Appendix IS-4

Geotechnical Investigation

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December 5, 2023
Revised March 4, 2024
File Number 22063-01

Hudson Pacific Properties
11601 Wilshire Boulevard, 6th Floor
Los Angeles, California 90025

Attention: Chris Pearson

<u>Subject</u>: Preliminary Geotechnical Engineering Investigation
Proposed Sunset Las Palmas Studios Lower Lot Enhancement Plan
6650 West Romaine Street, Los Angeles, California

Dear Mr. Pearson:

This letter transmits the Preliminary Geotechnical Engineering Investigation for the subject site prepared by Geotechnologies, Inc. This report provides preliminary geotechnical recommendations for the development of the site, including earthwork, seismic design, retaining walls, excavations, shoring and foundation design.

This report is preliminary in nature because it is based on limited subsurface investigation, and because it provides recommendations for several structures. Due to its preliminary nature, this report is not intended for submission to the building official for building permit purposes.

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.

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PRELIMINARY GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED SUNSET LAS PALMAS STUDIOS LOWER LOT ENHANCEMENT PLAN
6650 WEST ROMAINE STREET
LOS ANGELES, CALIFORNIA

INTRODUCTION

This report presents the results of the preliminary 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 report is preliminary in nature because it is based on limited subsurface investigation, and because it provides recommendations for several structures. Due to its preliminary nature, this report is not intended for submission to the building official for building permit purposes. Supplemental subsurface exploration, laboratory testing and analyses will be required for the preparation of a comprehensive design-level geotechnical investigation suitable for submission to the building official for building permit purposes. Engineering for the proposed structures should not begin until approval of the comprehensive geotechnical investigation is granted by the local building official. Certain changes in the geotechnical recommendations may result due to the building department review process.

This investigation included excavation of one exploratory boring, sufficient to understand subsurface soils at the site, 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 location is 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 Hudson Pacific Properties. In addition, the Sunset Las Palmas Studios Lower Lot Plan prepared by Gensler, dated June 6, 2023, was reviewed for the preparation of this report. The proposed Sunset Las Palmas Studios Enhancement Plan ("project") proposes the construction of five new structures. The proposed structures consist of four single-story sound stage structures, to be built at-grade, and one four-story mill/production support structure, to be built over two subterranean parking levels. The finished floor elevation of the lowest subterranean levels for the mill/production support structure is expected to extend to a depth of 21½ feet below the proposed ground level. The enclosed Plot Plan shows the location and alignment of the proposed development. The existing four buildings and parking areas would be demolished as part of the project.

Column loads for the proposed structures are estimated to be between 200 and 900 kips. Wall loads are estimated to be between 5 and 25 kips per lineal foot. These loads reflect the dead plus live load. Grading is expected to consist of excavations in the order of 5 to 25 feet in depth for the removal and recompaction of unsuitable materials, and for the construction of the proposed subterranean levels and foundation elements.

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 project site is composed of the Sunset Las Palmas Studios Lower Lot, which is located at 6650 West Romaine Street in the Hollywood area of the City of Los Angeles, California. The project site is rectangular in shape, and approximately 133,989 square feet in area. The project site is bounded by West Romaine Street to the north, a City of Los Angeles maintenance yard and an office building to the east, Barton Avenue to the south, and North Las Palmas Avenue to the west. The project site is shown relative to nearby topographic features in the enclosed Vicinity Map.

Based on review of the topographic map included in the Sunset Las Palmas Studios Lower Lot Plan, dated June 6, 2023, the existing grade observed around the project site descends gently to the south, ranging from approximate elevation 290 feet above mean sea level (AMSL) along Romaine Street to the north, to 283 feet AMSL along Barton Avenue to the south. Within the project site, the existing ground surface is typically higher than the surrounding grade, and ranges between approximate elevation 284 feet and 294.7 feet AMSL. The project site is currently developed with a two-story parking garage located along the southern property line, four one to two-story miscellaneous buildings, and a large, paved parking lot.

Vegetation at the project site is limited, and consists of a few mature trees, bushes and shrubs, contained in manicured planter areas. Drainage appears to be by sheetflow to the city streets to the south.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was explored on December 30, 2020, by drilling one boring. It is the opinion of this firm that this limited exploration is sufficient to understand the subsurface conditions at the site. The boring was drilled to a depth of 50 feet below the existing site grade, with the aid of a truck-mounted drilling machine using 8-inch diameter hollowstem augers. The boring location is shown on the Plot Plan and the geologic materials encountered are logged on Plate A-1.

The location of the exploratory boring was determined from hardscaped features shown on the attached Plot Plan. The elevation of the exploratory boring was determined from elevations presented in the topographic map included in the Sunset Las Palmas Studios Lower Lot Plan, dated June 6, 2023. The location and elevation of the exploratory boring should be considered accurate only to the degree implied by the method used.

Geologic Materials

Fill materials were encountered in the exploratory boring, to a depth of 3 feet below the existing grade. The fill consists of silty clay, which is dark gray in color, moist, and stiff.

The fill is in turn underlain by older alluvial soils, consisting of interlayered mixtures of sand, silt and clay. The native soils are generally yellowish brown, dark brown and dark gray in color, moist to wet, stiff, or medium dense, and fine to medium grained. More detailed descriptions of the geologic materials encountered may be obtained from the log of the subsurface excavation.

Groundwater

Groundwater was encountered in the exploratory boring, at a depth of 22.2 feet below the existing grade. This depth corresponds to an approximate elevation of 270.3 fee AMSL.

The historically highest groundwater level was established by review of the Hollywood 7½ Minute Quadrangle Seismic Hazard Evaluation Report, Plate 1.2, Historically Highest Ground Water Contours (CDMG, 2006). Review of this plate indicates that the historically highest groundwater level varies across the project site from a depth of approximately 18 feet on the north, to a depth of approximately 17 feet on the south. The table below provides a summary of the historically highest groundwater level expected for the proposed structures:

Building	Historically Highest Groundwater Level Below Natural Grade (feet)	Approximately Historically Highest Groundwater Level Elevation (feet)
Mill/Production Support Structure	18	272’
All Four Sounds Stages	17	272’

Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, and other factors not evident at the time of the measurements reported herein. Fluctuations also may occur across the site. High groundwater levels can result in changed conditions.

Caving

Caving could not be directly observed during drilling of the boring due to the continuously cased design of the hollowstem augers. Based on the general experience of this firm, large diameter excavations, excavations that encounter granular cohesionless soils, and excavations below the groundwater table will most likely experience caving.

Oil Wells

Based on review of the California Geologic Energy Management Division (CalGEM) On-line Mapping System, the site is not located within the limits of oil field. Review of the CalGEM On-line Mapping System also indicates that no oil or gas wells were drilled within the subject site. The nearest well was drilled approximately ⅓-mile to the south of the site.

Regional Subsidence

The site is not located within a zone of known subsidence due to oil or other fluid withdrawal. Temporary dewatering may be required during construction. The installation of a temporary dewatering system is unlikely to cause settlement of adjacent structures. In the unlikely event that settlement does occur, it would be negligible and unlikely to affect adjacent structures.

SEISMIC EVALUATION

REGIONAL GEOLOGIC SETTING

The project site is located within the Los Angeles Basin and 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.

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, and 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 (Yerkes, 1965).

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.

The enclosed "Southern California Fault Map" shows the location of many mapped faults in the greater Los Angeles area. A list of faults located within 60 miles of the project site has been provided in the enclosed table titled "Seismic Source Summary Table". The following sections describe some of the regional Holocene-active faults, Pre-Holocene faults, and blind thrust faults.

Holocene Active Faults

Hollywood Fault

The Hollywood Fault is part of the Transverse Ranges Southern Boundary fault system. The Hollywood fault is located approximately 1.2 miles to the north of the subject site. This fault trends east-west along the base of the Santa Monica Mountains from the West Beverly Hills Lineament in the West Hollywood–Beverly Hills area to the Los Feliz area of Los Angeles. The Hollywood Fault is the eastern segment of the reverse oblique Santa Monica–Hollywood Fault. Based on geomorphic evidence, stratigraphic correlation between exploratory borings, and fault trenching studies, this fault is classified as active.

Until recently, the approximately 9.3-mile long Hollywood Fault was considered to be expressed as a series of linear ground-surface geomorphic expressions and south-facing ridges along the south margin of the eastern Santa Monica Mountains and the Hollywood Hills. Multiple recent fault rupture hazard investigations have shown that the Hollywood Fault is located south of the ridges and bedrock outcroppings along portions of Sunset Boulevard. The Hollywood Fault has not produced any damaging earthquakes during the historical period and has had relatively minor micro-seismic activity. It is estimated that the Hollywood Fault is capable of producing a maximum 6.7 magnitude earthquake.

Newport-Inglewood Fault System

According to the USGS database (2008) the Newport-Inglewood Fault System is located 4.4 miles to the southwest of the subject site. The Newport-Inglewood Fault System is a broad zone of discontinuous north to northwestern echelon faults and northwest to west trending folds. The fault system extends southeastward from West Los Angeles, across the Los Angeles Basin, to Newport Beach and possibly offshore beyond San Diego (Barrows, 1974; Weber, 1982; Ziony, 1985).

The onshore segment of the Newport-Inglewood Fault System extends for about 37 miles from the Santa Ana River to the Santa Monica Mountains. Here it is overridden by, or merges with, the east-west trending Santa Monica zone of reverse faults.

The surface expression of the Newport-Inglewood Fault System is made up of a strikingly linear alignment of domal hills and mesas that rise on the order of 400 feet above the surrounding plains. From the northern end to its southernmost onshore expression, the Newport-Inglewood Fault System is made up of: Cheviot Hills, Baldwin Hills, Rosecrans Hills, Dominguez Hills, Signal Hill-Reservoir Hill, Alamitos Heights, Landing Hill, Bolsa Chica Mesa, Huntington Beach Mesa, and Newport Mesa. Several single and multiple fault strands, arranged in a roughly left stepping en echelon arrangement, make up the fault zone and account for the uplifted mesas.

The most significant earthquake associated with the Newport-Inglewood Fault System was the Long Beach earthquake of 1933 with a magnitude of 6.3 on the Richter scale. It is believed that the Newport-Inglewood Fault System is capable of producing a 7.5 magnitude earthquake.

Santa Monica Fault

According to the USGS database, a segment of the Santa Monica Fault is located approximately 0.2 miles from the site. However, as discussed in a following section of this report, this appears to be a segment of what Hill *et al* (1979) mapped to be the Southern Santa Monica Fault, which alternatively Hildenbrand *et al* (2001) labeled as the North Salt Lake Fault. In the 2014 Fault Evaluation Report FER 253 for the Hollywood Quadrangle, the California Geological Survey (CGS) concluded that there is no clear evidence that the North Salt Lake Fault is a surface fault. Furthermore, CGS found no indication in the literature, or their observations, of Holocene surface rupture along this fault projection. The enclosed "Fault Mapping of the Hollywood Quadrangle" (CGS, 2014) shows the location of this fault segment as mapped by Weber *et al* (1980).

Based on the USGS database, the nearest segment of the active portion of the Santa Monica Fault is located approximately 4.6 miles to the northwest of the site. The Santa Monica Fault is a part of the Transverse Ranges Southern Boundary fault system, extending east from the coastline in Pacific Palisades through Santa Monica and West Los Angeles and merges with the Hollywood fault at the West Beverly Hills Lineament in Beverly Hills where its strike is northeast. It is believed that at least six surface ruptures have occurred in the past 50 thousand years. In addition, a well-documented surface rupture occurred between 10 and 17 thousand years ago, although a more recent earthquake probably occurred 1 to 3 thousand years ago. This leads to an average earthquake recurrence interval of 7 to 8 thousand years.[a] It is thought that the Santa Monica Fault system may produce earthquakes with a maximum magnitude of 7.4.

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[a] *Southern California Earthquake Center, a National Science Foundation and U.S. Geological Survey Center. Active Faults in the Los Angeles Metropolitan Region, www.scec.org/research/special/SCEC001activefaultsLA.pdf; accessed November 2023.*

Raymond Fault

The Raymond Fault is located approximately 6.8 miles to the northeast of the subject site. The Raymond Fault is an effective groundwater barrier which divides the San Gabriel Valley into groundwater sub-basins. Much of the geomorphic evidence for the Raymond Fault has been obliterated by urbanization of the San Gabriel Valley. However, a discontinuous escarpment can be traced from Monrovia to the Arroyo Seco in South Pasadena. The very bold, “knife edge” escarpment in Monrovia parallel to Scenic Drive is believed to be a fault scarp of the Raymond Fault. Trenching of the Raymond Fault is reported to have revealed Holocene movement (Weaver and Dolan, 1997).

The recurrence interval for the Raymond Fault is probably slightly less than 3,000 years, with the most recent documented event occurring approximately 1,600 years ago (Crook, et al, 1978). However, historical accounts of an earthquake that occurred in July 1855 as reported by Toppozada and others, 1981, places the epicenter of a Richter Magnitude 6 earthquake within the Raymond Fault. It is believed that the Raymond Fault is capable of producing a 6.8 magnitude earthquake. The Raymond Fault is considered active by the California Geological Survey.

Verdugo Fault

The Verdugo Fault is located approximately 7.4 miles to the northeast of the subject site. The Verdugo Fault runs along the southwest edge of the Verdugo Mountains. The fault displays a reverse motion. According to Weber, et. al., (1980) 2 to 3 meter high scarps were identified in alluvial fan deposits in the Burbank and Glendale areas. Further to the northeast, in Sun Valley, a fault was reportedly identified at a depth of 40 feet in a sand and gravel pit. Although considered active by the County of Los Angeles, Department of Public Works (Leighton, 1990), and the United States Geological Survey, the fault is not designated with an Earthquake Fault Zone by the

California Geological Survey. It is estimated that the Verdugo Fault is capable of producing a maximum 6.9 magnitude earthquake.

Malibu Coast Fault

The Malibu Coast Fault is part of the Transverse Ranges Southern Boundary fault system, a west-trending system of reverse, oblique-slip, and strike-slip faults that extends for more than approximately 124 miles along the southern edge of the Transverse Ranges and includes the Hollywood, Raymond, Anacapa–Dume, Malibu Coast, Santa Cruz Island, and Santa Rosa Island faults.

The Malibu Coast Fault System runs in an east-west orientation onshore subparallel to and along the shoreline for a linear distance of about 17 miles through the Malibu City limits, but also extends offshore to the east and west for a total length of approximately 37.5 miles. The onshore Malibu Coast Fault System involves a broad, wide zone of faulting and shearing as much as 1 mile in width. While the Malibu Coast Fault System has not been officially designated as an active fault zone by the State of California and no Special Studies Zones have been delineated along any part of the fault zone under the Alquist-Priolo Act of 1972, evidence for Holocene activity (movement in the last 11,000 years) has been established in several locations along individual fault splays within the fault zone. Due to such evidence, several fault splays within the onshore portion of the fault System are identified as active.[b]

[b] *City of Malibu Planning Department, Malibu General Plan, Chapter 5.0, Safety and Health Element, http://qcode.us/codes/malibu-general-plan/; accessed November 2023.*

Large historic earthquakes along the Malibu Coast Fault include the 1979, 5.2 magnitude earthquake and the 1989, 5.0 magnitude earthquake.[c] The Malibu Coast Fault System is approximately 11.7 miles west of the subject site and is believed to be capable of producing a maximum 7.0 magnitude earthquake.

Sierra Madre Fault System

The Sierra Madre Fault alone forms the southern tectonic boundary of the San Gabriel Mountains in the northern San Fernando Valley. It consists of a system of faults approximately 75 miles in length. The individual segments of the Sierra Madre Fault System range up to 16 miles in length and display a reverse sense of displacement and dip to the north. The most recently active portions of the System include the Mission Hills, Sylmar and Lakeview segments, which produced an earthquake in 1971 of magnitude 6.4. Tectonic rupture along the Lakeview Segment during the San Fernando Earthquake of 1971 produced displacements of approximately 2½ to 4 feet upward and southwestward.

It is believed that the Sierra Madre Fault System is capable of producing an earthquake of magnitude 7.3. The closest trace of the fault is located approximately 11.8 miles northeast of the subject site.

Palos Verdes Fault

Studies indicate that there are several active on-shore extensions of the strike-slip Palos Verdes Fault, which is located approximately 15.1 miles southwest of the subject site. Geophysical data also indicate the off-shore extensions of the fault are active, offsetting Holocene age deposits. No historic large magnitude earthquakes are associated with this fault. However, the fault is

[c] *California Institute of Technology, Southern California Data Center. Chronological Earthquake Index, www.data.scec.org/significant/malibu1979.html; accessed November 2023.*

considered active by the California Geological Survey. It is estimated that the Palos Verdes Fault is capable of producing a maximum 7.7 magnitude earthquake.

San Gabriel Fault System

The San Gabriel Fault System is located approximately 16.1 miles northeast of the subject site. The San Gabriel Fault System comprises a series of subparallel, steeply north-dipping faults trending approximately north 40 degrees west with a right-lateral sense of displacement. There is also a small component of vertical dip-slip separation. The fault system exhibits a strong topographic expression and extends approximately 90 miles from San Antonio Canyon on the southeast to Frazier Mountain on the northwest. The estimated right lateral displacement on the fault varies from 34 miles (Crowell, 1982) to 40 miles (Ehlig, 1986), to 10 miles (Weber, 1982). Most scholars accept the larger displacement values and place the majority of activity between the Late Miocene and Late Pliocene Epochs of the Tertiary Era (65 to 1.8 million years before present).

Portions of the San Gabriel Fault System are considered active by California Geological Survey. Recent seismic exploration in the Valencia area (Cotton and others, 1983; Cotton, 1985) has established Holocene offset. Radiocarbon data acquired by Cotton (1985) indicate that faulting in the Valencia area occurred between 3,500 and 1,500 years before present.

It is hypothesized by Ehlig (1986) and Stitt (1986) that the Holocene offset on the San Gabriel Fault System is due to sympathetic (passive) movement as a result of north-south compression of the upper Santa Susana Thrust sheet. Seismic evidence indicates that the San Gabriel Fault System is truncated at depth by the younger, north-dipping Santa Susana-Sierra Madre Faults (Oakeshott, 1975; Namson and Davis, 1988).

Whittier-Elsinore Fault System

The Whittier Fault is located approximately 17.7 miles to the southeast of the subject site. The Whittier Fault together with the Chino Fault comprises the northernmost extension of the northwest trending Elsinore Fault System. The mapped surface of the Whittier Fault extends in a west-northwest direction for a distance of 20 miles from the Santa Ana River to the terminus of the Puente Hills. The Whittier Fault is essentially a strike-slip, northeast dipping fault zone which also exhibits evidence of reverse movement along with en echelon[d] fault segments, en echelon folds and anatomizing (braided) fault segments. Right lateral offsets of stream drainages of up to 8800 feet (Durham and Yerkes, 1964) and vertical separation of the basement complex of 6,000 to 12,000 feet (Yerkes, 1972), have been documented. It is believed that the Whittier Fault is capable of producing a 7.8 magnitude earthquake.

The Whittier Narrows earthquakes of October 1, 1987, and October 4, 1987, occurred in the area between the westernmost terminus of the mapped trace of the Whittier Fault and the Frontal Fault System. The main 5.9 magnitude shock of October 1, 1987 was not caused by slip on the Whittier Fault. The quake ruptured a gently dipping thrust fault with an east-west strike (Haukson, Jones, Davis and others, 1988). In contrast, the earthquake of October 4, 1987, is assumed to have occurred on the Whittier Fault as focal mechanisms show mostly strike-slip movement with a small reverse component on a steeply dipping northwest striking plane (Haukson, Jones, Davis and others, 1988).

Santa Susana Fault

The Santa Susana Fault extends approximately 17 miles west-northwest from the northwest edge of the San Fernando Valley into Ventura County and is at the surface high on the south flank of the Santa Susana Mountains. The fault ends near the point where it overrides the south-side-up

[d] *En echelon refers to closely-spaced, parallel or subparallel, overlapping or step-like minor structural features*

South strand of the Oak Ridge Fault. The Santa Susana Fault strikes northeast at the Fernando lateral ramp and turns east at the northern margin of the Sylmar Basin to become the Sierra Madre Fault. This fault is exposed near the base of the San Gabriel Mountains for approximately 46 miles from the San Fernando Pass at the Fernando lateral ramp east to its intersection with the San Antonio Canyon fault in the eastern San Gabriel Mountains, east of which the range front is formed by the Cucamonga Fault. The Santa Susana Fault has not experienced any recent major ruptures except for a slight rupture during the 6.5 magnitude 1971 Sylmar earthquake.[e] The Santa Susana Fault is considered to be active by the County of Los Angeles. It is believed that the Santa Susana Fault has the potential to produce a 6.9 magnitude earthquake. The closest trace of the fault is located approximately 18.4 miles north of the subject site.

San Andreas Fault System

The San Andreas Fault System forms a major plate tectonic boundary along the western portion of North America. The system is predominantly a series of northwest trending faults characterized by a predominant right lateral sense of movement. At its closest point the San Andreas Fault System is located approximately 34.2 miles to the northeast of the subject site.

The San Andreas and associated faults have had a long history of inferred and historic earthquakes. Cumulative displacement along the system exceeds 150 miles in the past 25 million years (Jahns, 1973). Large historic earthquakes have occurred at Fort Tejon in 1857, at Point Reyes in 1906, and at Loma Prieta in 1989. Based on single-event rupture length, the maximum Richter magnitude earthquake is expected to be approximately 8.25 (Allen, 1968). The recurrence interval for large earthquakes on the southern portion of the fault system is on the order of 100 to 200 years.

[e] *California Institute of Technology, Southern California Data Center. Chronological Earthquake Index, www.data.scec.org/significant/santasusana.html; accessed November 2023.*

Pre Holocene Faults

Anacapa-Dume Fault

The Anacapa–Dume Fault, located approximately 13.3 miles to the northwest of the subject site, is a near-vertical offshore escarpment exceeding 600 meters locally, with a total length exceeding 62 miles. This fault is also part of the Transverse Ranges Southern Boundary fault system. It occurs as close as 3.6 miles offshore south of Malibu at its western end, but trends northeast where it merges with the offshore segments of the Santa Monica Fault Zone. It is believed that the Anacapa–Dume Fault is responsible for generating the historic 1930 magnitude 5.2 Santa Monica earthquake, the 1973 magnitude 5.3 Point Mugu earthquake, and the 1979 and 1989 Malibu earthquakes, each of which possessed a magnitude of 5.0.[f] The Anacapa–Dume fault is thought to be capable of producing a maximum magnitude 7.2 earthquake.

Blind Thrusts Faults

Blind or buried thrust faults are faults without a surface expression but are a significant source of seismic activity. By definition, these faults have no surface trace, therefore the potential for ground surface rupture is considered remote. 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 sometimes not known until they produce an earthquake. Two blind thrust faults in the Los Angeles metropolitan area are the Puente Hills blind thrust and the Elysian Park blind thrust. Another blind thrust fault of note is the Northridge fault located in the northwestern portion of the San Fernando Valley.

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[f] *City of Malibu Planning Department. Malibu General Plan, Chapter 5.0, Safety and Health Element, http://qcode.us/codes/malibu-general-plan/; accessed November 2023.*

The Elysian Park anticline is thought to overlie the Elysian Park blind thrust. This fault has been estimated to cause an earthquake every 500 to 1,300 years in the magnitude range 6.2 to 6.7. The Elysian Park anticline is approximately 2.7 miles to the southeast of the subject site.

The Puente Hills blind thrust fault extends eastward from Downtown Los Angeles to the City of Brea in northern Orange County. The Puente Hills blind thrust fault includes three north-dipping segments, named from east to west as the Coyote Hills segment, the Santa Fe Springs segment, and the Los Angeles segment. These segments are overlain by folds expressed at the surface as the Coyote Hills, Santa Fe Springs Anticline, and the Montebello Hills. The Los Angeles segment of the Puente Hills blind thrust is located approximately 3.8 miles to the southeast of the subject site.

The Santa Fe Springs segment of the Puente Hills blind thrust fault is believed to be the cause of the October 1, 1987, Whittier Narrows Earthquake. Based on deformation of late Quaternary age sediments above this fault system and the occurrence of the Whittier Narrows earthquake, the Puente Hills blind thrust fault is considered an active fault capable of generating future earthquakes beneath the Los Angeles Basin. A maximum moment magnitude of 7.0 is estimated by researchers for the Puente Hills blind thrust fault.

The Mw 6.7 Northridge earthquake was caused by the sudden rupture of a previously unknown, blind thrust fault. This fault has since been named the Northridge Thrust, however it is also known in some of the literature as the Pico Thrust. It has been assigned a maximum magnitude of 6.9 and a 1,500 to 1,800 year recurrence interval. The Northridge thrust is located 15.5 miles to the north of the subject site.

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.

The CGS published the Earthquake Fault Zones Map of the Hollywood Quadrangle in November, 2014. Based on review of this map, the nearest Earthquake Fault Zone is located approximately 0.9 miles to the north of the site, for the Hollywood Fault. A copy of this map may be found in the Appendix of this report.

North Salt Lake Fault (Southern Santa Monica Fault)

Based on review of the enclosed "Previous Fault Mapping of the Hollywood Quadrangle" (CGS, 2014), Hill *et al* (1979) mapped the Southern Santa Monica Fault in the vicinity of the northwest corner of the site. Alternatively, Hildenbrand *et al* (2001) labeled this fault as the North Salt Lake Fault.

According to Hildenbrand *et al* (2001): *"Vertical movement associated with the opening of the Los Angeles Basin resulted in numerous folds and faults in the footwall of the Santa Monica-Hollywood fault zone. For example, Wright (1991) suggested that the North Salt Lake fault may have been formed at this time by extension behind the uplifted Las Cienega's basement block that was sagging literally into the deeply subsided basin to the southwest. The North Salt Lake fault, a steeply-dipping normal fault, may form the southern margin of the Hollywood Basin (Wright, 1991)."*

In the 2014 Fault Evaluation Report FER 253 for the Hollywood Quadrangle, the California Geological Survey (CGS) concluded that there is no clear evidence that the North Salt Lake fault is a surface fault. CGS opinion is that the zone of differential subsidence identified in the enclosed "Previous Fault Mapping of the Hollywood Quadrangle" may be attributed to causes other than active fault displacement. CGS found no indication in the literature, or their observations, of Holocene surface rupture along this fault projection. Furthermore, the USGS database indicates that the rupture top for this fault is expected to be located approximately ½-mile below the ground surface.

No Special Studies Zones have been delineated by the State of California, or the City of Los Angeles, along any part of the North Salt Lake Fault. Based on these considerations, the potential for surface ground rupture at the subject sites 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.

As shown in the enclosed Earthquake Fault Zone Map, the State of California 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 site-specific liquefaction analysis was performed following the Recommended Procedures for Implementation of the California Geologic Survey Special Publication 117A, Guidelines for Analyzing and Mitigating Seismic Hazards in California (CGS, 2008), and the EERI Monograph (MNO-12) by Idriss and Boulanger (2008). This semi-empirical method is based on a correlation between measured values of Standard Penetration Test (SPT) resistance and field performance data.

Groundwater was encountered during exploration, at a depth of 22.2 feet. Based on review of the seismic hazard zone report of the Hollywood 7½-minute quadrangle (CDMG, 2006), the historically highest groundwater level for the site ranges between 17 and 18 feet below the ground surface. Both the historically highest groundwater level and the current groundwater level were utilized for the enclosed liquefaction analysis.

Section 11.8.3 of ASCE 7-16 indicates that the potential for liquefaction shall be evaluated utilizing an acceleration consistent with the MCE_G PGA. Utilizing the OSHPD seismic utility program, this corresponds to a PGA_M of 0.99g. The USGS Probabilistic Seismic Hazard Deaggregation program (USGS, 2014) indicates a PGA of 0.86g (2 percent in 50 years ground motion) and a mean magnitude of 6.7 for the site. The liquefaction potential evaluations were performed by utilizing a magnitude 6.7 earthquake, and a peak horizontal acceleration of 0.99g.

The enclosed "Empirical Estimation of Liquefaction Potential" is based on results obtained from Boring B1. Standard Penetration Test (SPT) data were collected at 5-foot intervals. Samples of the collected materials were conveyed to the laboratory for testing and analysis. The percent passing a Number 200 sieve, Atterberg Limits, and the plasticity index (PI) of representative soil samples encountered during exploratory are presented on the enclosed E-Plate and F-Plate.

The procedure presented in the SP117A guidelines was followed in analyzing the liquefaction potential of the subject site in combination with the most recent Los Angeles Building Code requirements. The SP117A guidelines were developed based on a document titled, "Assessment of the Liquefaction Susceptibility of Fine-Grained Soils", by Bray and Sancio (2006). According to the SP117A and City of Los Angeles criteria, soils having a Plastic Index (PI) greater than 18 exhibit clay-like behavior, and the liquefaction potential of these soils are considered to be low.

The results of liquefaction analysis indicate the site soils would not be prone to liquefaction during the design basis earthquake. Furthermore, the site is not expected to be affected by potential impacts related to liquefaction, such as lateral spreading and surface manifestation.

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 older alluvium, 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. The site is high enough and far enough from the ocean to preclude being prone to hazards of a tsunami.

Review of the County of Los Angeles Flood and Inundation Hazards Map (Leighton, 1990), indicates the site lies within the inundation boundaries of the Mulholland Dam. A determination of whether a higher site elevation would remove the site from the potential inundation zones is beyond the scope of this investigation.

Review of the applicable Flood Insurance Rate Map (06037C1605F, 2008) 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 negligible 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 preliminary finding of Geotechnologies, Inc. that construction of the proposed project is considered feasible from a geotechnical engineering standpoint provided the advice and recommendations presented herein are followed and implemented during construction.

This report is preliminary in nature because it is based on limited subsurface investigation, and because it provides recommendations for several structures. Due to its preliminary nature, this report is not intended for submission to the building official for building permit purposes. Supplemental subsurface exploration, laboratory testing and analyses will be required for the preparation of a comprehensive design-level geotechnical investigation suitable for submission to the building official for building permit purposes. Engineering for the proposed structures should not begin until approval of the comprehensive design-level geotechnical investigation is granted by the local building official. Certain changes in the geotechnical recommendations may result due to the building department review process.

During exploration, fill materials were encountered to a depth of 3 feet below the existing grade. The existing fill materials are not suitable for support of foundations and concrete slabs-on-grade but may be reused for the preparation of a compacted fill pad. Groundwater was encountered at a depth of 22.2 feet below the existing grade, which corresponds to an approximate elevation of 270.3 feet AMSL. Based on the elevations presented in the Sunset Las Palmas Studios Lower Lot Plan, dated June 6, 2023, it is the opinion of this firm that the historically-highest groundwater level corresponds to an approximate elevation of 272.0 feet AMSL.

General Preliminary Recommendations for Sound Stage Structures

The proposed sound stage structures may be supported by conventional foundations bearing in a newly-built compacted fill pad. For the preparation of a compacted fill pad, all the existing fill materials and upper native soils shall be removed and recompacted to a minimum depth of 3 feet below the bottom of the proposed foundations, or 5 feet below the proposed subgrade, whichever is greater. In addition, the compacted fill should extend horizontally a minimum of 3 feet beyond the edge of foundations or for a distance equal to the depth of fill below the foundation, whichever is greater.

General Preliminary Recommendations for Four-Story Mill/Production Support Structure

The subterranean finished grade for this structure is expected to extend to a depth of 22½ feet below the proposed ground level. It is anticipated that the existing fill materials will be removed during excavation for the proposed subterranean levels, exposing native older alluvial soils at the subterranean subgrade.

The exact finished floor elevation of the proposed lowest subterranean level is unknown at this time. It is however anticipated that it will extend below the historically highest groundwater level, and potentially below the current groundwater level. Where elements of a proposed structure extend below the current or historically highest groundwater level, the structure should either be designed to resist potential hydrostatic forces, or a permanent dewatering system should be installed so that external water pressure does not develop against the proposed retaining walls and floor slabs. It is the recommendation of this firm that the proposed structure be designed to resist hydrostatic forces in lieu of installation of a permanent dewatering system. This eliminates the need for maintenance of a permanent dewatering system and continuous handling, testing, and possible treatment of waters pumped from the system. In addition, it would not be necessary to comply with future changes in water quality standards for collected and released groundwater.

Under the hydrostatic design approach, foundations and slabs-on-grade shall be designed to resist hydrostatic uplift based on the shallowest groundwater level. In addition, the proposed retaining walls should be designed to resist hydrostatic pressures. Hydrostatic forces are addressed in the "Foundation Design" and "Retaining Wall Design" sections of this report.

Based on the depth of this proposed structure, relative to the current and historically highest groundwater levels, it is recommended that it is supported on a mat foundation bearing in the undisturbed native alluvial soils expected at the subgrade of the proposed excavation. It is recommended that the bottom of the structure and retaining walls should be completely watertight in order to prevent water seepage through normal shrinkage cracks or construction joints. It is recommended that care should be taken in the design and installation of waterproofing to avoid moisture problems, and to prevent water seepage into the structure. The design and inspection of waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method which would provide protection to subterranean walls, floors, and foundations.

Excavation of the proposed subterranean levels will require the installation of temporary shoring in order to achieve a stable excavation. additionally, it is recommended that a qualified dewatering consultant be retained in order to determine a formal temporary dewatering program. The expected number and depths of well-points, expected flow rates, and expected pre-pumping time frames should be determined by the dewatering consultant. Where the subgrade exposed at the bottom of the subterranean excavation is wet and pumping, the subgrade soil shall be stabilized as recommended in the “Wet Soils” section of this report.

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.

SEISMIC DESIGN CONSIDERATIONS

California Building Code Seismic Parameters

Based on information derived from the subsurface investigation, the subject site is classified as Site Class D, which corresponds to a “Stiff Soil” Profile, according to Table 20.3-1 of ASCE 7-16. This information and the site coordinates were input into the OSHPD seismic utility program in order to calculate ground motion parameters for the site.

CALIFORNIA BUILDING CODE SEISMIC PARAMETERS	
California Building Code	2022
ASCE Design Standard	7-16
Site Class	D
Mapped Spectral Acceleration at Short Periods (S_S)	2.088g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	2.088g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.392g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.748g
Site Coefficient (F_v)	1.7*
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	1.272g*
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.848g*

* *According to ASCE 7-16, a Long Period Site Coefficient (F_v) of 1.7 may be utilized provided that the value of the Seismic Response Coefficient (C_s) is determined by Equation 12.8-2 for values of $T \leq 1.5T_s$ and taken as equal to 1.5 times the value computed in accordance with either Equation 12.8-3 for $T_L \geq T > 1.5T_s$ or equation 12.8-4 for $T > T_L$. Alternatively, a site-specific ground motion hazard analysis may be performed in accordance with ASCE 7-16 Section 21.1 and/or a ground motion hazard analysis in accordance with ASCE 7-16 Section 21.2 to determine ground motions for any structure.*

EXPANSIVE SOILS

The onsite geologic materials are in the moderate expansion range. The Expansion Index was found to be 86 for a representative bulk sample. Reinforcing in accordance with the City of Los Angeles Building Code is provided in the "Foundation Design" and "Slab-On-Grade" sections of this report.

COLLAPSIBLE SOILS

Based on review of the enclosed consolidation curves, the soils to underlain the proposed development are not considered prone to hydroconsolidation.

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 0.2% 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 moderate for geologic materials with less than 0.2% and Type II cement may be utilized for concrete foundations in contact with the site soils. In addition a water-cement ratio of 0.5 should be maintained in the poured concrete.

DEWATERING

Groundwater was observed at a depth of 22.2 feet below the existing grade, which corresponds to an approximate elevation of 270.3 feet AMSL. Depending on the final finished floor elevation of the proposed mill/production support structure, the subterranean subgrade of this structure may extend below the current groundwater level. If this subterranean subgrade does extend below the current groundwater level, it is recommended that a qualified dewatering consultant be retained to determine if a formal temporary dewatering program will be required.

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 within the limits of a City of Los Angeles Methane Zone. A qualified methane consultant should be retained to consider the requirements and implications of the City's Methane Zone designation.

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 for At-Grade Sound Stages

Preparation of a compacted fill pad will be required for the proposed sound stage structures, which will be built at-grade. For the preparation of a compacted fill pad, all existing fill and upper native alluvial soils shall be excavated to a minimum depth of 5 feet below the bottom of the proposed subgrade, or 3 feet below the bottom of the proposed foundations, whichever is greater. In addition, the excavation shall extend horizontally 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. It is very important that the position of the proposed structure is 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 materials having more than 15 percent finer than 0.005 millimeters may be compacted to a minimum of 90 percent of the maximum density.

All fill should be mechanically compacted in layers not more than 8 inches thick. Based on the moderate expansion index of the tested site soils, it is recommended that fill materials are moisture conditioned to 5 percent over 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 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) compaction is obtained.

Acceptable Materials

The excavated onsite materials are considered satisfactory for reuse in the controlled fills as long as any debris and/or organic matter is removed. Any imported materials 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 materials should consist of geologic materials with an expansion index of less than 50. The water-soluble sulfate content of the import materials should be less than 0.1% percentage by weight.

Imported materials 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 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) 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 92 percent.

Wet Soils

High-moisture content soils may be encountered where the proposed subterranean grade will extend below the existing groundwater level. Pumping (yielding or vertical deflection) of the high-moisture content soils at the bottom of the excavation may occur during operation of heavy equipment. Where pumping is encountered, angular minimum ¾-inch gravel should be placed and worked into the subgrade. The exact thickness of the gravel would be a trial and error procedure and would be determined in the field. It would likely be on the order of 1 to 2 feet thick.

The gravel will help to densify the subgrade as well as function as a stabilization material upon which heavy equipment may operate. It is not recommended that rubber tire construction equipment attempt to operate directly on the pumping subgrade soils prior to placing the gravel. Direct operation of rubber tire equipment on the soft subgrade soils will likely result in excessive disturbance to the soils, which in turn will result in a delay to the construction schedule since those disturbed soils would then have to be removed and properly recompacted. Extreme care should be utilized to place gravel as the subgrade becomes exposed.

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. 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.

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.

Abandoned Seepage Pits

No abandoned seepage pits were encountered during exploration and none are known to exist on the site. However, should such a structure be encountered during grading, options to permanently abandon seepage pits include complete removal and backfill of the excavation with compacted fill, or drilling out the loose materials and backfilling to within a few feet of grade with slurry, followed by a compacted fill cap.

If the subsurface structures are to be removed by grading, the entire structure should be demolished. The resulting void may be refilled with compacted soil. Concrete and brick generated during the seepage pit removal may be reused in the fill as long as all fragments are less than 6 inches in longest dimension and the debris comprises less than 15 percent of the fill by volume. All grading should comply with the recommendations of this report.

Where the seepage pit structure is to be left in place, the seepage pits should cleaned of all soil and debris. This may be accomplished by drilling. The pits should be filled with minimum 1-1/2 sack concrete slurry to within 5 feet of the bottom of the proposed foundations. In order to provide a more uniform foundation condition, the remainder of the void should be filled with controlled fill.

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

Based on the depth of the proposed mill/production support structure, relative to the current and historically highest groundwater levels, it is recommended this structure is supported on a mat foundation system bearing in the undisturbed native alluvial soils expected at the subgrade of the proposed excavation. Since the proposed sounds stage structures will be built at-grade, these structures may be supported on conventional foundations bearing in a newly built compacted fill pad.

Recommendations for both conventional foundations and a mat foundation system are provided herein.

MAT FOUNDATION

A mat foundation system is recommended for support of the mill/production support structure. The mat foundation may bear in the competent native alluvial soils expected at the subgrade of the proposed subterranean garage levels.

Structural information is not available at this time. For design purposes, an allowable bearing pressure of 2,500 pounds per square foot, with locally higher pressures up to 5,000 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 200 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.

Hydrostatic Considerations for Mat Foundations

Where constructed below the historically highest groundwater level, mat foundations shall be waterproofed and designed to withstand the hydrostatic uplift pressure based on the shallowest of the current and historically highest groundwater level. The uplift pressure to be used in design should be 62.4(H) pounds per square foot, where “H” is the vertical distance between the historically highest groundwater elevation and the elevation at the bottom of the mat foundation.

Lateral Mat Foundation Design

Resistance to lateral loading may be provided by friction acting at the base of the mat and by passive earth pressure. An allowable coefficient of friction of 0.34 may be used with the dead load forces.

Passive geologic pressure for the sides of foundations poured against undisturbed soil may be computed as an equivalent fluid having a density of 300 pounds per cubic foot with a maximum earth pressure of 1,800 pounds per square foot. The passive and friction components may be combined for lateral resistance without reduction. A one-third increase in the passive value may be used for short duration loading such as wind or seismic forces.

Mat Foundation Settlement

Settlement of a mat foundation is expected to occur on application of loading. The maximum settlement is expected to occur below the central portion of the mat and would not be expected to exceed 1 inch. The settlement along the edges of the mat would not be expected to exceed ½-inch. Therefore, the differential settlement anticipated across the mat is not expected to exceed ½-inch.

CONVENTIONAL FOUNDATIONS

The proposed sound stage structures may be supported by conventional foundations bearing in a new compacted fill pad.

Continuous foundations may be designed for a bearing capacity of 3,000 pounds per square foot and should be a minimum of 12 inches in width, 24 inches in depth below the lowest adjacent grade and 24 inches into the recommended bearing material.

Column foundations may be designed for a bearing capacity of 3,500 pounds per square foot, and should be a minimum of 24 inches in width, 24 inches in depth below the lowest adjacent grade and 24 inches into the recommended bearing material.

The bearing capacity increase for each additional foot of width is 100 pounds per square foot. The bearing capacity increase for each additional foot of depth is 400 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.

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.

Conventional Foundation Lateral Design

Resistance to lateral loading may be provided by friction acting at the base of foundations and by passive earth pressure. An allowable coefficient of friction of 0.3 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 1,500 pounds per square foot.

The passive and friction components may be combined for lateral resistance without reduction. A one-third increase in the passive value may be used for short duration loading such as wind or seismic forces.

Miscellaneous Conventional Foundations

Foundations for small miscellaneous outlying structures, such as property line fence walls, planters, exterior canopies, and trash enclosures, which will not be tied-in to the proposed hotel structure, may be supported on conventional foundations bearing in properly compacted fill and/or the native soils. Wall footings may be designed for a bearing value of 1,500 pounds per square foot and should be a minimum of 12 inches in width, 24 inches in depth below the lowest adjacent grade and 24 inches into the recommended bearing material. No bearing value increases are recommended.

Conventional Foundation Settlement

Settlement of the foundation system is expected to occur on initial application of loading. The maximum settlement is not expected to exceed 1 inch and occur below the heaviest loaded columns. Differential settlement 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

Retaining walls on the order of up to 22½ feet in height are anticipated for the proposed subterranean levels. As a precautionary measure, recommendations for the design of retaining walls up to a height of 25 feet are provided herein. It is anticipated these walls will be restrained. Foundations for these walls may be designed in accordance with the "Foundation Design" section above.

Retaining walls extending below the historically highest groundwater level shall be designed for to resist a full hydrostatic condition. Retaining walls located above the historically highest groundwater level may be designed for a drained condition, provided that a subdrain system is installed.

Additional active pressure should be added to the retaining wall design for any additional surcharge conditions, such as adjacent traffic and structures.

Restrained Retaining Walls

Restrained subterranean retaining walls up to 25 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. Wall pressures are provided in the following table for hydrostatic design. Pressures for drained conditions are also provided for designs that incorporate a subdrain above the historically highest groundwater level.

RESTRAINED BASEMENT WALLS ***(HYDROSTATIC DESIGN)***		
Height of Wall (Feet)	**AT-REST EARTH PRESSURE** Triangular Distribution of Pressure	**ACTIVE EARTH PRESSURE*** Triangular Distribution of Pressure
Up to 25	**94 pcf**	**83 pcf***

* To be combined with Dynamic Seismic Earth Pressure Active Hydrostatic Pressure

Retaining walls to be located above the historically highest groundwater level could be designed for a drained condition, provided that a subdrain is installed. Retaining walls which will be drained may be designed based on the following table:

RESTRAINED BASEMENT WALLS ABOVE THE GROUNDWATER LEVEL ***(DRAINED CONDITIONS)*****		
Height of Wall (Feet)	**AT-REST EARTH PRESSURE** Triangular Distribution of Pressure	**ACTIVE EARTH PRESSURE*** Triangular Distribution of Pressure
Up to 18 feet	**66 pcf**	**45 pcf***

* To be combined with Dynamic Seismic Earth Pressure Active Hydrostatic Pressure
**Where drained retaining wall pressures are utilized in the design, a subdrain system must be installed so that external water pressures cannot develop behind the walls.

Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures.

Dynamic (Seismic) Earth Pressure

For walls greater than 6 feet in height, retaining wall design shall consider the additional earth pressure caused by seismic ground shaking. A normal triangular pressure distribution should be utilized for the additional seismic loads, with an equivalent fluid pressure of 27 pounds per cubic foot. The seismic earth pressure should be combined with the lateral active earth pressure for analyses of restrained basement walls under seismic loading condition when using the load combination equations provided in the building code.

Surcharge Loads

It is anticipated that the proposed subterranean retaining walls may be surcharged by a neighboring single-story structure located to the east of the proposed building site. 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.

Vehicular traffic from adjacent streets, driveways and parking areas is expected in the vicinity of the proposed retaining walls. 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.

Waterproofing

Moisture affecting 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.

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

Retaining Wall Drainage

If the proposed walls are designed to fully resist hydrostatic forces, then retaining wall back drains may be omitted from the design.

If portions of the development incorporate a drained (or partially drained) design, retaining walls should be provided with a subdrain consisting of a perforated pipe, placed with perforations facing down, covered with a minimum of 12 inches of gravel, and a compacted fill blanket or other seal at the surface. 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 4 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 water 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.

It is recommended a qualified dewatering consultant be retained in order to establish design flow rates and ensure adequate sizing of subdrainage pipes and systems. Subdrainage pipes should outlet to an acceptable location.

Retaining Wall Backfill

Any required backfill should be mechanically compacted in layers not more than 8 inches thick, to at least 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) relative compaction, obtainable by the most recent revision of ASTM D 1557 method of compaction. Flooding should not be permitted. Proper compaction of the backfill will be necessary to reduce settlement of the backfill and 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.

Sump Pump Design

It is anticipated that sump pumps would not be necessary if the proposed retaining walls are designed to fully resist hydrostatic forces.

Should the proposed retaining walls be equipped with drainage just above the historic high-water level, then the only water which would be expected to affect the proposed retaining walls would be irrigation and precipitation. Additionally, the site grading will be such that all drainage will be directed to away from the structures, which will be designed with adequate non-erosive drainage devices. Based on these considerations, a retaining wall backdrainage system above the historic high-water level would not be expected to experience an appreciable flow of water, and in particular, no groundwater will affect it. However, for the purposes of design, a minimum flow of 20 gallons per minute may be assumed for sump design.

TEMPORARY EXCAVATIONS

Excavations up to 25 feet in depth may be required for construction of the proposed subterranean levels and foundation elements. 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. Vertical excavations exceeding 5 feet, or excavations which will be surcharged by adjacent traffic or structures should be shored.

Where sufficient space is available, temporary unsurcharged embankments could be cut at a uniform 1:1 slope gradient to a maximum depth of 30 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.

Excavation Observations

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.

SHORING DESIGN

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. The soldier piles may be designed as cantilevers or laterally braced utilizing drilled tied-back anchors or raker braces.

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 500 pounds per square foot per foot, up to a maximum of 5,000 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 0.34 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.

It is anticipated that the proposed shoring piles will extend below the current groundwater level and their installation will require the use of a tremie to place the concrete into the bottom of the hole. A tremie shall consist of a water-tight tube having a diameter of not less than 6 inches with a hopper at the top. The tube shall be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie shall be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end shall be closed at the start of the work to prevent water entering the tube and shall be entirely sealed at all times, except when the concrete is being placed. The tremie tube shall be kept full of concrete. The flow shall be continuous until the work is completed and the resulting concrete seal shall be monolithic and homogeneous. The tip of the tremie tube shall always be kept about five feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

A special concrete mix should be used for concrete to be placed below water. The design shall provide for concrete with a strength of 1,000 psi over the initial job specification. An admixture that reduces the problem of segregation of paste/aggregates and dilution of paste shall be included. The slump shall be commensurate to any research report for the admixture, provided that it shall also be the minimum for a reasonable consistency for placing when water is present.

Caving of the saturated granular earth materials below the groundwater level may occur during drilling of piles. Casing or polymer drilling fluid may be required during drilling in order to maintain open 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.

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 be 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.

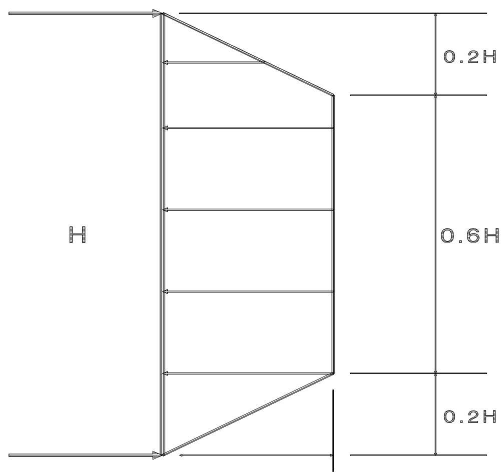
Lateral Pressures

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 20	28

A trapezoidal distribution of lateral earth pressure would be appropriate where shoring is to be restrained at the top by bracing or tie backs, with the trapezoidal distribution as shown in the diagram below.

TRAPEZOIDAL DISTRIBUTION OF PRESSURE



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

HEIGHT OF SHORING "H" (feet)	**DESIGN SHORING FOR (Where H is the height of the wall)**
Up to 20	18H
20 to 30	20H

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.

Tied-Back Anchors

Tied-back anchors may be used to resist lateral loads. Friction anchors are recommended. For design purposes, it may be assumed that the active wedge adjacent to the shoring is defined by a plane drawn 35 degrees with the vertical through the bottom plane of the excavation. Friction anchors should extend a minimum of 20 feet beyond the potentially active wedge. Anchors should be placed at least 6 feet on center to be considered isolated.

Drilled friction anchors may be designed for a skin friction of 450 pounds per square foot. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. Where belled anchors are utilized, the capacity of belled anchors may be designed by applying the skin friction over the surface area of the bonded anchor shaft. The diameter of the bell may be utilized as the diameter of the bonded anchor shaft when determining the surface area. This implies that in order for the belled anchor to fail, the entire parallel soil column must also fail.

Depending on the techniques utilized, and the experience of the contractor performing the installation, it is anticipated that a skin friction of 2,500 pounds per square foot could be utilized for post-grouted anchors. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads.

Anchor Installation

Tied-back anchors may be installed between 20 and 45 degrees below the horizontal. Caving of the anchor shafts, particularly within saturated sand deposits, should be anticipated and the following provisions should be implemented in order to minimize such caving. The anchor shafts should be filled with concrete by pumping from the tip out, and the concrete should extend from the tip of the anchor to the active wedge. In order to minimize the chances of caving, it is recommended that the portion of the anchor shaft within the active wedge be backfilled with sand before testing the anchor. This portion of the shaft should be filled tightly and flush with the face of the excavation. The sand backfill should be placed by pumping; the sand may contain a small amount of cement to facilitate pumping.

Tieback Anchor Testing

At least 10 percent of the anchors should be selected for "Quick", 200 percent tests. It is recommended that at least three anchors be selected for 24-hour, 200 percent tests. It is recommended that the 24-hour tests be performed prior to installation of additional tiebacks. The purpose of the 200 percent tests is to verify the friction value assumed in design. The anchors should be tested to develop twice the assumed friction value. Where satisfactory tests are not achieved on these initial anchors, the anchor diameter and/or length should be increased until satisfactory test results are obtained.

The total deflection during the 24-hour 200 percent test should not exceed 12 inches. During the 24-hour tests, the anchor deflection should not exceed 0.75 inches measured after the 200 percent test load is applied.

For the "quick" 200 percent tests, the 200 percent test load should be maintained for 30 minutes. The total deflection of the anchor during the 200 percent quick tests should not exceed 12 inches; the deflection after the 200 percent load has been applied should not exceed 0.25 inch during the 30-minute period.

All of the remaining anchors should be tested to at least 150 percent of design load. The total deflection during the 150 percent test should not exceed 12 inches. The rate of creep under the 150 percent test load should not exceed 0.1 inch over a 15 minute period in order for the anchor to be approved for the design loading.

After a satisfactory test, each anchor should be locked-off at the design load. This should be verified by rechecking the load in the anchor. The load should be within 10 percent of the design load. Where satisfactory tests are not attained, the anchor diameter and/or length should be increased or additional anchors installed until satisfactory test results are obtained. Where post-grouted anchors are utilized, additional post-grouting may be required. The installation and testing of the anchors should be observed by a representative of the soils engineer.

Internal Bracing

Rakers may be utilized to brace the soldier piles in lieu of tieback anchors. The raker bracing could be supported laterally by temporary concrete footings (deadmen) or by the permanent interior footings. An allowable bearing pressure of 4,000 pounds per square foot may be used for the design a raker foundations. This bearing pressure is based on a raker foundation a minimum of 24 inches in width and length as well as 18 inches in depth into native alluvial soils. The base of the raker foundations should be horizontal. Care should be employed in the positioning of raker foundations so that they do not interfere with the foundations for the proposed structure.

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 should be a minimum of 5 inches in thickness. Slabs-on-grade should be cast over undisturbed natural geologic materials or properly controlled fill materials. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum dry density.

Outdoor concrete flatwork should be a minimum of 4 inches in thickness. Outdoor concrete flatwork should be cast over undisturbed natural geologic materials or properly controlled fill materials. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) 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, where necessary, it is recommended that a qualified consultant should 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 on various components of the structure.

Where any dampness would be objectionable or where the slab will be cast below the historic high groundwater level, it is recommended that floor slabs should be waterproofed. A qualified waterproofing consultant should be engaged in order to recommend a product and/or method which would provide protection from unwanted moisture.

Based on ACI 302.2R-30, Chapter 7, for projects which do not have vapor sensitive coverings or humidity-controlled areas, a vapor retarder/barrier is not necessary. Where a vapor retarder/barrier is considered necessary, the design of the slab and the installation of the vapor retarder/barrier should comply with the most recent revisions of ASTM E 1643 and ASTM E 1745. The vapor retarder/barrier should comply with ASTM E 1745 Class A requirements. The necessity of a vapor retarder/barrier is not a geotechnical issue and should be confirmed by qualified members of the design team.

Based on ACI 302.2R-30, Chapter 7, for projects with vapor sensitive coverings, a vapor retarder/ barrier should be provided. Figure 7.1 shows that the slab should be poured on the vapor retarder/barrier. The ACI guide notes in 5.2.3.2 that the decision to locate the vapor retarder/barrier in direct contact with the slab's underside had long been debated. Experience has shown, however, that the greatest level of protection for floor coverings, coating, or building environments is provided when the vapor retarder/barrier is placed in direct contact with the slab. The necessity of a vapor retarder as well as the use of dry granular material, as discussed above is not a geotechnical issue and should be confirmed by qualified members of the design team.

Where a vapor retarder/barrier is used, it should be placed on a level and compact subgrade. Precautions should be taken to protect the vapor retarder/barrier from damage during installation of reinforcing, utilities and concrete. The use of stakes driven thought the vapor retarder/barrier should be avoided. Repair any damaged areas of the vapor retarder/barrier prior to concrete placement.

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 10 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 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) 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 18-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	5
Moderate Truck	4	7
Heavy Truck	5	9

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 10 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 18-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

The on-site soils consist mainly of fine-grained soils with poor infiltration capabilities. In addition, based on the current groundwater level and the anticipated depth of the proposed subterranean levels, it is the opinion of this firm that onsite stormwater infiltration is not suitable for the project.

Where infiltration of stormwater into the subgrade soils is not advisable, most Building Officials have allowed the stormwater to be filtered through soils in planter areas. Once the water has been filtered through a planter it may be released into the storm drain system. It is recommended that overflow pipes are incorporated into the design of the discharge system in the planters to prevent flooding. In addition, the planters shall be sealed and waterproofed to prevent leakage. Please be advised that adverse impact to landscaping and periodic maintenance may result due to excessive water and contaminants discharged into the planters.

It is recommended that the design team (including the structural engineer, waterproofing consultant, plumbing engineer, and landscape architect) be consulted in regards to the design and construction of filtration systems.

DESIGN REVIEW

Engineering of the proposed project should not begin until approval of the final design-level geotechnical report by the Building Official is obtained in writing. Certain 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 excavation 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 environmental review and preliminary design 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 effect 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 an automatic-trip 140-pound 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.

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 on 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 on Plate D of this report.

Grain Size Distribution

Sieve analysis, ASTM D6913, is used to determine the grain size distribution of the soil larger than the Number 200 sieve. The grain size distributions are plotted on the E-Plates presented in the Appendix of this report.

Atterberg Limits

Depending on their moisture content, cohesive soils can be solid, plastic, or liquid. The water contents corresponding to the transitions from solid to plastic or plastic to liquid are known as the Atterberg Limits. The transitions are called the plastic limit and liquid limit. The difference between the liquid and plastic limits is known as the plasticity index. ASTM D 4318 is utilized to determine the Atterberg Limits. The results are shown on the enclosed F-Plates.

REFERENCES

California Department of Conservation, 2008, Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117A, California Geological Survey.

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State of California Division of Oil, Gas, and Geothermal Resources, Online Mapping System, http://maps.conservation.ca.gov/doms/doms-app.html.

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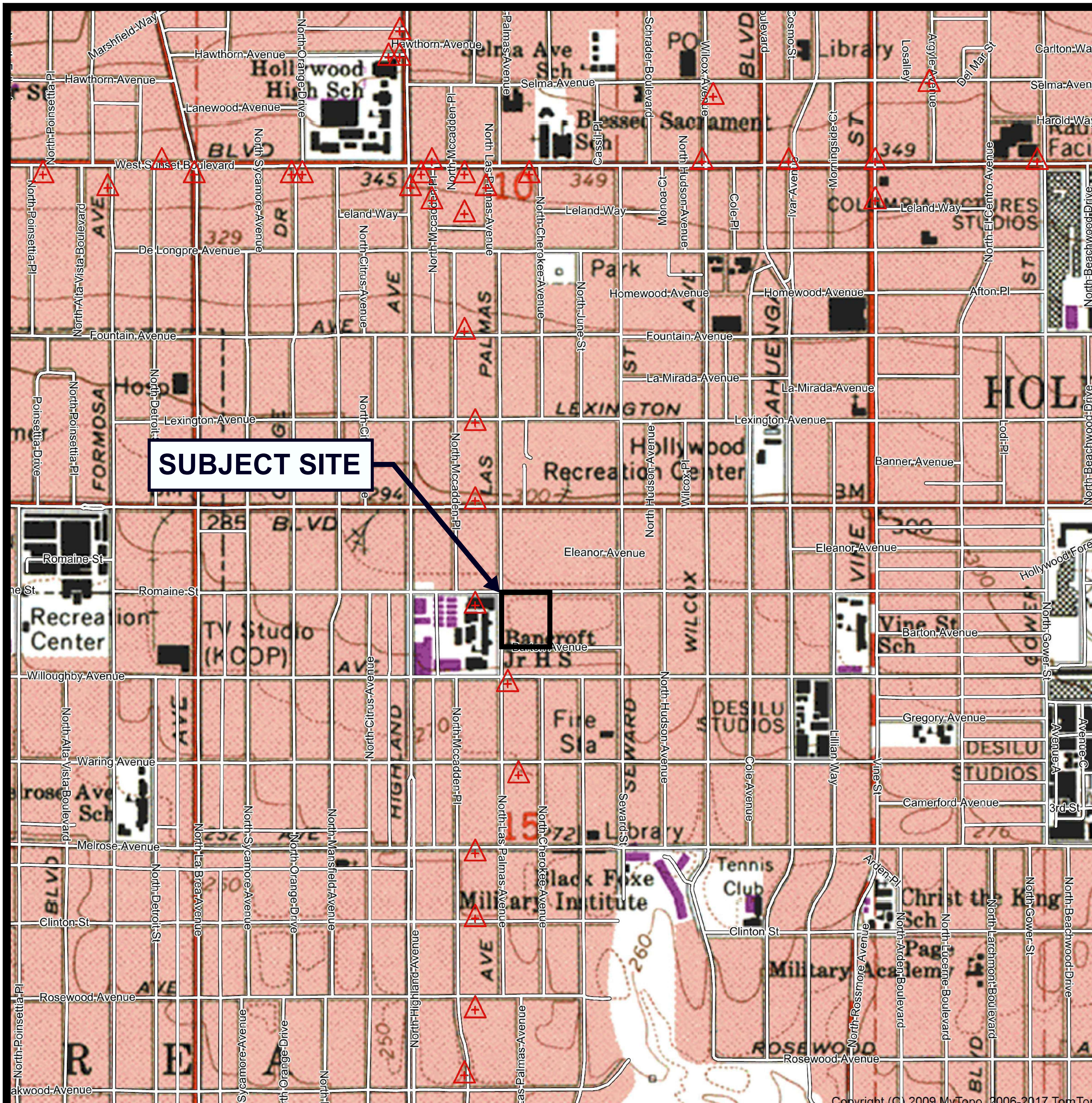


SCALE 1:12000

0.0 0.1 0.2 0.3 0.4 0.5 0.6
MILES

0 1000 2000 3000
FFFT

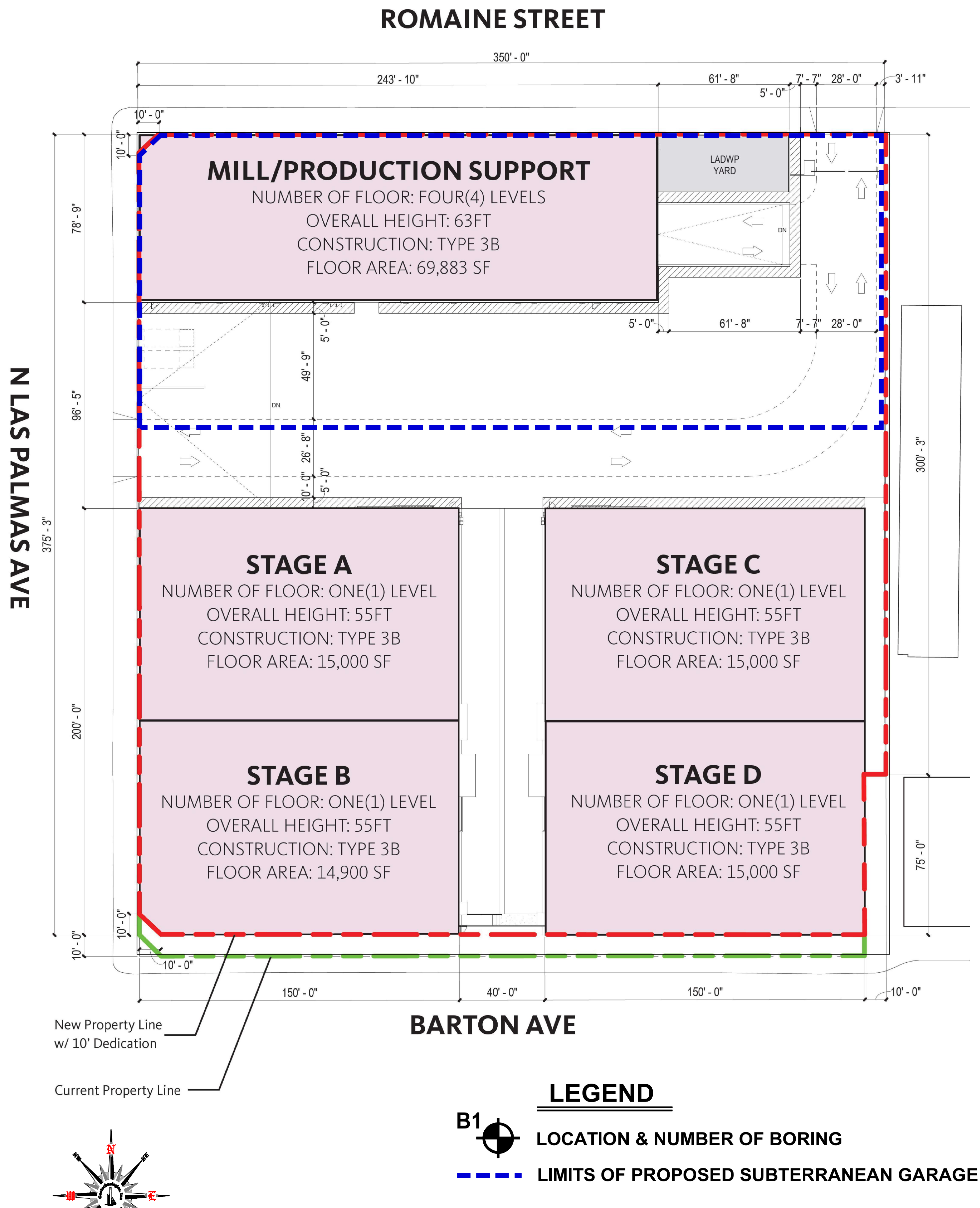
REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, HOLLYWOOD, CA QUADRANGLE

VICINITY MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

HUDSON PACIFIC PROPERTIES
SUNSET LAS PALMAS STUDIOS - LOWER LOT

FILE NO. 22063-01



REFERENCE: PLOT PLAN BY GENSLER
DATED JUNE 6, 2023

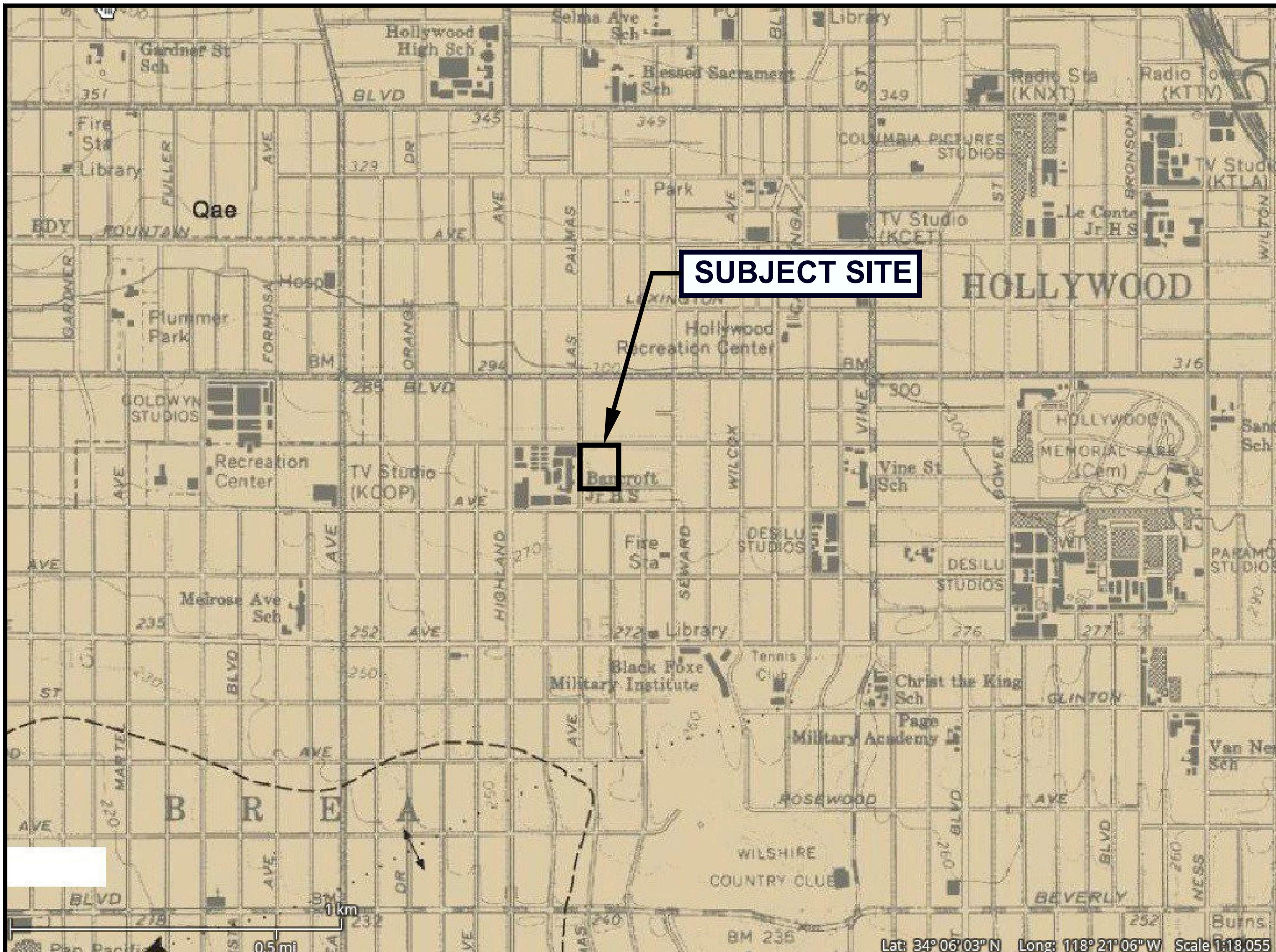
Geotechnologies, Inc.
Consulting Geotechnical Engineers

PLOT PLAN

HUDSON PACIFIC PROPERTIES
SUNSET LAS PALMAS STUDIOS - LOWER LOT

File No.: 22063-01

Date: December 2023



LEGEND

Qae: Older Surficial Sediments - alluvium: gravel, sand and clay, but slightly elevated and dissected

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., (1991) GEOLOGIC MAP OF THE HOLLYWOOD AND BURBANK (SOUTH HALF) QUADRANGLES (#DF-30)

LOCAL GEOLOGIC MAP - DIBBLEE

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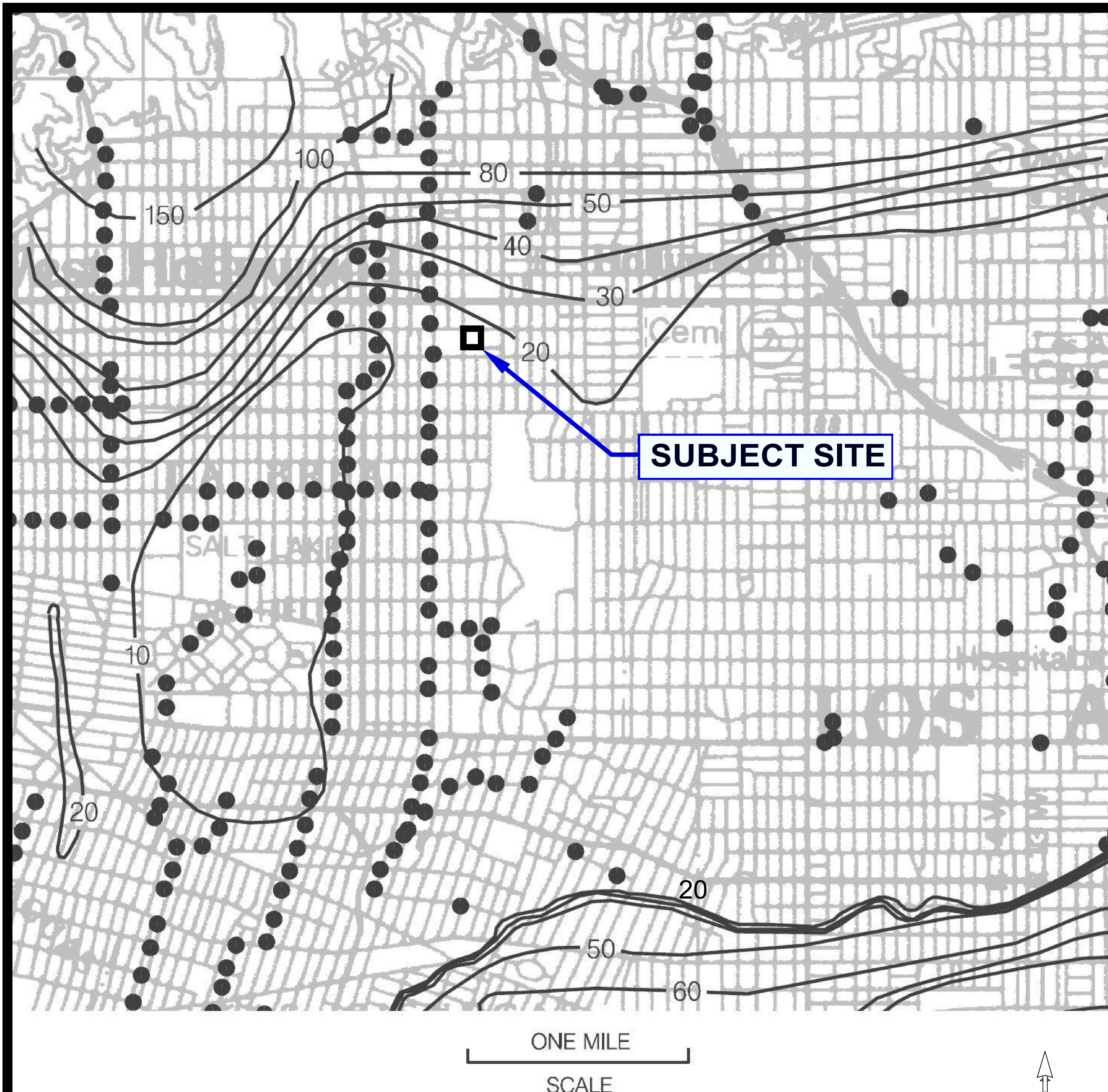
Consulting Geotechnical Engineers

HUDSON PACIFIC PROPERTIES
SUNSET LAS PALMAS STUDIOS - LOWER LOT

FILE NO. 22063-01



20 Depth to groundwater in feet

REFERENCE: CDMG, SEISMIC HAZARD ZONE REPORT, 026
HOLLYWOOD 7.5 - MINUTE QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA (1998, REVISED 2006)

HISTORICALLY HIGHEST GROUNDWATER LEVELS

Geotechnologies, Inc.
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HUDSON PACIFIC PROPERTIES
SUNSET LAS PALMAS STUDIOS - LOWER LOT

FILE NO. 22063-01

SEISMIC SOURCE SUMMARY TABLE

Geotechnologies, Inc.

(Based on USGS 2008 National Seismic Hazard Maps)

Hudson Pacific Priperties

File No.: 22063-01

Fault Name	Distance (Miles)	Preferred Dip (degrees)	Dip Direction	Slip Sense	Activity	Reference
Santa Monica (North Salt Lake)	0.20	44		strike slip	-	1, 2
Hollywood	1.23	70	N	strike slip	A (EFZ)	2
Elysian Park (Upper)	2.74	50	NE	reverse	-	1
Puente Hills (LA)	3.84	27	N	thrust	-	1
Newport-Inglewood	4.43	88		strike slip	A (EFZ)	2
Santa Monica	4.62	44		strike slip	A (EFZ)	2
Raymond	6.79	79	N	strike slip	A (EFZ)	2
Verdugo	7.40	55	NE	reverse	A	1,3
Malibu Coast	11.68	75	N	strike slip	A (EFZ)	2
Sierra Madre	11.84	53	N	reverse	A	3
Sierra Madre (San Fernando)	11.84	45	N	reverse	A (EFZ)	2
Anacapa-Dume	13.33	41	N	thrust	PA	3
Palos Verdes	15.10	90	V	strike slip	A	2
Northridge	15.45	35	S	thrust	A	3
San Gabriel	16.12	61	N	strike slip	A (EFZ)	2
Elsinore (Whittier)	17.66	75	NE	strike slip	A (EFZ)	2
Santa Susana	18.44	55	N	reverse	A	3
Clamshell-Sawpit	20.19	50	NW	reverse	PA	3
Simi-Santa Rosa	25.47	60		strike slip	A (EFZ)	2
Holser	25.75	58	S	reverse	-	1
San Jose	26.36	74	NW	strike slip	-	1
Oak Ridge	30.80	53		reverse	-	1
Chino	34.01	65	SW	strike slip		2
San Andreas	34.23	90	V	strike slip	A (EFZ)	2
San Cayetano	34.24	42	N	thrust	A (EFZ)	2
Cucamonga	34.82	45	N	reverse	A (EFZ)	2
San Joaquin Hills	35.61	23	SW	thrust	-	1
Newport-Inglewood (Offshore)	41.94	90	V	Strike Slip	A	3
San Jacinto	46.29	90	V	strike slip	-	1
Santa Ynez	47.18	70		strike slip	A	2
Ventura-Pitas Point	49.41	64	N	reverse	A (EFZ)	2
Pitas Point	49.41	55		reverse	A (EFZ)	2
Gleghorn	52.15	90	V	Strike Slip	-	1
Channel Islands Thrust	53.44	20	N	thrust	-	1
Santa Cruz Island	53.60	90	V	strike slip	A	2
Mission Ridge-Arroyo Parida	54.34	70	S	reverse	PA	2
Red Mountain	58.13	56	N	reverse	A (EFZ)	2
Garlock	59.39	90	V	strike slip	A (EFZ)	2

1 = United States Geological Survey

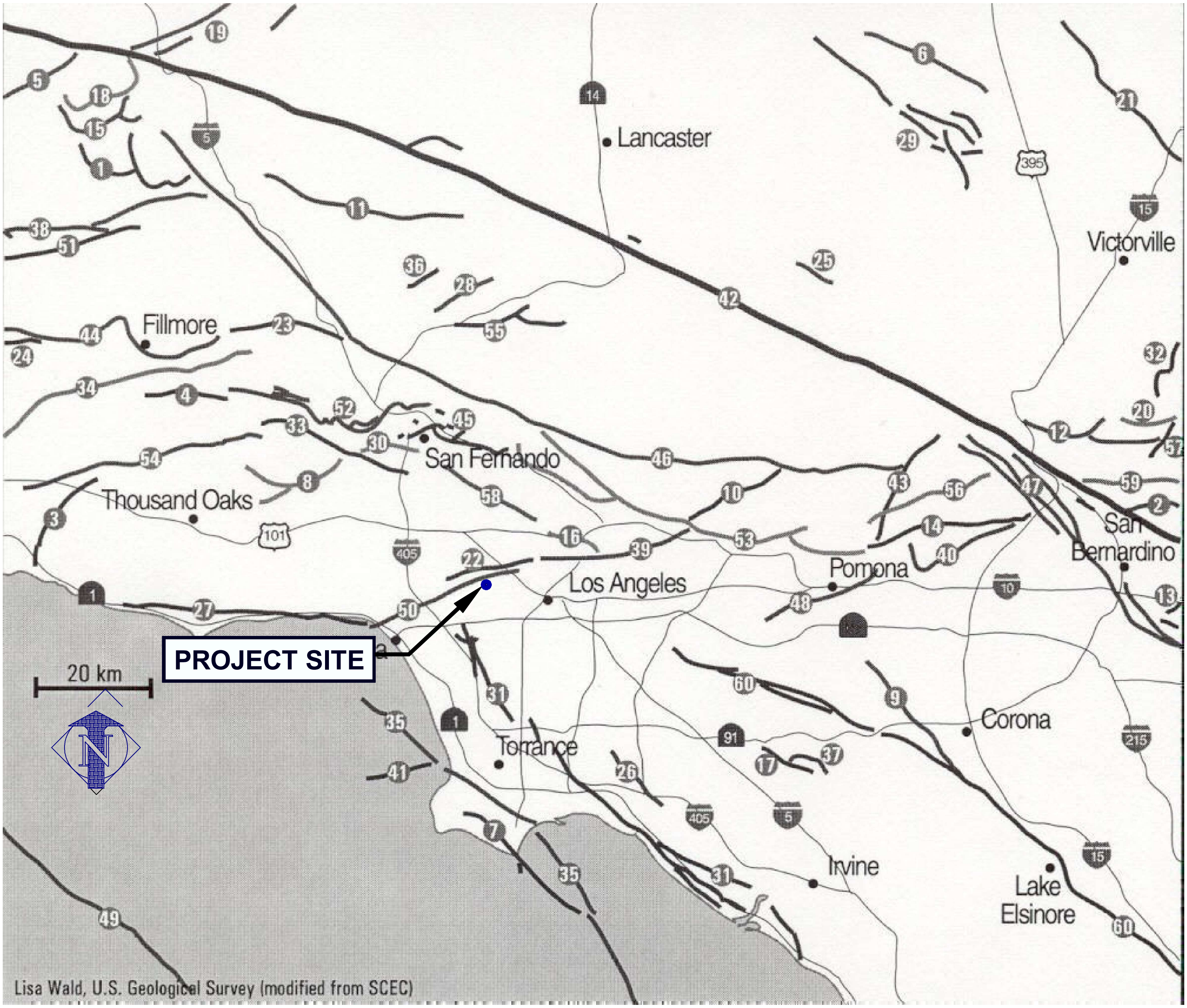
2 = California Geological Survey

3 = County of Los Angeles, Dept. of Public Works, 1990

A = Holocene Active

PA = Pre Holocene

A (EFZ) = Holocene Active (Earthquake Fault Zone)



1 Alamo thrust
2 Arrowhead fault
3 Bailey fault
4 Big Mountain fault
5 Big Pine fault
6 Blake Ranch fault
7 Cabrillo fault
8 Chatsworth fault
9 Chino fault
10 Clamshell-Sawpit fault
11 Clearwater fault
12 Cleghorn fault
13 Crafton Hills fault zone
14 Cucamonga fault zone
15 Dry Creek fault
16 Eagle Rock fault
17 El Modeno fault
18 Frazier Mountain thrust
19 Garlock fault zone
20 Grass Valley fault

24 Lion Canyon fault
25 Llano fault
26 Los Alamitos fault
27 Malibu Coast fault
28 Mint Canyon fault
29 Mirage Valley fault zone
30 Mission Hills fault
31 Newport Inglewood fault zone
32 North Frontal fault zone
33 Northridge Hills fault
34 Oak Ridge fault
35 Palos Verdes fault zone
36 Pelona fault
37 Peralta Hills fault
38 Pine Mountain fault
39 Raymond fault
40 Red Hill (Etiwanda Ave) fault

44 San Cayetano fault
45 San Fernando fault zone
46 San Gabriel fault zone
47 San Jacinto fault
48 San Jose fault
49 Santa Cruz-Santa Catalina Ridge f.z.
50 Santa Monica fault
51 Santa Ynez fault
52 Santa Susana fault zone
53 Sierra Madre fault zone
54 Simi fault
55 Soledad Canyon fault
56 Stoddard Canyon fault
57 Tunnel Ridge fault
58 Verdugo fault
59 Waterman Canyon fault
60 Whittier fault

REFERENCE: http://pasadena.wr.usgs.gov/info/images/LA%20Faults.pdf

SOUTHERN CALIFORNIA FAULT MAP

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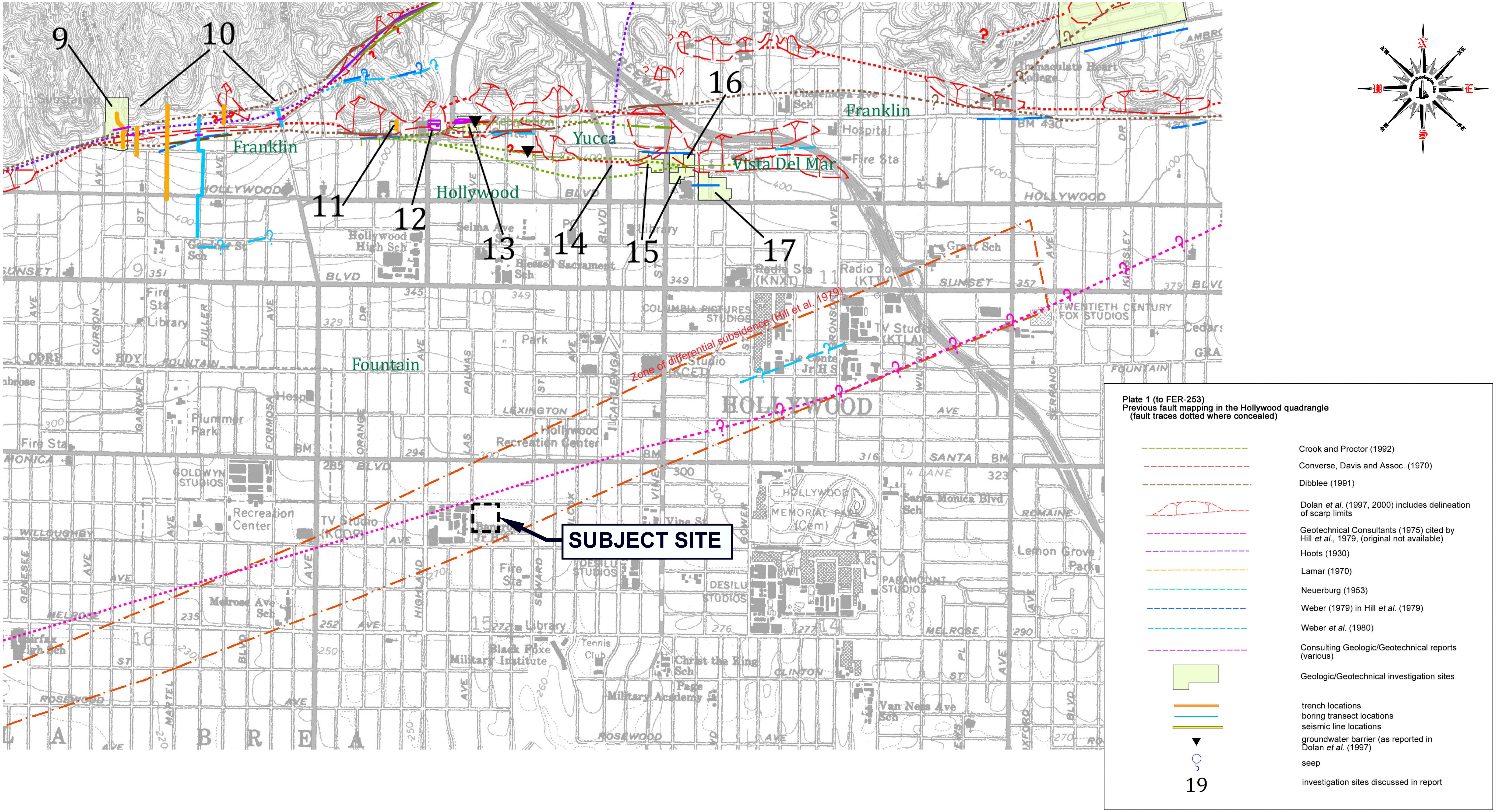
HUDSON PACIFIC PROPERTIES
SUNSET LAS PALMAS STUDIOS - LOWER LOT

FILE NO. 22063-01



PREVIOUS FAULT MAPPING OF THE HOLLYWOOD QUADRANGLE

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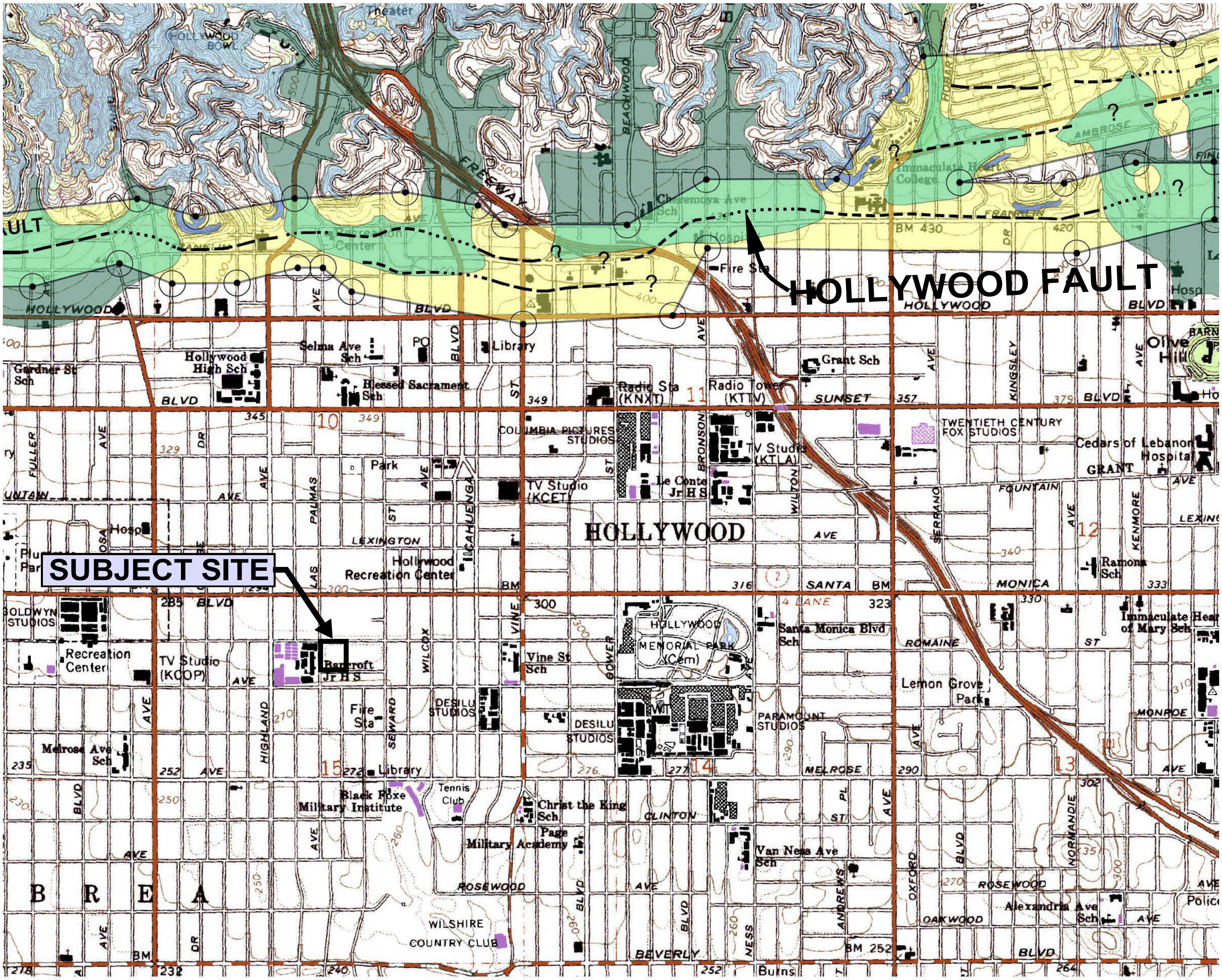
Consulting Geotechnical Engineers

HUDSON PACIFIC PROPERTIES
SUNSET LAS PALMAS STUDIOS - LOWER LOT

FILE No. 22063-01

REFERENCE: FAULT EVALUATION REPORT FER 253
CGS, 2014



1 0.5 0 1 2 MILES

1000 0 1000 2000 3000 4000 5000 6000 7000 8000 9000 10000 FEET

1 0.5 0 1 2 KILOMETERS

1000 500 0 1000 2000 METERS

Contour Interval 20 Feet

LIQUEFACTION AREA

Earthquake Fault Zones
Alquist-Priolo Earthquake Fault Zone

REFERENCE: EARTHQUAKE ZONES OF REQUIRED INVESTIGATION, HOLLYWOOD QUADRANGLE, CALIFORNIA GEOLOGICAAL SURVEY, NOVEMBER 6, 2014

EARTHQUAKE FAULT ZONE MAP

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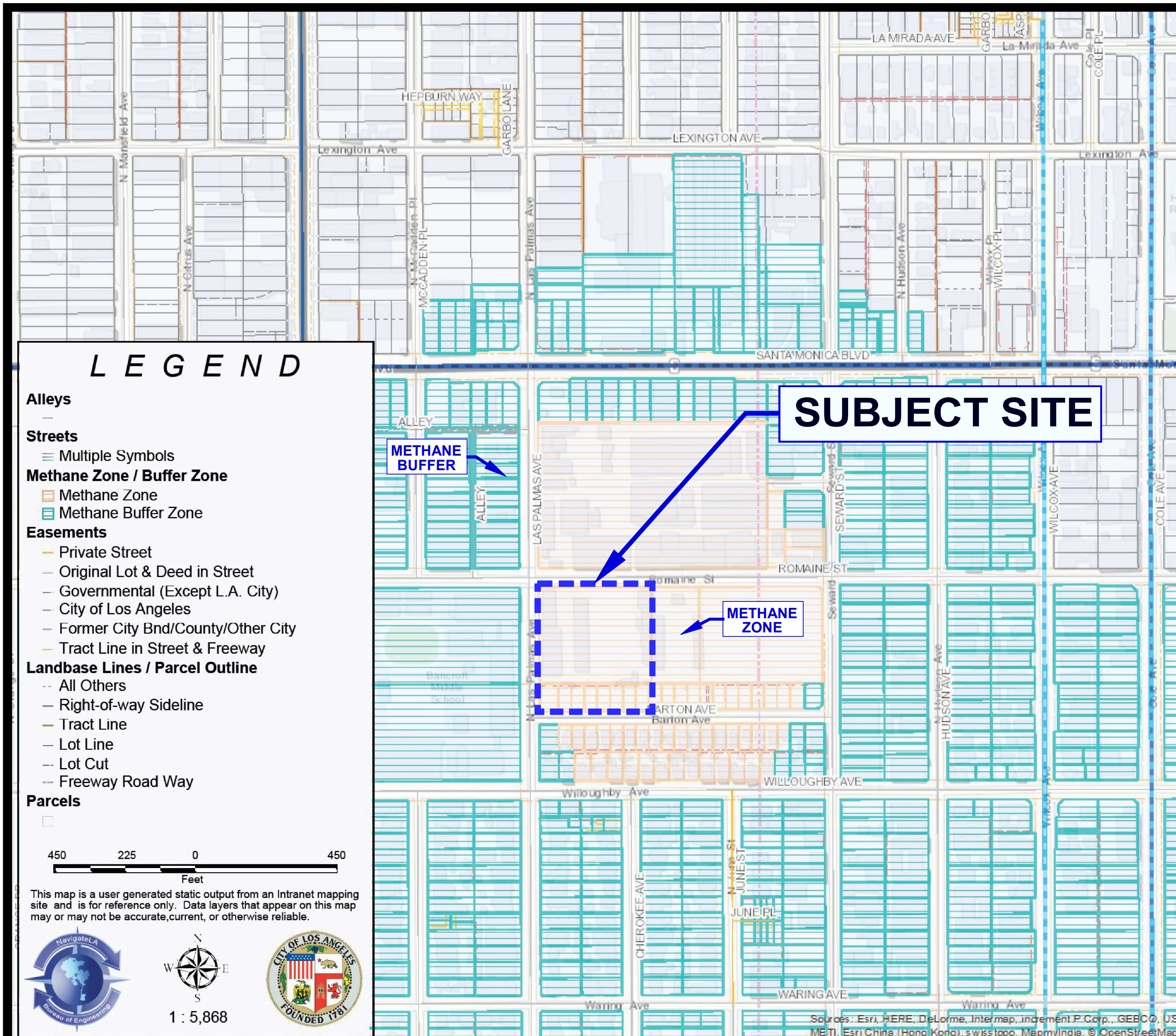
HUDSON PACIFIC PROPERTIES
SUNSET LAS PALMAS STUDIOS - LOWER LOT

FILE NO. 22063-01



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

METHANE ZONE RISK MAP

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HUDSON PACIFIC PROPERTIES
SUNSET LAS PALMAS STUDIOS - LOWER LOT

FILE NO. 22063-01

BORING LOG NUMBER 1

Hudson Pacific Properties — **Date: 12/30/20** — **Elevation: 292.5'***

File No. 22063-01 — **Method: 8-inch diameter Hollow Stem Auger**

km — *Reference: Topographic Map Included in the Entitlement Package

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		4-inch Asphalt, No Base
				-		
				1 --		FILL: Silty Clay, dark gray, moist, stiff
				-		
				2 --		
2.5	19	5.2	120.3	-		
				3 --		
				-	CL	NATIVE SOILS: Silty Clay, dark gray, moist, stiff
				4 --		
				-		
5	11	23.8	SPT	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	21	17.5	100.6	-		
				8 --	ML/CL	Sandy Silt to Silty Clay, dark gray and yellowish brown, moist, stiff
				-		
				9 --		
				-		
10	11	23.7	SPT	10 --		
				-		
				11 --		
				-		
				12 --		
12.5	37	21.4	103.8	-		
				13 --	CL	Silty Clay, dark gray, moist, stiff
				-		
				14 --		
				-		
15	19	17.2	SPT	15 --		
				-	ML	Sandy to Clayey Silt, dark brown, moist, stiff
				16 --		
				-		
				17 --		
17.5	38	19.9	111.2	-		
				18 --	SM/ML	Silty Sand to Sandy Silt, dark gray and gray, moist, medium dense, stiff, fine grained
				-		
				19 --		
				-		
20	16	25.9	SPT	20 --		
				-	ML/CL	Sandy Silt to Silty Clay, dark brown and gray, moist, stiff
				21 --		
				-		
				22 --		
22.5	27	21.2	96.1	-		
				23 --	SM	Silty Sand, dark brown, moist, medium dense, fine grained
				-		
				24 --		
				-		
25	23	15.4	SPT	25 --		
				-	SP	Sand, dark brown, wet, medium dense, fine to medium grained

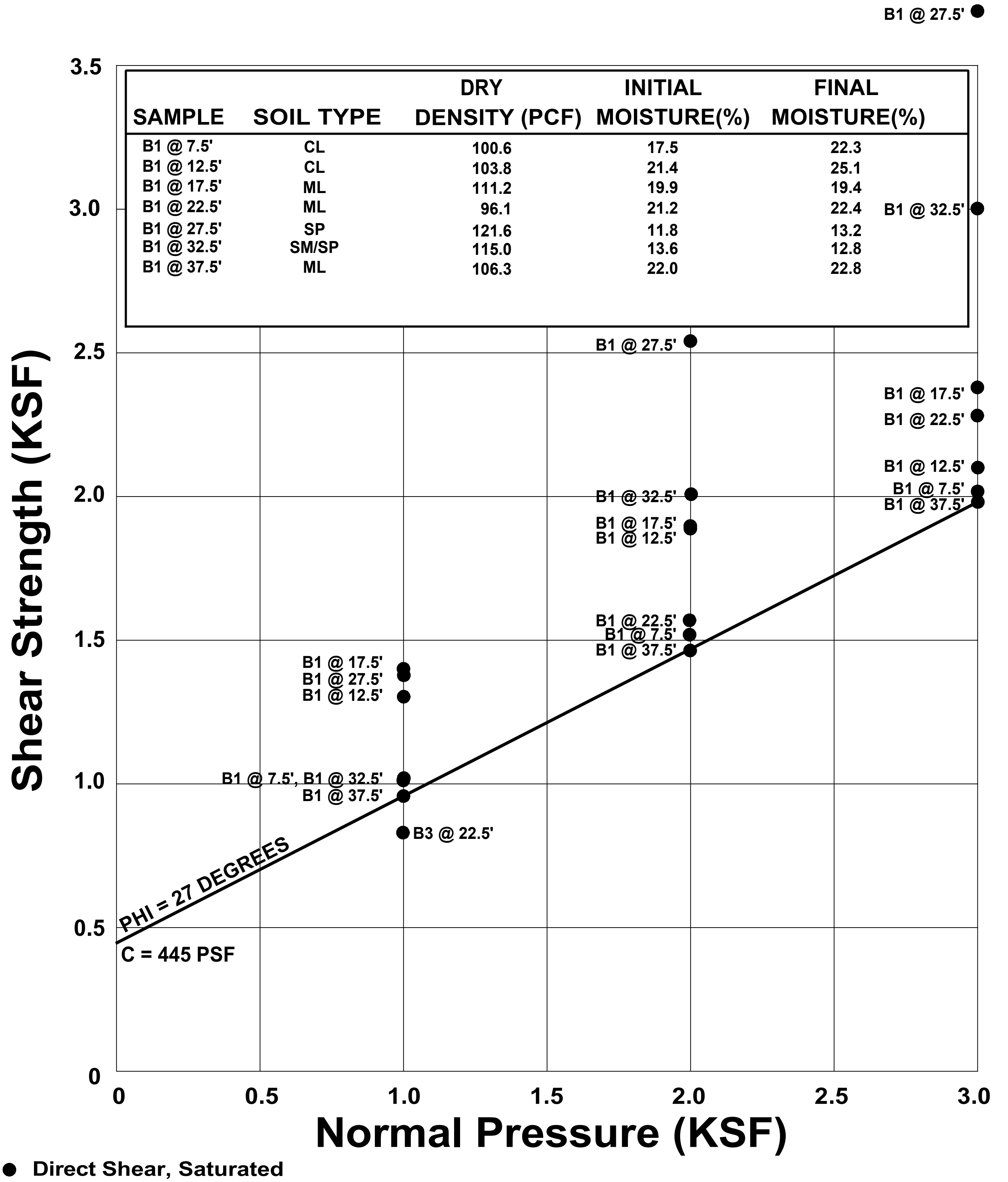
BORING LOG NUMBER 1

Hudson Pacific Properties

File No. 22063-01

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				27		
27.5	42	11.8	121.6			Used 8-inch diameter Hollow-Stem Auger
				28		140-lb. Automatic Hammer, 30-inch drop
						Modified California Sampler used unless otherwise noted
				29		
						SPT=Standard Penetration Test
30	21	12.6	SPT	30		
						minor rock fragments
				31		
				32		
32.5	31	13.6	115.0			
				33	SM/SP	Silty Sand to Sand, dark and yellowish brown, wet, medium dense, fine to medium grained
				34		
35	21	19.7	SPT	35		
					SP/ML	Sand to Sandy Silt with Clay, dark and grayish brown, wet, medium dense, stiff, fine grained
				36		
				37		
37.5	40	22.0	106.3			
				38	ML/SM	Sandy Silt to Silty Sand, dark brown and gray, moist to wet, medium dense, stiff, fine grained
				39		
40	15	19.3	SPT	40		
					ML/CL	Clayey Silt to Silty Clay, dark brown and gray, moist, stiff
				41		
				42		
42.5	39	16.6	115.0			
				43		
				44		
45	22	17.5	SPT	45		
					ML	Sandy to Clayey Silt, dark brown and gray, moist, stiff
				46		
				47		
47.5	37	18.4	113.9			
				48		
				49		Silty Sand to Clayey Sand, dark brown and gray, wet, medium
					SM/SC	dense, fine grained
50	25	18.4	SPT	50		
						Total Depth 50 feet Water at 22.2 feet Fill to 3 feet



SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B1 @ 7.5'	CL	100.6	17.5	22.3
B1 @ 12.5'	CL	103.8	21.4	25.1
B1 @ 17.5'	ML	111.2	19.9	19.4
B1 @ 22.5'	ML	96.1	21.2	22.4
B1 @ 27.5'	SP	121.6	11.8	13.2
B1 @ 32.5'	SM/SP	115.0	13.6	12.8
B1 @ 37.5'	ML	106.3	22.0	22.8

SHEAR TEST DIAGRAM

Geotechnologies, Inc.
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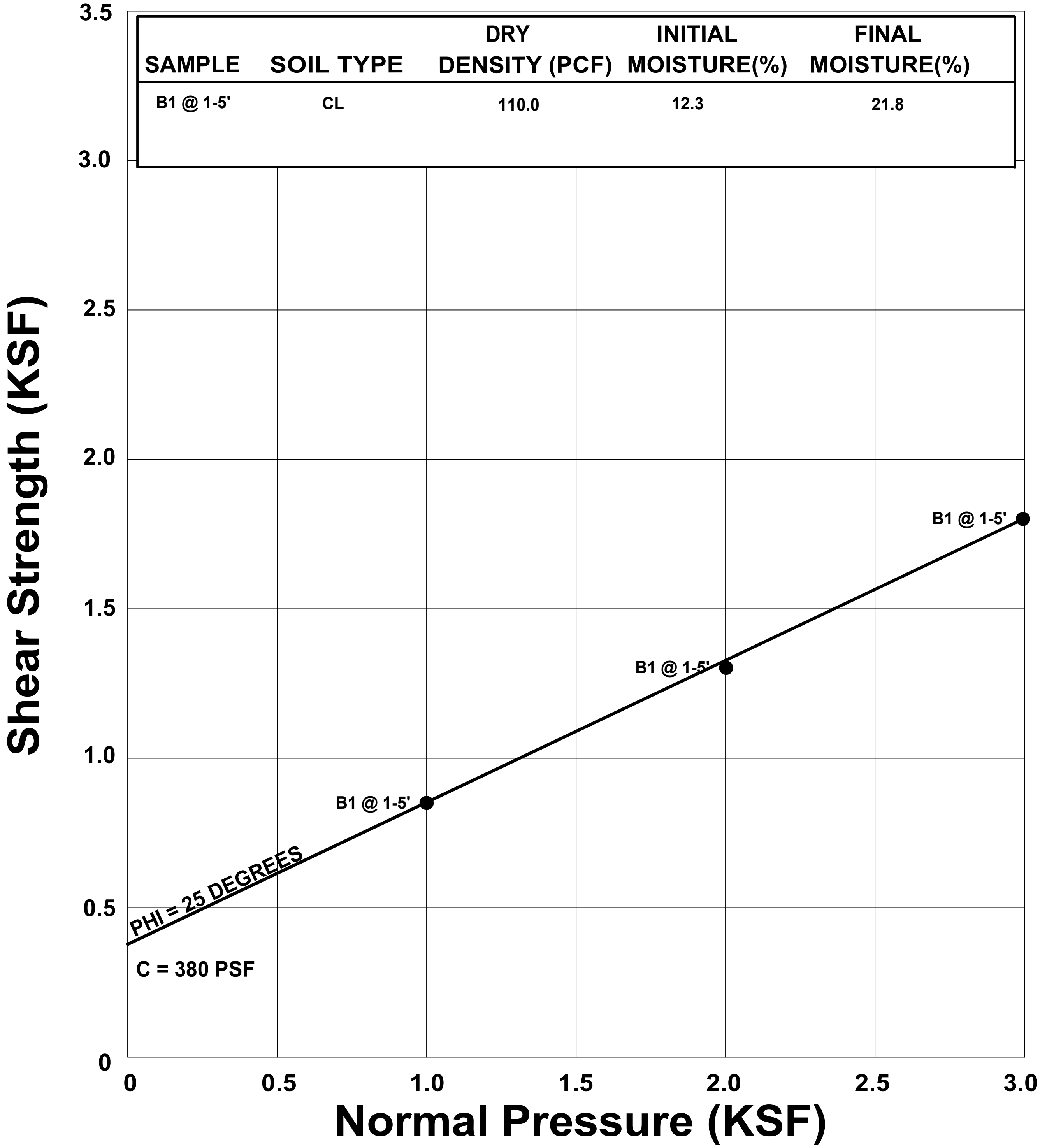
HUDSON PACIFIC PROPERTIES
SUNSET LAS PALMAS STUDIOS - LOWER LOT

FILE NO. 22063-01 | **PLATE: B-1**

BULK SAMPLE REMOLDED TO 90 PERCENT OF THE MAXIMUM LABORATORY DENSITY

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B1 @ 1-5'	CL	110.0	12.3	21.8



● Direct Shear, Saturated

SHEAR TEST DIAGRAM

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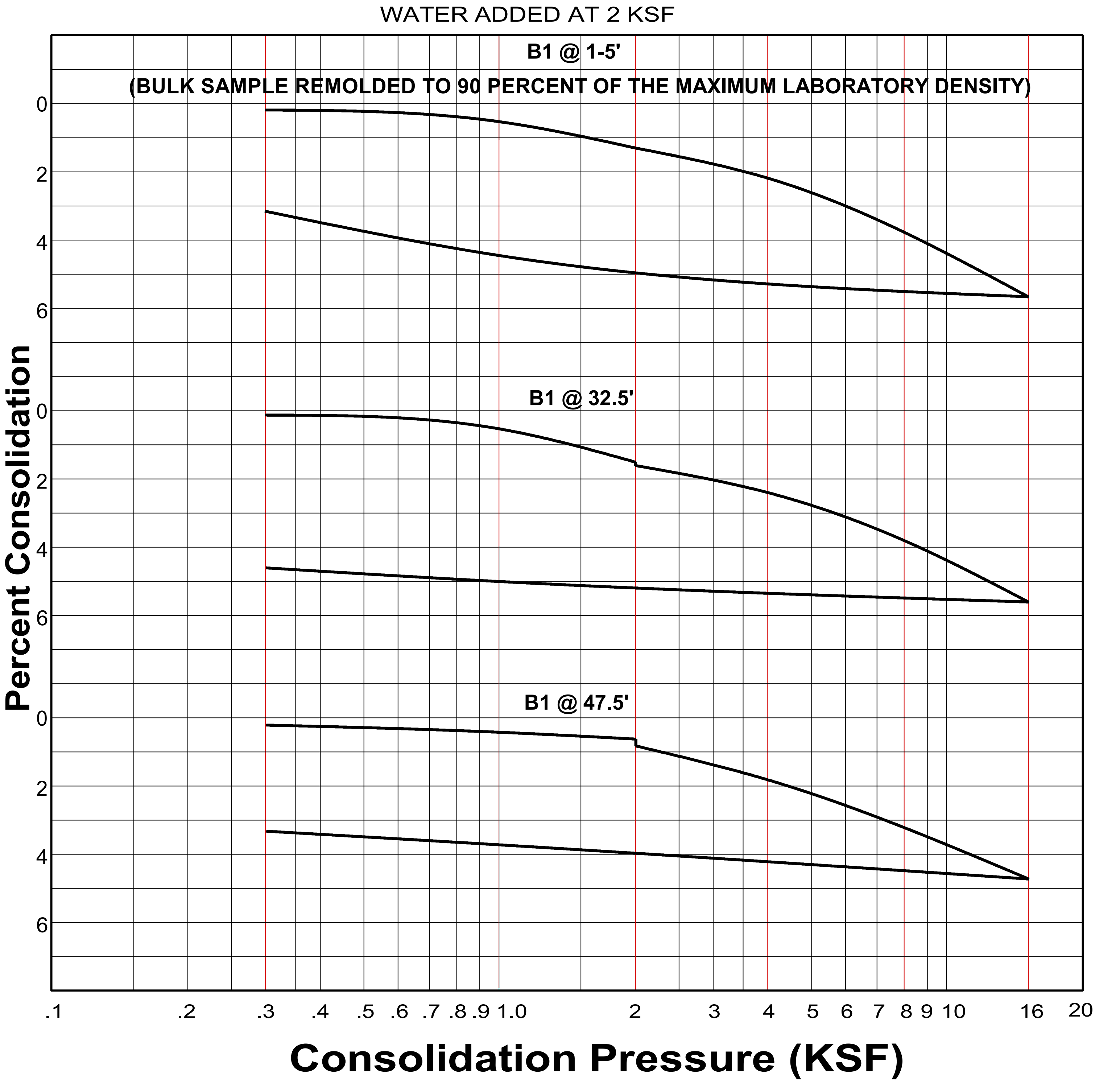
Consulting Geotechnical Engineers

HUDSON PACIFIC PROPERTIES
SUNSET LAS PALMAS STUDIOS - LOWER LOT

FILE NO. 22085	PLATE: B-2



CONSOLIDATION TEST

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HUDSON PACIFIC PROPERTIES
SUNSET LAS PALMAS STUDIOS - LOWER LOT

FILE NO. 22063-01 | **PLATE: C**

ASTM D 1557

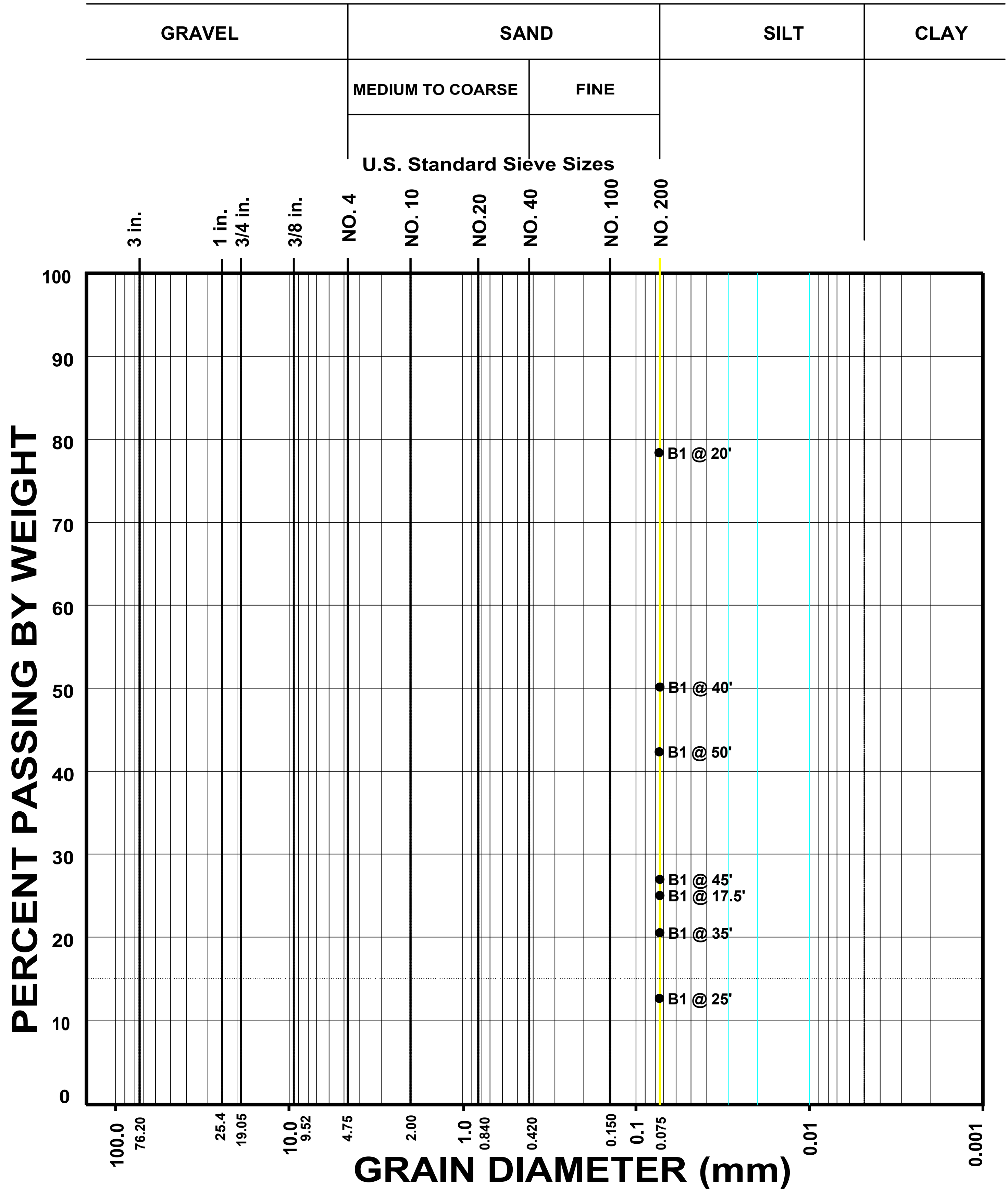
SAMPLE	B3 @ 1-5'
SOIL TYPE:	CL
MAXIMUM DENSITY pcf.	122.2
OPTIMUM MOISTURE %	12.3

ASTM D 4829

SAMPLE	B3 @ 1-5'
SOIL TYPE:	CL
EXPANSION INDEX UBC STANDARD 18-2	86
EXPANSION CHARACTER	MODERATE

SULFATE CONTENT

SAMPLE	B3 @ 1-5'	B3 @ 22.5'
SULFATE CONTENT: (percentage by weight)	< 0.20 %	< 0.20 %



GRAIN SIZE DISTRIBUTION

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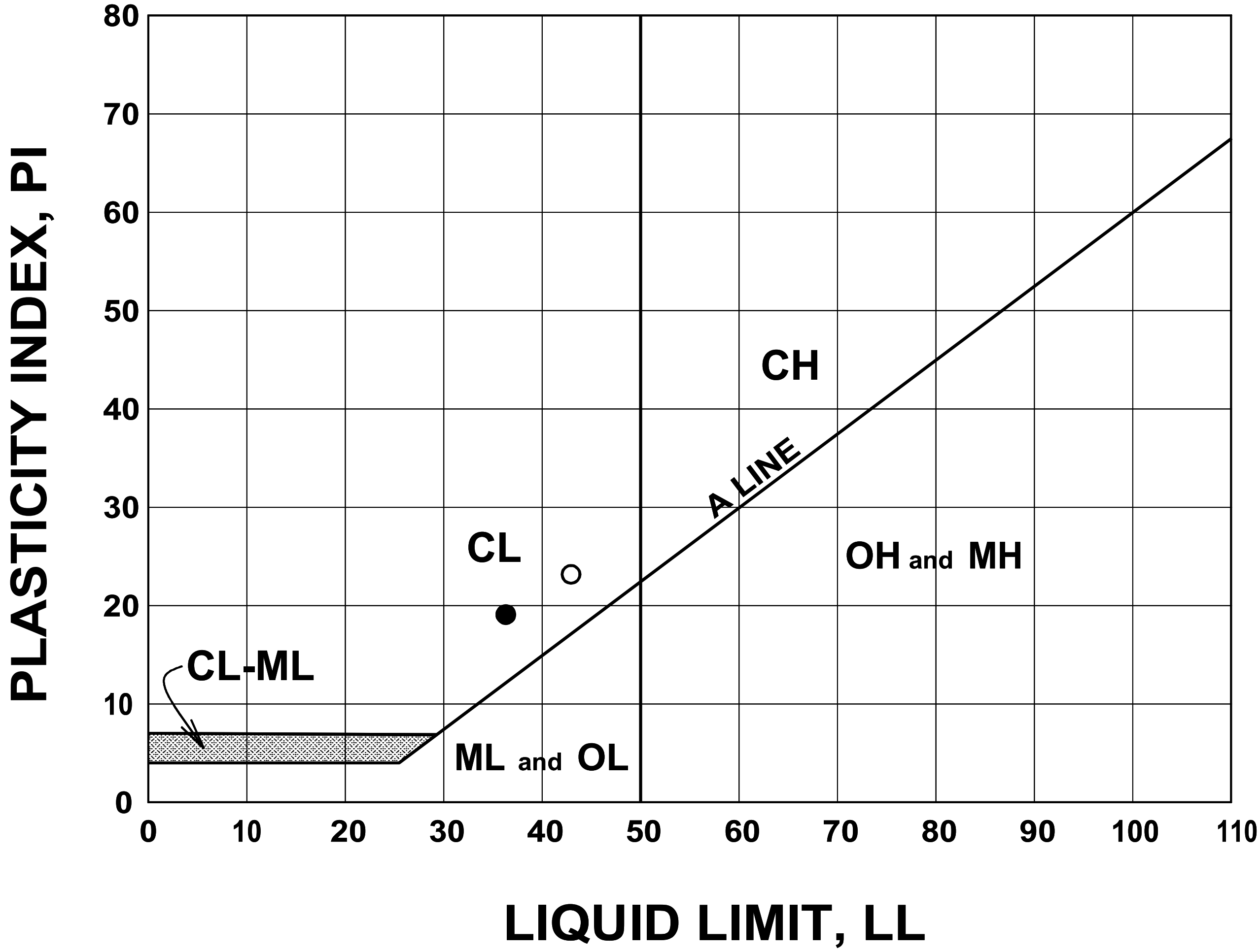
HUDSON PACIFIC PROPERTIES
SUNSET LAS PALMAS STUDIOS - LOWER LOT

FILE NO. 22063-01 | **PLATE: E**

ASTM D4318



BORING NUMBER	DEPTH (FEET)	TEST SYMBOL	LL	PL	PI	DESCRIPTION
B1	20	○	43	20	23	CL
B1	40	●	36	17	19	CL

ATTERBERG LIMITS DETERMINATION

Geotechnologies, Inc.
Consulting Geotechnical Engineers

HUDSON PACIFIC PROPERTIES
SUNSET LAS PALMAS STUDIOS - LOWER LOT

FILE NO. 22063-01 | **PLATE: F**

Geotechnologies, Inc.

Project: Hudson Pacific Properties
File No.: 22063-01
Description: Liquefaction Analysis
Boring No: B1

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	6.7
Peak Ground Horizontal Acceleration, PGA (g):	0.99
Calculated Mag.Wtg.Factor:	1.234
GROUNDWATER INFORMATION:	
Current Groundwater Level (ft):	22.2
Historically Highest Groundwater Level* (ft):	17.0
Unit Weight of Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

BOREHOLE AND SAMPLER INFORMATION:

Borehole Diameter (inches):	8
SPT Sampler with room for Liner (Y/N):	Y
LIQUEFACTION BOUNDARY:	
Plastic Index Cut Off (PI):	18
Minimum Liquefaction FS:	1.3

Depth to Base Layer (feet)	Total Unit Weight (pcf)	Current Water Level (feet)	Historical Water Level (feet)	Field SPT Blowcount N	Depth of SPT Blowcount (feet)	Fines Content #200 Sieve (%)	Plastic Index (PI)	Vetical Stress σ_{vc}, (psf)	Effective Vert. Stress σ_{vc}', (psf)	Fines Corrected $(N_1)_{60-cs}$	Stress Reduction Coeff, r_d	Cyclic Shear Ratio CSR	Mag. Scaling Factor (Sand) MSF	Overburden Corr. Factor K_σ	Cyclic Resist. Ratio $CRR_{M7.5,\sigma vc'=1}$	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Liquefaction Settlment ΔS_i (inches)
1	126.5	Unsaturated	Unsaturated	11	5	0.0	0	126.5	126.5	24.2	1.00	0.646	1.23	1.10	0.273	0.370	Non-Liq.	0.00
2	126.5	Unsaturated	Unsaturated	11	5	0.0	0	253.0	253.0	24.2	1.00	0.644	1.23	1.10	0.273	0.370	Non-Liq.	0.00
3	126.5	Unsaturated	Unsaturated	11	5	0.0	0	379.5	379.5	24.2	1.00	0.642	1.23	1.10	0.273	0.370	Non-Liq.	0.00
4	126.5	Unsaturated	Unsaturated	11	5	0.0	0	506.0	506.0	24.2	0.99	0.640	1.23	1.10	0.273	0.370	Non-Liq.	0.00
5	126.5	Unsaturated	Unsaturated	11	5	0.0	0	632.5	632.5	24.9	0.99	0.637	1.23	1.10	0.288	0.391	Non-Liq.	0.00
6	126.5	Unsaturated	Unsaturated	11	5	0.0	0	759.0	759.0	23.4	0.99	0.635	1.23	1.10	0.257	0.349	Non-Liq.	0.00
7	126.5	Unsaturated	Unsaturated	11	5	0.0	0	885.5	885.5	21.9	0.98	0.632	1.23	1.10	0.232	0.315	Non-Liq.	0.00
8	118.2	Unsaturated	Unsaturated	11	5	0.0	0	1003.7	1003.7	20.7	0.98	0.629	1.23	1.10	0.215	0.292	Non-Liq.	0.00
9	118.2	Unsaturated	Unsaturated	11	5	0.0	0	1121.9	1121.9	21.0	0.97	0.626	1.23	1.09	0.218	0.293	Non-Liq.	0.00
10	118.2	Unsaturated	Unsaturated	11	5	0.0	0	1240.1	1240.1	20.0	0.97	0.624	1.23	1.07	0.206	0.272	Non-Liq.	0.00
11	118.2	Unsaturated	Unsaturated	6	10	0.0	0	1358.3	1358.3	10.2	0.96	0.621	1.23	1.04	0.120	0.154	Non-Liq.	0.00
12	118.2	Unsaturated	Unsaturated	6	10	0.0	0	1476.5	1476.5	9.8	0.96	0.618	1.23	1.03	0.117	0.149	Non-Liq.	0.00
13	126.0	Unsaturated	Unsaturated	6	10	0.0	0	1602.5	1602.5	9.4	0.96	0.615	1.23	1.02	0.114	0.144	Non-Liq.	0.00
14	126.0	Unsaturated	Unsaturated	6	10	0.0	0	1728.5	1728.5	9.0	0.95	0.611	1.23	1.02	0.111	0.140	Non-Liq.	0.00
15	126.0	Unsaturated	Unsaturated	6	10	0.0	0	1854.5	1854.5	9.7	0.95	0.608	1.23	1.01	0.116	0.145	Non-Liq.	0.00
16	126.0	Unsaturated	Unsaturated	19	15	35.0	0	1980.5	1980.5	39.1	0.94	0.605	1.23	1.02	2.000	2.000	Non-Liq.	0.00
17	126.0	Unsaturated	Unsaturated	19	15	35.0	0	2106.5	2106.5	38.2	0.93	0.602	1.23	1.00	2.000	2.000	Non-Liq.	0.00
18	133.3	Unsaturated	Saturated	19	15	35.0	0	2239.8	2177.4	37.3	0.93	0.615	1.23	0.98	1.903	2.000	Non-Liq.	0.00
19	133.3	Unsaturated	Saturated	19	15	35.0	0	2373.1	2248.3	36.5	0.92	0.628	1.23	0.97	1.552	1.851	3.0	0.00
20	133.3	Unsaturated	Saturated	19	15	35.0	0	2506.4	2319.2	35.7	0.92	0.639	1.23	0.95	1.299	1.528	2.5	0.00
21	133.3	Unsaturated	Saturated	16	20	78.4	23	2639.7	2390.1	29.4	0.91	0.649	1.23	0.96	0.450	0.531	Non-Liq.	0.00
22	133.3	Unsaturated	Saturated	16	20	78.4	23	2773.0	2461.0	28.8	0.91	0.658	1.23	0.95	0.419	0.490	Non-Liq.	0.00
23	116.5	Saturated	Saturated	16	20	78.4	23	2889.5	2515.1	28.5	0.90	0.667	1.23	0.94	0.406	0.473	Non-Liq.	0.00
24	116.5	Saturated	Saturated	16	20	78.4	23	3006.0	2569.2	28.3	0.90	0.675	1.23	0.94	0.396	0.460	Non-Liq.	0.00
25	116.5	Saturated	Saturated	16	20	78.4	23	3122.5	2623.3	28.1	0.89	0.682	1.23	0.94	0.387	0.448	Non-Liq.	0.00
26	116.5	Saturated	Saturated	23	25	12.6	0	3239.0	2677.4	37.9	0.88	0.688	1.23	0.90	2.000	2.000	3.0	0.00
27	116.5	Saturated	Saturated	23	25	12.6	0	3355.5	2731.5	37.6	0.88	0.694	1.23	0.89	2.000	2.000	2.9	0.00
28	135.9	Saturated	Saturated	23	25	12.6	0	3491.4	2805.0	39.7	0.87	0.699	1.23	0.88	2.000	2.000	2.9	0.00
29	135.9	Saturated	Saturated	23	25	12.6	0	3627.3	2878.5	39.4	0.87	0.702	1.23	0.88	2.000	2.000	2.9	0.00
30	135.9	Saturated	Saturated	23	25	12.6	0	3763.2	2952.0	39.0	0.86	0.706	1.23	0.87	2.000	2.000	2.9	0.00
31	135.9	Saturated	Saturated	21	30	20.5	0	3899.1	3025.5	36.6	0.85	0.708	1.23	0.87	1.582	1.693	2.4	0.00
32	135.9	Saturated	Saturated	21	30	20.5	0	4035.0	3099.0	36.3	0.85	0.711	1.23	0.86	1.461	1.558	2.2	0.00
33	130.6	Saturated	Saturated	21	30	20.5	0	4165.6	3167.2	36.0	0.84	0.713	1.23	0.86	1.362	1.447	2.1	0.00
34	130.6	Saturated	Saturated	21	30	20.5	0	4296.2	3235.4	35.7	0.84	0.714	1.23	0.86	1.274	1.349	1.9	0.00
35	130.6	Saturated	Saturated	21	30	20.5	0	4426.8	3303.6	35.4	0.83	0.715	1.23	0.85	1.196	1.262	1.8	0.00
36	130.6	Saturated	Saturated	21	35	20.5	0	4557.4	3371.8	35.1	0.82	0.716	1.23	0.85	1.126	1.184	1.7	0.00
37	130.6	Saturated	Saturated	21	35	20.5	0	4688.0	3440.0	34.8	0.82	0.717	1.23	0.85	1.062	1.115	1.6	0.00
38	129.7	Saturated	Saturated	21	35	20.5	0	4817.7	3507.3	34.5	0.81	0.717	1.23	0.85	1.006	1.053	1.5	0.00
39	129.7	Saturated	Saturated	21	35	20.5	0	4947.4	3574.6	34.3	0.80	0.717	1.23	0.85	0.955	0.997	1.4	0.00
40	129.7	Saturated	Saturated	21	35	20.5	0	5077.1	3641.9	34.0	0.80	0.716	1.23	0.84	0.909	0.946	1.3	0.00
41	129.7	Saturated	Saturated	15	40	50.1	19	5206.8	3709.2	24.2	0.79	0.716	1.23	0.90	0.273	0.302	Non-Liq.	0.00
42	129.7	Saturated	Saturated	15	40	50.1	19	5336.5	3776.5	24.0	0.79	0.715	1.23	0.90	0.269	0.298	Non-Liq.	0.00
43	134.1	Saturated	Saturated	15	40	50.1	19	5470.6	3848.2	23.9	0.78	0.714	1.23	0.89	0.265	0.293	Non-Liq.	0.00
44	134.1	Saturated	Saturated	15	40	50.1	19	5604.7	3919.9	23.7	0.77	0.712	1.23	0.89	0.262	0.288	Non-Liq.	0.00
45	134.1	Saturated	Saturated	15	40	50.1	19	5738.8	3991.6	23.5	0.77	0.710	1.23	0.89	0.258	0.284	Non-Liq.	0.00
46	134.1	Saturated	Saturated	22	45	26.9	0	5872.9	4063.3	35.0	0.76	0.708	1.23	0.81	1.117	1.113	1.6	0.00
47	134.1	Saturated	Saturated	22	45	26.9	0	6007.0	4135.0	34.8	0.76	0.706	1.23	0.81	1.060	1.055	1.5	0.00
48	134.8	Saturated	Saturated	22	45	26.9	0	6141.8	4207.4	34.5	0.75	0.704	1.23	0.80	1.008	1.001	1.4	0.00
49	134.8	Saturated	Saturated	22	45	26.9	0	6276.6	4279.8	34.3	0.74	0.702	1.23	0.80	0.960	0.952	1.4	0.00
50	134.8	Saturated	Saturated	25	50	42.3	0	6411.4	4352.2	40.4	0.74	0.699	1.23	0.77	2.000	1.889	2.7	0.00