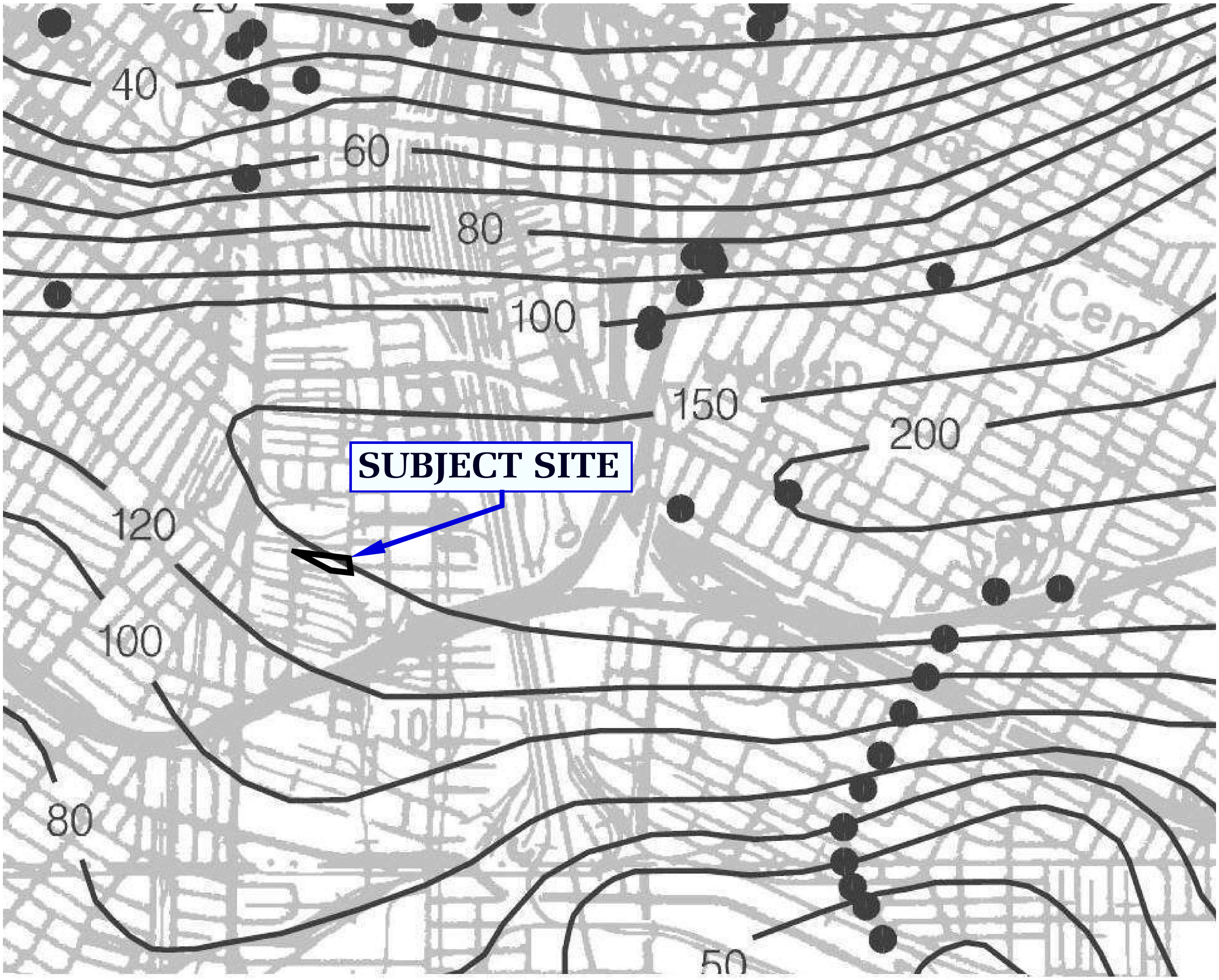
REFERENCE: DIBBLEE, T.W., (1989) GEOLOGIC MAP OF THE LOS ANGELES QUADRANGLE (#DF-22)

# LOCAL GEOLOGIC MAP - DIBBLEE

**Geotechnologies, Inc.**
*Consulting Geotechnical Engineers*

SCD 1811 SACRAMENTO, LLC

FILE NO. 21971




ONE MILE

SCALE

20 Depth to groundwater in feet

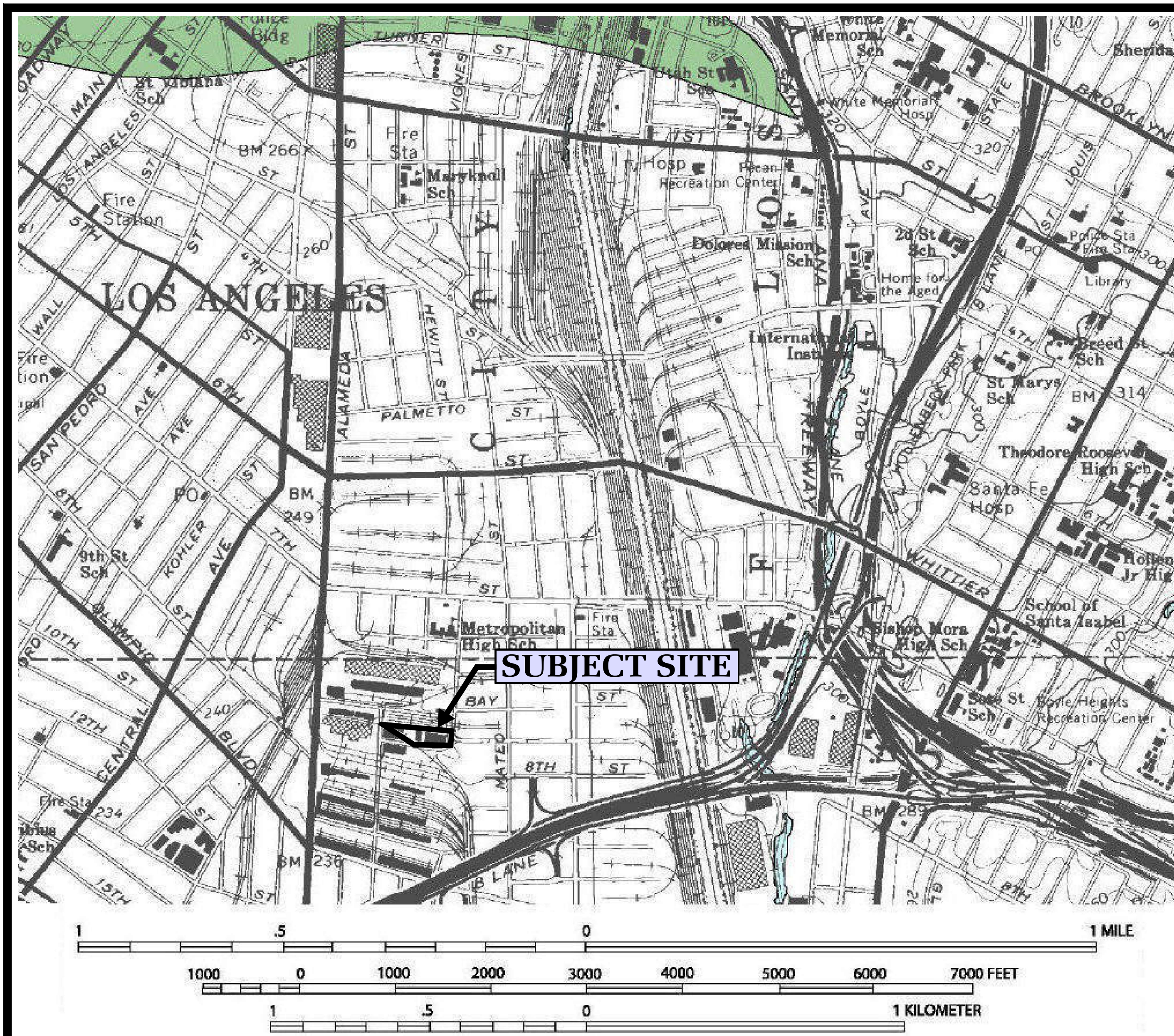
REFERENCE: CDMG, SEISMIC HAZARD ZONE REPORT, 029
LOS ANGELES 7.5 - MINUTE QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA (1998, REVISED 2006)

# HISTORICALLY HIGHEST GROUNDWATER LEVELS

**Geotechnologies, Inc.**
Consulting Geotechnical Engineers

SCD 1811 SACRAMENTO, LLC

FILE NO. 21971




LIQUEFACTION AREA

**REFERENCE:** SEISMIC HAZARD ZONES, LOS ANGELES QUADRANGLE OFFICIAL MAP (CDMG, 1999)

# SEISMIC HAZARD ZONE MAP

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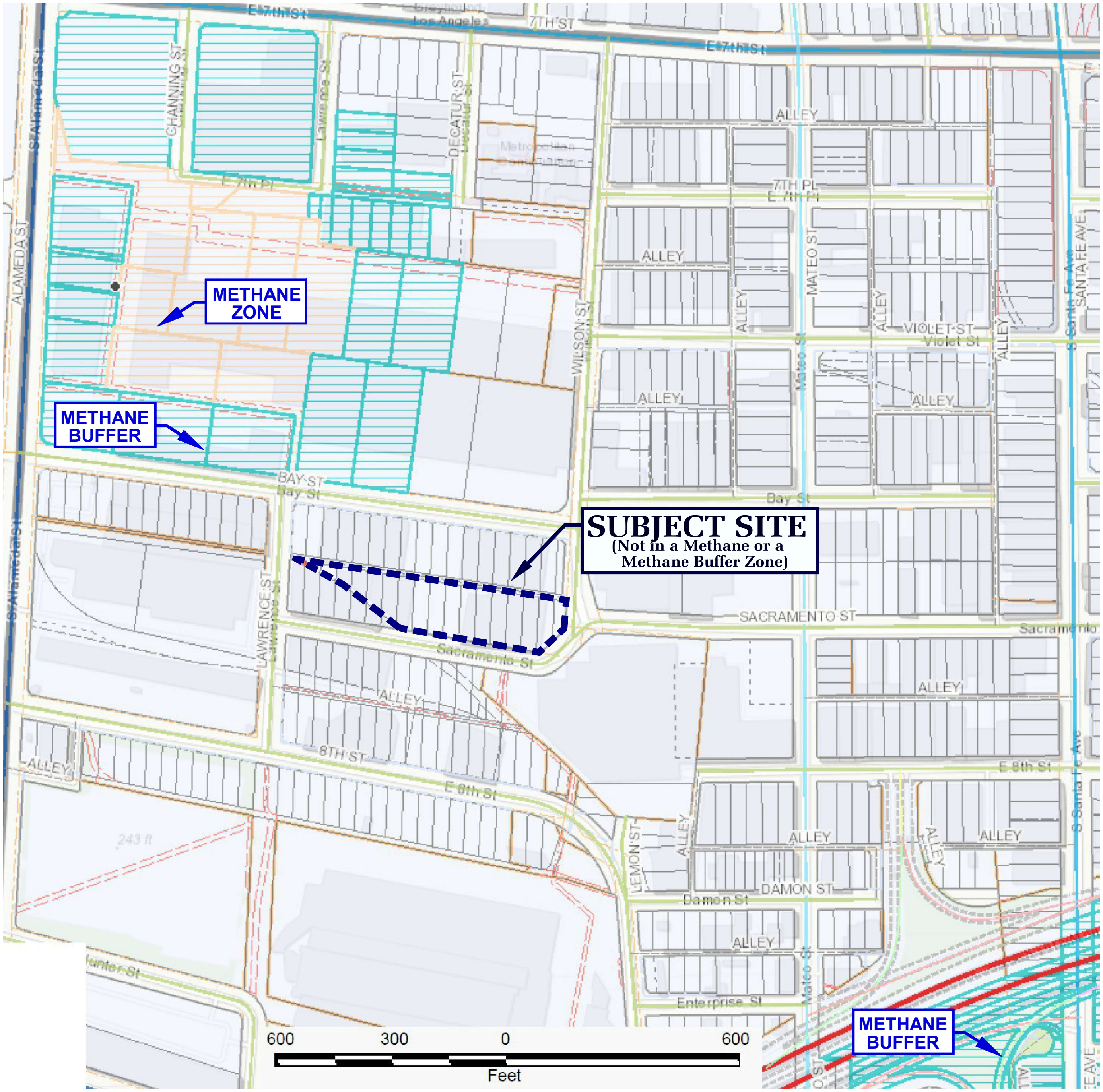
Consulting Geotechnical Engineers

SCD 1811 SACRAMENTO, LLC

FILE NO. 21971




REFERENCE: http://navigatela.lacity.org/NavigateLA/

# METHANE ZONE RISK MAP

**Geotechnologies, Inc.**
*Consulting Geotechnical Engineers*

SCD 1811 SACRAMENTO, LLC

FILE NO. 21971

# BORING LOG NUMBER 1

**SCD 1811 Sacramento, LLC** **Date: 12/10/21**

**File No. 21971** **Method: 8-Inch Diameter Hollow Stem Auger**

ln

| Sample Depth ft. | Blows per ft. | Moisture content % | Dry Density p.c.f. | Depth in feet | USCS Class. | Description<br>Surface Conditions: Asphalt For Parking |
|---|---|---|---|---|---|---|
| | | | | 0 -- | | 5 Inch Asphalt, 3 Inch Base |
| | | | | - | | |
| | | | | 1 -- | | |
| | | | | - | | FILL: Silty Sand, dark brown, moist, medium dense, fine grained |
| | | | | 2 -- | | |
| | | | | - | | |
| 3 | 16 | 1.6 | 104.7 | 3 -- | | |
| | | | | - | SP/ML | NATIVE SOILS: Sand to Sandy Silt, dark and yellowish brown, moist, medium dense, stiff, fine grained |
| | | | | 4 -- | | |
| | | | | - | | |
| 5 | 16 | 21.1 | 88.3 | 5 -- | | |
| | | | | - | | |
| | | | | 6 -- | | |
| | | | | - | | |
| | | | | 7 -- | | |
| | | | | - | | |
| | | | | 8 -- | | |
| | | | | - | | |
| | | | | 9 -- | | |
| | | | | - | | |
| 10 | 34 | 3.4 | 109.6 | 10 -- | | |
| | | | | - | SP/SM | Sand to Silty Sand, yellowish and grayish brown, moist, medium dense, fine to medium grained |
| | | | | 11 -- | | |
| | | | | - | | |
| | | | | 12 -- | | |
| | | | | - | | |
| | | | | 13 -- | | |
| | | | | - | | |
| | | | | 14 -- | | |
| | | | | - | | |
| 15 | 51 | 1.9 | 106.6 | 15 -- | | |
| | | | | - | SP | Sand, yellowish brown, moist, dense, fine to medium grained |
| | | | | 16 -- | | |
| | | | | - | | |
| | | | | 17 -- | | |
| | | | | - | | |
| | | | | 18 -- | | |
| | | | | - | | |
| | | | | 19 -- | | |
| | | | | - | | |
| 20 | 56 | 5.6 | 114.5 | 20 -- | | |
| | | | | - | | |
| | | | | 21 -- | | |
| | | | | - | | |
| | | | | 22 -- | | |
| | | | | - | | |
| | | | | 23 -- | | |
| | | | | - | | |
| | | | | 24 -- | | |
| | | | | - | | |
| 25 | 90 | 1.8 | 109.4 | 25 -- | | |
| | | | | - | | yellowish and dark brown, very dense, fine grained |

# BORING LOG NUMBER 1

**SCD 1811 Sacramento, LLC**

**File No. 21971**

ln

| Sample Depth ft. | Blows per ft. | Moisture content % | Dry Density p.c.f. | Depth in feet | USCS Class. | Description |
|---|---|---|---|---|---|---|
| | | | | - | | |
| | | | | 26 -- | | |
| | | | | - | | |
| | | | | 27 -- | | |
| | | | | - | | |
| | | | | 28 -- | | |
| | | | | - | | |
| | | | | 29 -- | | |
| | | | | - | | |
| 30 | 35 | 1.9 | 116.2 | 30 -- | | |
| | 50/4" | | | - | | Total Depth: 30 Feet |
| | | | | 31 -- | | No Water |
| | | | | - | | Fill To 3 Feet |
| | | | | 32 -- | | |
| | | | | - | | NOTE: The stratification lines represent the approximate |
| | | | | 33 -- | | boundary between earth types; the transition may be gradual. |
| | | | | - | | |
| | | | | 34 -- | | Used 8-inch diameter Hollow-Stem Auger |
| | | | | - | | 140-lb. Automatic Hammer, 30-inch drop |
| | | | | 35 -- | | Modified California Sampler used unless otherwise noted |
| | | | | - | | |
| | | | | 36 -- | | |
| | | | | - | | |
| | | | | 37 -- | | |
| | | | | - | | |
| | | | | 38 -- | | |
| | | | | - | | |
| | | | | 39 -- | | |
| | | | | - | | |
| | | | | 40 -- | | |
| | | | | - | | |
| | | | | 41 -- | | |
| | | | | - | | |
| | | | | 42 -- | | |
| | | | | - | | |
| | | | | 43 -- | | |
| | | | | - | | |
| | | | | 44 -- | | |
| | | | | - | | |
| | | | | 45 -- | | |
| | | | | - | | |
| | | | | 46 -- | | |
| | | | | - | | |
| | | | | 47 -- | | |
| | | | | - | | |
| | | | | 48 -- | | |
| | | | | - | | |
| | | | | 49 -- | | |
| | | | | - | | |
| | | | | 50 -- | | |
| | | | | - | | |

# BORING LOG NUMBER 2

**SCD 1811 Sacramento, LLC** **Date: 12/08/21**

**File No. 21971** **Method: 8-Inch Diameter Hollow Stem Auger**

In

| Sample Depth ft. | Blows per ft. | Moisture content % | Dry Density p.c.f. | Depth in feet | USCS Class. | Description<br>Surface Conditions: Asphalt For Parking |
|---|---|---|---|---|---|---|
| | | | | 0 -- | | 2 Inch Asphalt, No Base |
| | | | | - | | |
| | | | | 1 -- | | FILL: Silty Sand, dark brown, moist, medium dense, fine grained, minor brick fragments |
| | | | | - | | |
| | | | | 2 -- | | |
| | | | | - | | |
| | | | | 3 -- | | |
| | | | | - | | |
| | | | | 4 -- | | |
| | | | | - | | |
| 5 | 18 | 11.2 | SPT | 5 -- | | |
| | | | | - | | |
| | | | | 6 -- | | |
| | | | | - | | |
| 7.5 | 27 | 8.6 | 101.8 | 7 -- | | |
| | | | | - | SP | NATIVE SOILS: Sand, yellowish brown, moist, medium dense, fine grained |
| | | | | 8 -- | | |
| | | | | - | | |
| | | | | 9 -- | | |
| | | | | - | | |
| 10 | 18 | 19.6 | SPT | 10 -- | | |
| | | | | - | SM/SP | Silty Sand to Sand, grayish brown to yellowish brown, moist, medium dense, fine grained |
| | | | | 11 -- | | |
| | | | | - | | |
| | | | | 12 -- | | |
| 12.5 | 44 | 7.2 | 100.0 | - | | |
| | | | | 13 -- | SP/ML | Sand to Sandy Silt, yellowish and grayish brown, moist, medium dense, fine grained |
| | | | | - | | |
| | | | | 14 -- | | |
| | | | | - | | |
| 15 | 60 | 2.6 | SPT | 15 -- | | |
| | | | | - | SP | Sand, dark and yellowish brown, moist, medium dense to dense, fine to medium grained, minor cobbles |
| | | | | 16 -- | | |
| | | | | - | | |
| | | | | 17 -- | | |
| 17.5 | 84 | 4.8 | 107.9 | - | | |
| | | | | 18 -- | | |
| | | | | - | | |
| | | | | 19 -- | | |
| | | | | - | | |
| 20 | 46 | 4.6 | SPT | 20 -- | | |
| | | | | - | | |
| | | | | 21 -- | | |
| | | | | - | | |
| | | | | 22 -- | | |
| 22.5 | 60 | 2.2 | 109.0 | - | | |
| | | | | 23 -- | | |
| | | | | - | | |
| | | | | 24 -- | | |
| | | | | - | | |
| 25 | 43 | 7.3 | SPT | 25 -- | | |
| | | | | - | | fine to coarse grained |

# BORING LOG NUMBER 2

**SCD 1811 Sacramento, LLC**

**File No. 21971**

ln

| Sample Depth ft. | Blows per ft. | Moisture content % | Dry Density p.c.f. | Depth in feet | USCS Class. | Description |
|---|---|---|---|---|---|---|
| | | | | 26 | | |
| | | | | 27 | | |
| 27.5 | 77 | 2.9 | 114.2 | | | |
| | | | | 28 | | |
| | | | | 29 | | |
| 30 | 48 | 7.2 | SPT | 30 | | |
| | | | | | SM/SP | Sand to Silty sand, dark and yellowish brown, moist, dense, fine to medium grained, minor cobbles |
| | | | | 31 | | |
| | | | | 32 | | |
| 32.5 | 99 | 2.7 | 112.1 | | | |
| | | | | 33 | | |
| | | | | 34 | | |
| 35 | 52 | 2.7 | SPT | 35 | | |
| | | | | | SP/SW | Sand to Gravelly Sand, grayish brown, moist, dense, fine to coarse grained |
| | | | | 36 | | |
| | | | | 37 | | |
| 37.5 | 100/10" | 1.3 | 134.8 | | | |
| | | | | 38 | | very dense |
| | | | | 39 | | |
| 40 | 70 | 2.1 | SPT | 40 | | |
| | | | | 41 | | |
| | | | | 42 | | |
| 42.5 | 100 | 3.3 | 123.7 | | | |
| | | | | 43 | SM/SP | Cobbley Sand to Sand, yellowish brown, moist, very dense, fine to coarse grained |
| | | | | 44 | | |
| 45 | 50/6" | 1.7 | SPT | 45 | | |
| | | | | 46 | | |
| | | | | 47 | | |
| 47.5 | 77 | 3.1 | 124.3 | | | |
| | | | | 48 | | |
| | | | | 49 | | |
| 50 | 87 | 1.7 | SPT | 50 | | |

# BORING LOG NUMBER 2

**SCD 1811 Sacramento, LLC**

**File No. 21971**

ln

| Sample Depth ft. | Blows per ft. | Moisture content % | Dry Density p.c.f. | Depth in feet | USCS Class. | Description |
|---|---|---|---|---|---|---|
| | | | | - | | |
| | | | | 51 -- | | |
| | | | | - | | |
| | | | | 52 -- | | |
| 52.5 | 100/8" | 2.5 | 130.6 | - | | |
| | | | | 53 -- | | |
| | | | | - | | |
| | | | | 54 -- | | |
| | | | | - | | |
| 55 | 50/6" | 2.1 | SPT | 55 -- | | |
| | | | | - | | End At 55 Feet By Refusal |
| | | | | 56 -- | | No Water |
| | | | | - | | Fill To 7 Feet |
| | | | | 57 -- | | |
| | | | | - | | NOTE: The stratification lines represent the approximate |
| | | | | 58 -- | | boundary between earth types; the transition may be gradual. |
| | | | | - | | |
| | | | | 59 -- | | Used 8-inch diameter Hollow-Stem Auger |
| | | | | - | | 140-lb. Automatic Hammer, 30-inch drop |
| | | | | 60 -- | | Modified California Sampler used unless otherwise noted |
| | | | | - | | |
| | | | | 61 -- | | SPT=Standard Penetration Test |
| | | | | - | | |
| | | | | 62 -- | | |
| | | | | - | | |
| | | | | 63 -- | | |
| | | | | - | | |
| | | | | 64 -- | | |
| | | | | - | | |
| | | | | 65 -- | | |
| | | | | - | | |
| | | | | 66 -- | | |
| | | | | - | | |
| | | | | 67 -- | | |
| | | | | - | | |
| | | | | 68 -- | | |
| | | | | - | | |
| | | | | 69 -- | | |
| | | | | - | | |
| | | | | 70 -- | | |
| | | | | - | | |
| | | | | 71 -- | | |
| | | | | - | | |
| | | | | 72 -- | | |
| | | | | - | | |
| | | | | 73 -- | | |
| | | | | - | | |
| | | | | 74 -- | | |
| | | | | - | | |
| | | | | 75 -- | | |
| | | | | - | | |

# BORING LOG NUMBER 3

**SCD 1811 Sacramento, LLC** | **Date: 12/10/21**

**File No. 21971** | **Method: 8-Inch Diameter Hollow Stem Auger**

In

| Sample Depth ft. | Blows per ft. | Moisture content % | Dry Density p.c.f. | Depth in feet | USCS Class. | Description<br>Surface Conditions: Asphalt For Parking |
|---|---|---|---|---|---|---|
| | | | | 0 -- | | 6 Inch Asphalt, 3 Inch Base |
| | | | | - | | |
| | | | | 1 -- | | |
| | | | | - | | FILL: Silty Sand, dark and yellowish brown, moist, medium |
| | | | | 2 -- | | dense, fine grained |
| 2.5 | 13 | 15.1 | 91.3 | - | | |
| | | | | 3 -- | | |
| | | | | - | SM | NATIVE SOILS: Silty Sand, dark and yellowish brown, moist, |
| | | | | 4 -- | | medium dense, fine grained |
| | | | | - | | |
| 5 | 12 | 29.4 | 89.3 | 5 -- | | |
| | | | | - | SP/ML | Sand to Sandy Silt, dark and yellowish brown, moist, medium |
| | | | | 6 -- | | dense, stiff, fine grained |
| | | | | - | | |
| | | | | 7 -- | | |
| | | | | - | | |
| | | | | 8 -- | | |
| | | | | - | | |
| | | | | 9 -- | | |
| | | | | - | | |
| 10 | 35 | 3.1 | 97.3 | 10 -- | | |
| | | | | - | SP | Sand, dark and yellowish brown, moist, medium dense, fine |
| | | | | 11 -- | | grained |
| | | | | - | | |
| | | | | 12 -- | | |
| | | | | - | | |
| | | | | 13 -- | | |
| | | | | - | | |
| | | | | 14 -- | | |
| | | | | - | | |
| 15 | 100 | 3.3 | 125.7 | 15 -- | | |
| | | | | - | SP/SW | Sand to Pebbley Sand, gray and yellowish brown, moist, |
| | | | | 16 -- | | very dense, fine to coarse grained |
| | | | | - | | |
| | | | | 17 -- | | |
| | | | | - | | |
| | | | | 18 -- | | |
| | | | | - | | |
| | | | | 19 -- | | |
| | | | | - | | |
| 20 | 100 | 4.1 | 112.5 | 20 -- | | |
| | | | | - | | |
| | | | | 21 -- | | |
| | | | | - | | |
| | | | | 22 -- | | |
| | | | | - | | |
| | | | | 23 -- | | |
| | | | | - | | |
| | | | | 24 -- | | |
| | | | | - | | |
| 25 | 92 | 3.2 | 104.8 | 25 -- | | |
| | | | | - | SP | Sand, yellowish brown, moist, very dense, fine to medium grained |

# BORING LOG NUMBER 3

**SCD 1811 Sacramento, LLC**

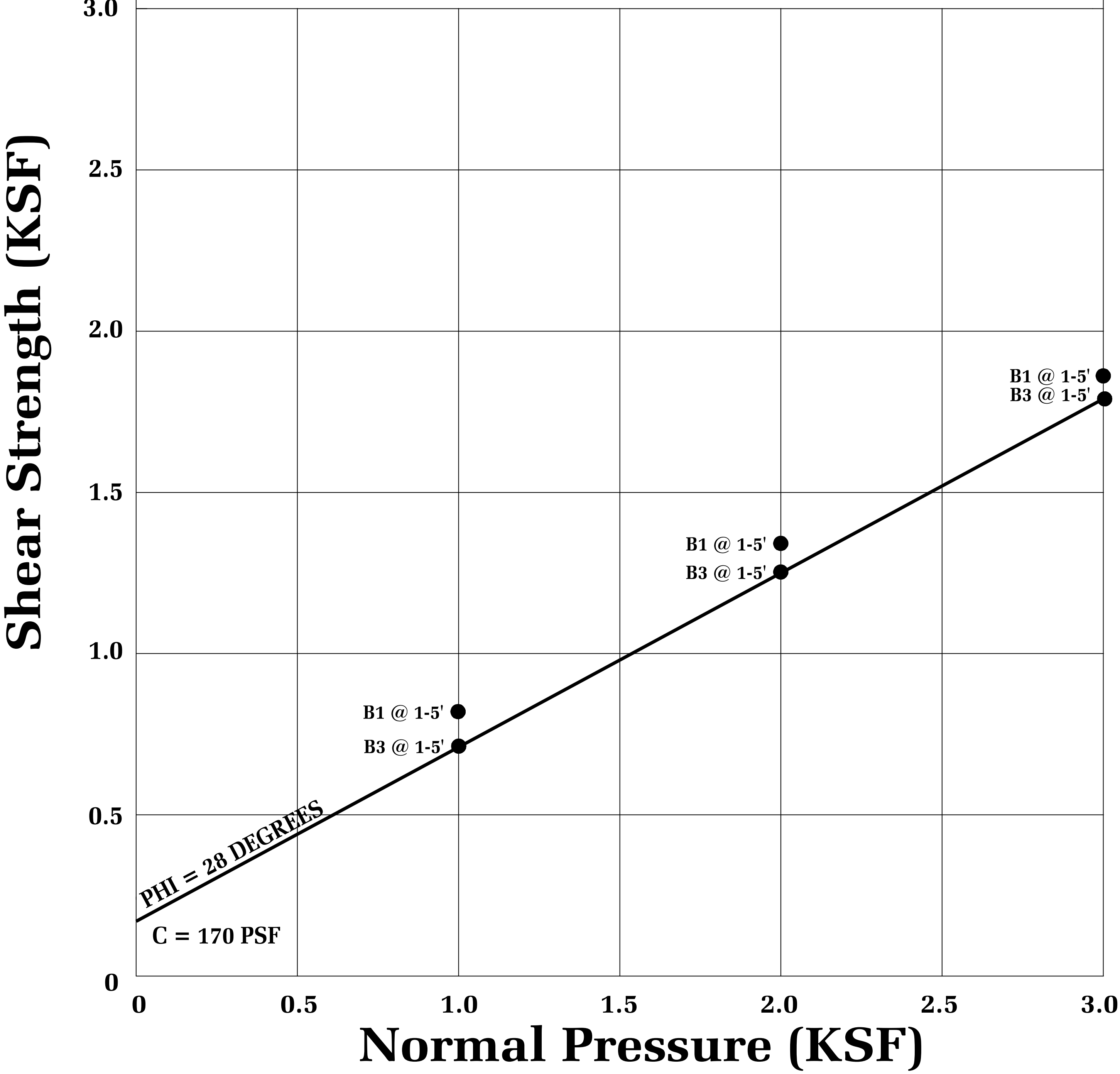
**File No. 21971**

ln

| Sample Depth ft. | Blows per ft. | Moisture content % | Dry Density p.c.f. | Depth in feet | USCS Class. | Description |
|---|---|---|---|---|---|---|
| | | | | 26 -- | | |
| | | | | 27 -- | | |
| | | | | 28 -- | | |
| | | | | 29 -- | | |
| 30 | 95 | 5.0 | 114.1 | 30 -- | | |
| | | | | 31 -- | | Total Depth: 30 Feet<br>No Water<br>Fill To 3 Feet |
| | | | | 32 -- | | |
| | | | | 33 -- | | NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual. |
| | | | | 34 -- | | Used 8-inch diameter Hollow-Stem Auger<br>140-lb. Automatic Hammer, 30-inch drop<br>Modified California Sampler used unless otherwise noted |
| | | | | 35 -- | | |
| | | | | 36 -- | | |
| | | | | 37 -- | | |
| | | | | 38 -- | | |
| | | | | 39 -- | | |
| | | | | 40 -- | | |
| | | | | 41 -- | | |
| | | | | 42 -- | | |
| | | | | 43 -- | | |
| | | | | 44 -- | | |
| | | | | 45 -- | | |
| | | | | 46 -- | | |
| | | | | 47 -- | | |
| | | | | 48 -- | | |
| | | | | 49 -- | | |
| | | | | 50 -- | | |

## BULK SAMPLE REMOLDED TO 90 PERCENT OF THE MAXIMUM LABORATORY DENSITY

| SAMPLE | SOIL TYPE | DRY DENSITY (PCF) | INITIAL MOISTURE(%) | FINAL MOISTURE(%) |
|---|---|---|---|---|
| B1 @ 1-5' | SM | 121.5 | 9.5 | 18.7 |
| B3 @ 1-5' | SM | 115.4 | 12.3 | 22.8 |




● Direct Shear, Saturated

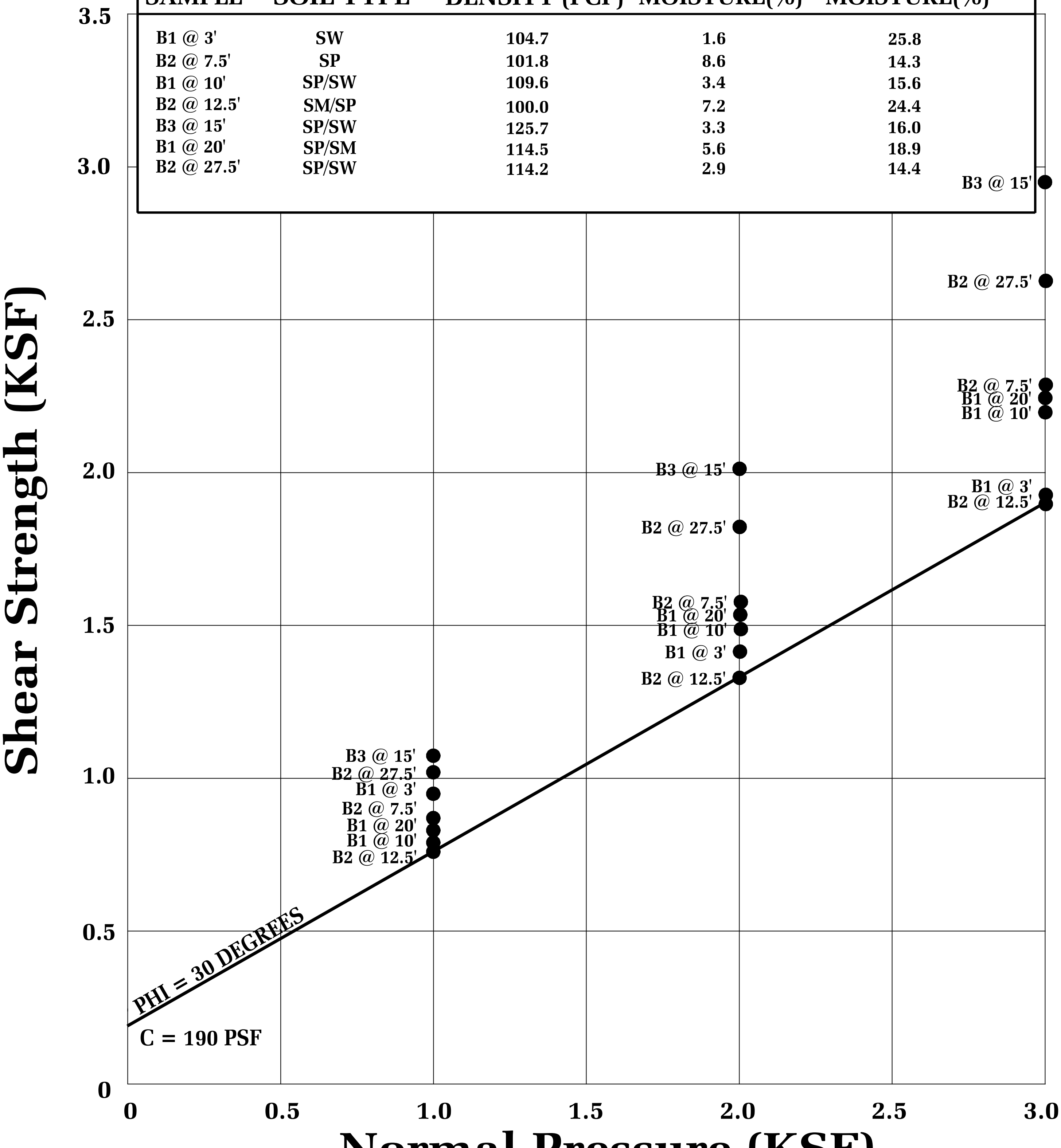
# SHEAR TEST DIAGRAM

**Geotechnologies, Inc.**
Consulting Geotechnical Engineers

SCD 1811 SACRAMENTO, LLC

FILE NO. 21971 | PLATE: B-1

| SAMPLE | SOIL TYPE | DRY DENSITY (PCF) | INITIAL MOISTURE(%) | FINAL MOISTURE(%) |
|---|---|---|---|---|
| B1 @ 3' | SW | 104.7 | 1.6 | 25.8 |
| B2 @ 7.5' | SP | 101.8 | 8.6 | 14.3 |
| B1 @ 10' | SP/SW | 109.6 | 3.4 | 15.6 |
| B2 @ 12.5' | SM/SP | 100.0 | 7.2 | 24.4 |
| B3 @ 15' | SP/SW | 125.7 | 3.3 | 16.0 |
| B1 @ 20' | SP/SM | 114.5 | 5.6 | 18.9 |
| B2 @ 27.5' | SP/SW | 114.2 | 2.9 | 14.4 |



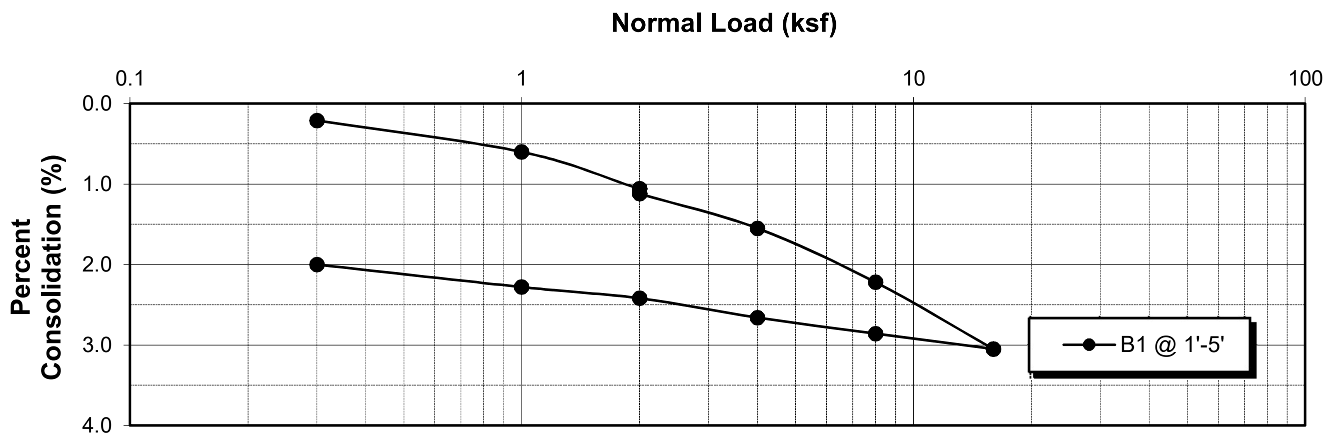

● Direct Shear, Saturated

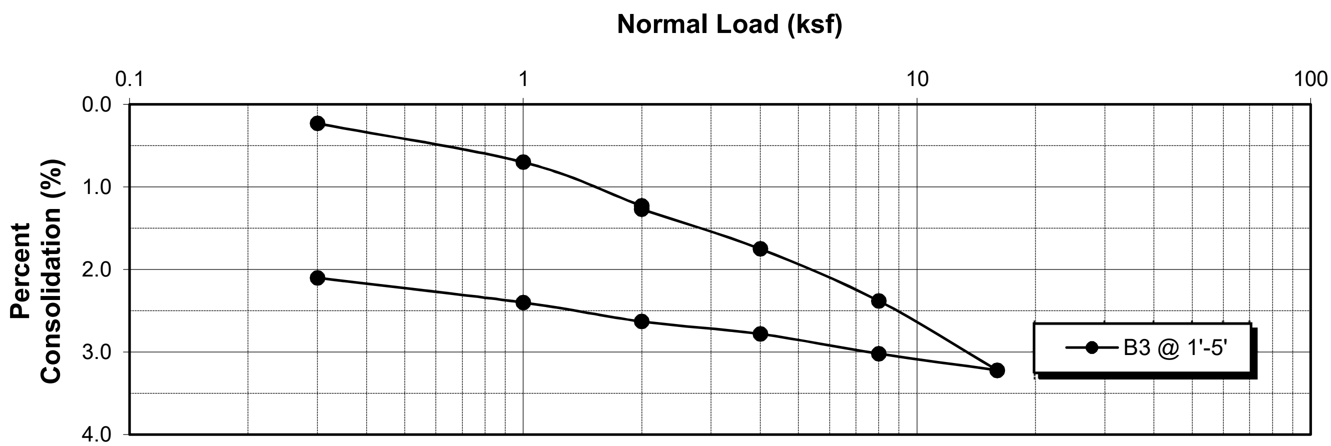
# SHEAR TEST DIAGRAM

**Geotechnologies, Inc.**
Consulting Geotechnical Engineers

SCD 1811 SACRAMENTO, LLC

FILE NO. 21971 | PLATE: B



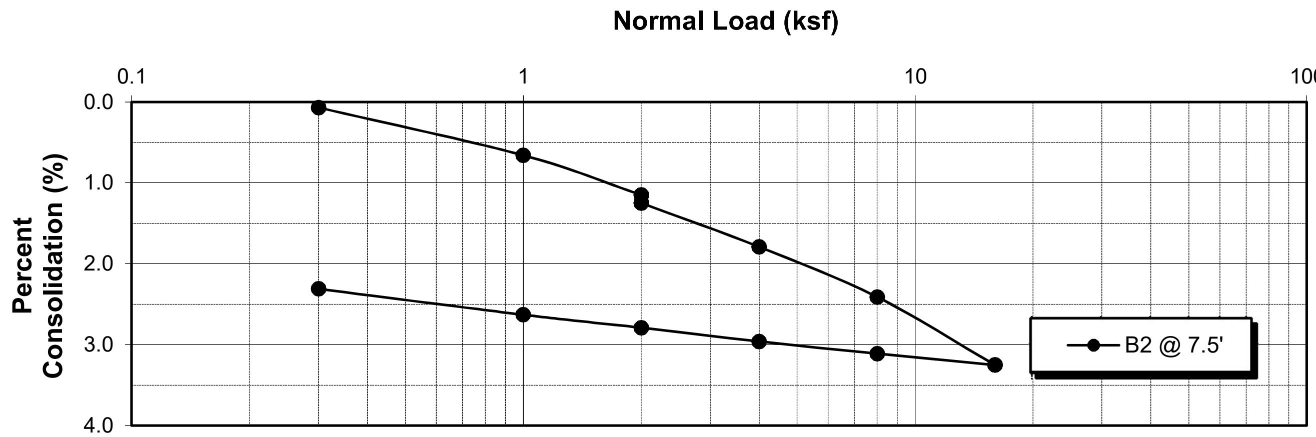



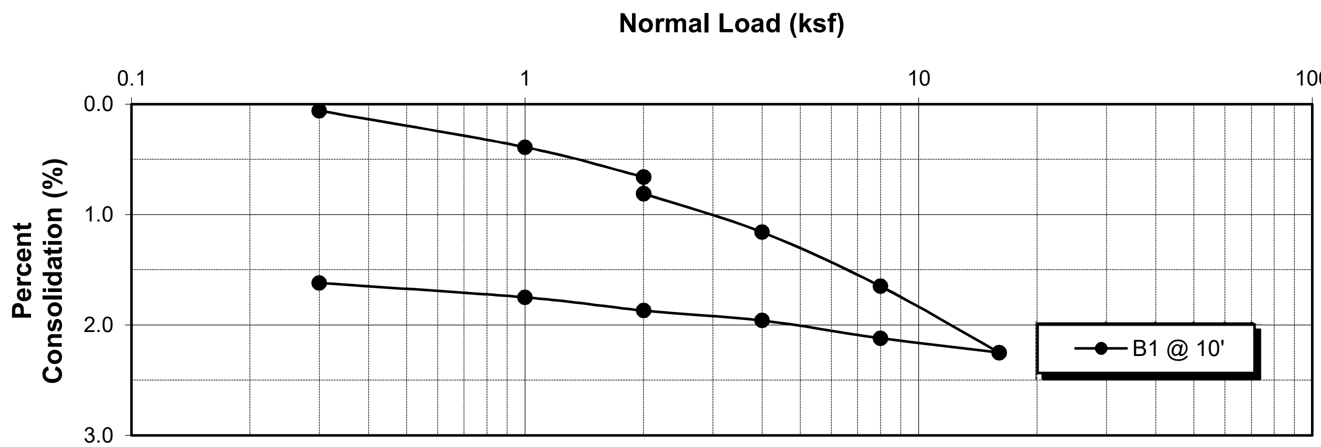

**Water added at 2 KSF**

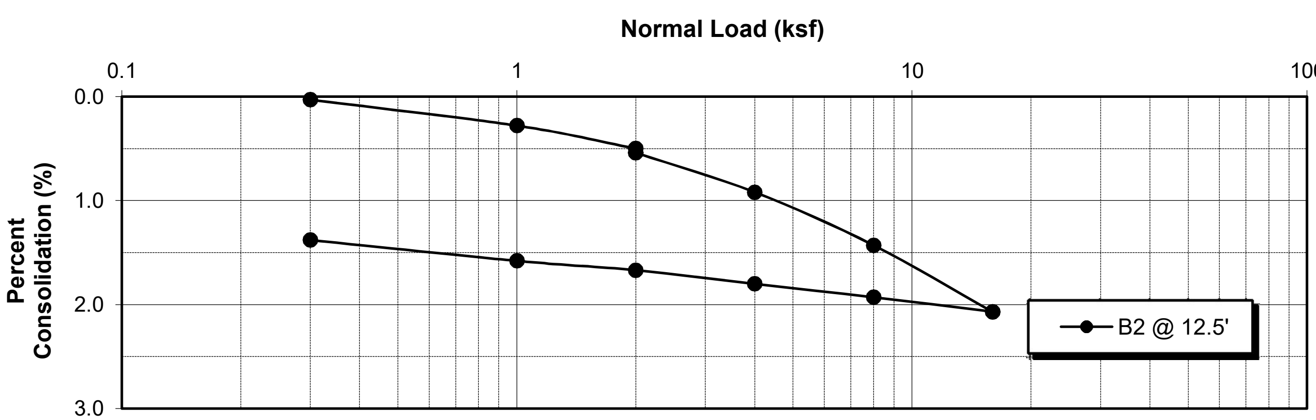
# CONSOLIDATION

**Geotechnologies, Inc.**
*Consulting Geotechnical Engineers*

| | |
|---|---|
| PROJECT: SCD 1811 SACRAMENTO, LLC | |
| FILE NO.: 21971 | PLATE: C-1 |



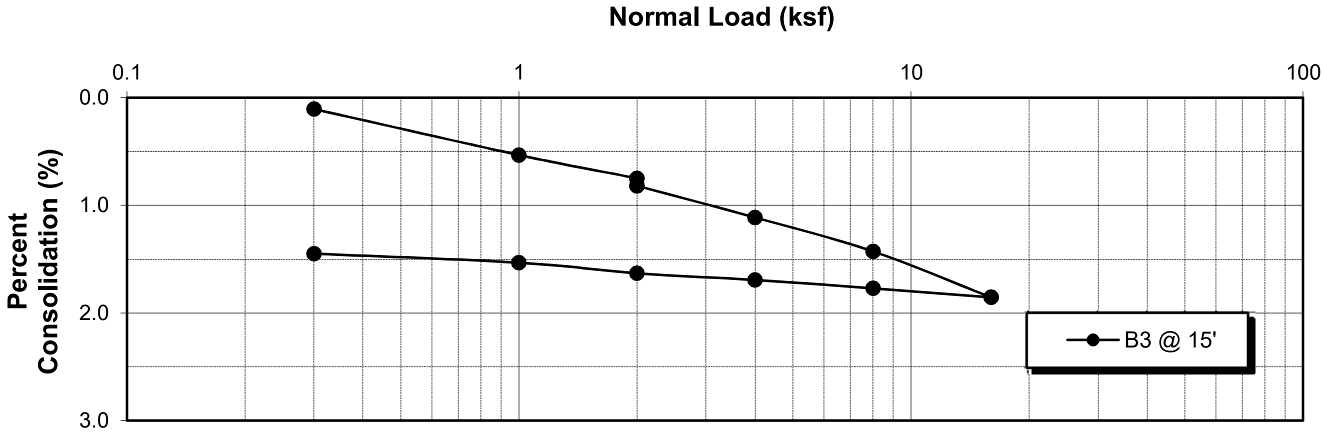

**Water added at 2 KSF**

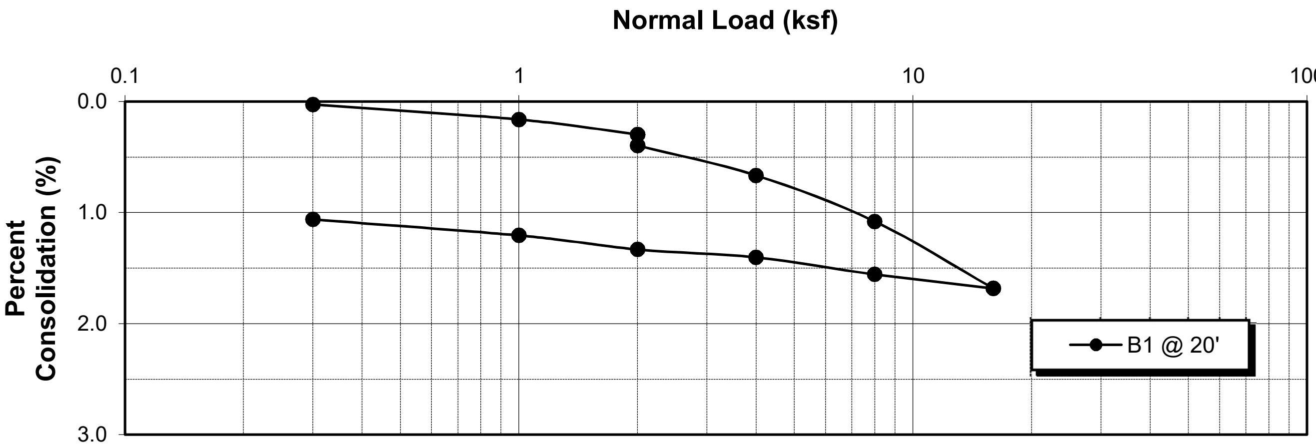
# CONSOLIDATION

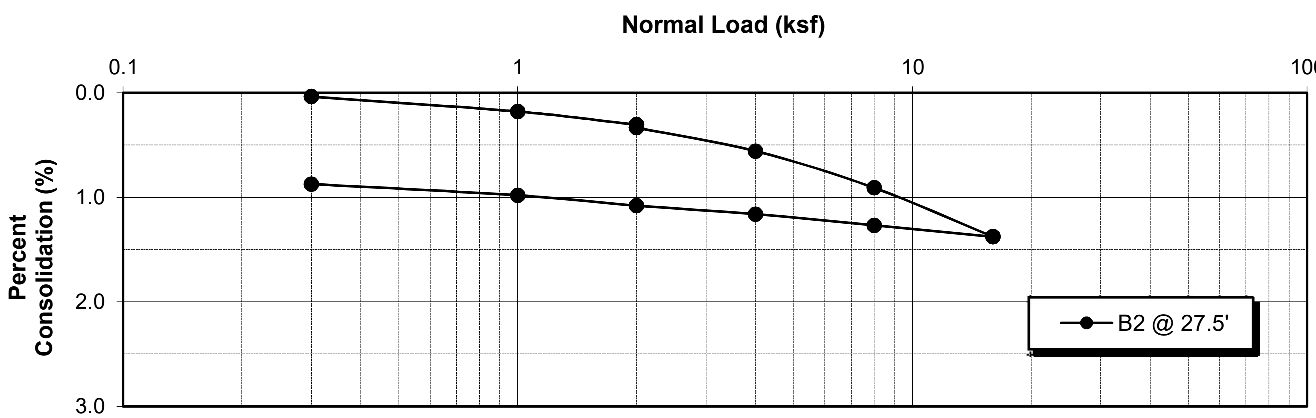
**Geotechnologies, Inc.**
*Consulting Geotechnical Engineers*

PROJECT: SCD 1811 SACRAMENTO, LLC

FILE NO.: 21971 | PLATE: C-2



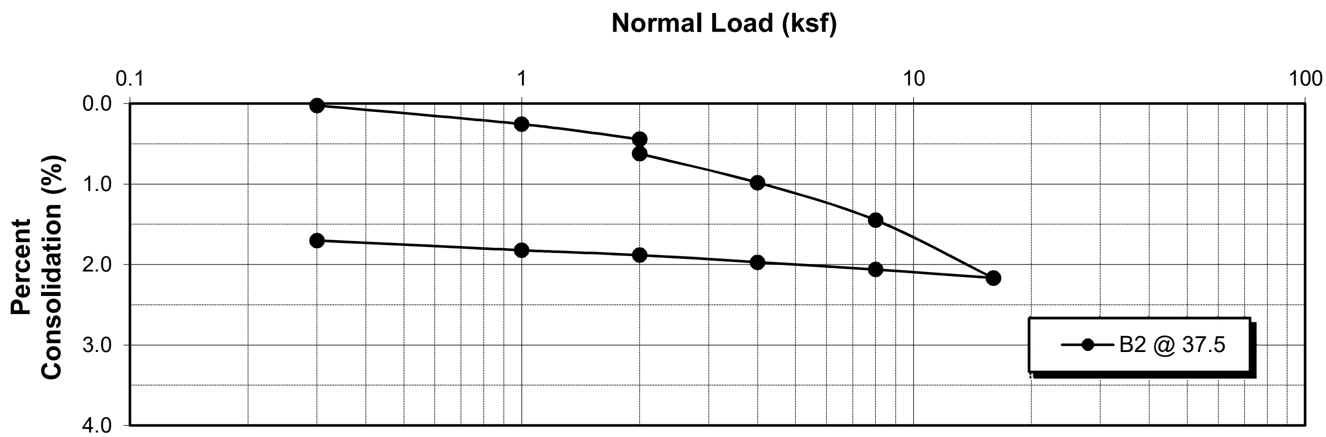



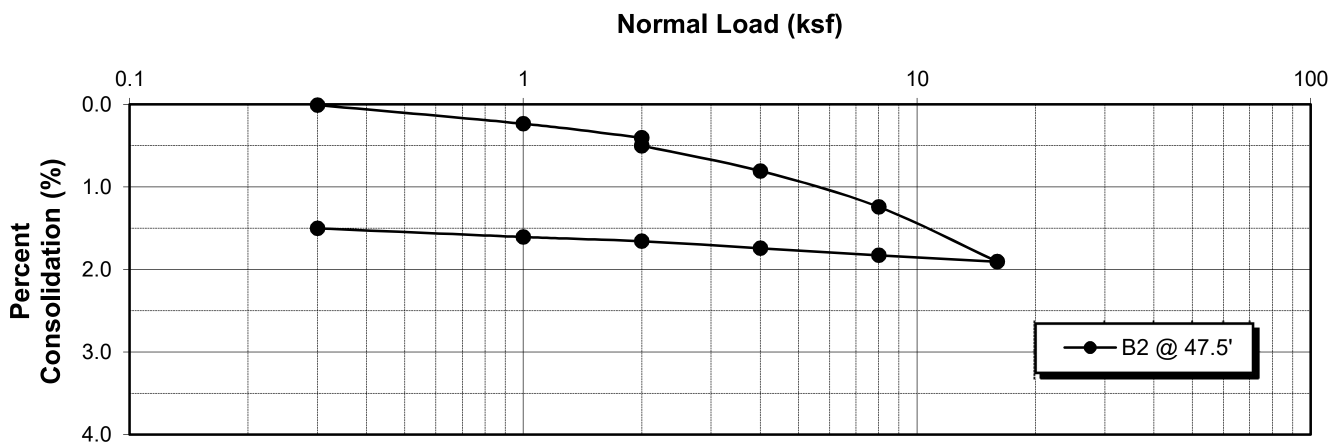

**Water added at 2 KSF**

# CONSOLIDATION

**Geotechnologies, Inc.**
*Consulting Geotechnical Engineers*

| | |
|---|---|
| PROJECT: SCD 1811 SACRAMENTO, LLC | |
| FILE NO.: 21971 | PLATE: C-3 |




Water added at 2 KSF

# CONSOLIDATION

**Geotechnologies, Inc.**
*Consulting Geotechnical Engineers*

| PROJECT: SCD 1811 SACRAMENTO, LLC | |
|---|---|
| FILE NO.: 21971 | PLATE: C-4 |

## ASTM D-1557

| SAMPLE | B1 @ 1-5' | B3 @ 1-5' |
|---|---|---|
| SOIL TYPE: | SM | SM |
| MAXIMUM DENSITY pcf. | 121.5 | 115.4 |
| OPTIMUM MOISTURE % | 9.5 | 12.3 |

## ASTM D 4829

| SAMPLE | B1 @ 1-5' | B3 @ 1-5' |
|---|---|---|
| SOIL TYPE: | SM | SM/SP |
| EXPANSION INDEX<br>UBC STANDARD 18-2 | 4 | 5 |
| EXPANSION CHARACTER | VERY LOW | VERY LOW |

## SULFATE CONTENT

| SAMPLE | B1 @ 1-5' | B3 @ 1-5' | B2 @ 7.5' | B2 @ 12.5' |
|---|---|---|---|---|
| SULFATE CONTENT:<br>(percentage by weight) | < 0.10% | < 0.10% | < 0.10% | < 0.10% |

# COMPACTION/EXPANSION/SULFATE DATA SHEET

**Geotechnologies, Inc.**
Consulting Geotechnical Engineers

SCD 1811 SACRAMENTO, LLC

FILE NO. 21971

PLATE: D

**Geotechnologies, Inc.**

Project: SCD 1811 Sacramento, LLC

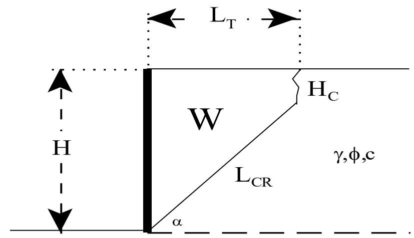
File No.: 21971

Description: Retaining Wall up to 12 feet High

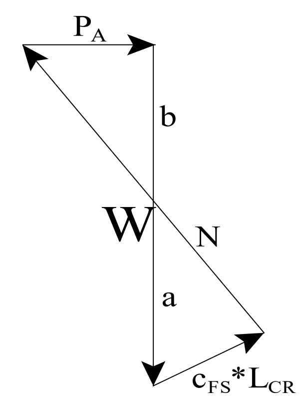
## Retaining Wall Design with Level Backfill
## (Vector Analysis)

Input:

| | | | |
|---|---|---|---|
| Retaining Wall Height | (H) | 12.00 | feet |
| Unit Weight of Retained Soils | ($\gamma$) | 120.0 | pcf |
| Friction Angle of Retained Soils | ($\phi$) | 30.0 | degrees |
| Cohesion of Retained Soils | (c) | 190.0 | psf |
| Factor of Safety | (FS) | 1.50 | |
| Factored Parameters: | ($\phi_{FS}$) | 21.1 | degrees |
| | ($c_{FS}$) | 126.7 | psf |




| Failure Angle ($\alpha$) degrees | Height of Tension Crack ($H_C$) feet | Area of Wedge (A) feet$^2$ | Weight of Wedge (W) lbs/lineal foot | Length of Failure Plane ($L_{CR}$) feet | a lbs/lineal foot | b lbs/lineal foot | Active Pressure ($P_A$) lbs/lineal foot |
|---|---|---|---|---|---|---|---|
| 40 | 4.0 | 76 | 9175.3 | 12.5 | 4553.4 | 4621.9 | 1586.8 |
| 41 | 3.8 | 74 | 8928.9 | 12.5 | 4317.1 | 4611.8 | 1673.9 |
| 42 | 3.7 | 72 | 8679.6 | 12.4 | 4097.5 | 4582.1 | 1754.2 |
| 43 | 3.6 | 70 | 8429.7 | 12.3 | 3893.7 | 4536.0 | 1827.9 |
| 44 | 3.5 | 68 | 8180.5 | 12.2 | 3704.5 | 4476.0 | 1895.2 |
| 45 | 3.4 | 66 | 7933.2 | 12.1 | 3528.7 | 4404.5 | 1956.2 |
| 46 | 3.4 | 64 | 7688.6 | 12.0 | 3365.4 | 4323.2 | 2011.2 |
| 47 | 3.3 | 62 | 7447.2 | 11.9 | 3213.4 | 4233.8 | 2060.2 |
| 48 | 3.2 | 60 | 7209.4 | 11.8 | 3071.8 | 4137.6 | 2103.5 |
| 49 | 3.2 | 58 | 6975.3 | 11.7 | 2939.7 | 4035.5 | 2141.1 |
| 50 | 3.2 | 56 | 6745.1 | 11.5 | 2816.4 | 3928.7 | 2173.1 |
| 51 | 3.1 | 54 | 6518.8 | 11.4 | 2701.0 | 3817.9 | 2199.6 |
| 52 | 3.1 | 52 | 6296.5 | 11.3 | 2592.9 | 3703.6 | 2220.8 |
| 53 | 3.1 | 51 | 6078.1 | 11.2 | 2491.4 | 3586.6 | 2236.7 |
| 54 | 3.1 | 49 | 5863.4 | 11.0 | 2396.0 | 3467.4 | 2247.3 |
| 55 | 3.1 | 47 | 5652.4 | 10.9 | 2306.2 | 3346.2 | 2252.6 |
| 56 | 3.1 | 45 | 5445.0 | 10.8 | 2221.5 | 3223.5 | 2252.8 |
| 57 | 3.1 | 44 | 5241.0 | 10.6 | 2141.5 | 3099.6 | 2247.7 |
| 58 | 3.1 | 42 | 5040.3 | 10.5 | 2065.6 | 2974.7 | 2237.4 |
| 59 | 3.1 | 40 | 4842.7 | 10.4 | 1993.6 | 2849.0 | 2221.8 |
| 60 | 3.1 | 39 | 4648.0 | 10.2 | 1925.1 | 2722.9 | 2200.9 |
| 61 | 3.2 | 37 | 4456.2 | 10.1 | 1859.8 | 2596.3 | 2174.6 |
| 62 | 3.2 | 36 | 4266.9 | 10.0 | 1797.4 | 2469.6 | 2142.9 |
| 63 | 3.2 | 34 | 4080.2 | 9.8 | 1737.4 | 2342.7 | 2105.6 |
| 64 | 3.3 | 32 | 3895.7 | 9.7 | 1679.8 | 2215.9 | 2062.6 |
| 65 | 3.4 | 31 | 3713.3 | 9.5 | 1624.1 | 2089.2 | 2013.9 |




Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 2252.8 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 31 pcf

**Design Wall for an Equivalent Fluid Pressure: 31 pcf**

**Geotechnologies, Inc.**

Project: SCD 1811 Sacramento, LLC
File No.: 21971

| | | | |
|---|---|---|---|
| Soil Weight | $\gamma$ | 120 | pcf |
| Internal Friction Angle | $\phi$ | 30 | degrees |
| Cohesion | c | 0 | psf |
| Height of Retaining Wall | H | 12 | feet |

**Restrained Retaining Wall Design based on At Rest Earth Pressure**

$\sigma'_h = K_o\sigma'_v$

| | | |
|---|---|---|
| $K_o = 1 - \sin\phi$ | 0.500 | |
| $\sigma'_v = \gamma H$ | 1440.0 | psf |

$\sigma'_h =$ 720.0 psf

EFP = 60 pcf

$P_o =$ 4320.0 lbs/ft (based on a triangular distribution of pressure)

Design wall for an EFP of 60 pcf

**Geotechnologies, Inc.**

Project: SCD 1811 Sacramento, LLC

File No.: 21971

## Seismically Induced Lateral Soil Pressure on Retaining Wall

**Input:**

| | | | |
|---|---|---|---|
| Height of Retaining Wall: | (H) | 12.0 | feet |
| Retained Soil Unit Weight: | ($\gamma$) | 120.0 | pcf |
| Horizontal Ground Acceleration: | ($k_h$) | 0.30 | g |

**Seismic Increment ($\Delta P_{AE}$):**

$\Delta P_{AE} = (0.5*\gamma*H^2)*(0.75*k_h)$

$\Delta P_{AE} =$ 1944.0 lbs/ft

Force applied at 0.6H above the base of the wall

Transfer load to 2/3 of the height of the wall

$T*(2/3)*H = \Delta P_{AE}*0.6*H$

T = 1749.6 lbs/ft

$EFP = 2*T/H^2$

**EFP = 24 pcf**

**triangular distribution of pressure**

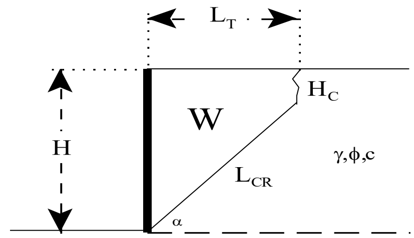
**Geotechnologies, Inc.**

Project: SCD 1811 Sacramento, LLC
File No.: 21971
Description: Shoring Walls up to 20 feet High

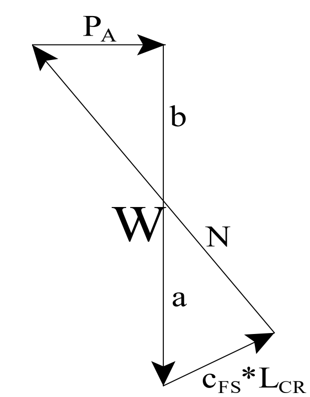
## Shoring Design with Level Backfill
## (Vector Analysis)

Input:

| | | | |
|---|---|---|---|
| Shoring Height | (H) | 12.00 | feet |
| Unit Weight of Retained Soils | ($\gamma$) | 120.0 | pcf |
| Friction Angle of Retained Soils | ($\phi$) | 30.0 | degrees |
| Cohesion of Retained Soils | (c) | 190.0 | psf |
| Factor of Safety | (FS) | 1.25 | |
| Factored Parameters: | ($\phi_{FS}$) | 24.8 | degrees |
| | ($c_{FS}$) | 152.0 | psf |




| Failure Angle ($\alpha$) degrees | Height of Tension Crack ($H_C$) feet | Area of Wedge (A) $feet^2$ | Weight of Wedge (W) lbs/lineal foot | Length of Failure Plane ($L_{CR}$) feet | a lbs/lineal foot | b lbs/lineal foot | Active Pressure ($P_A$) lbs/lineal foot |
|---|---|---|---|---|---|---|---|
| 40 | 5.7 | 66 | 7955.4 | 9.8 | 5137.3 | 2818.1 | 766.1 |
| 41 | 5.5 | 66 | 7882.6 | 10.0 | 4929.1 | 2953.5 | 858.6 |
| 42 | 5.2 | 65 | 7772.8 | 10.1 | 4718.9 | 3053.9 | 945.9 |
| 43 | 5.0 | 64 | 7636.2 | 10.2 | 4512.0 | 3124.2 | 1027.7 |
| 44 | 4.9 | 62 | 7480.2 | 10.3 | 4311.7 | 3168.5 | 1103.9 |
| 45 | 4.7 | 61 | 7310.2 | 10.3 | 4119.6 | 3190.6 | 1174.5 |
| 46 | 4.6 | 59 | 7130.3 | 10.3 | 3936.7 | 3193.6 | 1239.3 |
| 47 | 4.5 | 58 | 6943.6 | 10.3 | 3763.4 | 3180.2 | 1298.4 |
| 48 | 4.4 | 56 | 6752.1 | 10.3 | 3599.5 | 3152.6 | 1351.8 |
| 49 | 4.3 | 55 | 6557.7 | 10.2 | 3444.7 | 3113.0 | 1399.6 |
| 50 | 4.2 | 53 | 6361.6 | 10.2 | 3298.8 | 3062.8 | 1441.8 |
| 51 | 4.1 | 51 | 6164.8 | 10.1 | 3161.2 | 3003.7 | 1478.6 |
| 52 | 4.1 | 50 | 5968.1 | 10.0 | 3031.3 | 2936.7 | 1509.8 |
| 53 | 4.0 | 48 | 5771.9 | 10.0 | 2908.8 | 2863.1 | 1535.7 |
| 54 | 4.0 | 46 | 5576.7 | 9.9 | 2793.1 | 2783.6 | 1556.3 |
| 55 | 4.0 | 45 | 5382.8 | 9.8 | 2683.6 | 2699.2 | 1571.5 |
| 56 | 4.0 | 43 | 5190.3 | 9.7 | 2579.9 | 2610.4 | 1581.5 |
| 57 | 4.0 | 42 | 4999.5 | 9.6 | 2481.5 | 2517.9 | 1586.2 |
| 58 | 4.0 | 40 | 4810.3 | 9.5 | 2388.0 | 2422.3 | 1585.6 |
| 59 | 4.0 | 39 | 4622.9 | 9.4 | 2299.0 | 2323.9 | 1579.8 |
| 60 | 4.0 | 37 | 4437.1 | 9.3 | 2214.0 | 2223.1 | 1568.8 |
| 61 | 4.0 | 35 | 4253.0 | 9.1 | 2132.6 | 2120.4 | 1552.4 |
| 62 | 4.1 | 34 | 4070.6 | 9.0 | 2054.5 | 2016.1 | 1530.8 |
| 63 | 4.1 | 32 | 3889.6 | 8.9 | 1979.3 | 1910.3 | 1503.8 |
| 64 | 4.1 | 31 | 3710.1 | 8.7 | 1906.6 | 1803.5 | 1471.3 |
| 65 | 4.2 | 29 | 3531.9 | 8.6 | 1836.1 | 1695.8 | 1433.5 |




Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 1586.2 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 22 pcf

**Design Shoring for an Equivalent Fluid Pressure: 28 pcf**

**Geotechnologies, Inc.**

Project: SCD 1811 Sacramento, LLC
File No.: 21971
Description: Slot Cut

# Slot Cut Calculation

Input:

| | | | |
|---|---|---|---|
| Height of Slots | (H) | 7 | feet |
| Unit Weight of Soils | ($\gamma$) | 120.0 | pcf |
| Friction Angle of Soils | ($\phi$) | 30.0 | degrees |
| Cohesion of Soils | (c) | 190.0 | psf |
| Factor of Safety | (FS) | 1.25 | |

Factor of Safety = Resistance Force/Driving Force

| | | | |
|---|---|---|---|
| Coefficient of Lateral Earth Pressure At-Rest | $K_o$ | 0.5 | |

Surcharge Pressure:

| | | | |
|---|---|---|---|
| Line Load | ($q_L$) | 2000.0 | plf |
| Distance Away from Edge of Excavation | (X) | 0.0 | feet |

**Design Equations**

$b = H/(\tan\alpha)$

$A = 0.5*H*b$

$W = 0.5*H*b*\gamma$ *(per lineal foot of slot width)*

$F_1 = d*W*(\sin\alpha)*(\cos\alpha)$

$F_2 = d*L$

$R_1 = d*[W*(\cos^2\alpha)*(\tan\phi)+(c*b)]$

$R_2 = 2*\Delta F$

$\Delta F = A*[1/3*\gamma*H*K_o*(\tan\phi)+c]$

**FS = Resistance Force/Driving Force**

**FS = $(R_1+R_2)/(F_1+F_2)$**

| Failure Angle ($\alpha$) degrees | Base Width of Failure Wedge (b) feet | Area of Failure Wedge (A) feet2 | Weight of Failure Wedge (W) lbs/lineal foot | **Driving Force** Wedge + Surcharge per lineal foot of Slot Wdith | **Resisting Force** Failure Wedge per lineal foot of Slot Width | **Resisting Force** Side Resistance Force ($\Delta$F) lbs | Allowable Width of Slots* (d) feet |
|---|---|---|---|---|---|---|---|
| 60 | 4.0 | 14 | 1697.4 | 1601.0 | 1301.6 | 3922.4 | 11.3 |
| 61 | 3.9 | 14 | 1629.7 | 1539.1 | 1229.8 | 3765.8 | 11.0 |
| 62 | 3.7 | 13 | 1563.2 | 1477.0 | 1160.6 | 3612.3 | 10.7 |
| 63 | 3.6 | 12 | 1498.0 | 1415.0 | 1093.9 | 3461.6 | 10.4 |
| 64 | 3.4 | 12 | 1433.9 | 1353.0 | 1029.7 | 3313.5 | 10.1 |
| 65 | 3.3 | 11 | 1370.9 | 1291.1 | 967.8 | 3168.0 | 9.9 |
| 66 | 3.1 | 11 | 1309.0 | 1229.5 | 908.2 | 3024.8 | 9.7 |
| 67 | 3.0 | 10 | 1248.0 | 1168.2 | 850.8 | 2883.8 | 9.6 |
| 68 | 2.8 | 10 | 1187.8 | 1107.2 | 795.6 | 2744.8 | 9.4 |
| 69 | 2.7 | 9 | 1128.6 | 1046.7 | 742.5 | 2607.9 | 9.3 |
| 70 | 2.5 | 9 | 1070.1 | 986.7 | 691.4 | 2472.7 | 9.2 |
| 71 | 2.4 | 8 | 1012.3 | 927.3 | 642.3 | 2339.3 | 9.1 |
| 72 | 2.3 | 8 | 955.3 | 868.5 | 595.1 | 2207.4 | 9.1 |
| 73 | 2.1 | 7 | 898.8 | 810.5 | 549.7 | 2077.1 | 9.0 |
| 74 | 2.0 | 7 | 843.0 | 753.3 | 506.1 | 1948.1 | 9.0 |
| 75 | 1.9 | 7 | 787.8 | 696.9 | 464.2 | 1820.4 | 9.0 |
| 76 | 1.7 | 6 | 733.0 | 641.5 | 424.0 | 1693.9 | 9.0 |
| 77 | 1.6 | 6 | 678.8 | 587.1 | 385.3 | 1568.5 | 9.1 |
| 78 | 1.5 | 5 | 624.9 | 533.8 | 348.2 | 1444.1 | 9.1 |
| 79 | 1.4 | 5 | 571.5 | 481.6 | 312.6 | 1320.6 | 9.2 |
| 80 | 1.2 | 4 | 518.4 | 430.7 | 278.4 | 1197.9 | 9.3 |
| 81 | 1.1 | 4 | 465.7 | 381.0 | 245.5 | 1076.0 | 9.4 |
| 82 | 1.0 | 3 | 413.2 | 332.6 | 213.9 | 954.8 | 9.5 |
| 83 | 0.9 | 3 | 361.0 | 285.6 | 183.5 | 834.2 | 9.7 |
| 84 | 0.7 | 3 | 309.0 | 240.0 | 154.4 | 714.1 | 9.9 |
| 85 | 0.6 | 2 | 257.2 | 196.0 | 126.3 | 594.4 | 10.1 |

Critical Slot Width with Factor of Safety equal or exceeding 1.5:

$d_{allow}$ 9.0 feet

**The proposed excavation may be made using the A-B-C Slot-Cutting Method with a Maximum Allowable Slot Width of 8 Feet, and up to 7 Feet in Height, with a Factor of Safety Equal or Exceeding 1.25.**

# REPORT

# SURFACE WAVE MEASUREMENTS

# 1727–1829 E SACRAMENTO STREET
# LOS ANGELES, CALIFORNIA

**GEO***Vision* Project No. 22235

*Prepared for*

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*Prepared by*


**GEO***Vision*, Inc.
1124 Olympic Drive
Corona, California 92881
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Report 22235-01 Rev 0

June 15, 2022

# TABLE OF CONTENTS



# LIST OF TABLES



# LIST OF FIGURES

# 1 INTRODUCTION

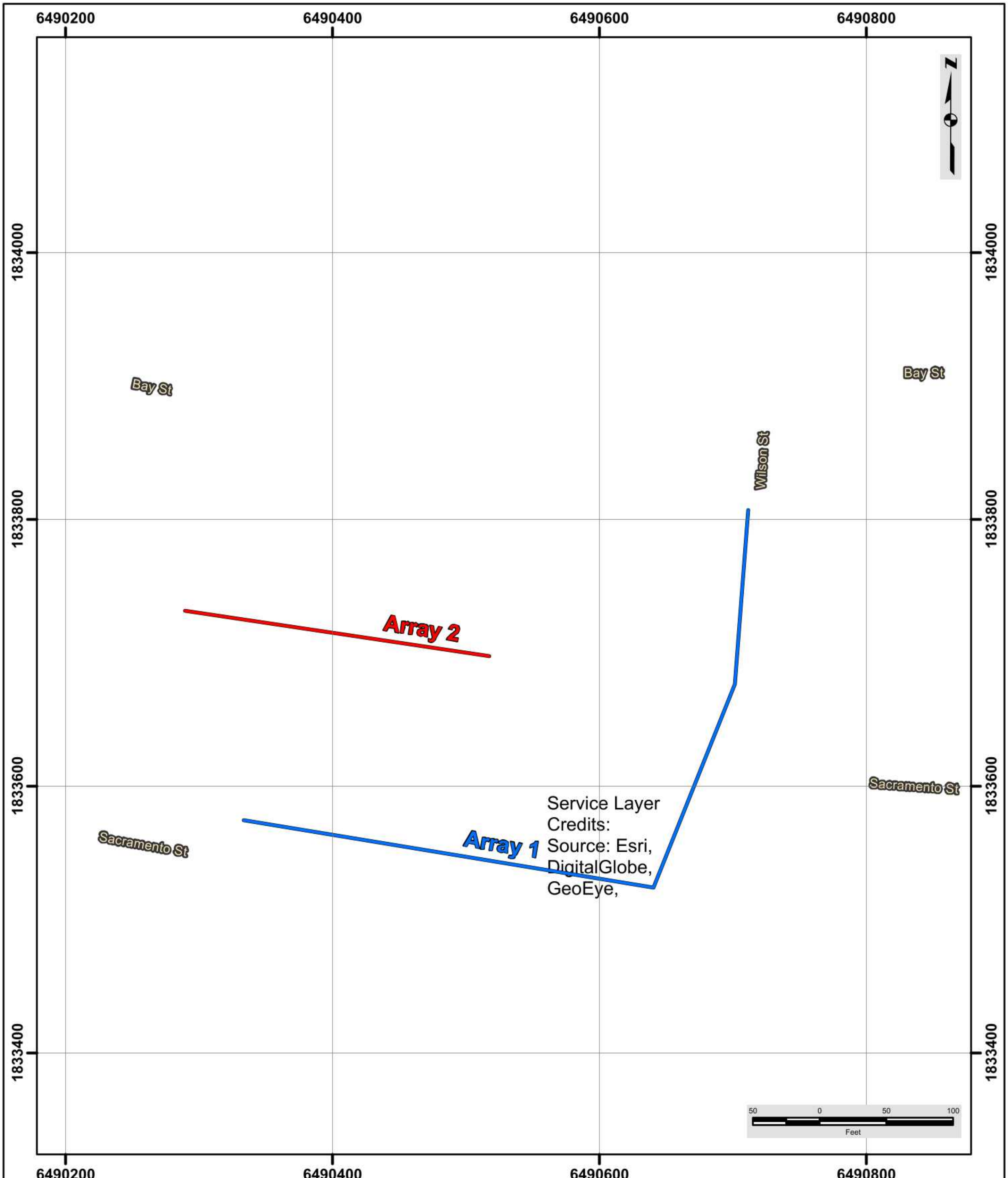
In-situ seismic measurements using active- and passive-source surface wave techniques were performed at 1727-1829 E. Sacramento Street, Los Angeles, California on June 1, 2022. The purpose of the investigation was to provide a shear (S) wave velocity profile to a depth of 30 m (100 ft), or greater, and estimate the average S-wave velocity of the upper 30 m ($V_{S30}$) and 100 ft ($V_{S100ft}$). The active-source surface wave technique utilized during this investigation consisted of the multi-channel analysis of surface waves (MASW) method. The passive-source surface wave technique consisted of the array microtremor method. The locations of the active- and passive-source surface wave testing locations are shown on Figure 1. Array microtremor measurements were made using an L-shaped array (Array 1) and MASW measurements were made on a linear array on the subject property (Array 2).

For seismic design, the 2019 California Buiilding Code (CBC) and 2018 International Building Code (IBC) reference the provisions in ASCE/SEI 7-16 (Minimum Design Loads and Associated Criteria for Buildings and Other Structures). The Site Classes and associated S-wave velocity ranges outlined in Table 20.3-1 of ASCE/SEI 7-16 are as follows:

Site Class A – Hard rock – $V_{S100ft} > 5{,}000$ ft/s
Site Class B – Rock –$2{,}500 < V_{S100ft} \leq 5{,}000$ ft/s
Site Class C – Very dense soil and soft rock –$1{,}200 < V_{S100ft} \leq 2{,}500$ ft/s
Site Class D – Stiff soil – $600 < V_{S100ft} \leq 1{,}200$ ft/s
Site Class E – Soft clay soil – $V_{S100ft} < 600$ ft/s (IBC)
Site Class F – Soils requiring site response analysis

At many sites, active surface wave techniques (MASW) with the utilization of portable energy sources, such as hammers and weight drops, are sufficient to obtain S-wave velocity sounding to 30 m (100 ft) depth. At sites with high ambient noise levels and/or very soft soils, these energy sources may not be sufficient to image to this depth and a larger energy source, such as a bulldozer, is necessary. Alternatively, passive surface wave techniques, such as the array microtremor technique can be used to extend the depth of investigation at sites that have adequate ambient noise conditions. It should be noted that two-dimensional passive-source surface wave arrays (e.g., triangular, circular, or L-shaped arrays) are expected to perform better than linear arrays.

This report contains the results of the active and passive surface wave measurements conducted at the site. An overview of the surface wave methods is given in Section 2. Field and data reduction procedures are discussed in Sections 3 and 4, respectively. Data modeling is presented in Section 5 and interpretation and results are presented in Section 6. References and our professional certification are presented in Sections 7 and 8, respectively.




Active Surface Wave Array (MASW)
Passive Surface Wave Array (L-Array)

NOTES:
1. Coordinate System: California State Plane, NAD83, Zone V (0405), US Survey Feet
2. Base map source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community

| | |
|---|---|
| Date: | 6/7/2022 |
| GV Project: | 22235 |
| Developed by: | A Martin |
| Drawn by: | T Rodriguez |
| Approved by: | A Martin |
| File Name: | 22235-1.MXD |

**FIGURE 1**
**SITE MAP**

**1727-1829 E SACRAMENTO ST**
**LOS ANGELES, CALIFORNIA**

**PREPARED FOR**
**ENGEO, INC.**

# 2 OVERVIEW OF SURFACE WAVE TECHNIQUES

## *2.1 Introduction*

Active- and passive-source (ambient vibration) surface wave techniques are routinely utilized for site characterization. Active surface wave techniques include the spectral analysis of surface waves (SASW) and multi-channel array surface wave (MASW) methods. Passive surface wave techniques include the horizontal over vertical spectral ratio (HVSR) technique and the array and refraction microtremor methods.

The basis of surface wave methods is the dispersive characteristic of Rayleigh and Love waves when propagating in a layered medium. Surface waves of different wavelengths ($\lambda$) or frequencies (f) sample different depth. As a result of the variance in the shear stiffness of the distinct layers, waves with different wavelengths propagate at different phase velocities; hence, dispersion. A surface wave dispersion curve is the variation of $V_R$ or $V_L$ with $\lambda$ or f. The Rayleigh wave phase velocity ($V_R$) depends primarily on the material properties ($V_S$, mass density, and Poisson's ratio or compression wave velocity) over a depth of approximately one wavelength. The Love wave phase velocity ($V_L$) depends primarily on $V_S$ and mass density. Rayleigh and Love wave propagation are also affected by damping or seismic quality factor (Q). Rayleigh wave techniques are utilized to measure vertically polarized S-waves ($S_V$-wave); whereas Love wave techniques are utilized to measure horizontally polarized S-waves ($S_H$-wave).

## *2.2 Surface Wave Techniques*

The MASW and array microtremor techniques were utilized during this investigation and are discussed below.

### 2.2.1 MASW Technique

A description of the MASW method is given by Park, 1999a and 1999b and Foti, 2000. Ground motions are typically recorded by 24, or more, geophones typically spaced 1 to 3 m apart along a linear array and connected to a seismograph. Energy sources for shallow investigations include various sized hammers and vehicle mounted weight drops. When applying the MASW technique to develop a one-dimensional (1-D) $V_S$ model, it is preferable to use multiple-source offsets from both ends of the array. The most commonly applied MASW technique is the Rayleigh-wave based MASW method, which we refer to as $MAS_RW$ to distinguish from Love-wave based MASW ($MAS_LW$). $MAS_RW$ and $MAS_LW$ acquisition can easily be combined with P- and S-wave seismic refraction acquisition, respectively. $MAS_RW$ data are generally recorded using a vertical source and vertical geophone but may also be recorded using a horizontal geophone with radial (in-line) orientation. $MAS_LW$ data are recorded using transversely orientated horizontal source and transverse horizontal geophone.

A wavefield transform is applied to the time-history data to convert the seismic record from time-offset space to frequency-wavenumber (f-k) space in which the fundamental or higher surface-wave modes can be easily identified as energy maxima and picked. Frequency and/or wavenumber can easily be mapped to phase velocity, slowness, or wavelength using the following properties: $k = 2\pi/\lambda$, $\lambda = v/f$. Common wave-field transforms include: the f-k transform (a 2D fast Fourier transform), slant-stack transform (also referred to as intercept-slowness or τ-p transform and equivalent to linear Radon transform), frequency domain beamformer, and phase-shift transform. The minimum wavelength that can be recovered from MASW data set without spatial aliasing is equal to the minimum receiver spacing. Occasionally, SASW analysis procedures are used to extract surface wave dispersion data, from fixed receiver pairs, at smaller wavelengths than can be recovered by wavefield transformation. Construction of a dispersion curve over the wide frequency/wavelength range necessary to develop a robust $V_S$ model while also limiting the maximum wavelength based on an established near-field criterion (e.g., Yoon and Rix, 2009; Li and Rosenblad, 2011), generally requires multiple source offsets.

Although the clear majority of MASW surveys record Rayleigh waves, it has been shown that Love wave techniques can be more effective in some environments, particularly shallow rock sites and sites with a highly attenuative, low velocity surface layer (Xia, et al., 2012; **GEO***Vision*, 2012; Yong, et al., 2013; Martin, et al., 2014). Rayleigh wave techniques, however, are generally more effective at sites where velocity gradually increases with depth because larger energy sources are readily available for the generation of Rayleigh waves. Rayleigh wave techniques are also more applicable to sites with high velocity layers and/or velocity inversions because the presence of such structures is more apparent in the Rayleigh wave dispersion curves than in Love wave dispersion curves. Rayleigh wave techniques are preferable at sites with a high velocity surface layer because Love waves do not theoretically exist in such environments. Occasionally, the horizontal radial component of a Rayleigh wave may yield higher quality dispersion data than the vertical component because different modes of propagation may have more energy in one component than the other. Recording both the vertical and horizontal components of the Rayleigh wave is particularly useful at sites with complex modes of propagation or when attempting to recover multiple Rayleigh wave modes for multi-mode modeling as demonstrated in Dal Moro, et al, 2015. Joint inversion of Rayleigh and Love wave data may yield more accurate $V_S$ models and offer a means to investigate anisotropy, where $S_V$- and $S_H$-wave velocity are not equal, as shown in Dal Moro and Ferigo, 2011.

### 2.2.2 Array Microtremor Technique

A detailed discussion of the array microtremor method can be found in Okada, 2003. Unlike active source techniques which use an active energy source (i.e., hammer), the array microtremor technique (also referred to as passive surface wave or array ambient vibration method) records background noise (ambient vibrations) emanating from ocean wave activity, wind noise, traffic, industrial activity, construction, etc. The technique uses 4, or more, receivers aligned in a 2-dimensional array. Triangle, circle, semi-circle, and "L" shaped arrays are commonly used, although any 2-dimensional arrangement of receivers can be used. For investigations of the upper 100 m, receivers typically consist of 1 to 4.5 Hz geophones. For deeper investigations, 5 to 120 s seismometers are generally utilized. The nested triangle array, which consists of several embedded equilateral triangles, is popular as it provides accurate dispersion curves with a relatively small number of geophones. The "L" array is useful at sites located at the corner of intersecting streets. The maximum receiver separation in an array should be at a minimum equal to the desired depth of investigation. Typically, 15 to 60 minutes of ambient vibration data is recorded depending on the size of the array, desired depth of investigation, and noise conditions. Investigations to depths on the order of 1 km may require that ambient vibrations are recorded for a much longer duration. The surface wave dispersion curve is typically estimated from array microtremor data using various f-k methods such as beamforming (Lacoss, et al., 1969), and maximum-likelihood (Capon, 1969), and the spatial-autocorrelation (SPAC) method. The beam-forming and maximum-likelihood methods are generally referred to as the frequency wavenumber (FK) and high-resolution frequency wavenumber (HRFK or HFK) methods. The SPAC method was originally based on work by Aki, 1957 and has since been extended and modified (Ling and Okada, 1993 and Ohori *et al*., 2002) to permit the use of noncircular arrays, and is now collectively referred to as extended spatial autocorrelation (ESPAC or ESAC). Further modifications to the SPAC method permit the use of irregular or random arrays (Bettig *et al*., 2001). Although it is common to apply SPAC methods to obtain a surface wave dispersion curve for modeling, other approaches involve direct modeling of the coherency data, also referred to as SPAC coefficients (Asten, 2006 and Asten, *et al*., 2015). The beam-forming and maximum-likelihood methods are generally referred to as the frequency wavenumber (FK) and high-resolution frequency wavenumber (HRFK or HFK) methods, respectively. More recently, a Rayleigh wave three-component beamforming method (RTBF) has been developed (Wathelet, et al., 2018) and appears to offer significant resolution enhancements over other methods.

FK, HRFK and RTBF methods are generally expected to perform better when ambient vibration sources are not azimuthally well-distributed (e.g., rural area where the primary noise source is a large industrial facility). SPAC methods are expected to perform better when noise sources are azimuthally well-distributed (e.g., in a large, urbanized area).

The minimum wavelength surface wave that can be extracted from an array microtremor dataset acquired utilizing a symmetric array is typically set equal to the minimum receiver spacing. The maximum wavelength is often set equal to twice the maximum receiver separation for SPAC analysis and the maximum receiver spacing for FK analysis.

## 2.3 Surface Wave Dispersion Curve Modeling

The dispersion curves generated from the active and passive surface wave soundings are generally combined and modeled using iterative forward and inverse modeling routines. The final model profile is assumed to represent actual site conditions. The theoretical model used to interpret the dispersion curve assumes horizontally layered, laterally invariant, homogeneous-isotropic material. Although these conditions are seldom strictly met at a site, the results of active and/or passive surface wave testing provide a good "global" estimate of the material properties along the array. The results may be more representative of the site than a borehole "point" estimate.

The surface wave forward problem is typically solved using the Thomson-Haskell transfer-matrix (Thomson, 1950; Haskell, 1953) later modified by Dunkin (1965) and Knopoff (1964), dynamic stiffness matrix (Kausel and Roësset, 1981), or reflection and transmission coefficient (Kennett, 1974) methods. All of these methods can determine fundamental- and higher-mode phase velocities, which correspond to plane waves in 2-D space. The transfer-matrix method is often used in MASW and passive surface-wave software packages, whereas the dynamic stiffness matrix is utilized in many SASW software packages. $MAS_RW$ and/or passive surface-wave modeling may involve modeling of the fundamental mode, some form of effective mode, or multiple individual modes (multi-mode). As outlined in Roësset et al. (1991), several options exist for forward modeling of Rayleigh wave SASW data. One formulation considers only fundamental mode plane Rayleigh-wave motion (called the 2-D solution), whereas another includes all stress waves (e.g., body, fundamental, and higher mode surface waves) and incorporates a generalized receiver geometry (3-D global solution) or actual receiver geometry (3-D array solution).

The fundamental mode assumption is generally applicable to modeling Rayleigh-wave dispersion data collected at normally dispersive sites, providing there are not abrupt increases in velocity or steep velocity gradients. Effective-mode or multi-mode approaches are often required for irregularly dispersive sites and sites with steep velocity gradients at shallow depth. If active and passive surface wave data are combined or $MAS_RW$ data are combined from multiple seismic records with different source offsets and receiver gathers, then effective-mode computations are limited to algorithms that assume far-field plane Rayleigh wave propagation. Local search (e.g., linearized matrix inversion methods) or global search methods (e.g., Monte Carlo approaches such as simulated annealing, generic algorithm and neighborhood algorithm) are typically used to solve the inverse problem.

The maximum wavelength ($\lambda_{max}$) recovered from a surface wave data set is typically used to estimate depth of investigation although a sensitivity analysis of the $V_S$ models would be a more robust means to estimate depth of investigation. For normally dispersive velocity profiles with a gradual increase in $V_S$ with depth, the maximum depth of investigation is on the order of $\lambda_{max}/2$ for both Rayleigh and Love wave dispersion data. For velocity profiles with an abrupt increase in $V_S$ at depth, the maximum depth of investigation is on the order of $\lambda_{max}/3$ for Rayleigh wave dispersion data but less than $\lambda_{max}/3$ for Love wave dispersion data. The depth of investigation can be highly variable for sites with complex velocity structure (e.g., high velocity layers).

As with all other surface geophysical methods, the inversion of surface wave dispersion data does not yield a unique $V_S$ model and multiple possible solutions may equally fit the experimental data. Based on experience at other sites, the shear wave velocity models ($V_S$ and layer thicknesses) determined by surface wave testing are within 20% of the velocities and layer thicknesses that would be determined by other seismic methods (Brown, 1998). The average velocity of the upper 30 m, however, is much more accurate, often to better than 5%, because it is not sensitive to the layering in the model. $V_{S30}$ does not appear to suffer from the non-uniqueness inherent in $V_S$ models derived from surface wave dispersion curves (Martin et al., 2006, Comina et al., 2011). Therefore, $V_{S30}$ is more accurately estimated from the inversion of surface wave dispersion data than the resulting $V_S$ models.

It may not always be possible to develop a coherent, fundamental mode dispersion curve over sufficient frequency range for modeling due to dominant higher modes with the higher modes not clearly identifiable for multi-mode modeling. It may, however, be possible to identify the Rayleigh wave phase velocity of the fundamental mode at 40 m wavelength ($V_{R40}$) in which case $V_{S30}$ can at least be estimated using the Brown et al., 2000 relationship:

$$V_{S30} = 1.045V_{R40}$$

This relationship was established based on a statistical analysis of many surface wave data sets from sites with control by velocities measured in nearby boreholes and has been further evaluated by Martin and Diehl, 2004, and Albarello and Gargani, 2010. Further investigation of this approach has revealed that $V_{S30}$ is generally between $V_{R40}$ and $V_{R45}$ with $V_{R40}$ often being most appropriate for shallow groundwater sites and $V_{R45}$ for deep ground water sites. A detailed study of such an approach for Love wave dispersion data has not been conducted; however, preliminary analysis demonstrates that $V_{S30}$ is generally between $V_{L50}$ and $V_{L55}$. Although we do not recommend that these empirical $V_{S30}$ estimates replace modeling of surface wave dispersion data, they do offer a means of cost effectively evaluating $V_{S30}$ over a large area. $V_{R40}$ or $V_{L55}$ can also be used to quantify error in $V_{S30}$ by evaluating the scatter in the dispersion data at these wavelengths.

# 3 FIELD PROCEDURES

The active- and passive-source surface wave sounding locations at the site were established by **GEO***Vision* personnel and are shown in Figure 1. Two types of surface wave data were acquired at the site: an active-source surface wave array to characterize near-surface velocity structure and a passive-source surface wave array to characterize deeper velocity structure. Passive-source surface wave data were acquired along Array 1 using the array microtremor method. Active-source surface wave data were acquired along Array 2 using the MASW technique. The locations of the surface wave arrays were surveyed using a Trimble R10 GPS system with the RTX differential correction service.

MASW equipment used during this investigation consisted of a Geometrics Geode signal enhancement seismograph, 4.5 Hz vertical geophones, seismic cable, a 4-lb hammer, 10-lb sledgehammer, and accelerated weight drop (AWD). MASW data were acquired along Array 2, a linear array of 48 geophones spaced 1.5 m (4.9 ft) apart for a length of 70.5 m (231 ft). Shot points were located between about 1.5 and 30 m (5 and 100 ft) from the end geophone locations, as space permitted, and at 8 station intervals in the interior of the array. The AWD was used as the energy source at all off-end source locations. The 4- and 10-lb hammers were utilized at the near-offset and interior source locations. Data from the transient impacts (hammers) were generally averaged 5 to 10 times to improve the signal-to-noise ratio. All field data were saved to hard disk and documented on field data acquisition forms.

The passive surface wave equipment consisted of Geometrics Atom nodal seismographs and 2 Hz vertical geophones. The L-shaped Array 1 consisted to two coincidently located 17-station L-shaped arrays with variable geophone spacing and maximum 89 and 95 m (292 and 311 ft) lengths for the legs of the array. Ambient noise measurements were made along each passive array for about 60 minutes with a 4-millisecond sample rate. Passive surface wave data were downloaded to a laptop computer for later processing. The field geometry and associated files names were documented in field notes.

# 4 DATA REDUCTION

The MASW data were reduced using the software Seismic Pro Surface V9 developed by Geogiga and multiple in-house scripts for various data extraction and formatting tasks, with all data reduction documented in a Microsoft Excel spreadsheet.

The following steps were used for data reduction:

- Input seismic records to be used for analysis into software package.
- Check and correct source and receiver geometry as necessary.
- Select offset range used for analysis (multiple offset ranges utilized for each seismic record as discussed below) and document in spreadsheet.
- Apply phase shift transform to seismic record to convert the data from time – offset to frequency – phase velocity space.
- Identify, pick, save, and document dispersion curve.
- Change the receiver offset range and repeat process.
- Repeat process for all seismic records.
- Use in-house script to apply near-field criteria with maximum wavelength set equal to 1.0 times the source to midpoint of receiver array distance.
- Use in-house script to merge multiple dispersion curves extracted from the MASW data collected along each seismic line for a specific source type (different source locations, different receiver offset ranges, etc.).
- Edit dispersion data, as necessary (e.g., delete poor quality curves and outliers).
- Calculate a representative dispersion curve at equal log-frequency or log-wavelength spacing for the MASW dispersion data using a moving average, polynomial curve fitting routine.

This unique data reduction strategy, which can involve combination of over 50 dispersion curves for a 1D sounding, is designed for characterizing sites with complex velocity structure that do not yield surface wave dispersion data over a wide frequency range from a single source type or source location. The data reduction strategy ensures that the dispersion curve selected for modeling is representative of average conditions beneath the array and spans as broad a frequency/wavelength range as possible while considering near field effects.

The array microtremor data were reduced using the Seisimager software package developed by Oyo Corporation/Geometrics, Inc. The processing sequence for implementation of the ESAC method in the SeisImager software package is as follows:

- Input all seismic records for a dataset into software.
- Apply time-segmentation routine to break the data file into multiple ~60 second time blocks.
- Load receiver geometry (x and y positions) for each channel in seismic record.
- Calculate the SPAC coefficients for each time block and average.
- Optionally, select a subset of receiver offset ranges for analysis (e.g., only select receiver pairs with multiple azimuths).

- For each frequency calculate the RMS error between the SPAC coefficients and a Bessel function of the first kind and order zero over a user defined phase velocity range and velocity step.
- Plot an image of RMS error as a function for frequency (f) and phase velocity (v).
- Identify and pick the dispersion curve as the continuous trend on the f-v image with the lowest RMS error.
- Repeat the process for all arrays.
- Use an in-house script to convert dispersion curves to appropriate format for editing.
- Edit dispersion data, as necessary, and use in-house script to combine all dispersion data after setting maximum wavelength to about 2 times the maximum receiver spacing.
- Calculate a representative dispersion curve for the passive dispersion data from each array using a moving average polynomial curve fitting routine.

The representative dispersion curves from the active and passive surface wave data were combined and the moving average polynomial curve fitting routine in WinSASW V3 was used to generate a composite representative dispersion curve for modeling. During this process, the active and passive surface wave dispersion data were given equal weights. An equal logarithm wavelength sample rate was used for the representative dispersion curve to reflect the gradual loss in model resolution with depth.

# 5 DATA MODELING

Surface wave data were modeled using the effective mode inversion routine in the Seisimager WaveEq software package. During this process, an initial velocity model was generated based on general characteristics of the dispersion curve and the inverse modeling routine utilized to adjust the layer $V_S$ until an acceptable agreement with the observed data was obtained. Layer thicknesses were adjusted, and the inversion process repeated until a $V_S$ model was developed with low RMS error between the observed and calculated dispersion curves. In many cases, once an acceptable $V_S$ model is developed, layer thicknesses are again adjusted, and the inversion process repeated to develop an ensemble of $V_S$ models with similar RMS error to quantify non-uniqueness. Because the primary purpose of this investigation was to estimate $V_{S30}$ ($V_{S100ft}$), it was not considered necessary to develop multiple $V_S$ models. Data inputs into the modeling software include layer thickness, S-wave velocity, P-wave velocity or Poisson's ratio, and mass density. P-wave velocity and mass density only have a very small influence (i.e., less than 10%) on the S-wave velocity model generated from a surface wave dispersion curve. However, realistic assumptions for P-wave velocity, which is significantly impacted by the location of the saturated zone, and mass density will slightly improve the accuracy of the S-wave velocity model.

Constant mass density values of 1.79 to 2.03 g/cm$^3$ (112 to 127 lb/ft$^3$) were used in the velocity profiles for subsurface sediments depending on P- and S-wave velocity. Within the normal range encountered in geotechnical engineering, variation in mass density has a negligible (±2%) effect on the estimated $V_S$ from surface wave dispersion data. During modeling of Rayleigh wave dispersion data, the compression wave velocity, $V_P$, for unsaturated sediments was estimated using a Poisson's ratio, *v*, of 0.33 and the relationship:

$$V_P = V_S\,[(2(1-v))/(1-2v)]^{0.5}$$

Poisson's ratio has a larger effect than density on the estimated $V_S$ from Rayleigh wave dispersion data. Achenbach (1973) provides approximate relationship between Rayleigh wave velocity ($V_R$), $V_S$ and *v:*

$$V_R = V_S\,[(0.862 + 1.14\,v)/(1+ v)]$$

Using this relationship, it can be shown that $V_S$ derived from $V_R$ only varies by about 10% over possible 0 to 0.5 range for Poisson's ratio where:

$$V_S = 1.16V_R \text{ for } v = 0$$
$$V_S = 1.05V_R \text{ for } v = 0.5$$

The realistic range of Poisson's ratio for typical unsaturated sediments is about 0.25 to 0.35. Over this range, $V_S$ derived from modeling of Rayleigh wave dispersion data will vary by about 5%. There is no evidence of shallow groundwater based on the seismic refraction first arrival data. For the purpose of data modeling, it was assumed that the depth to the high Poisson's ratio, saturated zone is about 30 m (100 ft).

# 6 INTERPRETATION AND RESULTS

The fit of the calculated fundamental mode dispersion curve to the experimental data collected along Arrays 1 and 2 and the modeled $V_S$ profile for the surface wave sounding are presented as Figure 2. The resolution decreases gradually with depth due to the loss of sensitivity of the dispersion curve to changes in $V_S$ at greater depth. The $V_S$ profile used to match the field data is provided in tabular form as Tables 1 and 2.

The $V_S$ models were developed from the surface wave dispersion data derived from array microtremor and MASW data acquired along Arrays 1 and 2, respectively. The Rayleigh wave phase velocities from the passive surface wave array are in good agreement with those from the MASW data in the region where they overlap. Scatter in dispersion data from the two methods are expected to be associated with lateral velocity variability beneath the arrays. The estimated depth of investigation for the combined active and passive surface wave sounding is about 100 m (330 ft), about 50% of the maximum Rayleigh wave wavelength.

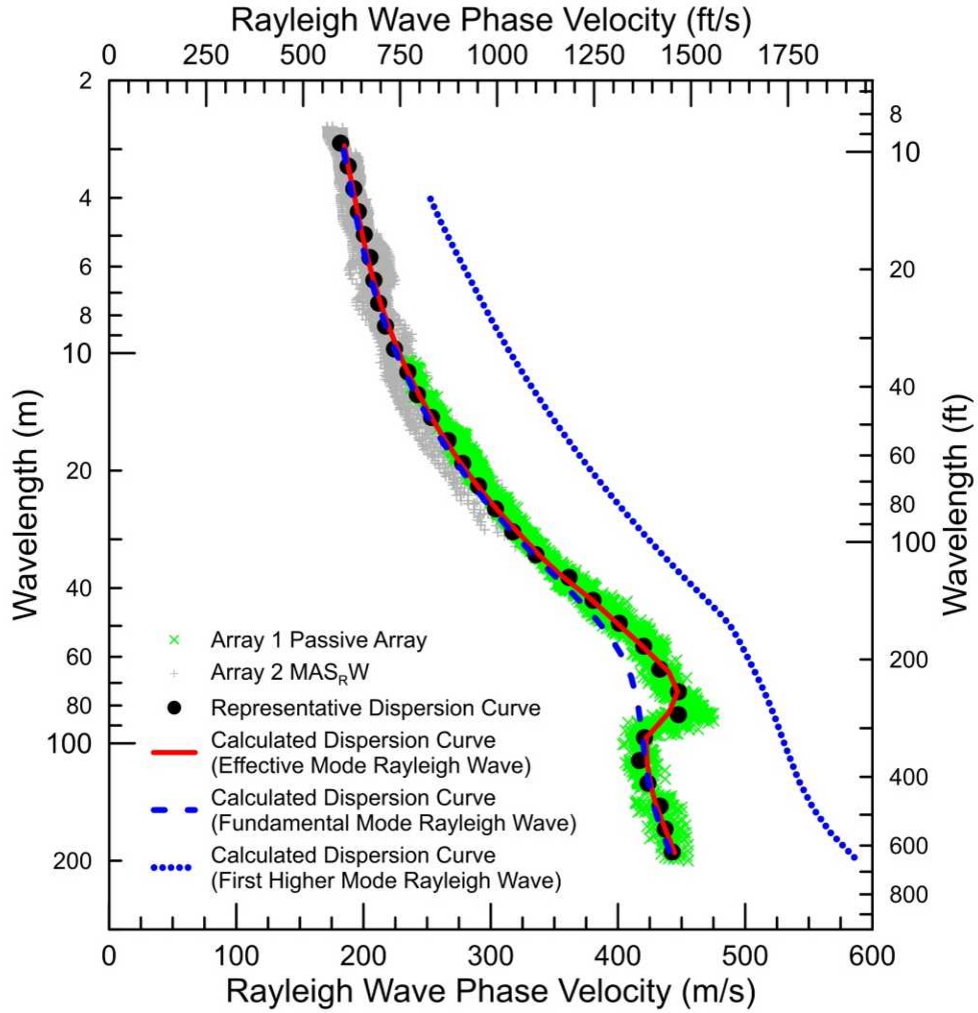
$V_{S30}$ and $V_{S100ft}$ are 366 m/s and 1,206 ft/s, respectively.

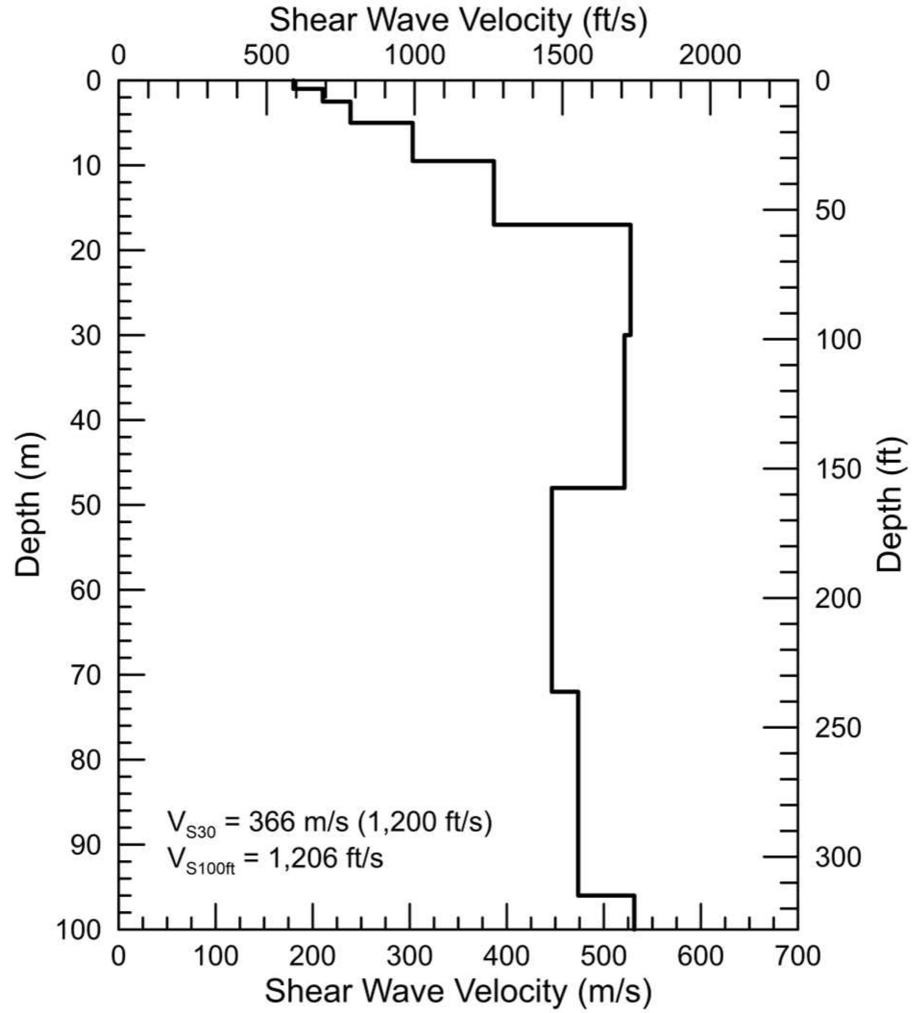
**Table 1 Arrays 1 and 2 $V_S$ Model (Metric Units)**

| Depth to Top of Layer (m) | Layer Thickness (m) | S-Wave Velocity (m/s) | Inferred P-Wave Velocity (m/s) | Inferred Poisson's Ratio | Inferred Density (g/cm$^3$) |
|---|---|---|---|---|---|
| 0 | 1 | 180 | 359 | 0.332 | 1.79 |
| 1 | 1.5 | 210 | 421 | 0.334 | 1.83 |
| 2.5 | 2.5 | 239 | 478 | 0.334 | 1.88 |
| 5 | 4.5 | 303 | 606 | 0.333 | 1.93 |
| 9.5 | 7.5 | 387 | 774 | 0.334 | 1.97 |
| 17 | 13 | 528 | 1055 | 0.333 | 2.03 |
| 30 | 18 | 521 | 1869 | 0.458 | 2.03 |
| 48 | 24 | 446 | 1785 | 0.467 | 2.00 |
| 72 | 24 | 474 | 1816 | 0.464 | 2.01 |
| 96 | Half Space | 531 | 1881 | 0.457 | 2.03 |

**Table 2 Arrays 1 and 2 Vs Model (Imperial Units)**

| Depth to Top of Layer (ft) | Layer Thickness (ft) | S-Wave Velocity (ft/s) | Inferred P-Wave Velocity (ft/s) | Inferred Poisson's Ratio | Inferred Density (lb/ft$^3$) |
|---|---|---|---|---|---|
| 0.0 | 3.3 | 591 | 1179 | 0.332 | 112 |
| 3.3 | 4.9 | 690 | 1382 | 0.334 | 114 |
| 8.2 | 8.2 | 784 | 1569 | 0.334 | 117 |
| 16.4 | 14.8 | 994 | 1988 | 0.333 | 120 |
| 31.2 | 24.6 | 1269 | 2540 | 0.334 | 123 |
| 55.8 | 42.7 | 1731 | 3461 | 0.333 | 127 |
| 98.4 | 59.1 | 1710 | 6131 | 0.458 | 127 |
| 157.5 | 78.7 | 1465 | 5857 | 0.467 | 125 |
| 236.2 | 78.7 | 1553 | 5958 | 0.464 | 125 |
| 315.0 | Half Space | 1743 | 6170 | 0.457 | 127 |

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Project No: 22235
Date: JUN 14, 2022
Drawn By: A MARTIN
Approved By: Antony J Martin

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FIGURE 2
SURFACE WAVE MODEL - ARRAYS 1 AND 2

1727-1829 E SACRAMENTO STREET
LOS ANGELES, CALIFORNIA

PREPARED FOR
ENGEO, INC.

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# 8 CERTIFICATION

All geophysical data, analysis, interpretations, conclusions, and recommendations in this document have been prepared under the supervision of and reviewed by a **GEO*Vision*** California Professional Geophysicist.

Reviewed and approved by,

6/15/2022

Antony J. Martin — Date
California Professional Geophysicist, P. Gp.
**GEO***Vision* Geophysical Services

* This geophysical investigation was conducted under the supervision of a California Professional Geophysicist using industry standard methods and equipment. A high degree of professionalism was maintained during all aspects of the project from the field investigation and data acquisition, through data processing interpretation and reporting. All original field data files, field notes and observations, and other pertinent information are maintained in the project files and are available for the client to review for a period of at least one year.

A professional geophysicist's certification of interpreted geophysical conditions comprises a declaration of his/her professional judgment. It does not constitute a warranty or guarantee, expressed or implied, nor does it relieve any other party of its responsibility to abide by contract documents, applicable codes, standards, regulations, or ordinances.

# Appendix IS-6.2

## Response to City of Los Angeles Soils Report Review Letter

**Geotechnologies, Inc.**
*Consulting Geotechnical Engineers*

439 Western Avenue
Glendale, California 91201-2837
818.240.9600 • Fax 818.240.9675

Celebrating 50 YEARS OF SERVICE 1971-2021

October 13, 2022
File No. 21971

SCD 1811 Sacramento, LLC
633 West 5th Street, Floor 68
Los Angeles, California 90071

Attention: Fei Ye

Subject: Response to City of Los Angeles Soils Report Review Letter
Proposed Office Development
1727 through 1829 East Sacramento Street, Los Angeles, California

References: *Reports by Geotechnologies, Inc.:*
Geotechnical Engineering Investigation dated January 25, 2022, updated August 22, 2022.

*City of Los Angeles Department of Building and Safety Correspondence:*
Soils Report Review Letter, dated October 7, 2022, Log # 123199.

Dear Ms. Ye:

This firm is in receipt of the referenced Soils Report Review Letter, dated October 7, 2022, issued by the City of Los Angeles, Department of Building and Safety. Therein, two comments are made which require input from this office. The comments are repeated below and the response immediately follows. A copy of the correction letter has been enclosed for reference.

Comment 1: *The consultant shall provide a statement that reference report by GEOVision dated 06/15/2022 was reviewed, that they concur with or do not concur with the findings contained therein, and that they will accept professional responsibility for the use of any data from others. P/BC 2020-113.*

Response: The Surface Wave Measurement Report prepared by GEOVision, dated June 15, 2022 was reviewed by Geotechnologies, Inc. Geotechnologies, Inc. concurs with the findings of this report. Furthermore, Geotechnologies, Inc. accepts professional responsibility for the use of any data from this report.

Comment 2: *Verify the current legal description and addresses (for all lots part of the project site) with the Address Section of the Bureau of Engineering located on the Third Floor of the 201 N. Figueroa Street, City of Los Angeles offices.*

Response: Based on information obtained from the City of Los Angeles Bureau of Engineering, the site's legal description and site address is as follows:

**Site Legal Description:**

TRACT: Thomas Leahy's Subdivision of the Eight Street Tract (MR 55-93/95)
BLOCK: 2
LOTS: 19 (arb 1), 20 (arb 1), 21 (arb 1), 22 (arb 1), 23 (arb 1), 24 (arb 1), 25 (arb 1), 26 (arb 1), 27, 28, 29, 30, 31, 32, 33, 34, FR 35 and FR 36.

**Site Address:**

1727, 1731, 1735, 1801, 1805, 1811, 1815, 1817, 1821, 1825, and 1829 East Sacrament Street, Los Angeles, California

Geotechnologies, Inc. appreciates the opportunity to provide our services on this project. Should you have any questions please contact this office.

Respectfully submitted,
GEOTECHNOLOGIES, INC.

REGISTERED PROFESSIONAL ENGINEER
GREGORIO VARELA
No. 81201
Exp. 9/30/23
CIVIL
STATE OF CALIFORNIA

GREGORIO VARELA
R.C.E. 81201

GV:km

Enclosure: Soils Report Review Letter dated October 7, 2022 (2 pages)

Distribution: (2) City of Los Angeles, Department of Building and Safety

CITY OF LOS ANGELES
CALIFORNIA

ERIC GARCETTI
MAYOR

DEPARTMENT OF
BUILDING AND SAFETY
201 NORTH FIGUEROA STREET
LOS ANGELES, CA 90012

OSAMA YOUNAN, P.E.
GENERAL MANAGER
SUPERINTENDENT OF BUILDING

JOHN WEIGHT
EXECUTIVE OFFICER

# SOILS REPORT REVIEW LETTER

October 7, 2022

LOG # 123199
SOILS/GEOLOGY FILE - 2

SCD 1811 Sacramento, LLC
633 West 5th Street, 68th Floor
Los Angeles, CA 90071

| | |
|---|---|
| TRACT: | Thomas Leahy's Subdivision of the Eighth Street Tract (MR 55-93/95) |
| BLOCK: | 2 |
| LOT(S): | 19 (Arb 1), 20 (Arb 1), 21 (Arb 1), 22 (Arb 1), 23 (Arb 1), 24 (Arb 1), 25 (Arb 1), 26 (Arb 1), 27, 28, 29, 30, 31, 32, 33, 34, FR 35 & FR 36 |
| LOCATION: | 1727 to 1829 E. Sacramento Street |

| CURRENT REFERENCE REPORT/LETTER(S) | REPORT No. | DATE OF DOCUMENT | PREPARED BY |
|---|---|---|---|
| Soils Report | 21971 | 08/22/2022 | Geotechnologies, Inc. |
| Surface Wave Measurements Report | 22235-01 | 06/15/2022 | GEOVision |

The Grading Division of the Department of Building and Safety has reviewed the referenced reports that provide recommendations for the proposed office development consisting of a 15-story office building to be constructed at or near existing site grade. Levels 1 through 6 will consist of a podium that will include parking, offices and mixed-use space. Levels 6 through 15 will be on the eastern portion of the structure and will consist of office space, as shown on the Plot Plan in the 08/22/2022 report. A storm water infiltration system consisting of a dry well is recommended by the consultants. According to the consultants, the site is currently developed with three (3) 1-story warehouse buildings at grade and paved parking lots.

Three borings were drilled to depths ranging from 30 to 55 feet. The earth materials at the subsurface exploration locations consist of up to 7 feet of uncertified fill underlain by native alluvium soils. According to the consultants, groundwater was not encountered to the maximum depth explored of 55 feet, and historically highest groundwater level is at about 145 feet below the ground surface. The site is relatively level.

The consultants recommend to support the proposed structure(s) on conventional (6-story portion of structure) and/or mat (15-story portion of structure) foundations bearing on a blanket of properly placed fill, a minimum of 3 feet thick below the bottom of the foundations and/or native undisturbed alluvial soils (see pgs. 8, 9 & 16 of the 08/22/2022 report).

The review of the subject reports cannot be completed at this time and will be continued upon submittal of an addendum to the reports which shall include, but not be limited to, the following:

(Note: Numbers in parenthesis ( ) refer to applicable sections of the 2020 City of LA Building Code. P/BC numbers refer the applicable Information Bulletin. Information Bulletins can be accessed on the internet at LADBS.ORG.)

1. The consultants shall provide a statement that referenced report by GEOVision dated 06/15/2022 was reviewed, that they either concur with or do not concur with the findings contained therein, and that they will accept professional responsibility for the use of any data from others. P/BC 2020-113

2. Verify the current legal description and addresses (for all lots part of the project site) with the Address Section of the Bureau of Engineering located on the Third Floor of the 201 N. Figueroa Street, City of Los Angeles offices.

The soils engineer shall prepare a report containing an itemized response to the review items indicated in this letter. If clarification concerning the review letter is necessary, the report review engineer may be contacted. Two copies of the response report, including one unbound wet-signed original for archiving purposes, a pdf-copy of the complete report in flash drive, and the appropriate fees will be required for submittal.

GLEN RAAD
Geotechnical Engineer I

Log No. 123199
213-482-0480

cc: Geotechnologies, Inc., Project Consultant
GEOVision, Project Consultant
LA District Office

# Appendix IS-6.3

## Site-Specific Seismic Hazard Analysis

**Geotechnologies, Inc.**
*Consulting Geotechnical Engineers*

439 Western Avenue
Glendale, California 91201-2837
818.240.9600 • Fax 818.240.9675

Celebrating 1971-2021 50 YEARS OF SERVICE

July 7, 2022
Revised October 13, 2022
File No. 21971

SCD 1811 Sacramento, LLC
633 West 5th Street, Floor 68
Los Angeles, California 90071

Attention: Fei Ye

Subject: Site-Specific Seismic-Hazard Analysis
Proposed Office Development
1727 through 1829 East Sacramento Street, Los Angeles, California

Reference: *Report by Geotechnologies, Inc.:*
Geotechnical Engineering Investigation, dated January 25, 2022, revised August 22, 2022.

Dear Ms. Ye:

## INTRODUCTION

The intention of this letter is to present the site specific seismic-hazard analysis recently prepared for the site. The site-specific seismic-hazard analysis was performed by ENGEO, who specializes in ground motion studies. ENGEO's report is appended. Their results are summarized below.

ENGEO's involvement with this project is limited to the preparation of the enclosed Site-Specific Seismic-Hazard Analysis. Geotechnologies, Inc. remains the project's Geotechnical Engineer of Record. Geotechnologies, Inc. has reviewed the enclosed Site-Specific Seismic-Hazard Analysis prepared by ENGEO, dated June 30, 2022. Geotechnologies, Inc. concurs with the findings of this analysis. Furthermore, Geotechnologies, Inc. accepts professional responsibility for the use of any data from this analysis.

### Seismic Shearwave Velocity Measurements

At the direction of ENGEO, a Surface Wave Measurement has been recently performed at the site by GEOVision. The intention of this work was to measure the shear-wave velocity at the site, to obtain the corresponding site classification. This work was summarized in the Report of Surface Wave Measurement, dated June 15, 2022, enclosed in the attached report.

Based on the shear-wave velocities measured by GEOVision, the shear-wave velocity on the upper 30 meters of the site ($V_{S30}$) below the existing ground surface was determined to be 366 m/s. This $V_{S30}$ value corresponds to a site classification for seismic design of Site Class C, which corresponds to a "Very Dense Soil and Soft Rock" Profile, according to NEHRP.

Geotechnologies, Inc. has reviewed the report prepared by GEOVision. Geotechnologies, Inc. concurs with the findings of this report. Furthermore, Geotechnologies, Inc. accepts professional responsibility for the use of any data from this report.

**Site-Specific Ground Motion Evaluation (Response Spectrum)**

The table below provides a summary of the site-specific design acceleration parameters derived by ENGEO. These site-specific parameters may be used for the design of the proposed structure. A detailed discussion of the ground motion evaluation methodology and assumptions is provided in the enclosed Seismic-Hazards Analysis report.

| SITE SPECIFIC DESIGN ACCELERATION PARAMETERS | |
|---|---|
| Seismic Parameters | ASCE 7-16 Site Specific Site Class C |
| $S_S$ | 1.91g |
| $S_1$ | 0.68g |
| $S_{MS}$ | 2.12g |
| $S_{M1}$ | 1.25g |
| $S_{DS}$ | 1.41g |
| $S_{D1}$ | 0.84g |
| $PGA_M$ | 0.87g |

## CLOSURE

Except as supplemented herein, all other recommendations contained in our referenced geotechnical engineering investigation remain applicable for the proposed project.

Geotechnologies, Inc. appreciates the opportunity to provide our services on this project. Should you have any questions please contact this office.

Respectfully Submitted,
GEOTECHNOLOGIES, INC.

REGISTERED PROFESSIONAL ENGINEER
GREGORIO VARELA
No. 81201
Exp. 9/30/23
CIVIL
STATE OF CALIFORNIA

GREGORIO VARELA
R.C.E. 81201

GV:km

Enclosures: Seismic-Hazard Analysis by ENGEO revised June 30, 2022 (44 pages)

Distribution: (2) City of Los Angeles, Department of Building and Safety

# 1811-1825 SACRAMENTO STREET
LOS ANGELES, CALIFORNIA

## SEISMIC-HAZARD ANALYSIS

SUBMITTED TO
Mr. Bryan Haworth
Development Associate
Skanska USA Commercial Development
633 West 5th Street, Floor 68
Los Angeles, CA 90071

PREPARED BY
ENGEO Incorporated

June 29, 2022
Revised June 30, 2022

PROJECT NO.
20460.000.001



ENGEO
Expect Excellence

Project No.
**20460.000.001**

June 29, 2022
Revised June 30, 2022

Mr. Bryan Haworth
Development Associate
Skanska USA Commercial Development
633 West 5th Street, Floor 68
Los Angeles, CA 90071

Subject: 1811–1825 Sacramento Street
Los Angeles, California

**SEISMIC-HAZARD ANALYSIS**

Dear Mr. Haworth:

We are pleased to present the enclosed results of our seismic-hazard analysis (SHA) for the design of the proposed development located at 1811-1825 Sacramento Street in Los Angeles, California. We performed our SHA in accordance with the 2019 California Building Code (CBC), which references the 2016 version of the American Society of Civil Engineers document titled "Minimum Design Loads and Associated Criteria for Buildings and Other Structures," (ASCE 7-16).

If you have any questions or comments regarding this report, please call and we will be glad to discuss them with you.

Sincerely,

ENGEO Incorporated

REGISTERED PROFESSIONAL ENGINEER
DAVID P. TEAGUE
No. 90014
CIVIL
STATE OF CALIFORNIA

David Teague, PhD, PE

REGISTERED PROFESSIONAL ENGINEER
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GEOTECHNICAL
STATE OF CALIFORNIA

Jeff Fippin, GE

dpt/jaf/jf

# TABLE OF CONTENTS

# 1.0 INTRODUCTION

In this report, we discuss the development of seismic design parameters for the proposed project located at 1811-1825 Sacramento Street in Los Angeles, California. We developed seismic design parameters in accordance with the criteria in the 2019 California Building Code (CBC), which references the 2016 version of the American Society of Civil Engineers document titled "Minimum Design Loads and Associated Criteria for Buildings and Other Structures," (ASCE 7-16). We performed a site-specific seismic-hazard analysis (SHA) in accordance with Section 21.2 of ASCE 7-16 to develop the seismic design parameters.

## 1.1 PROJECT LOCATION AND DESCRIPTION

Figure 1 displays a Site Vicinity Map. The proposed project is bounded by existing warehouses on the north, Wilson Street to the east, Sacramento Street on the south, and an existing warehouse on the west. We understand from our discussions with Skanska USA Commercial Development (Skanska) and the project Structural Engineer, DCI Engineers (DCI), that the project consists of a new 15-story structure.

## 1.2 PURPOSE AND SCOPE

Our scope involved the following steps, which we describe in detail in this report.

- Perform geophysical testing at the site to obtain shear-wave velocity data to support our seismic-hazard analysis
- Perform seismic-hazard analysis to develop Risk-Targeted, Maximum Considered Earthquake ($MCE_R$) and Design Earthquake (DE) response spectra for the project site

We prepared this report for the exclusive use of Skanska and their consultants for design of this project. This document may not be reproduced in whole or in part by any means whatsoever, nor may it be quoted or excerpted without our express written consent.

# 2.0 GEOPHYSICAL TESTING

We retained the services of GEOVision Geophysical (GEOVision) to perform non-invasive surface-wave testing at the project site. A thorough explanation of GEOVision's data acquisition and interpretation is provided in their report, which is included in Appendix A. We provide a summary below.

Surface-wave testing generally involves: (1) measurement of wavefields with strong Rayleigh wave energy, (2) calculation of the dispersion of the measured Raleigh waves (i.e., phase velocity versus frequency), and (3) using the dispersion data to solve an inverse problem to develop shear-wave velocity (Vs) profiles. GEOVision performed both active- and passive-source surface-wave testing at the project site. Active-source testing utilized sledgehammers and an accelerated weight drop as seismic sources, while passive-source testing relied on ambient vibrations. The test locations are shown in Figure 2. GEOVision calculated representative dispersion data from each test array and combined them to develop a representative dispersion curve for the site. They then performed an inversion of the dispersion curve using an effective mode approach to develop a representative $V_S$ profile for the site. This profile is provided in Figure 2 of Appendix A.

# 3.0 SEISMIC-HAZARD ANALYSIS

In this section, we describe the seismic-hazard analysis (SHA) that we performed in accordance with Sections 21.2 through 21.4 of ASCE 7-16. Specifically, we completed the following tasks to develop $MCE_R$ and DE response spectra.

- Perform probabilistic seismic-hazard analysis (PSHA) to develop a risk-targeted, maximum-rotated uniform hazard response spectrum (UHS) corresponding to a 2-percent probability of exceedance in 50 years (2,475-year return period).

- Perform deterministic seismic-hazard analysis (DSHA) to develop a maximum rotated 84$^{th}$ percentile response spectrum considering scenario earthquakes. Ensure that the DSHA response spectrum satisfies the lower limit requirements of Supplement 1 of ASCE 7-16.

- Develop a site-specific $MCE_R$ response spectrum by taking the lesser of the results of the PSHA and DSHA.

- Compare the $MCE_R$ response spectrum developed in the previous step with 80 percent of the associated general response spectrum (i.e., the code minimum) to develop the recommended site-specific $MCE_R$ response spectrum.

- Take two-thirds of the $MCE_R$ response spectrum to develop the DE response spectrum.

- Develop design acceleration parameters in accordance with Section 21.4 of ASCE 7-16.

## 3.1 GROUND-MOTION MODELS AND SITE PARAMETERS

We used four semi-empirical ground-motion models (GMMs) from the Next Generation Attenuation West 2 (NGA West 2) project (Ancehta et al., 2014) in the seismic-hazard analysis for this project. These include Abrahamson et al. (2014) [ASK], Boore et al. (2014) [BSSA], Campbell and Bozorgnia (2014) [CB], and Chiou and Youngs (2014) [CY]. We performed our analysis using all four GMMs for a spectral damping of 5 percent of critical damping. We used a logic-tree approach and assigned equal weight (0.25) to the four GMMs in our analysis.

The ground-motion models incorporate “site parameters” to model how subsurface soil will amplify or attenuate ground motions as they propagate from underlying bedrock. These site parameters include:

- Time-averaged shear-wave velocity in the upper 100 feet or 30 meters, $V_{S30.}$
- Depth at which the shear-wave velocity ($V_S$) reaches 3,280 feet per second (ft/s) or 1.0 kilometers per second (km/s) ($z_{1.0}$).
- Depth at which $V_S$ reaches 8,200 ft/s or 2.5 km/s ($z_{2.5}$).

We calculated a $V_{S30}$ value of 1,206 ft/s or 366 meters per second (m/s). This $V_{S30}$ falls within the range of Site Class C as defined in Section 20.3 of ASCE 7-16. We used the Southern California Earthquake Center (SCEC) velocity model Version 4, as implemented in the USGS Site Data Application Software (OpenSHA), to estimate a $z_{1.0}$ and $z_{2.5}$ of 1,480 feet (0.45 kilometers, km) and 7,550 feet (2.3 km), respectively.

## 3.2 PROBABILISTIC SEISMIC-HAZARD ANALYSIS

### 3.2.1 Fault Database and Probabilistic Model

We performed a probabilistic seismic-hazard analysis (PSHA) to develop a uniform hazard response spectrum for a return period of 2,475 years for the project site. We utilized the Third California Earthquake Rupture Forecast model (UCERF3). We show the fault sources for this model in Figure 3. This is the most up-to-date rupture forecast model for the state of California and, as such, is required by ASCE 7-16. We calculated the seismic hazard using the standard methodology for hazard analysis (McGuire, 2004). The seismic-hazard calculations can be represented by the following equation, which is an application of the total-probability theorem.

$$H(a) = \sum_i v_i \iint P[A > a|m, r]\, f_{Mi}(\mathrm{m})\, f_{Ri|Mi}(\mathrm{r}, \mathrm{m})\, dr\, dm$$

In this equation, the hazard **H*(a)*** is the annual frequency of earthquakes that produce a ground motion amplitude ***A*** higher than ***a***. Amplitude ***A*** may represent peak ground acceleration, velocity, or it may represent spectral pseudo-spectral acceleration (PSa) at a given frequency. The summation in the equation shown extends over all sources (i.e., over all faults and areas). In the above equation, $v_i$ is the annual rate of earthquakes (with magnitude higher than some threshold $M_i$) in source ***i***, and $f_{Mi}(m)$ and $f_{Ri|Mi}(r,m)$ are the probability density functions on magnitude and distance, respectively. ***P[A > a|m, r]*** is the probability that an earthquake of magnitude ***m*** at distance ***r*** produces a ground-motion amplitude ***A*** at the site that is greater than ***a***. Seismic sources may be either faults or background seismicity zones; the specification of source geometries and the calculation of $f_{Ri|Mi}$, are performed differently for these two types of sources.

In Figure 4 we show the median component (RotD50) 2,475-year UHS for each GMM. We considered the weighted mean of all GMMs in our analyses. To convert the mean RotD50 response spectrum to maximum-rotated response spectrum, we applied the maximum rotation factors discussed in Shahi and Baker (2014). We also applied the mapped risk factors defined in Section 21.2.1.1 of ASCE 7-16 in order to develop a risk-targeted spectrum. We show the maximum-rotated, risk-targeted PSHA response spectrum in Figure 4. We tabulated these values in Table 3.2.1-1.

**TABLE 3.2.1-1: Development of Risk-Targeted, Maximum Rotated PSHA Response Spectrum**

| PERIOD (seconds) | MEAN RotD50 2,475-YEAR UHS (g) | MAXIMUM ROTATION FACTOR | RISK COEFFICIENT | MAX.-ROTATED, RISK TARGETED PSHA (g) |
|---|---|---|---|---|
| 0.010 | 0.871 | 1.190 | 0.902 | 0.935 |
| 0.020 | 0.878 | 1.190 | 0.902 | 0.943 |
| 0.030 | 0.925 | 1.190 | 0.902 | 0.993 |
| 0.050 | 1.095 | 1.190 | 0.902 | 1.176 |
| 0.075 | 1.380 | 1.190 | 0.902 | 1.481 |
| 0.083 | 1.464 | 1.190 | 0.902 | 1.572 |
| 0.100 | 1.619 | 1.190 | 0.902 | 1.738 |
| 0.150 | 1.904 | 1.200 | 0.902 | 2.061 |
| 0.178 | 1.987 | 1.206 | 0.902 | 2.160 |

| PERIOD (seconds) | MEAN RotD50 2,475-YEAR UHS (g) | MAXIMUM ROTATION FACTOR | RISK COEFFICIENT | MAX.-ROTATED, RISK TARGETED PSHA (g) |
|---|---|---|---|---|
| 0.200 | 2.043 | 1.210 | 0.902 | 2.230 |
| 0.250 | 2.110 | 1.220 | 0.902 | 2.321 |
| 0.300 | 2.136 | 1.220 | 0.902 | 2.350 |
| 0.400 | 2.014 | 1.230 | 0.901 | 2.232 |
| 0.415 | 1.985 | 1.230 | 0.901 | 2.200 |
| 0.500 | 1.838 | 1.230 | 0.901 | 2.037 |
| 0.750 | 1.416 | 1.240 | 0.900 | 1.580 |
| 0.890 | 1.237 | 1.240 | 0.899 | 1.380 |
| 1.000 | 1.115 | 1.240 | 0.899 | 1.243 |
| 1.500 | 0.727 | 1.240 | 0.899 | 0.811 |
| 2.000 | 0.563 | 1.240 | 0.899 | 0.627 |
| 3.000 | 0.386 | 1.250 | 0.899 | 0.434 |
| 4.000 | 0.273 | 1.260 | 0.899 | 0.309 |
| 5.000 | 0.205 | 1.260 | 0.899 | 0.233 |
| 7.500 | 0.117 | 1.280 | 0.899 | 0.135 |
| 8.000 | 0.108 | 1.282 | 0.899 | 0.124 |
| 10.000 | 0.074 | 1.290 | 0.899 | 0.086 |

### 3.2.2 Disaggregation of the Seismic Hazard

We disaggregated the seismic hazard associated with the 2,475-year return period at the peak ground acceleration and at spectral periods of 0.5, 1.0, and 2.0 seconds. Based on our discussions with DCI, we understand that the approximate fundamental period of the proposed structure is 1 second; therefore, this range of spectral periods encompasses the structural period range of interest. We present disaggregation results are in Appendix A. We summarize the dominant scenarios and their relative contributions to the hazard at each period in Table 3.2.2-1. Background seismicity zones are not presented. The rupture distance ($R_{RUP}$) and mean moment magnitude ($M_W$) are listed for each scenario. Note that the mean $M_W$ for each scenario varies with spectral period; thus, we show the maximum of these mean $M_W$ values from those periods where the source contributes significantly to the hazard. The numbers in brackets represent unique sub-sections on a given fault. Regional faulting and seismicity are shown in Figure 3.

**TABLE 3.2.2-1: Summary of Disaggregation Results for a 2,475-Year Return Period***

| SOURCE | $R_{RUP}$ | | $M_W$ | PERCENT CONTRIBUTION | | | |
|---|---|---|---|---|---|---|---|
| | (miles) | (km) | | PGA | 0.5 second | 1.0 second | 2.0 seconds |
| Compton [2] | 8.7 | 14.0 | 7.39 | 10.0 | 10.2 | 9.9 | 8.5 |
| Elysian Park (Upper) [1] | 4.1 | 6.6 | 7.18 | 21.2 | 22.1 | 22.3 | 20.4 |
| Newport-Inglewood alt 1 [6] | 8.0 | 12.9 | 7.60 | 3.1 | 3.4 | 3.7 | 3.9 |
| Newport-Inglewood alt 1 [8] | 7.8 | 12.5 | 7.11 | 2.0 | 2.1 | 2.1 | 1.9 |
| Newport-Inglewood alt 2 [6] | 7.9 | 12.7 | 7.61 | 2.5 | 2.7 | 3.0 | 3.1 |
| Newport-Inglewood alt 2 [7] | 7.4 | 11.9 | 7.21 | 2.7 | 3.0 | 3.0 | 2.8 |
| Puente Hills (LA) [1] | 2.6 | 4.2 | 7.20 | 9.9 | 10.4 | 10.6 | 10.0 |

| SOURCE | $R_{RUP}$ (miles) | $R_{RUP}$ (km) | $M_W$ | PERCENT CONTRIBUTION PGA | 0.5 second | 1.0 second | 2.0 seconds |
|---|---|---|---|---|---|---|---|
| Puente Hills (Santa Fe Springs) [1] | 7.1 | 11.4 | 7.16 | 4.0 | 4.3 | 4.1 | 3.7 |
| Puente Hills [3] | 3.8 | 6.1 | 7.37 | 1.1 | 1.2 | 1.3 | 1.4 |
| Puente Hills [4] | 3.6 | 5.8 | 7.23 | 7.3 | 7.6 | 8.1 | 8.1 |
| Raymond [1] | 8.3 | 13.3 | 7.55 | < 1.0 | < 1.0 | < 1.0 | 2.0 |
| Raymond [2] | 6.6 | 10.6 | 7.35 | 3.0 | 3.7 | 4.0 | 4.5 |
| San Andreas (Mojave S) [8] | 35.3 | 56.9 | 8.09 | < 1.0 | 2.2 | 2.7 | 5.5 |
| San Pedro Escarpment [0] | 7.7 | 12.3 | 7.61 | < 1.0 | 1.1 | 1.2 | 1.2 |
| Sierra Madre [5] | 13.4 | 21.6 | 7.76 | < 1.0 | 1.0 | 2.3 | 3.4 |

*Based on USGS Unified Hazard Tool: Dynamic Conterminous U.S. 2014 (update) (v4.2.0)

## 3.3 DETERMINISTIC SEISMIC-HAZARD ANALYSIS

The deterministic seismic-hazard analysis (DSHA) involves developing the 84$^{th}$-percentile (i.e., lognormal mean plus one standard deviation) maximum-rotated response spectrum for a spectral damping of 5 percent of critical damping considering characteristic magnitudes of significant faults, without background seismicity, and the aforementioned ground-motion models. However, it is important to note that the definition of the characteristic magnitude is ambiguous when using the UCERF3 model due to its complexity. Based on the 2020 NEHRP Provisions, in deterministic analyses, "scenario" earthquakes with significant contribution to hazard should be used in lieu of "characteristic" earthquakes when using UCERF3. We identified the scenario earthquakes by considering the results of the disaggregation of the PSHA results. Accordingly, we considered the scenarios in Table 3.2.2-1, as described below. In accordance with the 2020 NEHRP Provisions, we did not consider those scenarios contributing less than 10 percent of the largest contributor at each period.

We considered the magnitudes in Table 3.2.2-1 and associated distances ($R_{RUP}$, $R_{JB}$, $R_X$) to calculate 84$^{th}$ percentile deterministic response spectra. We estimated additional ground-motion model parameters (e.g., rupture width, depth to top of rupture, etc.) for each fault/scenario based on fault-specific information published by the United States Geologic Survey (USGS). We show the RotD50, 84$^{th}$ percentile deterministic response spectra in Figure 5. Note that each response spectrum represents the weighted average of the four GMMs for each source. Per Section 21.2.2 of ASCE 7-16, we enveloped the 84$^{th}$ percentile response spectra developed from the applicable sources listed in Table 3.2.2-1. At all periods, the Puente Hills (LA) [1] scenario governed. Similar to the probabilistic response spectrum, we applied the maximum rotation factors discussed in Shahi and Baker (2014) to develop a maximum-rotated 84$^{th}$ percentile deterministic response spectrum. We show this spectrum in Figure 5, and we tabulate the results in Table 3.3-1.

We compared the maximum-rotated 84$^{th}$ percentile deterministic response spectrum with the lower limit defined in Section 21.2.2 of ASCE 7-16 and Supplement No. 1. Per Supplement 1, the lower limit is defined as 1.5$F_a$, where $F_a$ is the short-period site coefficient corresponding to a short-period mapped acceleration ($S_1$) of 1.5 g. For Site Class C, the value of $F_a$ is 1.2 and the lower limit is 1.8. Since the maximum PSa of the maximum-rotated 84$^{th}$ percentile deterministic response spectrum exceeds 1.8, no scaling of the DSHA response spectrum is required.

**TABLE 3.3-1: Development of Maximum Rotated DSHA Response Spectrum**

| PERIOD (seconds) | 84th PERCENTILE RotD50 DSHA ENVELOPE (g) | MAXIMUM ROTATION FACTOR | MAXIMUM-ROTATED DSHA (g) |
|---|---|---|---|
| 0.010 | 0.907 | 1.190 | 1.080 |
| 0.020 | 0.912 | 1.190 | 1.085 |
| 0.030 | 0.946 | 1.190 | 1.125 |
| 0.050 | 1.086 | 1.190 | 1.292 |
| 0.075 | 1.294 | 1.190 | 1.540 |
| 0.083 | 1.359 | 1.190 | 1.617 |
| 0.100 | 1.478 | 1.190 | 1.759 |
| 0.150 | 1.764 | 1.200 | 2.117 |
| 0.178 | 1.916 | 1.206 | 2.310 |
| 0.200 | 2.019 | 1.210 | 2.443 |
| 0.250 | 2.178 | 1.220 | 2.657 |
| 0.300 | 2.304 | 1.220 | 2.811 |
| 0.400 | 2.279 | 1.230 | 2.803 |
| 0.415 | 2.249 | 1.230 | 2.767 |
| 0.500 | 2.101 | 1.230 | 2.584 |
| 0.750 | 1.689 | 1.240 | 2.094 |
| 0.890 | 1.475 | 1.240 | 1.828 |
| 1.000 | 1.329 | 1.240 | 1.647 |
| 1.500 | 0.878 | 1.240 | 1.088 |
| 2.000 | 0.687 | 1.240 | 0.852 |
| 3.000 | 0.480 | 1.250 | 0.600 |
| 4.000 | 0.323 | 1.260 | 0.407 |
| 5.000 | 0.230 | 1.260 | 0.290 |
| 7.500 | 0.103 | 1.280 | 0.131 |
| 8.000 | 0.093 | 1.282 | 0.119 |
| 10.000 | 0.059 | 1.290 | 0.077 |

## 3.4 NEAR-FAULT EFFECTS

Given the proximity of the site to several active faults, the site is considered “near-fault” per ASCE 7-16. Near-fault sites may be subject to rupture directivity effects. Rupture directivity effects can increase long-period ground motions relative to the ground-motion model predictions (Somerville et al., 1997; Abrahamson 2000). Accordingly, we utilized the Bayless and Somerville (2013) model to adjust the long-period ground motions to account for these effects. The PSHA and DSHA response spectra presented above incorporate these adjustments. Note that these adjustments only influenced periods greater than 1.5 seconds.

# 4.0 SITE-SPECIFIC $MCE_R$ AND DE RESPONSE SPECTRA

Per Section 21.2.3 of ASCE 7-16, the site-specific $MCE_R$ is the lesser of the maximum-rotated and risk-targeted probabilistic and the 84th percentile maximum-rotated deterministic response spectra. Additionally, the $MCE_R$ is not permitted to be lower than 80 percent of the general $MCE_R$ response spectrum for Site Class C (i.e., the code minimum). In Figure 6, we depict the recommended $MCE_R$ response spectrum. We also show the DE response spectrum, which is defined as two-thirds of the $MCE_R$. We tabulate our recommended site-specific $MCE_R$ and DE response spectra for the project site in Table 4.0-1. We provide the associated site-specific design acceleration parameters in accordance with Section 21.4 of ASCE 7-16 in Table 4.0-2.

**TABLE 4.0-1: Site-Specific $MCE_R$ and DE Response Spectra**

| PERIOD (seconds) | PSEUDO-SPECTRAL ACCELERATION (g) | | | | | |
|---|---|---|---|---|---|---|
| | GENERAL $MCE_R$ | 80% GENERAL $MCE_R$ | PSHA (MAX.-ROTATED & RISK ARGETED) | DSHA (MAX.-ROTATED) | SITE-SPECIFIC $MCE_R$ | SITE-SPECIFIC DE |
| 0.010 | 1.082 | 0.866 | 0.935 | 1.080 | 0.935 | 0.623 |
| 0.020 | 1.248 | 0.998 | 0.943 | 1.085 | 0.998 | 0.666 |
| 0.030 | 1.413 | 1.131 | 0.993 | 1.125 | 1.131 | 0.754 |
| 0.050 | 1.745 | 1.396 | 1.176 | 1.292 | 1.396 | 0.930 |
| 0.075 | 2.158 | 1.727 | 1.481 | 1.540 | 1.727 | 1.151 |
| 0.083 | 2.292 | 1.834 | 1.572 | 1.617 | 1.834 | 1.222 |
| 0.100 | 2.292 | 1.834 | 1.738 | 1.759 | 1.834 | 1.222 |
| 0.150 | 2.292 | 1.834 | 2.061 | 2.117 | 2.061 | 1.374 |
| 0.178 | 2.292 | 1.834 | 2.160 | 2.310 | 2.160 | 1.440 |
| 0.200 | 2.292 | 1.834 | 2.230 | 2.443 | 2.230 | 1.486 |
| 0.250 | 2.292 | 1.834 | 2.321 | 2.657 | 2.321 | 1.547 |
| 0.300 | 2.292 | 1.834 | 2.350 | 2.811 | 2.350 | 1.567 |
| 0.400 | 2.292 | 1.834 | 2.232 | 2.803 | 2.232 | 1.488 |
| 0.415 | 2.292 | 1.834 | 2.200 | 2.767 | 2.200 | 1.467 |
| 0.500 | 1.904 | 1.523 | 2.037 | 2.584 | 2.037 | 1.358 |
| 0.750 | 1.269 | 1.015 | 1.580 | 2.094 | 1.580 | 1.053 |
| 0.890 | 1.070 | 0.856 | 1.380 | 1.828 | 1.380 | 0.920 |
| 1.000 | 0.952 | 0.762 | 1.243 | 1.647 | 1.243 | 0.829 |
| 1.500 | 0.635 | 0.508 | 0.811 | 1.088 | 0.811 | 0.540 |
| 2.000 | 0.476 | 0.381 | 0.627 | 0.852 | 0.627 | 0.418 |
| 3.000 | 0.317 | 0.254 | 0.434 | 0.600 | 0.434 | 0.289 |
| 4.000 | 0.238 | 0.190 | 0.309 | 0.407 | 0.309 | 0.206 |
| 5.000 | 0.190 | 0.152 | 0.233 | 0.290 | 0.233 | 0.155 |
| 7.500 | 0.127 | 0.102 | 0.135 | 0.131 | 0.131 | 0.088 |
| 8.000 | 0.119 | 0.095 | 0.124 | 0.119 | 0.119 | 0.080 |
| 10.000 | 0.076 | 0.061 | 0.086 | 0.077 | 0.077 | 0.051 |

**TABLE 4.0-2: Site-Specific Design Acceleration Parameters Based on ASCE 7-16 Section 21.4 (Latitude: 34.0307, Longitude: -118.2353)**

| ACCELERATION PARAMETER | VALUE (g) |
|---|---|
| Mapped $MCE_R$ Spectral Response Acceleration at Short Periods, $S_S$ | 1.91 |
| Mapped $MCE_R$ Spectral Response Acceleration at 1-second Period, $S_1$ | 0.68 |
| $MCE_R$ Spectral Response Acceleration at Short Periods, $S_{MS}$ | 2.12 |
| $MCE_R$ Spectral Response Acceleration at 1-second Period, $S_{M1}$ | 1.25 |
| Design Spectral Response Acceleration at Short Periods, $S_{DS}$ | 1.41 |
| Design Spectral Response Acceleration at 1-second Period, $S_{D1}$ | 0.84 |
| $MCE_G$ peak ground acceleration adjusted for site class effects, $PGA_M$ | 0.87 |

## 5.0 CLOSURE

The conclusions and recommendation presented herein are professional opinions based on the geotechnical and geologic data at the date of the submittal and our understanding of the project as described. The report is intended solely for use by the client and its design consultants for the design of the planned redevelopment of 1811-1825 Sacramento Street in Los Angeles, California and should not be used for any other purposes, including, but not limited to, a higher level of seismic performance and the design of any additional structures. If the project changes from the description, we should be given the opportunity to evaluate such data and to modify these conditions and recommendations as appropriate.

The findings and professional opinions presented in this report are presented within the limits prescribed by the client, in accordance with generally accepted professional engineering and geologic practices.

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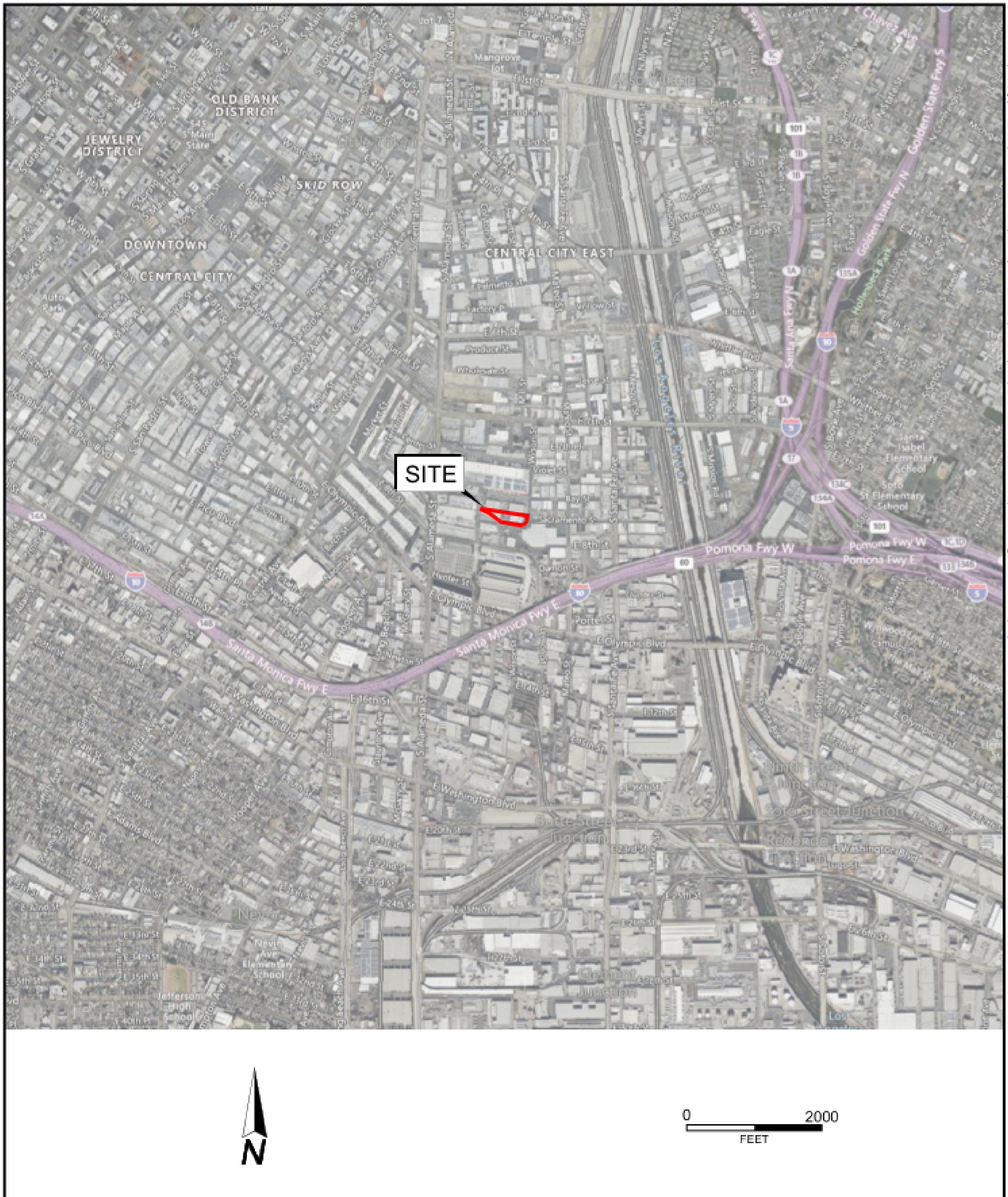
## FIGURES




BASE MAP SOURCE: BING MAPPING SERVICE

| ENGEO Expect Excellence | VICINITY MAP<br>1811 - 1825 SACRAMENTO STREET<br>LOS ANGELES, CALIFORNIA | PROJECT NO.: 20460.000.001<br>SCALE: AS SHOWN<br>DRAWN BY: SPPE CHECKED BY: JAF | FIGURE NO.<br>1 |
|---|---|---|---|

ORIGINAL FIGURE PRINTED IN COLOR

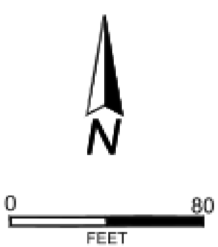
FILE PATH: G:\Drafting\PROJECTS_20000 to 21999\20460\20460000001\Geotech\GEX\20460000001-GEX-2-SitePlan-0622.dwg SAVE DATE: 6/17/2022 5:12:11 PM SAVED BY: SEspinoza


## EXPLANATION

*ALL LOCATIONS ARE APPROXIMATE*

ACTIVE SURFACE WAVE ARRAY (GEOVISION, 2022)

PASSIVE SURFACE WAVE ARRAY (GEOVISION, 2022)




*BASE MAP SOURCE: BING MAPPING SERVICE*

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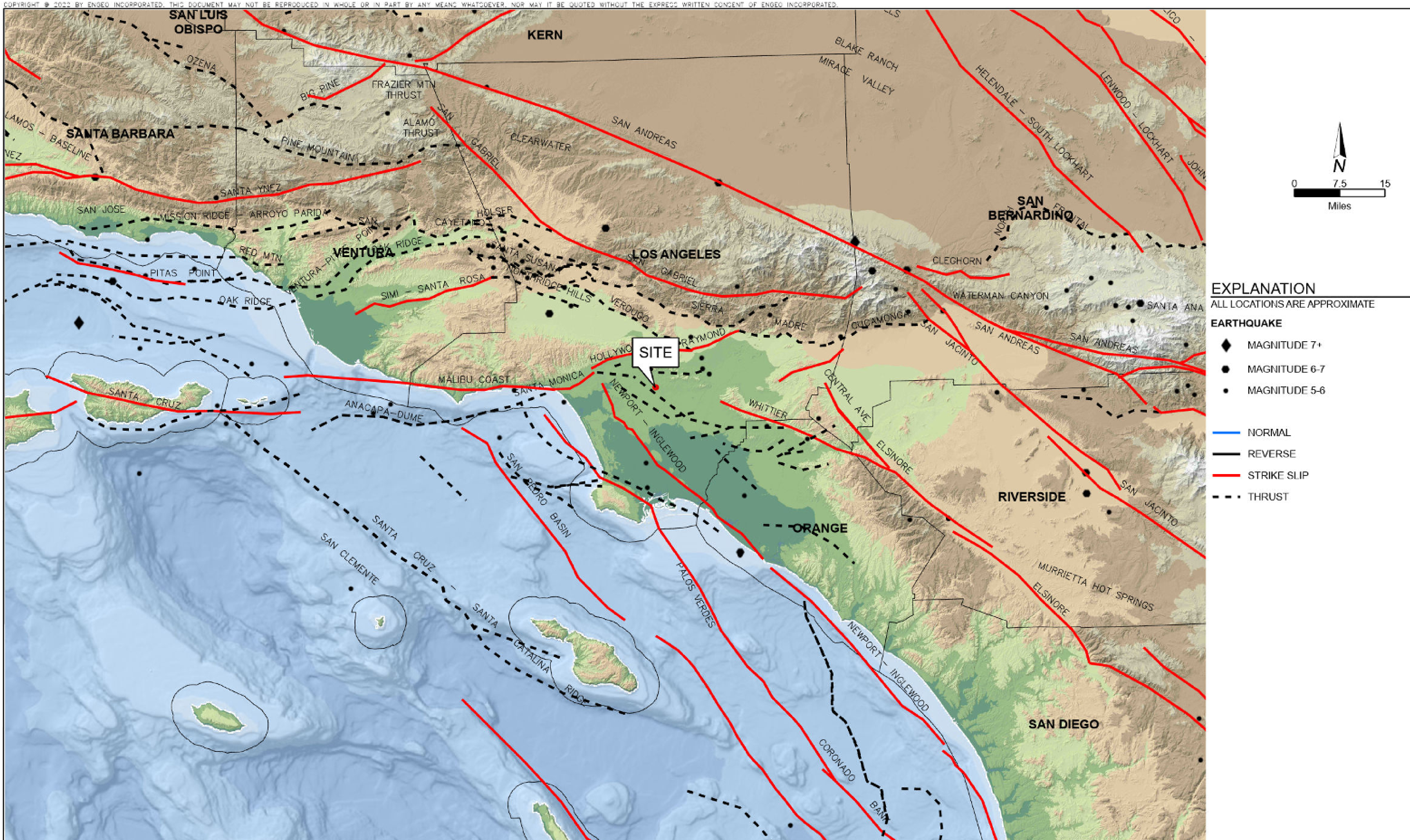
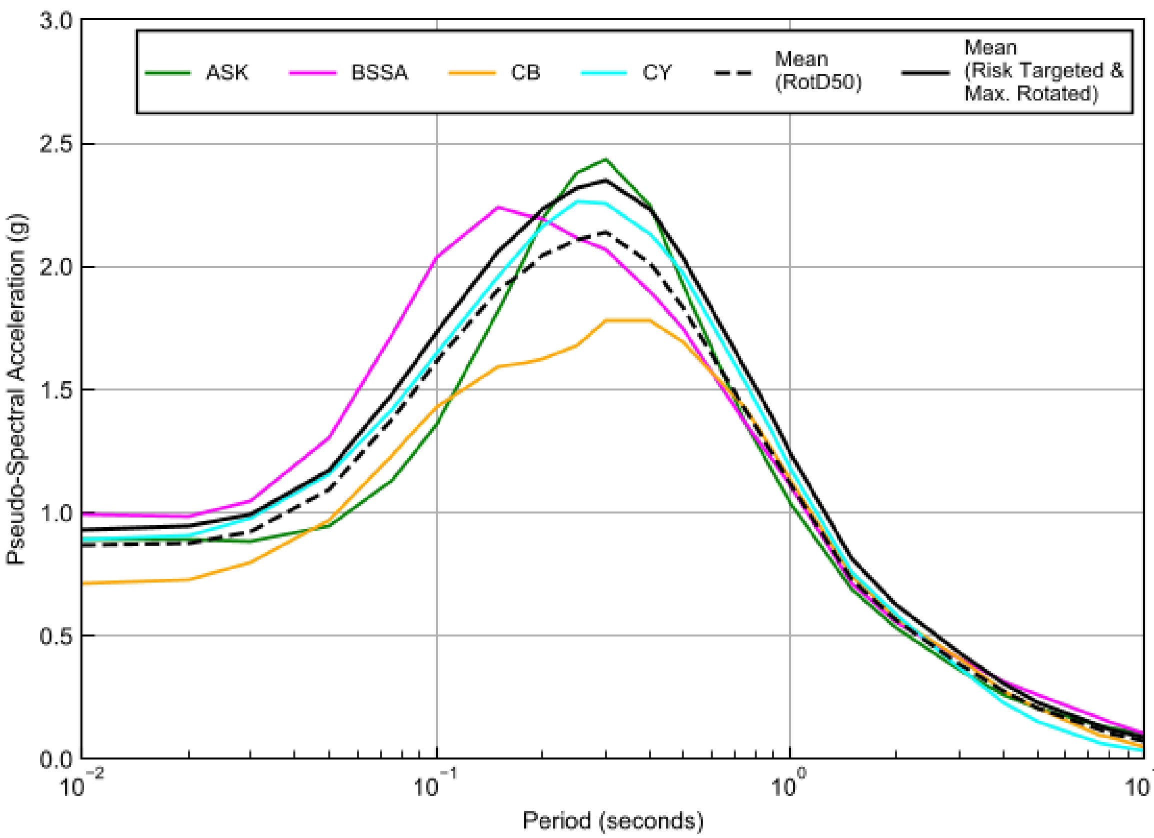



BASE MAP SOURCE
ESRI, GEBCO, DELORME, NATURALVUE
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATA SET (NED) AT 30 METER RESOLUTION
U.S.G.S OPEN-FILE REPORT 2014-1091

ENGEO
Expect Excellence

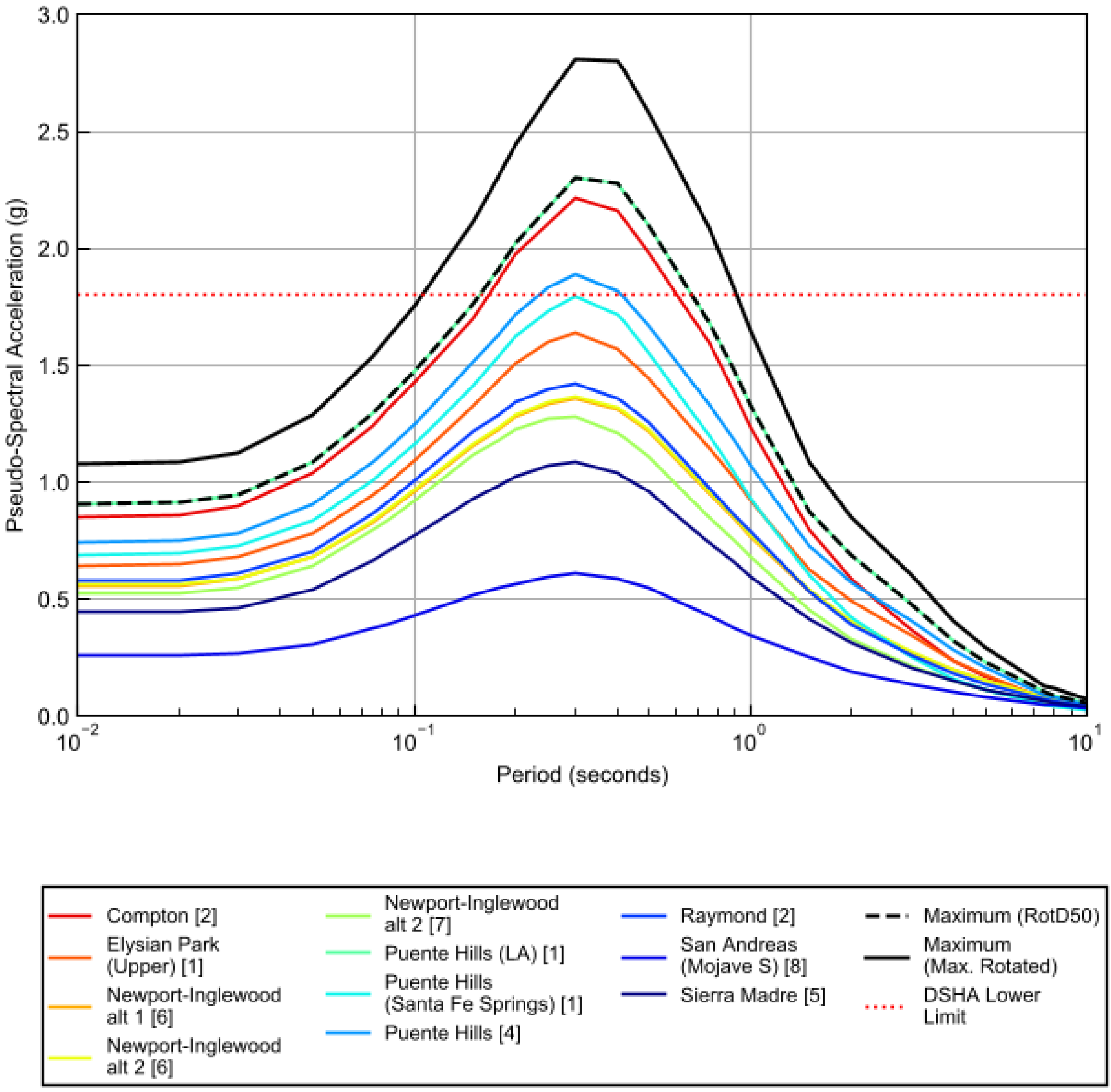
REGIONAL FAULTING AND SEISMICITY
1811 - 1825 SACRAMENTO STREET
LOS ANGELES, CALIFORNIA

| | |
|---|---|
| PROJECT NO.: 20460.000.001 | FIGURE NO. 3 |
| SCALE: AS SHOWN | |
| DRAWN BY: SPPE | CHECKED BY: JAF |






| ENGEO — Expect Excellence — | 2,475-YEAR UNIFORM HAZARD SPECTRA<br>1811-1825 SACRAMENTO STREET<br>LOS ANGELES, CALIFORNIA | PROJECT NO: 20460.000.001<br>SCALE: AS SHOWN<br>DRAWN BY: DPT CHECKED BY: JAF | FIGURE NO.<br>4 |
| --- | --- | --- | --- |



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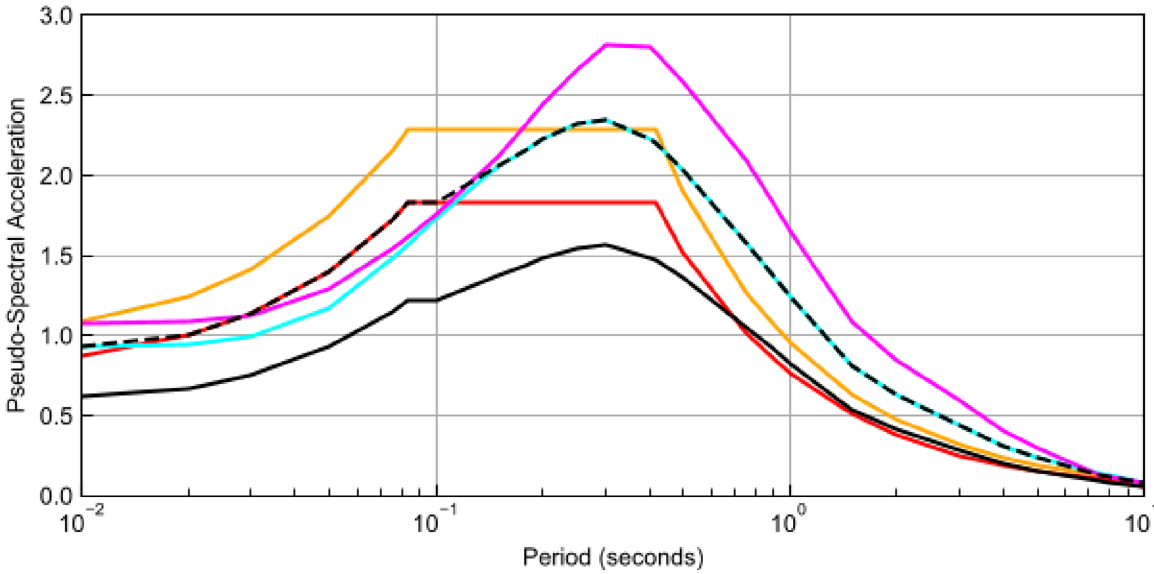
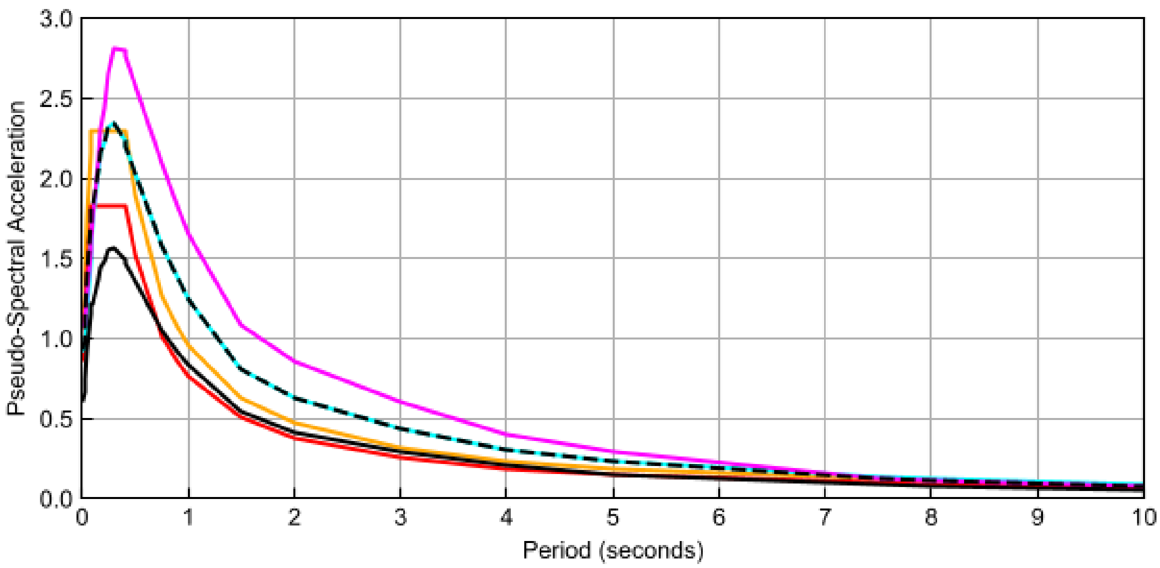
| | | | |
|---|---|---|---|
| 84$^{th}$ PERCENTILE DSHA RESPONSE SPECTRA<br>1811-1825 SACRAMENTO STREET<br>LOS ANGELES, CALIFORNIA | PROJECT NO: 20460.000.001 | | FIGURE NO. 5 |
| | SCALE: AS SHOWN | | |
| | DRAWN BY: DPT | CHECKED BY: JAF | |




| | | | | | |
|---|---|---|---|---|---|
| General $MCE_R$ | 80% General $MCE_R$ | PSHA (Max. Rotated & Risk Targeted) | DSHA (Max. Rotated) | Site-Specific $MCE_R$ | Site-Specific DE |



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SITE-SPECIFIC $MCE_R$ AND DE RESPONSE SPECTRA
1811-1825 SACRAMENTO STREET
LOS ANGELES, CALIFORNIA

| | | | |
|---|---|---|---|
| PROJECT NO: 20460.000.001 | | FIGURE NO. 6 | |
| SCALE: AS SHOWN | | | |
| DRAWN BY: DPT | CHECKED BY: JAF | | |

# APPENDIX A

## GEOPHYSICAL TEST RESULTS

# REPORT

# SURFACE WAVE MEASUREMENTS

# 1727–1829 E SACRAMENTO STREET
# LOS ANGELES, CALIFORNIA

**GEO***Vision* Project No. 22235

*Prepared for*

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(925) 866-9000

*Prepared by*


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Report 22235-01 Rev 0

June 15, 2022

# TABLE OF CONTENTS



# LIST OF TABLES



# LIST OF FIGURES

# 1 INTRODUCTION

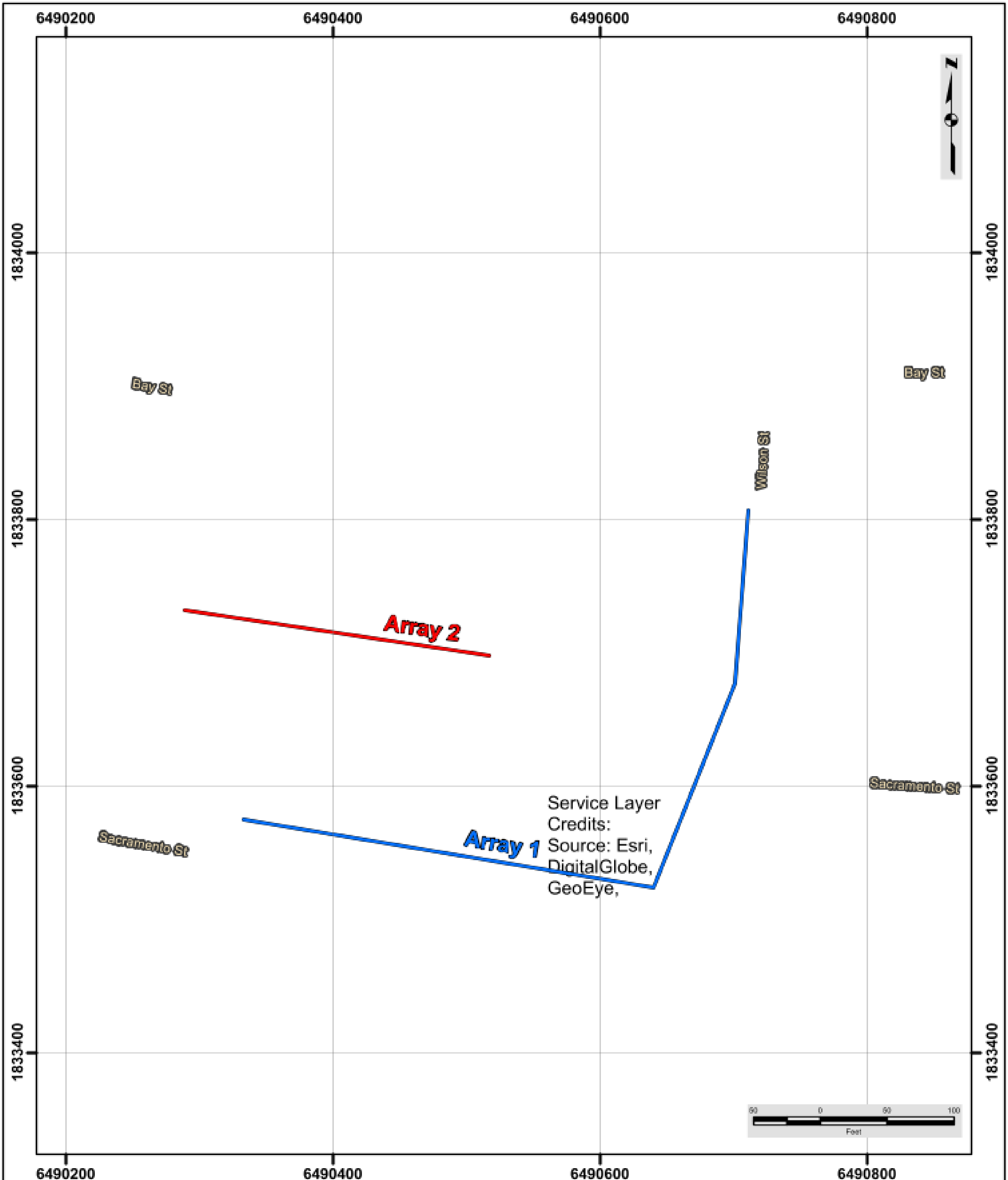
In-situ seismic measurements using active- and passive-source surface wave techniques were performed at 1727-1829 E. Sacramento Street, Los Angeles, California on June 1, 2022. The purpose of the investigation was to provide a shear (S) wave velocity profile to a depth of 30 m (100 ft), or greater, and estimate the average S-wave velocity of the upper 30 m ($V_{S30}$) and 100 ft ($V_{S100ft}$). The active-source surface wave technique utilized during this investigation consisted of the multi-channel analysis of surface waves (MASW) method. The passive-source surface wave technique consisted of the array microtremor method. The locations of the active- and passive-source surface wave testing locations are shown on Figure 1. Array microtremor measurements were made using an L-shaped array (Array 1) and MASW measurements were made on a linear array on the subject property (Array 2).

For seismic design, the 2019 California Buiilding Code (CBC) and 2018 International Building Code (IBC) reference the provisions in ASCE/SEI 7-16 (Minimum Design Loads and Associated Criteria for Buildings and Other Structures). The Site Classes and associated S-wave velocity ranges outlined in Table 20.3-1 of ASCE/SEI 7-16 are as follows:

Site Class A – Hard rock – $V_{S100ft} > 5,000$ ft/s
Site Class B – Rock –$2,500 < V_{S100ft} \leq 5,000$ ft/s
Site Class C – Very dense soil and soft rock –$1,200 < V_{S100ft} \leq 2,500$ ft/s
Site Class D – Stiff soil – $600 < V_{S100ft} \leq 1,200$ ft/s
Site Class E – Soft clay soil – $V_{S100ft} < 600$ ft/s (IBC)
Site Class F – Soils requiring site response analysis

At many sites, active surface wave techniques (MASW) with the utilization of portable energy sources, such as hammers and weight drops, are sufficient to obtain S-wave velocity sounding to 30 m (100 ft) depth. At sites with high ambient noise levels and/or very soft soils, these energy sources may not be sufficient to image to this depth and a larger energy source, such as a bulldozer, is necessary. Alternatively, passive surface wave techniques, such as the array microtremor technique can be used to extend the depth of investigation at sites that have adequate ambient noise conditions. It should be noted that two-dimensional passive-source surface wave arrays (e.g., triangular, circular, or L-shaped arrays) are expected to perform better than linear arrays.

This report contains the results of the active and passive surface wave measurements conducted at the site. An overview of the surface wave methods is given in Section 2. Field and data reduction procedures are discussed in Sections 3 and 4, respectively. Data modeling is presented in Section 5 and interpretation and results are presented in Section 6. References and our professional certification are presented in Sections 7 and 8, respectively.




Active Surface Wave Array (MASW)
Passive Surface Wave Array (L-Array)

NOTES:
1. Coordinate System: California State Plane, NAD83, Zone V (0405), US Survey Feet
2. Base map source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community

| | |
|---|---|
| Date: | 6/7/2022 |
| GV Project: | 22235 |
| Developed by: | A Martin |
| Drawn by: | T Rodriguez |
| Approved by: | A Martin |
| File Name: | 22235-1.MXD |

**FIGURE 1**
**SITE MAP**

**1727-1829 E SACRAMENTO ST**
**LOS ANGELES, CALIFORNIA**

**PREPARED FOR**
**ENGEO, INC.**

# 2 OVERVIEW OF SURFACE WAVE TECHNIQUES

## 2.1 Introduction

Active- and passive-source (ambient vibration) surface wave techniques are routinely utilized for site characterization. Active surface wave techniques include the spectral analysis of surface waves (SASW) and multi-channel array surface wave (MASW) methods. Passive surface wave techniques include the horizontal over vertical spectral ratio (HVSR) technique and the array and refraction microtremor methods.

The basis of surface wave methods is the dispersive characteristic of Rayleigh and Love waves when propagating in a layered medium. Surface waves of different wavelengths ($\lambda$) or frequencies (f) sample different depth. As a result of the variance in the shear stiffness of the distinct layers, waves with different wavelengths propagate at different phase velocities; hence, dispersion. A surface wave dispersion curve is the variation of $V_R$ or $V_L$ with $\lambda$ or f. The Rayleigh wave phase velocity ($V_R$) depends primarily on the material properties ($V_S$, mass density, and Poisson's ratio or compression wave velocity) over a depth of approximately one wavelength. The Love wave phase velocity ($V_L$) depends primarily on $V_S$ and mass density. Rayleigh and Love wave propagation are also affected by damping or seismic quality factor (Q). Rayleigh wave techniques are utilized to measure vertically polarized S-waves ($S_V$-wave); whereas Love wave techniques are utilized to measure horizontally polarized S-waves ($S_H$-wave).

## 2.2 Surface Wave Techniques

The MASW and array microtremor techniques were utilized during this investigation and are discussed below.

### 2.2.1 MASW Technique

A description of the MASW method is given by Park, 1999a and 1999b and Foti, 2000. Ground motions are typically recorded by 24, or more, geophones typically spaced 1 to 3 m apart along a linear array and connected to a seismograph. Energy sources for shallow investigations include various sized hammers and vehicle mounted weight drops. When applying the MASW technique to develop a one-dimensional (1-D) $V_S$ model, it is preferable to use multiple-source offsets from both ends of the array. The most commonly applied MASW technique is the Rayleigh-wave based MASW method, which we refer to as $MAS_RW$ to distinguish from Love-wave based MASW ($MAS_LW$). $MAS_RW$ and $MAS_LW$ acquisition can easily be combined with P- and S-wave seismic refraction acquisition, respectively. $MAS_RW$ data are generally recorded using a vertical source and vertical geophone but may also be recorded using a horizontal geophone with radial (in-line) orientation. $MAS_LW$ data are recorded using transversely orientated horizontal source and transverse horizontal geophone.

A wavefield transform is applied to the time-history data to convert the seismic record from time-offset space to frequency-wavenumber (f-k) space in which the fundamental or higher surface-wave modes can be easily identified as energy maxima and picked. Frequency and/or wavenumber can easily be mapped to phase velocity, slowness, or wavelength using the following properties: $k = 2\pi/\lambda$, $\lambda = v/f$. Common wave-field transforms include: the f-k transform (a 2D fast Fourier transform), slant-stack transform (also referred to as intercept-slowness or τ-p transform and equivalent to linear Radon transform), frequency domain beamformer, and phase-shift transform. The minimum wavelength that can be recovered from MASW data set without spatial aliasing is equal to the minimum receiver spacing. Occasionally, SASW analysis procedures are used to extract surface wave dispersion data, from fixed receiver pairs, at smaller wavelengths than can be recovered by wavefield transformation. Construction of a dispersion curve over the wide frequency/wavelength range necessary to develop a robust $V_S$ model while also limiting the maximum wavelength based on an established near-field criterion (e.g., Yoon and Rix, 2009; Li and Rosenblad, 2011), generally requires multiple source offsets.

Although the clear majority of MASW surveys record Rayleigh waves, it has been shown that Love wave techniques can be more effective in some environments, particularly shallow rock sites and sites with a highly attenuative, low velocity surface layer (Xia, et al., 2012; **GEO***Vision*, 2012; Yong, et al., 2013; Martin, et al., 2014). Rayleigh wave techniques, however, are generally more effective at sites where velocity gradually increases with depth because larger energy sources are readily available for the generation of Rayleigh waves. Rayleigh wave techniques are also more applicable to sites with high velocity layers and/or velocity inversions because the presence of such structures is more apparent in the Rayleigh wave dispersion curves than in Love wave dispersion curves. Rayleigh wave techniques are preferable at sites with a high velocity surface layer because Love waves do not theoretically exist in such environments. Occasionally, the horizontal radial component of a Rayleigh wave may yield higher quality dispersion data than the vertical component because different modes of propagation may have more energy in one component than the other. Recording both the vertical and horizontal components of the Rayleigh wave is particularly useful at sites with complex modes of propagation or when attempting to recover multiple Rayleigh wave modes for multi-mode modeling as demonstrated in Dal Moro, et al, 2015. Joint inversion of Rayleigh and Love wave data may yield more accurate $V_S$ models and offer a means to investigate anisotropy, where $S_V$- and $S_H$-wave velocity are not equal, as shown in Dal Moro and Ferigo, 2011.

### 2.2.2 Array Microtremor Technique

A detailed discussion of the array microtremor method can be found in Okada, 2003. Unlike active source techniques which use an active energy source (i.e., hammer), the array microtremor technique (also referred to as passive surface wave or array ambient vibration method) records background noise (ambient vibrations) emanating from ocean wave activity, wind noise, traffic, industrial activity, construction, etc. The technique uses 4, or more, receivers aligned in a 2-dimensional array. Triangle, circle, semi-circle, and "L" shaped arrays are commonly used, although any 2-dimensional arrangement of receivers can be used. For investigations of the upper 100 m, receivers typically consist of 1 to 4.5 Hz geophones. For deeper investigations, 5 to 120 s seismometers are generally utilized. The nested triangle array, which consists of several embedded equilateral triangles, is popular as it provides accurate dispersion curves with a relatively small number of geophones. The "L" array is useful at sites located at the corner of intersecting streets. The maximum receiver separation in an array should be at a minimum equal to the desired depth of investigation. Typically, 15 to 60 minutes of ambient vibration data is recorded depending on the size of the array, desired depth of investigation, and noise conditions. Investigations to depths on the order of 1 km may require that ambient vibrations are recorded for a much longer duration. The surface wave dispersion curve is typically estimated from array microtremor data using various f-k methods such as beamforming (Lacoss, et al., 1969), and maximum-likelihood (Capon, 1969), and the spatial-autocorrelation (SPAC) method. The beam-forming and maximum-likelihood methods are generally referred to as the frequency wavenumber (FK) and high-resolution frequency wavenumber (HRFK or HFK) methods. The SPAC method was originally based on work by Aki, 1957 and has since been extended and modified (Ling and Okada, 1993 and Ohori *et al*., 2002) to permit the use of noncircular arrays, and is now collectively referred to as extended spatial autocorrelation (ESPAC or ESAC). Further modifications to the SPAC method permit the use of irregular or random arrays (Bettig *et al*., 2001). Although it is common to apply SPAC methods to obtain a surface wave dispersion curve for modeling, other approaches involve direct modeling of the coherency data, also referred to as SPAC coefficients (Asten, 2006 and Asten, *et al*., 2015). The beam-forming and maximum-likelihood methods are generally referred to as the frequency wavenumber (FK) and high-resolution frequency wavenumber (HRFK or HFK) methods, respectively. More recently, a Rayleigh wave three-component beamforming method (RTBF) has been developed (Wathelet, et al., 2018) and appears to offer significant resolution enhancements over other methods.

FK, HRFK and RTBF methods are generally expected to perform better when ambient vibration sources are not azimuthally well-distributed (e.g., rural area where the primary noise source is a large industrial facility). SPAC methods are expected to perform better when noise sources are azimuthally well-distributed (e.g., in a large, urbanized area).

The minimum wavelength surface wave that can be extracted from an array microtremor dataset acquired utilizing a symmetric array is typically set equal to the minimum receiver spacing. The maximum wavelength is often set equal to twice the maximum receiver separation for SPAC analysis and the maximum receiver spacing for FK analysis.

## 2.3 Surface Wave Dispersion Curve Modeling

The dispersion curves generated from the active and passive surface wave soundings are generally combined and modeled using iterative forward and inverse modeling routines. The final model profile is assumed to represent actual site conditions. The theoretical model used to interpret the dispersion curve assumes horizontally layered, laterally invariant, homogeneous-isotropic material. Although these conditions are seldom strictly met at a site, the results of active and/or passive surface wave testing provide a good "global" estimate of the material properties along the array. The results may be more representative of the site than a borehole "point" estimate.

The surface wave forward problem is typically solved using the Thomson-Haskell transfer-matrix (Thomson, 1950; Haskell, 1953) later modified by Dunkin (1965) and Knopoff (1964), dynamic stiffness matrix (Kausel and Roësset, 1981), or reflection and transmission coefficient (Kennett, 1974) methods. All of these methods can determine fundamental- and higher-mode phase velocities, which correspond to plane waves in 2-D space. The transfer-matrix method is often used in MASW and passive surface-wave software packages, whereas the dynamic stiffness matrix is utilized in many SASW software packages. $MAS_RW$ and/or passive surface-wave modeling may involve modeling of the fundamental mode, some form of effective mode, or multiple individual modes (multi-mode). As outlined in Roësset et al. (1991), several options exist for forward modeling of Rayleigh wave SASW data. One formulation considers only fundamental mode plane Rayleigh-wave motion (called the 2-D solution), whereas another includes all stress waves (e.g., body, fundamental, and higher mode surface waves) and incorporates a generalized receiver geometry (3-D global solution) or actual receiver geometry (3-D array solution).

The fundamental mode assumption is generally applicable to modeling Rayleigh-wave dispersion data collected at normally dispersive sites, providing there are not abrupt increases in velocity or steep velocity gradients. Effective-mode or multi-mode approaches are often required for irregularly dispersive sites and sites with steep velocity gradients at shallow depth. If active and passive surface wave data are combined or $MAS_RW$ data are combined from multiple seismic records with different source offsets and receiver gathers, then effective-mode computations are limited to algorithms that assume far-field plane Rayleigh wave propagation. Local search (e.g., linearized matrix inversion methods) or global search methods (e.g., Monte Carlo approaches such as simulated annealing, generic algorithm and neighborhood algorithm) are typically used to solve the inverse problem.

The maximum wavelength ($\lambda_{max}$) recovered from a surface wave data set is typically used to estimate depth of investigation although a sensitivity analysis of the $V_S$ models would be a more robust means to estimate depth of investigation. For normally dispersive velocity profiles with a gradual increase in $V_S$ with depth, the maximum depth of investigation is on the order of $\lambda_{max}/2$ for both Rayleigh and Love wave dispersion data. For velocity profiles with an abrupt increase in $V_S$ at depth, the maximum depth of investigation is on the order of $\lambda_{max}/3$ for Rayleigh wave dispersion data but less than $\lambda_{max}/3$ for Love wave dispersion data. The depth of investigation can be highly variable for sites with complex velocity structure (e.g., high velocity layers).

As with all other surface geophysical methods, the inversion of surface wave dispersion data does not yield a unique $V_S$ model and multiple possible solutions may equally fit the experimental data. Based on experience at other sites, the shear wave velocity models ($V_S$ and layer thicknesses) determined by surface wave testing are within 20% of the velocities and layer thicknesses that would be determined by other seismic methods (Brown, 1998). The average velocity of the upper 30 m, however, is much more accurate, often to better than 5%, because it is not sensitive to the layering in the model. $V_{S30}$ does not appear to suffer from the non-uniqueness inherent in $V_S$ models derived from surface wave dispersion curves (Martin et al., 2006, Comina et al., 2011). Therefore, $V_{S30}$ is more accurately estimated from the inversion of surface wave dispersion data than the resulting $V_S$ models.

It may not always be possible to develop a coherent, fundamental mode dispersion curve over sufficient frequency range for modeling due to dominant higher modes with the higher modes not clearly identifiable for multi-mode modeling. It may, however, be possible to identify the Rayleigh wave phase velocity of the fundamental mode at 40 m wavelength ($V_{R40}$) in which case $V_{S30}$ can at least be estimated using the Brown et al., 2000 relationship:

$$V_{S30} = 1.045V_{R40}$$

This relationship was established based on a statistical analysis of many surface wave data sets from sites with control by velocities measured in nearby boreholes and has been further evaluated by Martin and Diehl, 2004, and Albarello and Gargani, 2010. Further investigation of this approach has revealed that $V_{S30}$ is generally between $V_{R40}$ and $V_{R45}$ with $V_{R40}$ often being most appropriate for shallow groundwater sites and $V_{R45}$ for deep ground water sites. A detailed study of such an approach for Love wave dispersion data has not been conducted; however, preliminary analysis demonstrates that $V_{S30}$ is generally between $V_{L50}$ and $V_{L55}$. Although we do not recommend that these empirical $V_{S30}$ estimates replace modeling of surface wave dispersion data, they do offer a means of cost effectively evaluating $V_{S30}$ over a large area. $V_{R40}$ or $V_{L55}$ can also be used to quantify error in $V_{S30}$ by evaluating the scatter in the dispersion data at these wavelengths.

# 3 FIELD PROCEDURES

The active- and passive-source surface wave sounding locations at the site were established by **GEO***Vision* personnel and are shown in Figure 1. Two types of surface wave data were acquired at the site: an active-source surface wave array to characterize near-surface velocity structure and a passive-source surface wave array to characterize deeper velocity structure. Passive-source surface wave data were acquired along Array 1 using the array microtremor method. Active-source surface wave data were acquired along Array 2 using the MASW technique. The locations of the surface wave arrays were surveyed using a Trimble R10 GPS system with the RTX differential correction service.

MASW equipment used during this investigation consisted of a Geometrics Geode signal enhancement seismograph, 4.5 Hz vertical geophones, seismic cable, a 4-lb hammer, 10-lb sledgehammer, and accelerated weight drop (AWD). MASW data were acquired along Array 2, a linear array of 48 geophones spaced 1.5 m (4.9 ft) apart for a length of 70.5 m (231 ft). Shot points were located between about 1.5 and 30 m (5 and 100 ft) from the end geophone locations, as space permitted, and at 8 station intervals in the interior of the array. The AWD was used as the energy source at all off-end source locations. The 4- and 10-lb hammers were utilized at the near-offset and interior source locations. Data from the transient impacts (hammers) were generally averaged 5 to 10 times to improve the signal-to-noise ratio. All field data were saved to hard disk and documented on field data acquisition forms.

The passive surface wave equipment consisted of Geometrics Atom nodal seismographs and 2 Hz vertical geophones. The L-shaped Array 1 consisted to two coincidently located 17-station L-shaped arrays with variable geophone spacing and maximum 89 and 95 m (292 and 311 ft) lengths for the legs of the array. Ambient noise measurements were made along each passive array for about 60 minutes with a 4-millisecond sample rate. Passive surface wave data were downloaded to a laptop computer for later processing. The field geometry and associated files names were documented in field notes.

# 4 DATA REDUCTION

The MASW data were reduced using the software Seismic Pro Surface V9 developed by Geogiga and multiple in-house scripts for various data extraction and formatting tasks, with all data reduction documented in a Microsoft Excel spreadsheet.

The following steps were used for data reduction:

- Input seismic records to be used for analysis into software package.
- Check and correct source and receiver geometry as necessary.
- Select offset range used for analysis (multiple offset ranges utilized for each seismic record as discussed below) and document in spreadsheet.
- Apply phase shift transform to seismic record to convert the data from time – offset to frequency – phase velocity space.
- Identify, pick, save, and document dispersion curve.
- Change the receiver offset range and repeat process.
- Repeat process for all seismic records.
- Use in-house script to apply near-field criteria with maximum wavelength set equal to 1.0 times the source to midpoint of receiver array distance.
- Use in-house script to merge multiple dispersion curves extracted from the MASW data collected along each seismic line for a specific source type (different source locations, different receiver offset ranges, etc.).
- Edit dispersion data, as necessary (e.g., delete poor quality curves and outliers).
- Calculate a representative dispersion curve at equal log-frequency or log-wavelength spacing for the MASW dispersion data using a moving average, polynomial curve fitting routine.

This unique data reduction strategy, which can involve combination of over 50 dispersion curves for a 1D sounding, is designed for characterizing sites with complex velocity structure that do not yield surface wave dispersion data over a wide frequency range from a single source type or source location. The data reduction strategy ensures that the dispersion curve selected for modeling is representative of average conditions beneath the array and spans as broad a frequency/wavelength range as possible while considering near field effects.

The array microtremor data were reduced using the Seisimager software package developed by Oyo Corporation/Geometrics, Inc. The processing sequence for implementation of the ESAC method in the SeisImager software package is as follows:

- Input all seismic records for a dataset into software.
- Apply time-segmentation routine to break the data file into multiple ~60 second time blocks.
- Load receiver geometry (x and y positions) for each channel in seismic record.
- Calculate the SPAC coefficients for each time block and average.
- Optionally, select a subset of receiver offset ranges for analysis (e.g., only select receiver pairs with multiple azimuths).

- For each frequency calculate the RMS error between the SPAC coefficients and a Bessel function of the first kind and order zero over a user defined phase velocity range and velocity step.
- Plot an image of RMS error as a function for frequency (f) and phase velocity (v).
- Identify and pick the dispersion curve as the continuous trend on the f-v image with the lowest RMS error.
- Repeat the process for all arrays.
- Use an in-house script to convert dispersion curves to appropriate format for editing.
- Edit dispersion data, as necessary, and use in-house script to combine all dispersion data after setting maximum wavelength to about 2 times the maximum receiver spacing.
- Calculate a representative dispersion curve for the passive dispersion data from each array using a moving average polynomial curve fitting routine.

The representative dispersion curves from the active and passive surface wave data were combined and the moving average polynomial curve fitting routine in WinSASW V3 was used to generate a composite representative dispersion curve for modeling. During this process, the active and passive surface wave dispersion data were given equal weights. An equal logarithm wavelength sample rate was used for the representative dispersion curve to reflect the gradual loss in model resolution with depth.

# 5 DATA MODELING

Surface wave data were modeled using the effective mode inversion routine in the Seisimager WaveEq software package. During this process, an initial velocity model was generated based on general characteristics of the dispersion curve and the inverse modeling routine utilized to adjust the layer $V_S$ until an acceptable agreement with the observed data was obtained. Layer thicknesses were adjusted, and the inversion process repeated until a $V_S$ model was developed with low RMS error between the observed and calculated dispersion curves. In many cases, once an acceptable $V_S$ model is developed, layer thicknesses are again adjusted, and the inversion process repeated to develop an ensemble of $V_S$ models with similar RMS error to quantify non-uniqueness. Because the primary purpose of this investigation was to estimate $V_{S30}$ ($V_{S100ft}$), it was not considered necessary to develop multiple $V_S$ models. Data inputs into the modeling software include layer thickness, S-wave velocity, P-wave velocity or Poisson's ratio, and mass density. P-wave velocity and mass density only have a very small influence (i.e., less than 10%) on the S-wave velocity model generated from a surface wave dispersion curve. However, realistic assumptions for P-wave velocity, which is significantly impacted by the location of the saturated zone, and mass density will slightly improve the accuracy of the S-wave velocity model.

Constant mass density values of 1.79 to 2.03 g/cm$^3$ (112 to 127 lb/ft$^3$) were used in the velocity profiles for subsurface sediments depending on P- and S-wave velocity. Within the normal range encountered in geotechnical engineering, variation in mass density has a negligible (±2%) effect on the estimated $V_S$ from surface wave dispersion data. During modeling of Rayleigh wave dispersion data, the compression wave velocity, $V_P$, for unsaturated sediments was estimated using a Poisson's ratio, $\nu$, of 0.33 and the relationship:

$$V_P = V_S\,[(2(1-\nu))/(1-2\nu)]^{0.5}$$

Poisson's ratio has a larger effect than density on the estimated $V_S$ from Rayleigh wave dispersion data. Achenbach (1973) provides approximate relationship between Rayleigh wave velocity ($V_R$), $V_S$ and $\nu$:

$$V_R = V_S\,[(0.862 + 1.14\,\nu)/(1+\nu)]$$

Using this relationship, it can be shown that $V_S$ derived from $V_R$ only varies by about 10% over possible 0 to 0.5 range for Poisson's ratio where:

$$V_S = 1.16V_R \text{ for } \nu = 0$$
$$V_S = 1.05V_R \text{ for } \nu = 0.5$$

The realistic range of Poisson's ratio for typical unsaturated sediments is about 0.25 to 0.35. Over this range, $V_S$ derived from modeling of Rayleigh wave dispersion data will vary by about 5%. There is no evidence of shallow groundwater based on the seismic refraction first arrival data. For the purpose of data modeling, it was assumed that the depth to the high Poisson's ratio, saturated zone is about 30 m (100 ft).

# 6 INTERPRETATION AND RESULTS

The fit of the calculated fundamental mode dispersion curve to the experimental data collected along Arrays 1 and 2 and the modeled $V_S$ profile for the surface wave sounding are presented as Figure 2. The resolution decreases gradually with depth due to the loss of sensitivity of the dispersion curve to changes in $V_S$ at greater depth. The $V_S$ profile used to match the field data is provided in tabular form as Tables 1 and 2.

The $V_S$ models were developed from the surface wave dispersion data derived from array microtremor and MASW data acquired along Arrays 1 and 2, respectively. The Rayleigh wave phase velocities from the passive surface wave array are in good agreement with those from the MASW data in the region where they overlap. Scatter in dispersion data from the two methods are expected to be associated with lateral velocity variability beneath the arrays. The estimated depth of investigation for the combined active and passive surface wave sounding is about 100 m (330 ft), about 50% of the maximum Rayleigh wave wavelength.

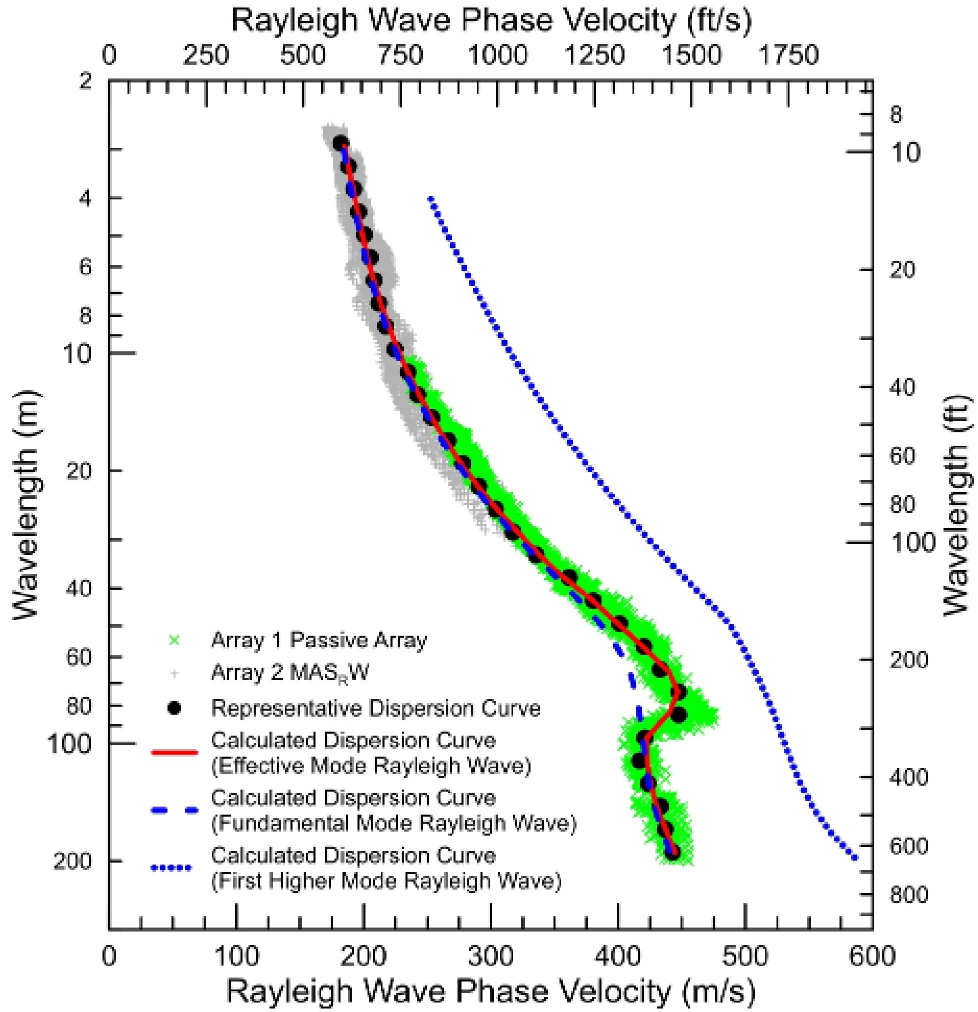
$V_{S30}$ and $V_{S100ft}$ are 366 m/s and 1,206 ft/s, respectively.

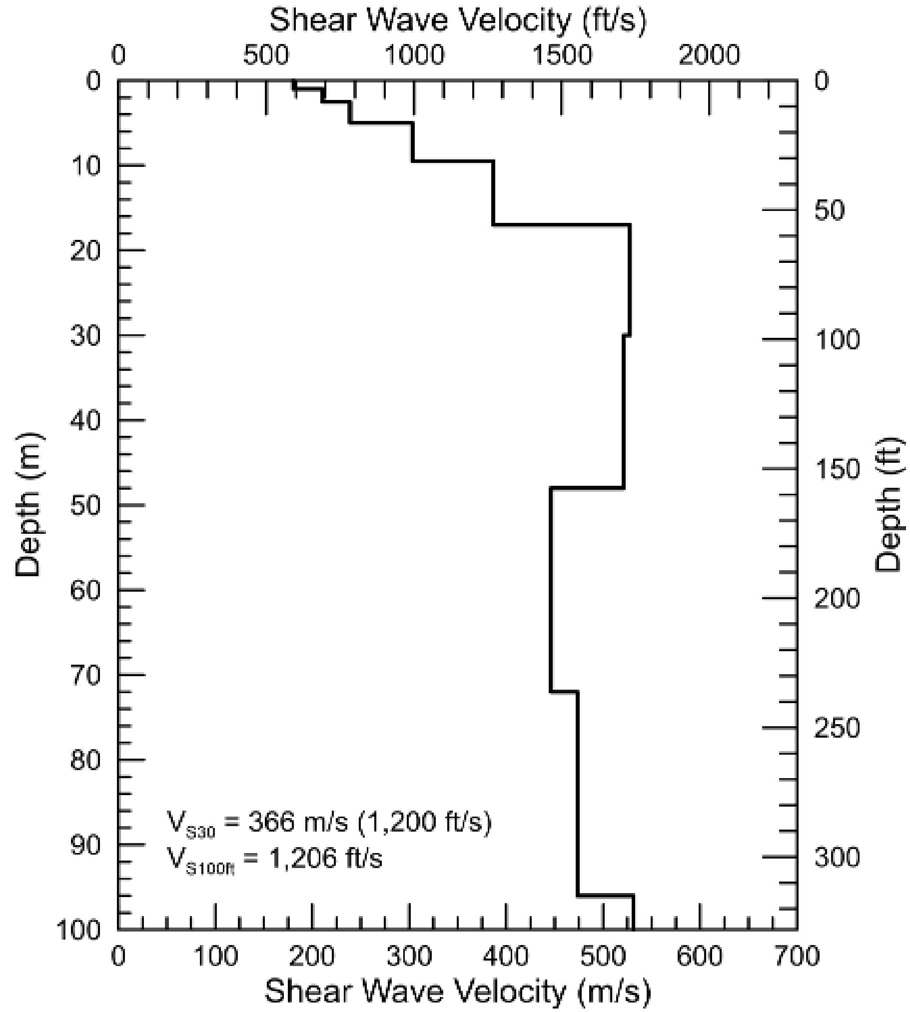
**Table 1 Arrays 1 and 2 $V_S$ Model (Metric Units)**

| Depth to Top of Layer (m) | Layer Thickness (m) | S-Wave Velocity (m/s) | Inferred P-Wave Velocity (m/s) | Inferred Poisson's Ratio | Inferred Density (g/cm$^3$) |
|---|---|---|---|---|---|
| 0 | 1 | 180 | 359 | 0.332 | 1.79 |
| 1 | 1.5 | 210 | 421 | 0.334 | 1.83 |
| 2.5 | 2.5 | 239 | 478 | 0.334 | 1.88 |
| 5 | 4.5 | 303 | 606 | 0.333 | 1.93 |
| 9.5 | 7.5 | 387 | 774 | 0.334 | 1.97 |
| 17 | 13 | 528 | 1055 | 0.333 | 2.03 |
| 30 | 18 | 521 | 1869 | 0.458 | 2.03 |
| 48 | 24 | 446 | 1785 | 0.467 | 2.00 |
| 72 | 24 | 474 | 1816 | 0.464 | 2.01 |
| 96 | Half Space | 531 | 1881 | 0.457 | 2.03 |

**Table 2 Arrays 1 and 2 Vs Model (Imperial Units)**

| Depth to Top of Layer (ft) | Layer Thickness (ft) | S-Wave Velocity (ft/s) | Inferred P-Wave Velocity (ft/s) | Inferred Poisson's Ratio | Inferred Density ($lb/ft^3$) |
|---|---|---|---|---|---|
| 0.0 | 3.3 | 591 | 1179 | 0.332 | 112 |
| 3.3 | 4.9 | 690 | 1382 | 0.334 | 114 |
| 8.2 | 8.2 | 784 | 1569 | 0.334 | 117 |
| 16.4 | 14.8 | 994 | 1988 | 0.333 | 120 |
| 31.2 | 24.6 | 1269 | 2540 | 0.334 | 123 |
| 55.8 | 42.7 | 1731 | 3461 | 0.333 | 127 |
| 98.4 | 59.1 | 1710 | 6131 | 0.458 | 127 |
| 157.5 | 78.7 | 1465 | 5857 | 0.467 | 125 |
| 236.2 | 78.7 | 1553 | 5958 | 0.464 | 125 |
| 315.0 | Half Space | 1743 | 6170 | 0.457 | 127 |

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|---|---|
| Project No: 22235 | FIGURE 2<br>SURFACE WAVE MODEL - ARRAYS 1 AND 2 |
| Date: JUN 14, 2022 | 1727-1829 E SACRAMENTO STREET<br>LOS ANGELES, CALIFORNIA |
| Drawn By: A MARTIN | |
| Approved By: | PREPARED FOR<br>ENGEO, INC. |

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# 8 CERTIFICATION

All geophysical data, analysis, interpretations, conclusions, and recommendations in this document have been prepared under the supervision of and reviewed by a **GEO*Vision*** California Professional Geophysicist.

Reviewed and approved by,

6/15/2022

Antony J. Martin — Date

California Professional Geophysicist, P. Gp.

**GEO***Vision* Geophysical Services

* This geophysical investigation was conducted under the supervision of a California Professional Geophysicist using industry standard methods and equipment. A high degree of professionalism was maintained during all aspects of the project from the field investigation and data acquisition, through data processing interpretation and reporting. All original field data files, field notes and observations, and other pertinent information are maintained in the project files and are available for the client to review for a period of at least one year.

A professional geophysicist's certification of interpreted geophysical conditions comprises a declaration of his/her professional judgment. It does not constitute a warranty or guarantee, expressed or implied, nor does it relieve any other party of its responsibility to abide by contract documents, applicable codes, standards, regulations, or ordinances.

# APPENDIX B

## SEISMIC-HAZARD ANALYSIS DISAGGREGATION RESULTS

**EXHIBIT B-1: Disaggregation results for a 2475-year return period at the PGA**

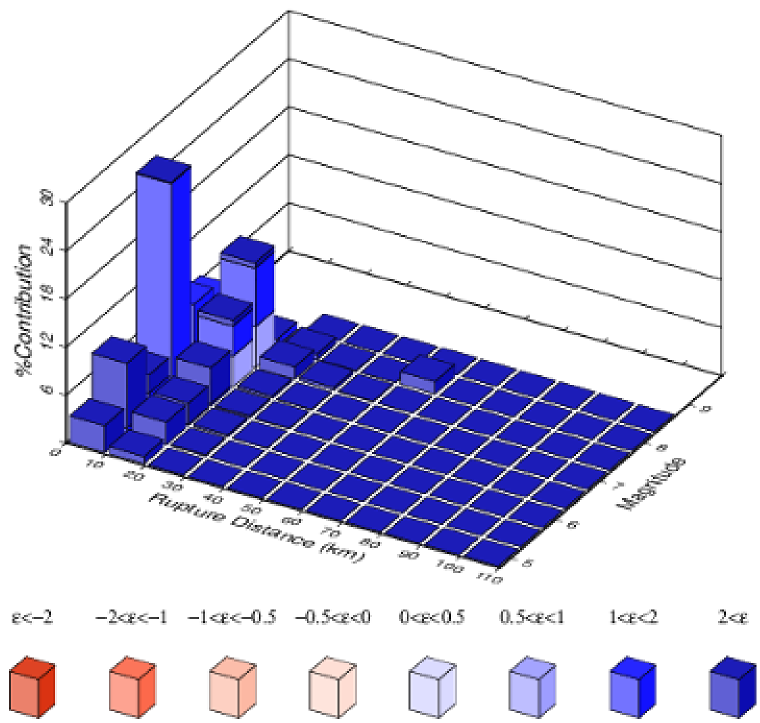



**EXHIBIT B-2: Disaggregation results for a 2475-year return period at a 0.5-second period**

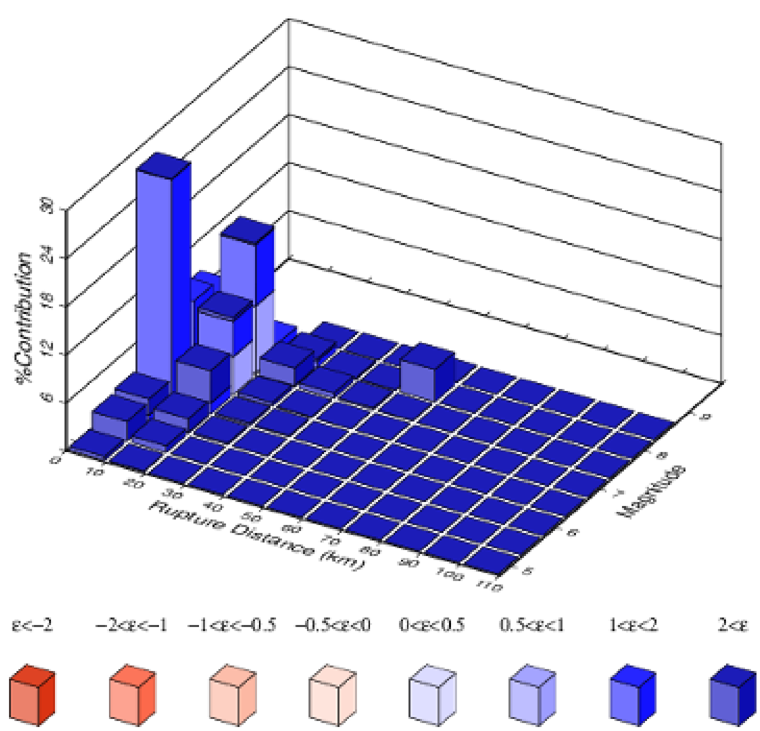


**EXHIBIT B-3: Disaggregation results for a 2475-year return period at a 1.0-second period**

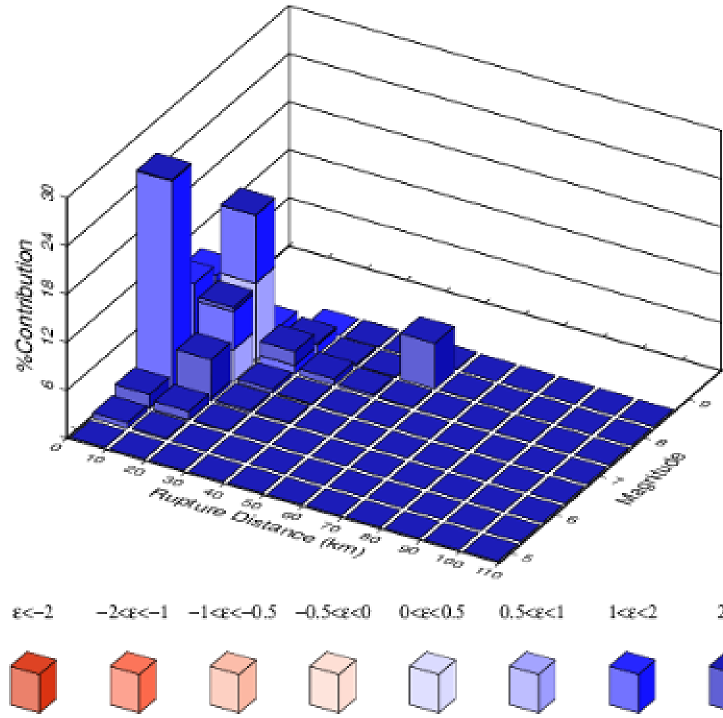
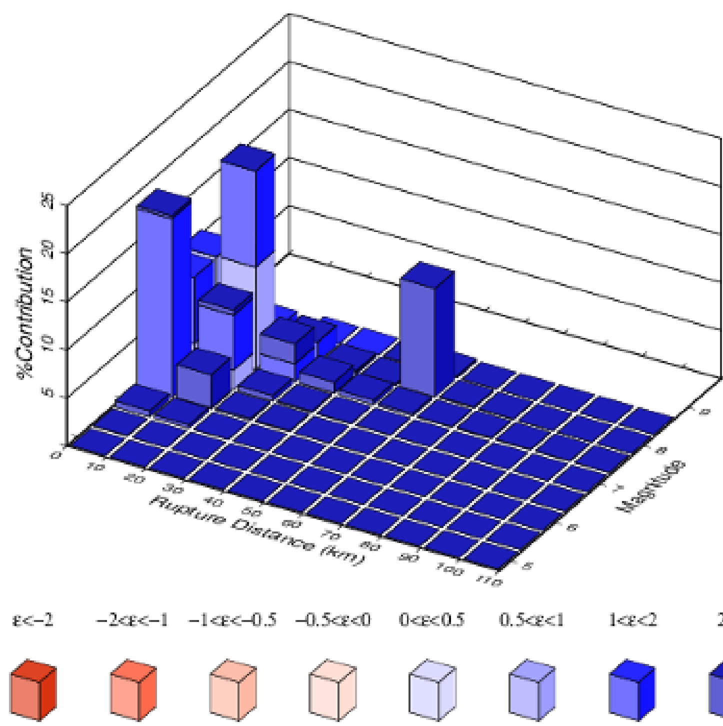



**EXHIBIT B-4: Disaggregation results for a 2475-year return period at a 2.0-second period**



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Expect Excellence

# Appendix IS-6.4

Soils Approval Letter

# CITY OF LOS ANGELES

CALIFORNIA

KAREN BASS
MAYOR

DEPARTMENT OF
BUILDING AND SAFETY
201 NORTH FIGUEROA STREET
LOS ANGELES, CA 90012

OSAMA YOUNAN, P.E.
GENERAL MANAGER
SUPERINTENDENT OF BUILDING

JOHN WEIGHT
EXECUTIVE OFFICER

## SOILS REPORT APPROVAL LETTER

October 6, 2023

LOG # 127932
SOILS/GEOLOGY FILE - 2

SCD 1811 Sacramento, LLC
633 West 5th Street, 68th Floor
Los Angeles, CA 90071

| | |
|---|---|
| TRACT: | Thomas Leahy's Subdivision of the Eighth Street Tract (MR 55-93/95) |
| BLOCK: | 2 |
| LOTS: | 19 (Arb 1), 20 (Arb 1), 21 (Arb 1), 22 (Arb 1), 23 (Arb 1), 24 (Arb 1), 25 (Arb 1), 26 (Arb 1), 27, 28, 29, 30, 31, 32, 33, 34, FR 35 & FR 36 |
| LOCATION: | 1727 to 1829 E. Sacramento Street |

| CURRENT REFERENCE REPORT/LETTER(S) | REPORT No. | DATE OF DOCUMENT | PREPARED BY |
|---|---|---|---|
| Update Report | 21971 | 09/19/2023 | Geotechnologies, Inc. |

| PREVIOUS REFERENCE REPORT/LETTER(S) | REPORT No. | DATE OF DOCUMENT | PREPARED BY |
|---|---|---|---|
| Dept. Approval Letter | 123701 | 11/30/2022 | LADBS |
| Soils Report | 21971 | 10/13/2022 | Geotechnologies, Inc. |
| Site-Specific Seismic-Hazard Report | 20460.000.001 | 06/30/2022 | ENGEO |
| Dept. Approval Letter | 123199-01 | 11/14/2022 | LADBS |
| Response Report | 21971 | 10/13/2022 | Geotechnologies, Inc. |
| Dept. Review Letter | 123199 | 10/07/2022 | LADBS |
| Soils Report | 21971 | 08/22/2022 | Geotechnologies, Inc. |
| Surface Wave Measurements Report | 22235-01 | 06/15/2022 | GEOVision |

The Grading Division of the Department of Building and Safety has reviewed the current referenced 09/19/2023 report that provides updated mat foundation recommendations, as described on pages 1 and 2 of the 09/19/2023 report. According to the consultants, the mat is expected to be deeper (about 11 feet bgs) in the central portion of the proposed mat foundation, and may be supported on native alluvial soils, whereas the remainder of the mat will be shallower and will be supported on compacted fill, and is not expected to be deeper than 8 feet bgs.

The Department previously conditionally approved the above referenced reports for the proposed structure in a letter dated 11/30/2022, Log #123701. It is proposed to construct an office development consisting of a 15-story office building to be constructed at or near existing site grade. Levels 1 through 6 will consist of a podium that will include parking, offices and mixed-use space.

Levels 6 through 15 will be on the eastern portion of the structure and will consist of office space, as shown on the Plot Plan in the 08/22/2022 report. A storm water infiltration system consisting of a dry well is recommended by the consultants. According to the consultants, the site is currently developed with three (3) 1-story warehouse buildings at grade and paved parking lots.

Previously, three borings were drilled to depths ranging from 30 to 55 feet. The earth materials at the subsurface exploration locations consisted of up to 7 feet of uncertified fill underlain by native alluvium soils. According to the consultants, groundwater was not encountered to the maximum depth explored of 55 feet, and historically highest groundwater level is at about 145 feet below the ground surface. The site is relatively level.

The consultants recommended to support the proposed structure(s) on conventional (6-story portion of structure) and/or mat (15-story portion of structure) foundations bearing on a blanket of properly placed fill, a minimum of 3 feet thick below the bottom of the foundations and/or native undisturbed alluvial soils (see pgs. 8, 9 & 16 of the 08/22/2022 report).

The current referenced 09/19/2023 report is acceptable, provided the following conditions are complied with during site development:

(Note: Numbers in parenthesis ( ) refer to applicable sections of the 2020 City of LA Building Code. P/BC numbers refer to the applicable Information Bulletin. Information Bulletins can be accessed on the internet at LADBS.ORG.)

1. All conditions of the above referenced Department approval letter dated 11/30/2022 (Log#123701) shall apply, except as specifically modified herein.

2. All the latest recommendations of the report(s) that are in addition to or more restrictive than the conditions contained herein shall be incorporated into the plans.

3. All foundations shall derive entire support from native undisturbed soils, or a blanket of properly placed fill a minimum of 3 feet thick, as recommended and approved by the soils engineer by inspection.

4. A supplemental report shall be provided in the event any deviation to the currently proposed project configuration, as presented and as shown in the plans and cross sections included in the approved reports, is made. This shall include but not limited to: relocation, change in any dimension, change in the number of stories above or below grade of any of the proposed structures; addition of any structure(s), such as retaining walls, decks, swimming pools, driveways, access roads, living quarters, etc.; or, additional permanent grading or temporary grading for construction purposes that are not described and not shown in the plans and cross sections included in the approved reports.

GLEN RAAD
Geotechnical Engineer I

Log No. 127932
213-482-0480

cc: Geotechnologies, Inc., Project Consultant
LA District Office

**CITY OF LOS ANGELES**
DEPARTMENT OF BUILDING AND SAFETY
Grading Division

| District LA | Log No. 127932 |
|---|---|

## APPLICATION FOR REVIEW OF TECHNICAL REPORTS

INSTRUCTIONS

A. Address all communications to the Grading Division, LADBS, 201 N. Figueroa St., 3rd Fl., Los Angeles, CA 90012
Telephone No. (213)482-0480.

B. Submit two copies (three for subdivisions) of reports, one "pdf" copy of the report on a CD-Rom or flash drive, and one copy of application with items "1" through "10" completed.

C. Check should be made to the City of Los Angeles.

1. LEGAL DESCRIPTION
Tract: Thomas Leahy's Subdivision of the Eight Street Tract (MR55-93/95)
Block: 2 Lots: 19-26 (arb1), 27-34, FR35 and FR36

2. PROJECT ADDRESS: 1727 through 1829 East Sacramento Street

3. OWNER: SCD 1811 Sacramento, LLC
Address: 633 W. 5th Street, Floor 68
City: Los Angeles Zip: 90071
Phone (Daytime):

4. APPLICANT Geotechnologies, Inc.
Address: 439 Western Avenue
City: Glendale Zip: 91201
Phone (Daytime): (818) 240-9600
E-mail address: Pymt:accounting@geoteq.com/Eng:gvarela@geoteq.com

5. Report(s) Prepared by: Geotechnologies, Inc. (File No. 21971)

6. Report Date(s): 01/25/22 Rev 08/22/22, 10/13/2022 Rev. 10/13/2022

7. Status of project: ☑ Proposed ☐ Under Construction ☐ Storm Damage

8. Previous site reports? ☑ YES if yes, give date(s) of report(s) and name of company who prepared report(s)
Geotechnical Engineering Investigation Updated 08/22/2022 Geotechnologies, Inc.

9. Previous Department actions? ☑ YES if yes, provide dates and attach a copy to expedite processing.
Dates: Soils Report Review Letter, Log #123199,Log# 123199-01, Log# 123701 10/07/22,11/14/22,11/30/22

10. Applicant Signature: [signature] Position: Engineer

(DEPARTMENT USE ONLY)

| REVIEW REQUESTED | FEES | REVIEW REQUESTED | | FEES |
|---|---|---|---|---|
| ☒ Soils Engineering | 181.50 | No. of Lots | | |
| ☐ Geology | | No. of Acres | | |
| ☐ Combined Soils Engr. & Geol. | | ☐ Division of Land | | |
| ☐ Supplemental | | Other | | |
| ☐ Combined Supplemental | | ☒ Expedite | | 90.75 |
| ☐ Import-Export Route | | ☐ Response to Correction | | |
| Cubic Yards: | | ☐ Expedite ONLY | | |
| | | Sub-total | | 272.25 |
| | | One-Stop Surcharge | | 69.91 |
| | | TOTAL FEE | | 342.16 |

Fee Due: 342.16
Fee Verified By: M Date: 9/21/23
(Cashier Use Only)

receipt # 1678353

ACTION BY:

THE REPORT IS: ☐ NOT APPROVED

☐ APPROVED WITH CONDITIONS ☐ BELOW ☐ ATTACHED

For Geology Date

For Soils Date