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Consulting Geotechnical Engineers

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August 12, 2021
File Number 22157

Radha Hotels USA, LLC
9 Rapallo
Irvine, California 92614

Attention: Srinivasan Radhakrishnan

<u>Subject</u>: Geotechnical Engineering Investigation
Proposed Residential Development
11903 through 11913 West Wilshire Boulevard, Los Angeles, California

Dear Mr. Radhakrishnan:

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

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

Should you have any questions please contact this office.

Respectfully submitted,
GEOTECHNOLOGIES, INC.

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LOS ANGELES, CALIFORNIA

INTRODUCTION

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

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

SITE CONDITIONS

The property is located at 11903 through 11913 West Wilshire Boulevard in the City of Los Angeles, California. The site is bounded by an alleyway to the north, by South Westgate Avenue to the east, by West Wilshire Boulevard to the south, and by a one-story commercial structure to the west. The site is shown relative to nearby topographic features in the enclosed Vicinity Map.

The topography observed across the site generally descends to the southeast. According to available survey, an approximate high elevation of 272.0 feet above Mean Sea Level Elevation (MSL) is recorded near the northern end of the site and an approximate low elevation of 267.0 feet above MSL is recorded near the southern end of the site. This corresponds to an approximate elevation difference of 5 feet across the site for an overall site gradient of 30 to 1 (horizontal to vertical). The enclosed Plot Plan provides site elevations.

The site is currently developed with a single-story, commercial structure near the center of the site, with asphalt-paved parking lots on the northeast and southwest sides. The vegetation on the site consists of isolated trees and shrubbery contained in planters. The neighboring development consists of residential and commercial developments. Drainage across the site within the parking lots appears to be by sheetflow to the city streets to the southeast.

PROPOSED DEVELOPMENT

Information concerning the proposed development was furnished by the client. The proposed project consists of a 7-story, residential development, constructed over one level of subterranean parking. The first two above grade stories are podium levels consisting of parking, retail, and amenities. The five stories above the podium will contain residential units. The lowest finish floor elevation is at about 260 feet above MSL.

With respect to the structural loading from this development, column loads are estimated to be between 600 and 800 kips. Wall loads are estimated to be between 5 and 20 kips per lineal foot. Grading will consist of excavations on the order of 15 feet for the proposed subterranean level, including 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.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was explored on June 30 and July 1, 2021, by excavating three exploratory borings. The exploratory borings varied in depth from 40 to 60 feet. The exploration was prosecuted with the aid of a truck-mounted drilling machine using 8-inch diameter hollowstem augers. The exploration locations are shown on the Plot Plan and the geologic materials encountered are logged on Plates A-1 through A-3.

The location of exploratory borings was determined by the survey provided by PSOMAS, dated June 22, 2021, measurements relative to hardscape features onsite and are shown on the enclosed Plot Plan. Elevations of the exploratory excavations were determined by interpolation of elevation contours shown on the enclosed Plot Plan. The location and elevation of the exploratory excavations should be considered accurate only to the degree implied by the method used.

Geologic Materials

Fill materials encountered in the exploratory borings consist of sandy silt and silty sand, which are dark brown in color, moist, medium dense, and stiff. The fill was found to be 3 feet in depth during exploration. Deeper fill may be present within the area of the existing structure.

The fill is in turn underlain by alluvial soils consisting of sandy to clayey silts, clays, silty sand and sands, which are yellowish to dark brown, and gray to dark gray in color, moist to very moist, dense to very dense, stiff to very stiff, and fine to coarse grained with variable amounts of slate fragments. The upper 20 feet of alluvium is interspersed with gravel sized slate fragments. A layer of sand with gravel and cobbles was identified in Boring B3 at a depth of 22.5 feet that is approximately 2.5 feet thick. More detailed descriptions of the earth materials encountered may be obtained from individual logs of the subsurface excavations.

Groundwater

Groundwater was encountered during exploration, in Borings B2 and B3 at depths of 27.5 and 27.8 feet below the ground surface, corresponding to elevations 240.5 and 242.2 feet above MSL, respectively. Groundwater was not encountered in Boring B1, which was excavated to a maximum depth of 40 feet below the existing site grade. However, a very moist layer was identified in Boring B1 at a depth of 25 feet below grade.

Review of California Geological Survey Seismic Hazard Evaluation Report of the Beverly Hills 7½-minute Quadrangle, (CDMG, 1998, Revised 2005), indicates that the historically highest groundwater level at the site is on the order of 20 feet below ground surface. A copy of this plate is included in the Appendix of this report.

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 exploration due to the type of excavation equipment utilized. Based on the experience of this firm, large diameter excavations, excavations that encounter granular, cohesionless soils and excavations below the groundwater table will most likely experience caving.

SEISMIC EVALUATION

REGIONAL GEOLOGIC SETTING

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

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

REGIONAL FAULTING

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

Buried thrust faults are faults without a surface expression but are a significant source of seismic activity. They are typically broadly defined based on the analysis of seismic wave recordings of hundreds of small and large earthquakes in the southern California area. Due to the buried nature of these thrust faults, their existence is usually not known until they produce an earthquake. The risk for surface rupture potential of these buried thrust faults is inferred to be low (Leighton, 1990).

However, the seismic risk of these buried structures in terms of recurrence and maximum potential magnitude is not well established. Therefore, the potential for surface rupture on these surface-verging splays at magnitudes higher than 6.0 cannot be precluded.

Santa Monica Fault

The closest known fault to the site which could cause surface rupture is the Santa Monica Fault which is mapped at approximately 0.41 miles to the south of the site as indicated on the enclosed Seismic Hazard Zone Map (USGS, 2008). The Santa Monica Fault is part of the Transverse Ranges Southern Boundary fault system. 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 Santa Monica Fault is the western 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.

Based on recent work by several engineering consultants and information compiled by the California Geological Survey, the Santa Monica-Hollywood Fault has been found to be sufficiently active and well-defined based on the criteria established by the California Geological Survey. As a result, an earthquake fault zone has been designated for this fault.

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.

Surface 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 nearest earthquake fault zone is located approximately 0.12 miles to the south as indicated on the attached Seismic Hazard Zone Map in the Appendix of this report. Based on these considerations, the potential for surface ground rupture at the subject site is considered low.

<u>Liquefaction</u>

Liquefaction is a phenomenon in which saturated silty to cohesionless soils below the groundwater table are subject to a temporary loss of strength due to the buildup of excess pore pressure during cyclic loading conditions such as those induced by an earthquake. Liquefaction-related effects include loss of bearing strength, amplified ground oscillations, lateral spreading, and flow failures. The Seismic Hazards Maps of the State of California (CDMG, 1999), does not classify the site as part of the potentially “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, in Borings B2 and B3 at depths of 27.5 and 27.8 feet below the ground surface, corresponding to elevations 240.5 and 242.2 feet above MSL, respectively. Review of the Seismic Hazard Zone Report (SHZR) for the Beverly Hills 7½-Minute Quadrangle, (CDMG, 1998, Revised 2006), indicates that the historic-high groundwater level at the site is on the order of 20 feet below ground surface. The historic highest groundwater level was conservatively utilized for the enclosed liquefaction analysis.

The peak ground acceleration (PGA_M) and modal magnitude were obtained from the USGS websites, using the Probabilistic Seismic Hazard Deaggregation program (USGS, 2014) and the U.S. Seismic Design Maps tool (USGS, 2018). A Site Class “D” (Stiff Soil Profile) and a published shear wave velocity of 259 meters per second were utilized for V_S30 (Tinsley and Fumal, 1985) in the USGS seismic programs. A modal magnitude (M_W) of 6.86 is obtained using the USGS Probabilistic Seismic Hazard Deaggregation program (USGS, 2014). A peak ground acceleration of 0.937g, which corresponds to the site’s PGA_M, was obtained using the U.S. Seismic Design Maps tool. These parameters are used in the enclosed liquefaction analyses.

The enclosed “Liquefaction Evaluation” is based on results obtained from Boring B3. 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. Fines content, as defined by percentage passing the #200 sieve, were utilized for the fines correction factor in computing the corrected blow counts of selected soil layers. In addition, Atterberg Limit tests were performed for selected samples. The results of these tests are presented on the enclosed E-Plate and F-Plate. Based on CGS Special Publication 117A (CDMG, 2008), the vast majority of liquefaction hazards are associated with sandy soils and silty soils of low plasticity.

The procedure presented in the SP117A guidelines was followed in analyzing the liquefaction potential of the subject site. The SP117A guidelines were developed based on a paper titled, "Assessment of the Liquefaction Susceptibility of Fine-Grained Soils", by Bray and Sancio (2006). According to the SP117A, soils having a Plastic Index greater than 18 exhibit clay-like behavior, and the liquefaction potential of these soils are considered to be low. Therefore, where the results of Atterberg Limits testing showed a Plastic Index greater than 18, the soils would be considered non-liquefiable, and the analysis of these soil layers was turned off in the liquefaction susceptibility column.

Based on the adjusted blow count data, results of laboratory testing, and the calculated factor of safety against the occurrence of liquefaction, it is the opinion of this firm that the potential for liquefaction at the site is considered to be remote.

Dynamic Dry Settlement

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

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

Tsunamis, Seiches and Flooding

Tsunamis are large ocean waves generated by sudden water displacement caused by a submarine earthquake, landslide, or volcanic eruption. This site is high enough and far enough from the ocean to preclude being prone to hazards of a tsunami.

Seiches are oscillations generated in enclosed bodies of water which can be caused by ground shaking associated with an earthquake. No major water-retaining structures are located immediately up gradient from the project site. Therefore, the risk of flooding from a seismically-induced seiche is considered to be remote.

Review of the County of Los Angeles Flood and Inundation Hazards Map (Leighton, 1990), indicates the site does not lie within an inundation boundary due to a seiche or a breached upgradient reservoir.

Landsliding

The probability of seismically-induced landslides occurring on the site is considered to be negligible due to the general lack of substantive elevation difference across or adjacent to the site. Therefore, potential impacts related to landsliding would be less than significant.

CONCLUSIONS AND RECOMMENDATIONS

Based upon the exploration, laboratory testing, and research, it is the 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.

The site is not located in an earthquake fault zone or within a Methane or Methane Buffer zone. Based on the site-specific liquefaction analysis, it is the opinion of this firm that the potential for liquefaction at the site is considered to be remote.

The site is underlain by fill materials and native alluvial soils. The fill was observed to extend to a maximum depth of 3 feet below the ground surface during exploration. The existing fill materials are not suitable for support of the proposed foundations, floor slabs or additional fill. The proposed development will be constructed over 1 subterranean level. Therefore, it is anticipated that excavations of the proposed subterranean level will remove the fill materials in the building area and expose the underlying native soils. The proposed structure may be supported on conventional foundations bearing in the underlying native soils found at the level of the proposed excavations.

Groundwater was encountered during exploration in Borings B2 and B3 at depths of 27.5 and 27.8 feet below the ground surface, corresponding to elevations 240.5 and 242.2 feet above MSL, respectively. Groundwater was not encountered in Boring B1, which was excavated to a maximum depth of 40 feet below the existing site grade. Review of California Geological Survey Seismic Hazard Evaluation Report of the Beverly Hills 7½-minute Quadrangle, (CDMG, 1998, Revised 2005), indicates that the historically highest groundwater level at the site is on the order of 20 feet below ground surface. The proposed structure is expected to extend 12 feet below grade. Therefore, the historic high groundwater level would be 8 feet below the base of the structure and the observed groundwater would be 15 feet below the base of the proposed structure. Since the proposed structure will remain above the historically highest groundwater level, the proposed slab-on-grade will not need to be designed for hydrostatic pressure.

It is anticipated that excavations on the order of 15 feet in depth will be required for the proposed subterranean levels and foundation elements. Excavation of the proposed subterranean levels will require shoring measures to provide a stable working area due to the proposed depth, the nature of the onsite soils, and the proximity of adjacent structures and property lines. Soldier piles and lagging should be anticipated for shoring.

Foundations for small outlying structures, such as property line walls, planters, trash enclosures, and canopies, which will not be tied-in to the proposed apartment structure, may be supported on compacted fill and/or the underlying native soils.

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 borings on the site as indicated and should in no way be construed to reflect any variations which may occur between these borings or which may result from changes in subsurface conditions. Any changes in the design or location of any structure, as outlined in this report, should be reviewed by this office. The recommendations contained herein should not be considered valid until reviewed and modified or reaffirmed subsequent to such review.

SEISMIC DESIGN CONSIDERATIONS

California Building Code Seismic Parameters

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	2019
ASCE Design Standard	7-16
Risk Category	II
Site Class	D
Mapped Spectral Acceleration at Short Periods (S_S)	1.996g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	1.996g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.331g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.714g
Site Coefficient (F_v)	1.7*
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	1.214g*
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.809g*

* *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 within the upper 5 feet are in the low expansion range. The Expansion Index was found to be 35 for bulk sample remolded to 90 percent of the laboratory maximum density. The onsite geologic materials within the basement level are in the very low expansion range. The Expansion Index was found to range between 1 to 17 for samples tested. Recommended reinforcing is noted in the "Foundation Design" and "Slabs on Grade" sections of this report.

WATER-SOLUBLE SULFATES

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

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

The water-soluble sulfate content of the onsite geologic materials was tested by California Test 417. The water-soluble sulfate content was determined to be less than 0.2% 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.

The design of the concrete mix is not within the area of expertise of the geotechnical engineer. It is recommended that a competent engineer familiar with concrete mix design should develop the recommendations for this project based on the tested severe sulfate exposure indicated above.

CITY OF LOS ANGELES METHANE ZONE

Based on review of the Navigate LA Website, developed by the City of Los Angeles, Bureau of Engineering, Department of Public Works, the subject site is not located within the limits of a City of Los Angeles Methane Zone or Methane Buffer Zone. A copy of the City of Los Angeles Methane and Methane Buffer Zone Map is attached.

GRADING GUIDELINES

The following guidelines are provided for any miscellaneous compaction that may be required, such as retaining wall or trench backfill, or subgrade preparation.

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.

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. Comparative compaction is defined, for purposes of these guidelines, as the ratio of the in-place density to the maximum density as determined by applicable ASTM testing.

All fill should be mechanically compacted in layers not more than 8 inches thick. The materials placed should be moisture conditions to within 3 percent of the optimum moisture content of the particular material placed. All fill shall be compacted to at least 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum laboratory density for the materials used. The maximum density shall be determined by the laboratory operated by Geotechnologies, Inc. in general accordance with the most recent revision of ASTM D 1557.

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. Materials larger than 6 inches in maximum dimension shall not be used in the fill.

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 general accordance with the most recent revision of ASTM D 1557.

Wet Soils

At the time of exploration, the soils which will be exposed at the bottom of the excavation were locally above optimum moisture content. It is anticipated that the excavated materials exposed at the bottom of excavated plane may require drying and aeration.

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 1-inch gravel and/or crushed concrete 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.

Shrinkage

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

Weather Related Grading Considerations

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

Temporary drainage devices should be installed to collect and transfer excess water to the street in non-erosive drainage devices. 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.

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.

FOUNDATION DESIGN

It is recommended that the proposed structure be supported on a system of conventional foundations bearing in the underlying dense Alluvium.

Conventional Foundations

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, 18 inches in depth below the lowest adjacent grade and 18 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, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material.

The bearing capacity increase for each additional foot of width is 150 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 6,000 pounds per square foot.

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.

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.

Miscellaneous Foundations

Conventional foundations for structures such as privacy walls or trash enclosures which will not be rigidly connected to the proposed residential structure may bear in native soils. Continuous footings may be designed for a bearing capacity of 1,500 pounds per square foot, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material. No bearing capacity increases are recommended.

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 0.28 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 300 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.

Foundation Settlement

Settlement of the foundation system is expected to occur on initial application of loading. The maximum settlement is expected to be ¾-inch and occur below the heaviest loaded columns. Differential settlement is not expected to exceed ½-inch across a distance of 30 feet.

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 12 feet in height are anticipated for the proposed subterranean parking level. As a precautionary measure, recommendations for retaining walls up to 15 feet in height are provided below. Retaining walls may be designed as indicated below, depending on whether the walls will be restrained or cantilevered. Retaining wall foundations may be designed in accordance with the provisions of the "Foundation Design" section of this report.

Additional pressure should be added for a surcharge condition due to vehicular traffic or adjacent structures. For traffic surcharge, the upper ten feet of the 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 surcharge behind the walls due to normal street traffic. If the traffic is kept back at least ten feet from the retaining walls, the traffic surcharge may be neglected.

Restrained Retaining Walls

Restrained subterranean retaining walls up to 15 feet in height and supporting a level back slope may be designed to resist a triangular distribution of earth pressure. It is recommended the walls be designed to resist the greater of the at-rest pressure, or the active pressure plus the seismic pressure, as discussed in the "Dynamic (Seismic) Earth Pressure" section below.

RESTRAINED BASEMENT WALLS		
	AT-REST EARTH PRESSURE	**ACTIVE EARTH PRESSURE *(To be Combined with Dynamic Seismic Earth Pressure)**
Height of Wall (Feet)	Triangular Distribution of Pressure (Pounds per Cubic Foot)	Triangular Distribution of Pressure (Pounds per Cubic Foot)
Up to 15 feet	74	30*

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

Dynamic (Seismic) Earth Pressure

Retaining walls exceeding 6 feet in height shall be designed to resist the additional earth pressure caused by seismic ground shaking. A triangular pressure distribution should be utilized for the additional seismic loads, with an equivalent fluid pressure of 26 pounds per cubic foot. When using the code based loading equations, the seismic earth pressure should be combined with the lateral active earth pressure for analyses of restrained basement walls under seismic loading condition. The dynamic earth pressure may be omitted where the retaining wall is 6 feet in height or less.

Cantilever Retaining Walls

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

HEIGHT OF WALL (feet)	EQUIVALENT FLUID PRESSURE (pounds per cubic foot)
Up to 15	30

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

Surcharge from Adjacent Structures

As indicated herein, additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures for retaining walls and shoring design. 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.

Waterproofing

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

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

Retaining Wall Drainage

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

A compacted fill blanket or other seal shall be provided at the surface. Retaining walls may be backfilled with gravel adjacent to the wall to within 2 feet of the ground surface. The onsite earth materials are acceptable for use as retaining wall backfill as long as they are compacted to a minimum of 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum density as determined by the latest revision of ASTM D 1557.

As an alternative, 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 rock, wrapped in filter fabric. A collector is placed within the gravel which directs collected waters through the wall to a sump or standard pipe and gravel system constructed under the slab. This method should be approved by the retaining wall designer prior to implementation.

Certain types of subdrain pipe are not acceptable to the various municipal agencies, it is recommended that prior to purchasing subdrainage pipe, the type and brand is cleared with the proper municipal agencies. Subdrainage pipes should outlet to an acceptable location.

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

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) of the maximum density in general accordance with the most recent revision of ASTM D 1557 method of compaction. Flooding should not be permitted. Compaction within 5 feet, measured horizontally, behind a retaining structure should be achieved by use of light weight, hand operated compaction equipment.

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

Sump Pump Design

The purpose of the recommended retaining wall backdrainage system is to relieve hydrostatic pressure. Groundwater was encountered at depths ranging from 27.5 to 27.8 feet below the existing ground surface. Therefore, it is anticipated that the only water which could affect the proposed retaining walls would be irrigation waters and precipitation. Additionally, the proposed site grading is such that all drainage is directed to the street and the structure has been designed with adequate non-erosive drainage devices.

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

TEMPORARY EXCAVATIONS

It is anticipated that excavations on the order of 15 feet in vertical height will be required for the proposed subterranean levels and foundation elements. As a precautionary measure, recommendations for excavations up to 20 feet in height are provided below. 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. Excavations which will be surcharged by adjacent traffic or structures should be shored.

Where sufficient space is available, temporary unsurcharged embankments could be sloped back without shoring. Excavations over 5 feet in height may be excavated at a uniform 1:1 (h:v) slope gradient in its entirety to a maximum height of 20 feet. A uniform sloped excavation does not have a vertical component.

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

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 geologic materials. For design purposes, an allowable passive value for the geologic materials below the bottom plane of excavation may be assumed to be 500 pounds per square foot per foot. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed geologic materials.

Casing may be required should caving be experienced in the granular geologic materials. 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.

The frictional resistance between the soldier piles and retained geologic material may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.28 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 600 pounds per square foot. The minimum depth of embedment for shoring piles is 5 feet below the bottom of the footing excavation or 7 feet below the bottom of excavated plane whichever is deeper.

Groundwater was encountered during exploration at depths between 27.5 to 27.8 feet below the existing site grade. Therefore, it is anticipated that the proposed piles in excess of 27.5 feet in depth may encounter water. Piles placed below the water level 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 10 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 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 homogenous. The tip of the tremie tube shall always be kept above 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 p.s.i. of 1,000 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 is shall also be the minimum for a reasonable consistency for placing when water is present.

Lagging

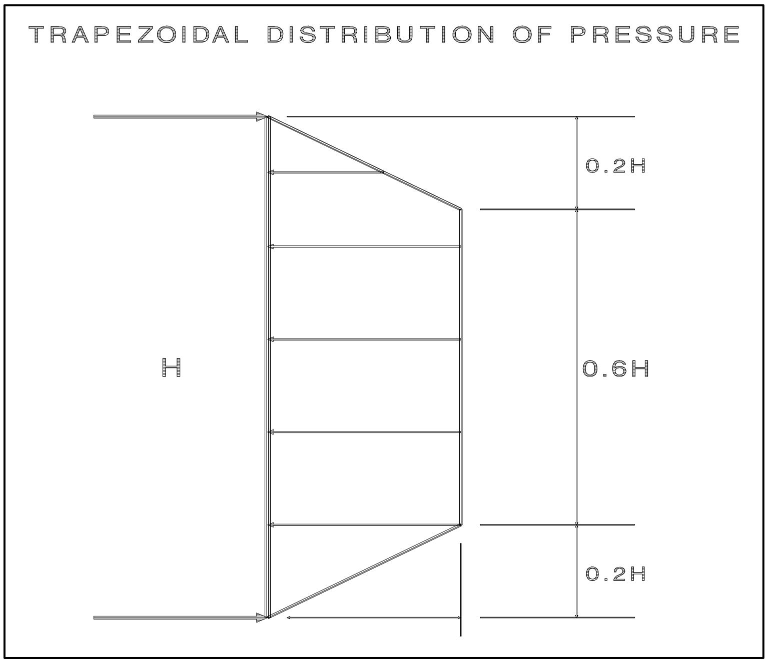
Soldier piles and anchors should be designed for the full anticipated pressures. Due to arching in the geologic materials, the pressure on the lagging will be less. It is recommended that the lagging should be designed for the full design pressure but may be limited to a maximum of 400 pounds per square 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

It is anticipated that excavations on the order of 15 feet in depth will be required for the proposed subterranean levels and foundation elements. As a precautionary measure, recommendations for shoring walls up to 20 feet in height are provided below. A triangular distribution of lateral earth pressure should be utilized for the design of a cantilevered shoring system. A trapezoidal distribution of lateral earth pressure would be appropriate where shoring is to be restrained at the top by bracing or tie backs. The design of trapezoidal distribution of pressure is shown in the diagram below. Equivalent fluid pressures for the design of cantilever and restrained shoring are presented in the following table:

LATERAL SHORING WALL PRESSURES		
Height of Shoring Wall (feet)	Cantilever Shoring System Equivalent Fluid Pressure (pcf) Triangular Distribution of Pressure	Restrained Shoring System Lateral Earth Pressure (psf)* Trapezoidal Distribution of Pressure
Up to 20	28	18H

*Where H is the height of the shoring in feet.



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. Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination.

The upper ten feet of the 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 surcharge behind the walls due to normal street traffic. If the traffic is kept back at least ten feet from the retaining walls, the traffic surcharge may be neglected.

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 500 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 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.

Tie-back Anchor Testing

At the 10 percent of the anchors should be selected for “Quick”, 200 percent tests. It is recommended that at least three of these 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.

Deflection

It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized that some deflection will occur. The City of Los Angeles Department of Building and Safety requires limiting shoring deflection to ½ inch at the top of the shored embankment where a structure is within a 1:1 (h:v) 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 (h:v) 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.

Raker Brace Foundations

An allowable bearing pressure of 5,000 pounds per square foot may be used for the design of raker foundations bearing in native alluvial soils. The existing uncertified fill materials shall not be used for support of raker foundations. The bearing pressure is based on a raker foundation that is a minimum of 4 feet in width and length as well as 2 feet in depth. The base of the raker foundations should be horizontal. Care should be employed in the position of raker foundations so that they do not interfere with the foundations for the proposed structure.

Monitoring

Because of the depth of the excavation, some mean 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 existing structures on the adjacent properties should be made prior to, and during construction to record any movement or change due to vibration 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 slabs-on-grade above the historically highest groundwater level should be a minimum of 4 inches in thickness, and should be reinforced with a minimum of #4 steel bars on 16-inch centers each way. All 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, and should be reinforced with a minimum of #3 steel bars on 24-inch centers each way. 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, it is recommended that a qualified consultant be engaged to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. The qualified consultant should provide recommendations for mitigation of potential adverse impacts of moisture vapor transmission on various components of the structure.

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

All concrete slabs-on-grade should be supported on vapor retarder, except for exposed garage slabs without any floor finishes. The design of the slab and the installation of the vapor retarder should comply with the most recent revisions of ASTM E 1643 and ASTM E 1745. The vapor retarder should comply with ASTM E 1745 Class A requirements.

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

Concrete Crack Control

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

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

Complete removal of the existing fill soils beneath outdoor flatwork such as walkways or patio areas, is not required, however, due to the rigid nature of concrete, some cracking, a shorter design life and increased maintenance costs should be anticipated. In order to provide uniform support beneath the flatwork it is recommended that a minimum of 12 inches of the exposed subgrade beneath the flatwork be scarified and recompacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) relative compaction.

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 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) 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 shorter design life and increased maintenance costs. The following pavement sections are recommended:

Service	**Asphalt Pavement Thickness Inches**	**Base Course Inches**
Passenger Cars (TI=4)	3	5
Moderate Truck (TI=6)	4	7

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 consist of Crushed Aggregate Base which conform with Section 200.2.2 or 200.2.4 of the most recent edition of “Standard Specifications for Public Works Construction”, (Green Book).

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

The performance of pavement in highly dependent upon providing positive surface drainage away from the edges. Ponding 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 proposed building is expected to be constructed over 1 subterranean level extending on the order of 12 feet below the exiting grade, when considering the proposed foundation system. In addition, groundwater was encountered at depths ranging from 27.5 to 27.8 feet below the existing site grade. Due to the depth of the proposed subterranean structure, and the presence of groundwater, it is the opinion of this firm that stormwater infiltration at the site is not feasible for the planned development.

Where infiltration of stormwater into the subgrade is not advisable, 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 geotechnical report by the Building Official is obtained in writing. Significant changes in the geotechnical recommendations may result during the building department review process.

It is recommended that the geotechnical aspects of the project be reviewed by this firm during the design process. This review provides assistance to the design team by providing specific recommendations for particular cases, as well as review of the proposed construction to evaluate whether the intent of the recommendations presented herein are satisfied.

CONSTRUCTION MONITORING

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

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

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

EXCAVATION CHARACTERISTICS

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

CLOSURE AND LIMITATIONS

The purpose of this report is to aid in the design and completion of the described project. Implementation of the advice presented in this report is intended to reduce certain risks associated with construction projects. The professional opinions and geotechnical advice contained in this report are sought because of special skill in engineering and geology and were prepared in accordance with generally accepted geotechnical engineering practice.

Geotechnologies, Inc. has a duty to exercise the ordinary skill and competence of members of the engineering profession. Those who hire Geotechnologies, Inc. are not justified in expecting infallibility, but can expect reasonable professional care and competence.

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

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

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

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

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

EXCLUSIONS

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

GEOTECHNICAL TESTING

Classification and Sampling

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

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

Moisture and Density Relationships

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

Direct Shear Testing

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

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

Consolidation Testing

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

Expansion Index Testing

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

Grain Size Distribution

These tests cover the quantitative determination of the distribution of particle sizes in soils. Sieve analysis is used to determine the grain size distribution of the soil larger than the Number 200 sieve.

General accordance with the most recent revision of ASTM D 422 is used to determine particle sizes smaller than the Number 200 sieve. A hydrometer is used to determine the distribution of particles sizes by a sedimentation process. The grain size distributions are plotted on the E-Plate 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 for solid to plastic are known as the Atterberg Limits. The transitions are called the plastic and liquid limit. The difference between the liquid and the 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-Plate presented in the Appendix 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.

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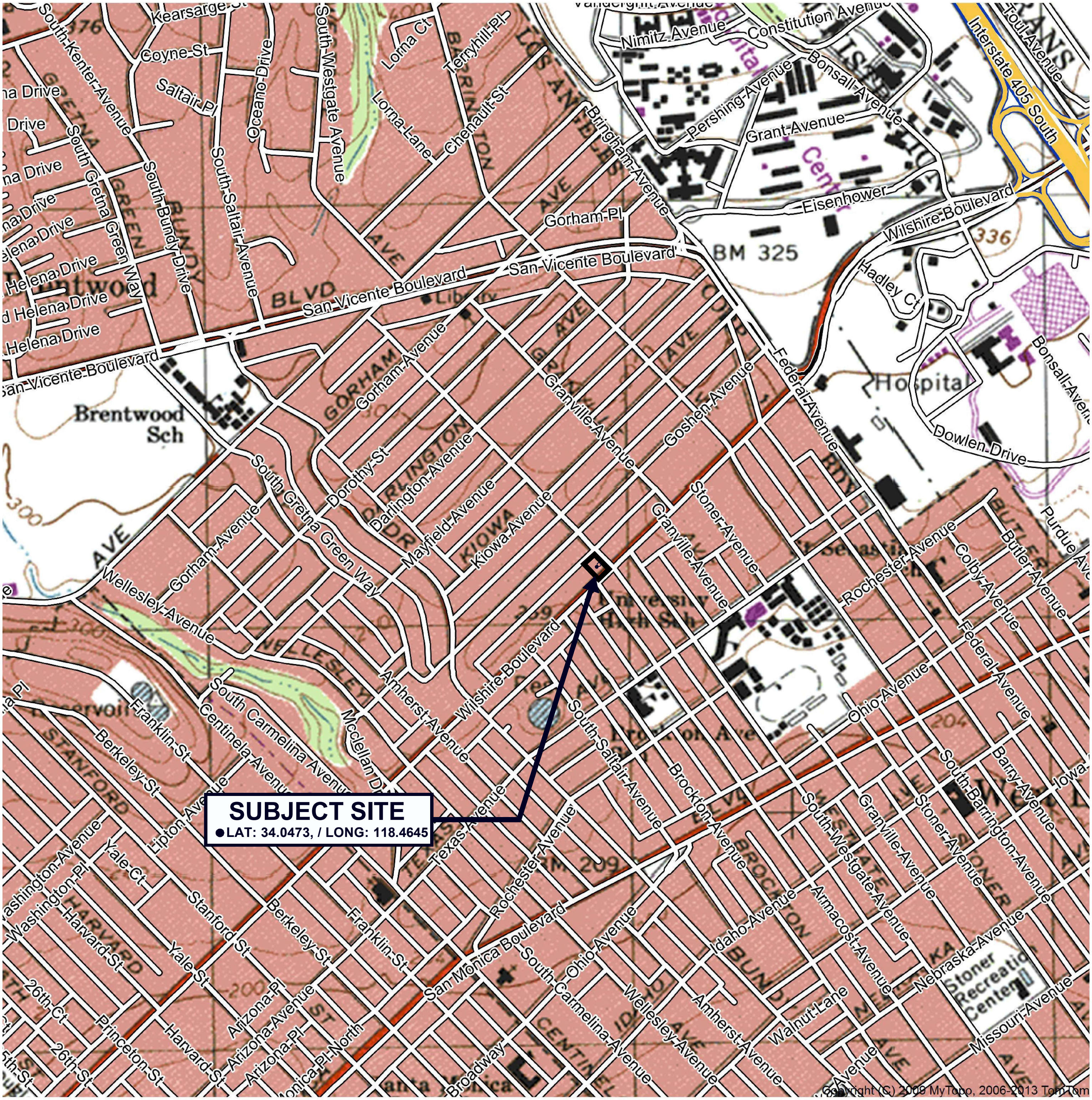


SCALE 1:12000

0.0 0.1 0.2 0.3 0.4 0.5 0.6
MILES

0 1000 2000 3000
FEET

N

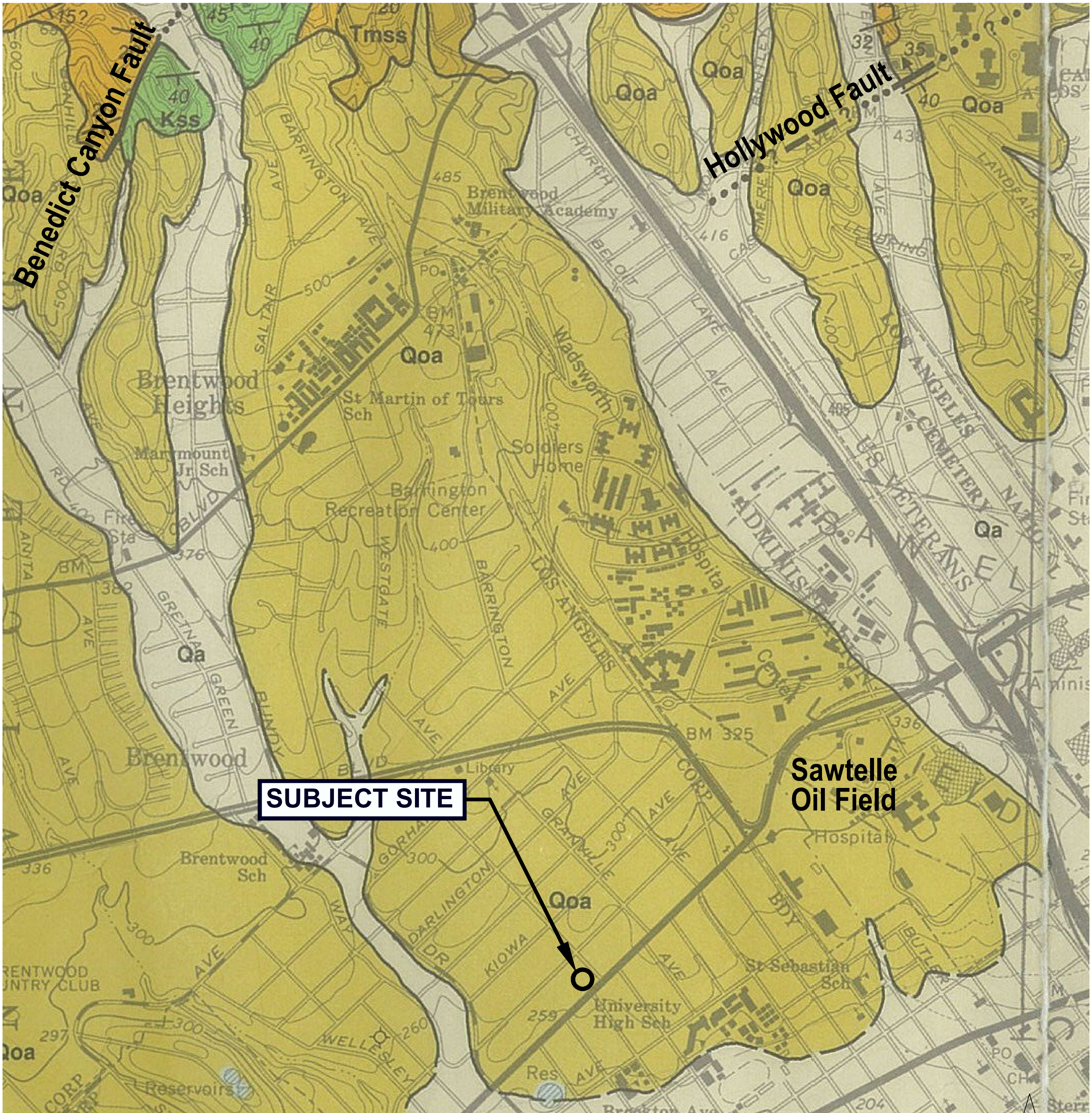
REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, BEVERLY HILLS, CA QUADRANGLE

VICINITY MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

RADHA HOTELS USA, LLC
11905 WILSHIRE BLVD., LOS ANGELES

FILE NO. 22157



SCALE IN FEET

0 500 1000 2000

N

LEGEND

Qa: Surficial Sediments: alluvium - gravel, sand and clay

Qoa: Older Surficial Sediments: older alluvium of gray to light brown pebble-gravel, sand and silt-clay

Tmss: Monterey Formation:tan to light gray semi-friable bedded sandstone

Tmu: Monterey Formation - similar to Tmss

Kss: Unnamed Strata - tan moderately hard sandstone

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 BEVERLY HILLS & VAN NUYS (SOUTH HALF) QUADRANGLES (#DF-31)

LOCAL GEOLOGIC MAP - DIBBLEE

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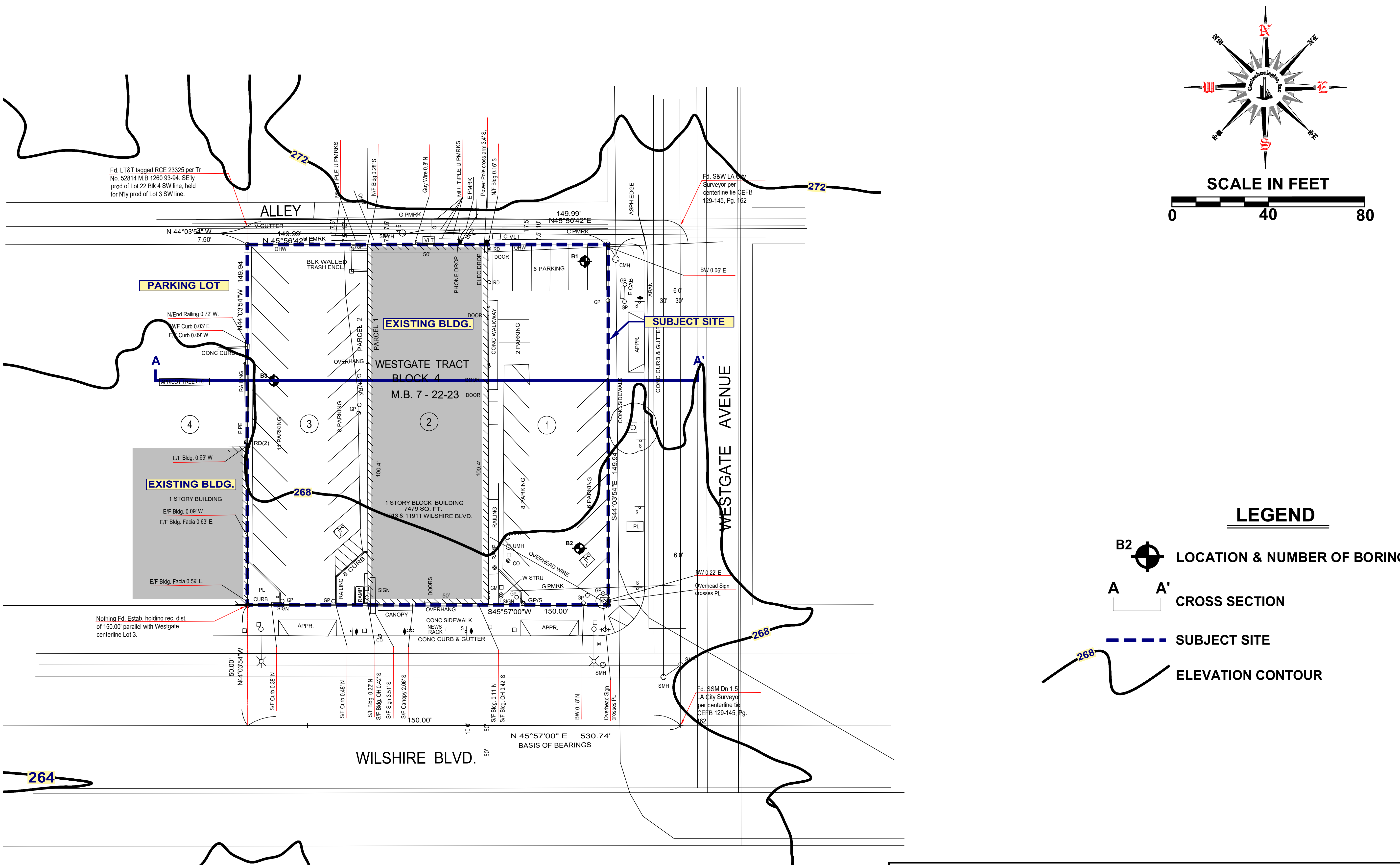
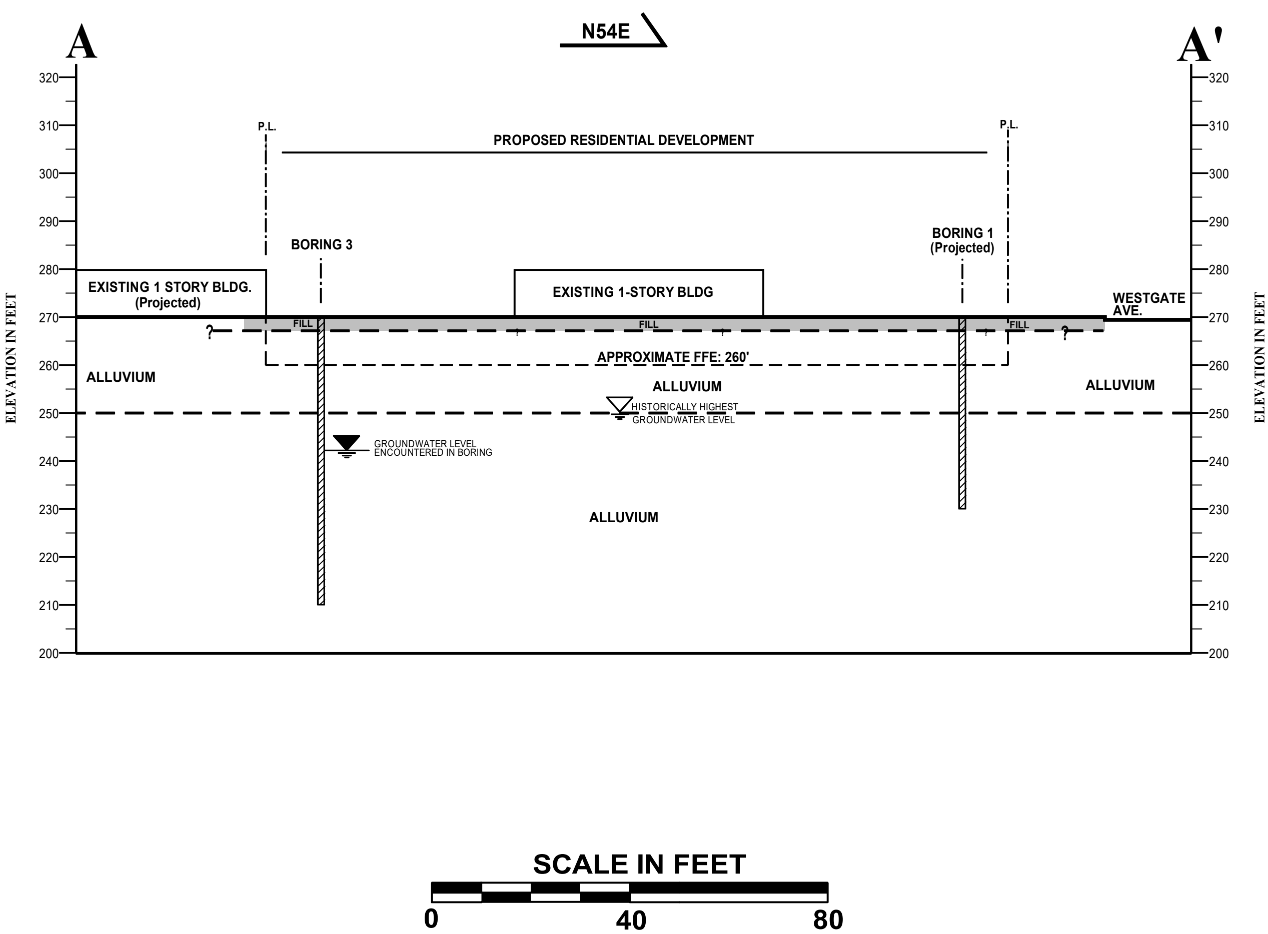
FILE NO. 22157



SURVEYORS MAP PROVIDED BY PSOMAS
DATED JUNE 22, 2021

PLOT PLAN	
Geotechnologies, Inc. *Consulting Geotechnical Engineers*	**RADHA HOTELS USA, LLC** 11905 WILSHIRE BLVD., LOS ANGELES
FILE No. 22157	DRAWN BY: AL
	DATE: August 2021



CROSS SECTION A-A'

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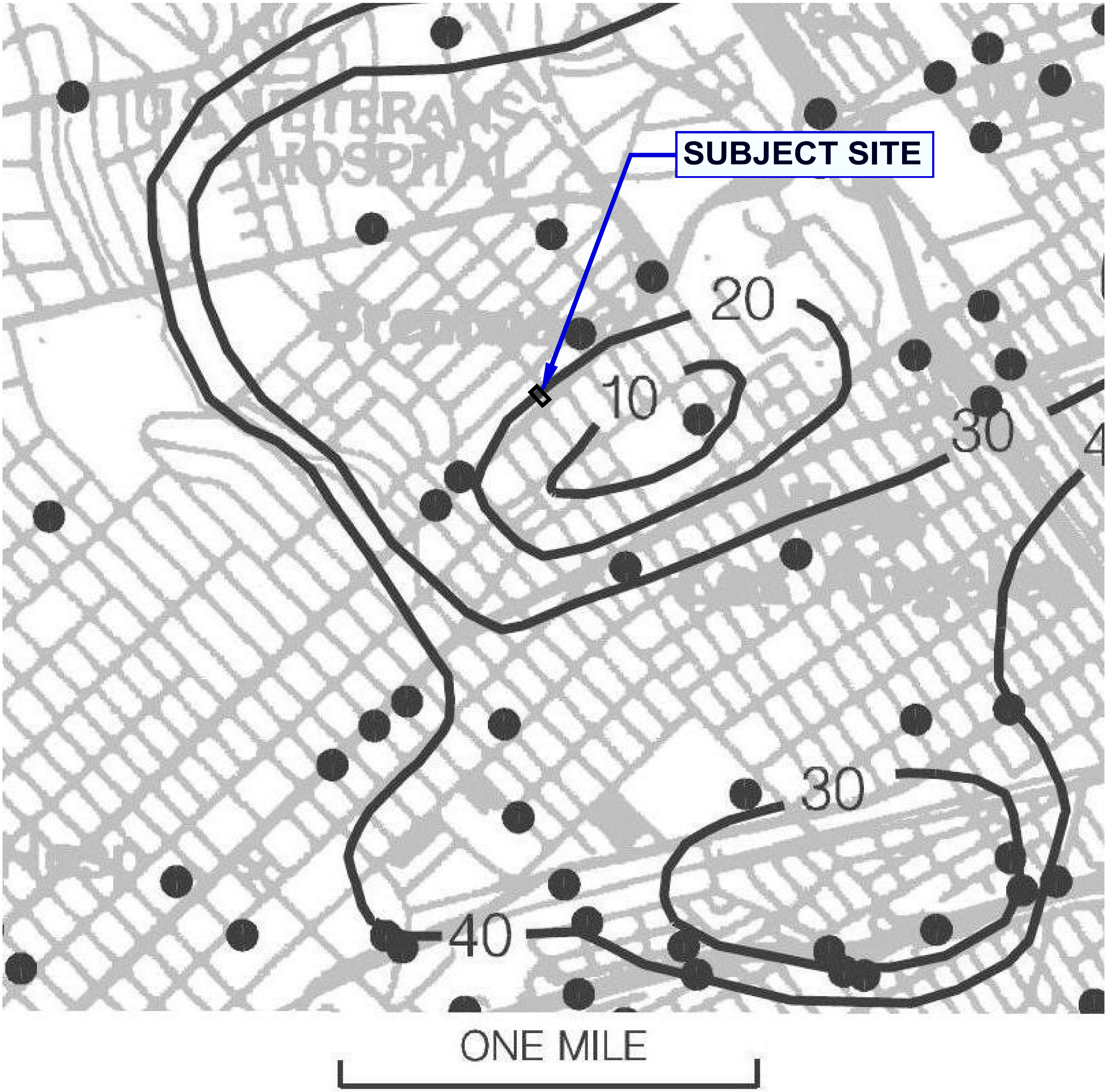
FILE No. 22157 | **DRAWN BY: YD**

DATE: August 2021



20 Depth to groundwater in feet

REFERENCE: CDMG, SEISMIC HAZARD ZONE REPORT, 023
BEVERLY HILLS 7.5 - MINUTE QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA (1998, REVISED 2005)

HISTORICALLY HIGHEST GROUNDWATER LEVELS

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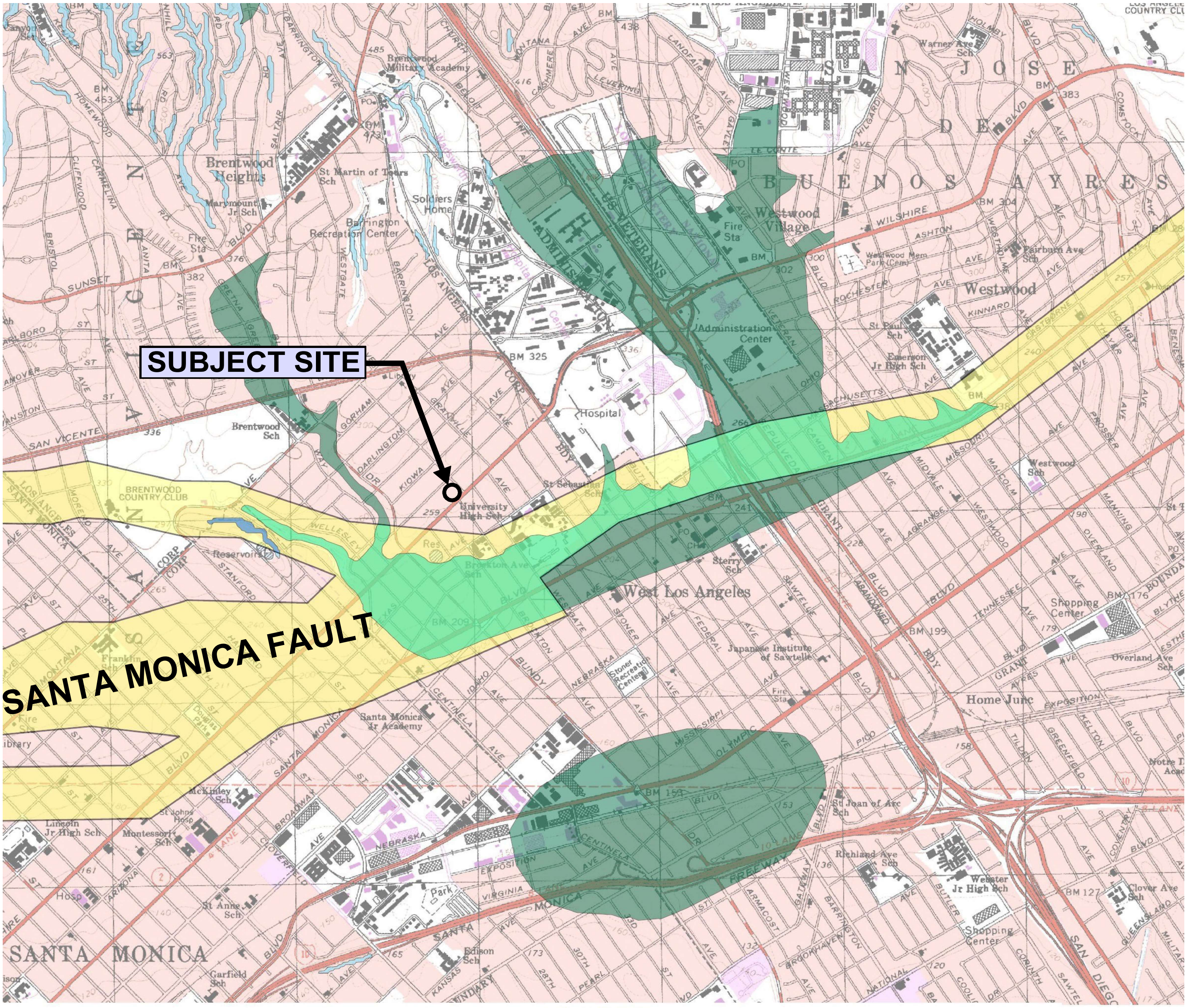
FILE No. 22157



Scale 1: 24,000

1 0.5 0 1 2 Miles

1,000 500 0 1,000 2,000 3,000 4,000 5,000 6,000 7,000 8,000 9,000 10,000 Feet

1 0.5 0 1 2 Kilometers

1,000 500 0 1,000 2,000 Meters

Liquefaction Zone

Earthquake Fault Zones
Alquist-Priolo Earthquake Fault Zone

REFERENCE: EARTHQUAKE FAULT ZONES, BEVERLY HILLS QUADRANGLE, CALIFORNIA GEOLOGICAL SURVEY, JANUARY 2018

SEISMIC HAZARD ZONE MAP

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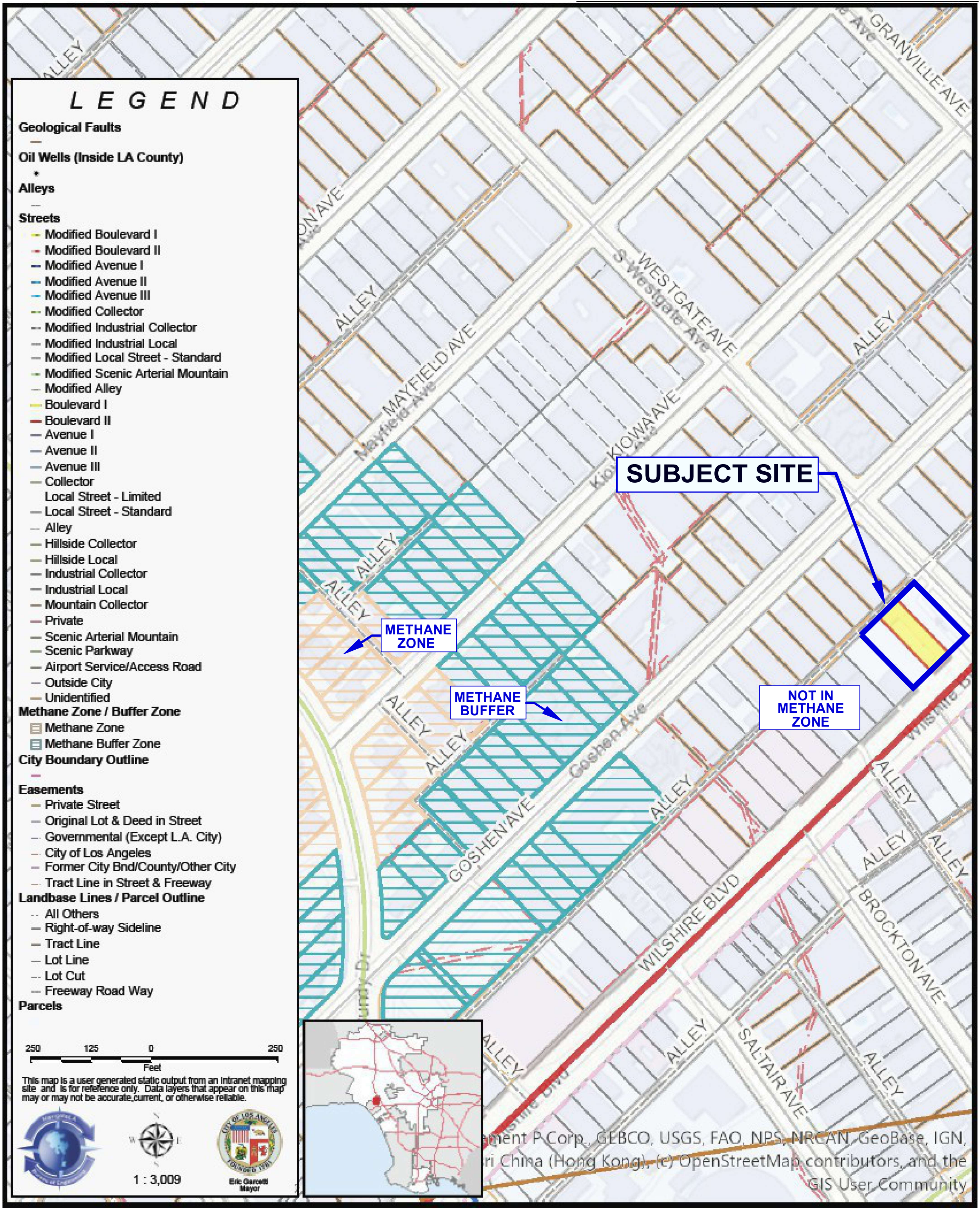
FILE NO. 22157



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

METHANE ZONE RISK MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

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11905 WILSHIRE BLVD, LOS ANGELES

FILE NO. 22157

BORING LOG NUMBER 1

Radha Hotels USA, LLC. **Date: 07/01/21** **Elevation: 270'***

File No. 22157 **Method: 8-Inch Diameter Hollowstem Auger**

ln

***Reference: City of Los Angeles Navigate LA Website 2006 Elevation Contours**

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt Parking Lot
				0 --		4-Inch Asphalt, No Base
				-		Fill: Sandy Silt, dark brown, moist, stiff
				1 --		
				-		
				2 --		
2.5	48/6"	12.0	101.8	-		
	50/5"			3 --		
				-	ML	ALLUVIUM: Sandy Silt, dark and yellowish brown, moist, very stiff
				4 --		
				-		
5	48/6"	10.9	117.6	5 --		
	50/4"			-	SM/ML	Silty Sand to Sandy Silt, dark and yellowish brown, moist, very dense, very stiff, fine grained, minor slate fragments
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	44/6"	13.5	116.6	10 --		
	50/3"			-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	45/6"	16.4	116.5	15 --		
	50/3"			-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	100/9"	7.3	123.0	20 --		
				-	SM/SP	Silty Sand to Sand with slate fragments, dark and yellowish brown, moist, very dense, fine to medium grained
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	40/6"	28.0	98.9	25 --		
	50/5"			-	SM/ML	Silty Sand to Sandy Silt, dark brown, very moist, medium dense, stiff, fine grained

BORING LOG NUMBER 1

Radha Hotels USA, LLC.

File No. 22157

ln

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26 --		
				27 --		
				28 --		
				29 --		
30	80	19.7	109.6	30 --		
				31 --	SM/ML	Silty Sand to Sandy silt, dark brown, moist, medium dense, stiff, fine grained
				32 --		
				33 --		
				34 --		
35	88	19.6	109.2	35 --		
				36 --	ML/CL	Clayey Silt to Silty Clay, dark and grayish brown, moist, stiff
				37 --		
				38 --		
				39 --		
40	45/6" 50/4"	17.3	112.4	40 --		
				41 --		Total Depth 40 Feet No Water Fill To 3 Feet
				42 --		
				43 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				44 --		
				45 --		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				46 --		
				47 --		
				48 --		
				49 --		
				50 --		

BORING LOG NUMBER 2

Radha Hotels USA, LLC. **Date: 06/30/21** **Elevation: 268***

File No. 22157 **Method: 8-Inch Diameter Hollowstem Auger**

ln ***Reference: City of Los Angeles Navigate LA Website 2006 Elevation Contours**

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt Parking Lot
				0 --		5-Inch Asphalt, No Base
				-		Fill: Silty Sand to Sandy Silt, dark brown, moist, medium dense, stiff
				1 --		
				-		
				2 --		
2.5	36	17.4	108.9	-		
				3 --		
				-	SM/ML	ALLUVIUM: Silty Sand to Sandy Silt dark brown, moist, medium dense, stiff, fine grained
				4 --		
				-		
5	76	12.2	118.6	5 --		
				-	ML	Sandy Silt, dark brown, stiff
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	90	14.9	118.8	10 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark brown, moist, medium dense, stiff, fine grained
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	88	15.4	117.0	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	58	25.6	102.3	20 --		
				-	ML/CL	Clayey Silt to Silty Clay, dark brown, moist, stiff
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	42	26.0	99.2	25 --		
				-	SM/ML	Silty Sand to Sandy Silt, gray and dark brown, very moist, stiff, fine grained

BORING LOG NUMBER 2

Radha Hotels USA, LLC.

File No. 22157

ln

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
30	88	16.9	111.1	30 --		
				-	ML	Sandy Silt, dark brown, moist, very stiff
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
35	42/6"	18.3	114.1	35 --		
	50/5"			-		
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		Sandy Silt, dark gray, slightly moist, very stiff
40	45/6"	20.1	104.8	40 --		
	50/4"			-		Total Depth 40 Feet
				41 --		Water at 27.5 Feet four hours after drilling
				-		Fill to 3
				42 --		
				-		NOTE: The stratification lines represent the approximate
				43 --		boundary between earth types; the transition may be gradual.
				-		
				44 --		Used 8-inch diameter Hollow-Stem Auger
				-		140-lb. Automatic Hammer, 30-inch drop
				45 --		Modified California Sampler used unless otherwise noted
				-		
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
				50 --		
				-		

BORING LOG NUMBER 3

Radha Hotels USA, LLC. **Date: 06/30/21** **Elevation: 270'***

File No. 22157 **Method: 8-Inch Diameter Hollowstem Auger**

ln **Reference: City of Los Angeles Navigate LA Website 2006 Elevation Contours**

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt Parking Lot
				0 --		3.5 Inch Asphalt Light Base
				-		Fill: Sandy Silt, dark brown, moist, stiff
				1 --		
				-		
				2 --		
2.5	38/6" 50/3"	12.6	118.8	-		
				3 --		
				-	SM/ML	ALLUVIUM: Silty Sand to Sandy Silt, dark brown, moist, very dense, fine grained, stiff, minimum slate fragments
				4 --		
				-		
5	83	11.7	SPT	5 --		
				-	SM	Silty Sand with Slate fragments, dark brown and gray, moist, medium dense, fine grained
				6 --		
				-		
				7 --		
7.5	100/10"	12.1	121.0	-		Silty Sand, dark brown and slightly moist, very dense, fine grained, some slate fragments
				8 --		
				-		
				9 --		
				-		
10	52	14.7	SPT	10 --		
				-		
				11 --		
				-		
				12 --		
12.5	89	18.4	114.1	-		
				13 --	ML	Sandy Silt, dark brown, moist, very stiff
				-		
				14 --		
				-		
15	66	16.9	SPT	15 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark brown, moist, dense, stiff, fine grained
				16 --		
				-		
				17 --		
17.5	100/9"	6.0	106.0	-		
				18 --	SP	Sand dark brown, moist, very dense, fine to medium grained
				-		
				19 --		
				-		
20	82	6.0	SPT	20 --		
				-	SM/SP	Silt Sand to Sand with slate fragments, dark brown and gray, moist, very dense, fine to medium grained
				21 --		
				-		
				22 --		
22.5	100/9"	5.1	110.7	-		
				23 --	SP/SW	Sand to Cobbley Sand, dark and grayish brown, moist, very dense, fine to coarse grained
				-		
				24 --		
				-		
25	79	8.1	SPT	25 --		
				-	SP	Sand, dark and grayish brown, slightly moist, very dense, fine to medium grained

BORING LOG NUMBER 3

Radha Hotels USA, LLC.

File No. 22157

ln

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				- 26 -- - 27 -- -	SP	Sand, dark and grayish brown, slightly moist, very dense, fine to medium grained
27.5	100/9"	13.9	117.6			
				28 -- - 29 -- -	SM/SP	Silty Sand to Sand, very moist, very dense, fine to medium grained
30	80	18.0	SPT	30 --		
				- 31 -- - 32 -- -	ML	Sandy to Clayey Silt, dark brown, moist, stiff
32.5	69	20.5	108.2			
				33 -- - 34 -- -	SM/ML	Silty Sand to Sandy Silt, dark brown, moist, medium dense to dense, stiff, fine grained
35	21	23.6	SPT	35 --		
				- 36 -- - 37 -- -	ML/CL	Clayey Silt to Silty Clay, dark yellow, moist, stiff
37.5	72	19.1	112.1			
				38 -- - 39 -- -	ML	Sandy to Clayey Silt, dark brown, moist, stiff
40	40	20.8	SPT	40 -- - 41 -- - 42 -- -		
42.5	98	18.4	113.8			
				43 -- - 44 -- -	ML/CL	Clayey Silt to Silty Clay, dark and grayish brown, moist, very stiff
45	79	12.2	SPT	45 --		
				- 46 -- - 47 -- -	SM/ML	Silty Sand to Sandy Silt, dark brown and gray, wet, very dense, stiff, fine to medium grained, minor slate fragments
47.5	48/6" 50/4"	24.0	105.2			
				48 -- - 49 -- -	ML	Sandy Silt, dark brown and gray, moist, stiff
50	45/6" 50/4"	17.6	SPT	50 -- -		

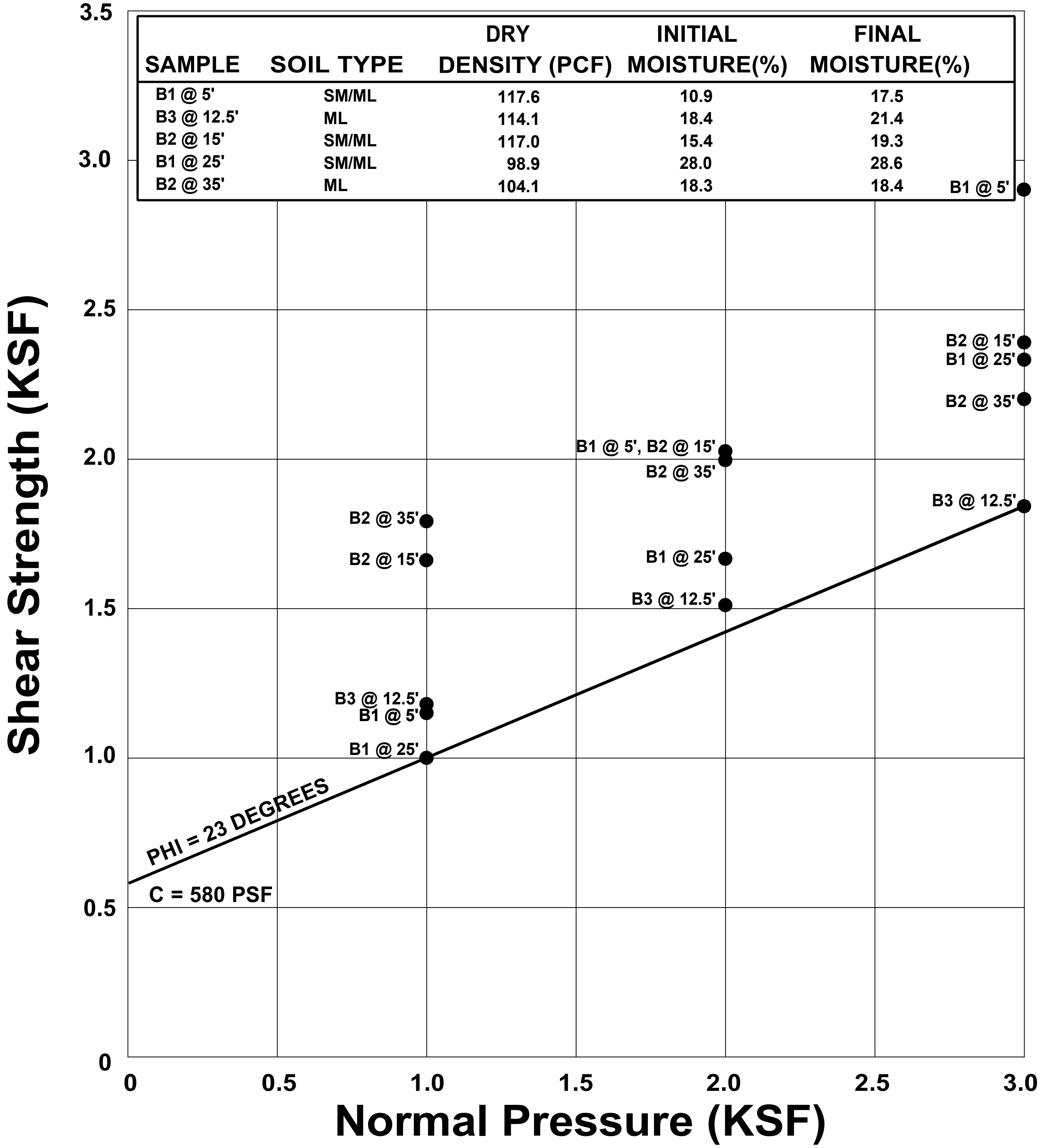
BORING LOG NUMBER 3

Radha Hotels USA, LLC.

File No. 22157

ln

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
52.5	100/10"	18.2	111.7	-		
				53 --	ML	Sandy Silt, dark brown and gray, moist, stiff
				-		
				54 --		
				-		
55	77	17.7	SPT	55 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark brown, moist, dense, stiff, medium grained
				56 --		
				-		
				57 --		
57.5	100/9"	22.1	106.3	-		
				58 --	ML/CL	Clayey Silt to Silty Clay, dark and grayish brown, moist, very stiff
				-		
				59 --		
				-		
60	73	18.4	SPT	60 --		
				-		Total Depth 60 Feet
				61 --		Water at 27 Feet 10 Inches.
				-		Fill to 3 Feet
				62 --		
				-		
				63 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				64 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				65 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				66 --		
				-		SPT=Standard Penetration Test
				67 --		
				-		
				68 --		
				-		
				69 --		
				-		
				70 --		
				-		
				71 --		
				-		
				72 --		
				-		
				73 --		
				-		
				74 --		
				-		
				75 --		
				-		



SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B1 @ 5'	SM/ML	117.6	10.9	17.5
B3 @ 12.5'	ML	114.1	18.4	21.4
B2 @ 15'	SM/ML	117.0	15.4	19.3
B1 @ 25'	SM/ML	98.9	28.0	28.6
B2 @ 35'	ML	104.1	18.3	18.4

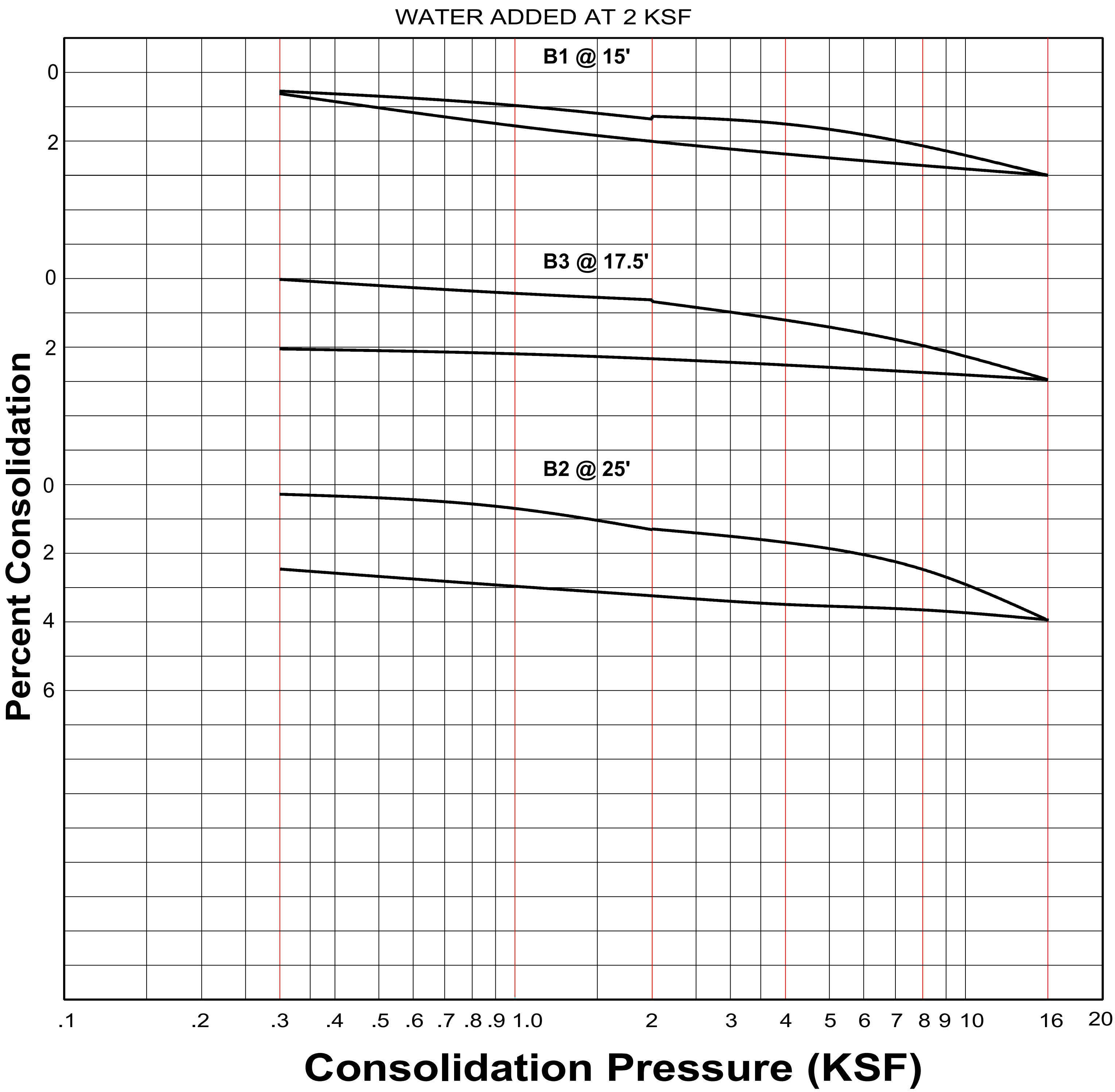
● Direct Shear, Saturated

SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

RADHA HOTELS USA, LLC
11905 WILSHIRE BLVD., LOS ANGELES

FILE NO. 22157 | PLATE: B



CONSOLIDATION TEST

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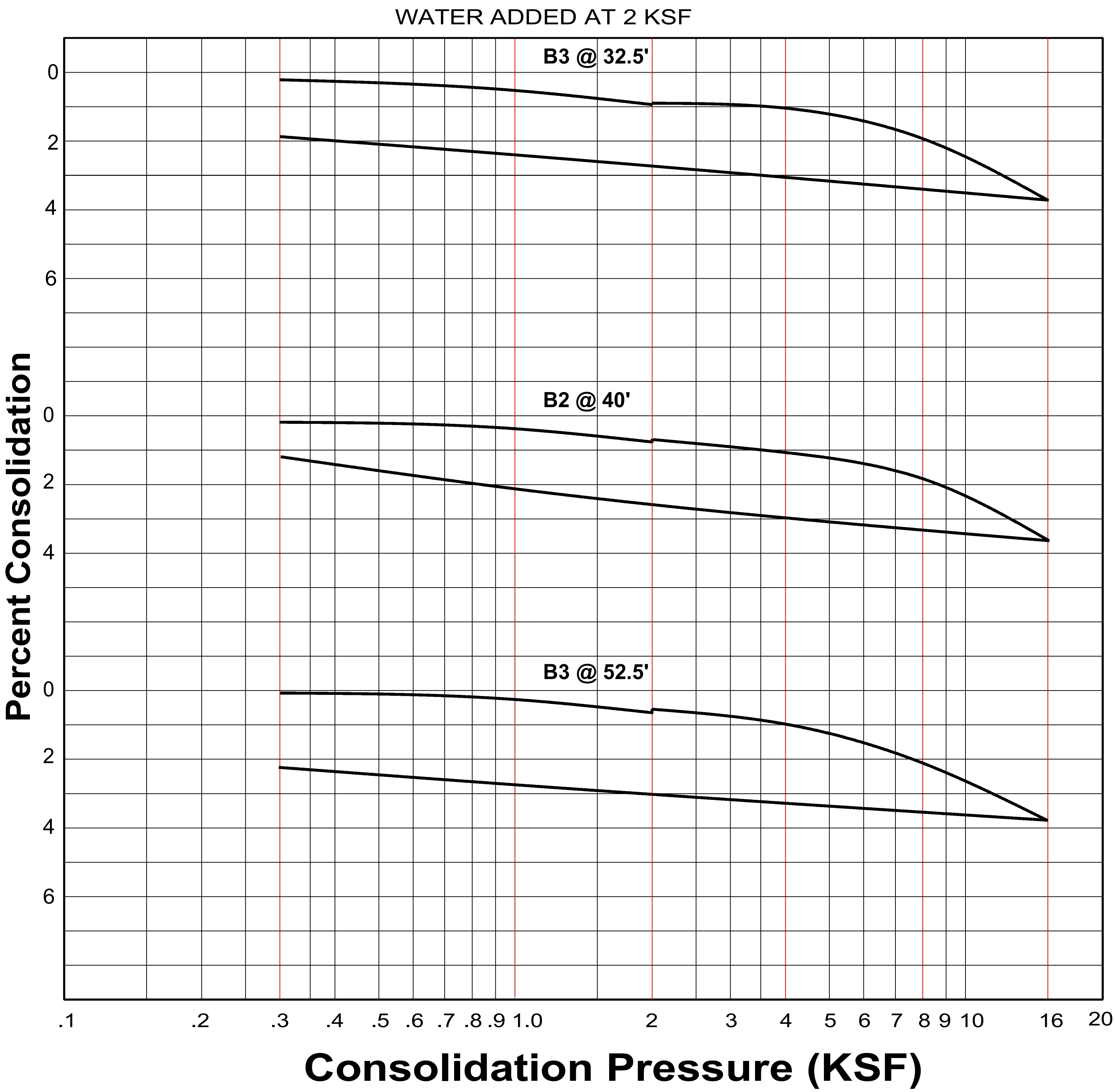
FILE NO. 22157	**PLATE: C-1**



CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

RADHA HOTELS USA, LLC
11905 WILSHIRE BLVD., LOS ANGELES

FILE NO. 22157	**PLATE: C-2**

ASTM D 1557

SAMPLE	B2 @ 1'-5'
SOIL TYPE:	SM
MAXIMUM DENSITY pcf.	124.9
OPTIMUM MOISTURE %	10.7

ASTM D 4829

SAMPLE	B2 @ 1-5'	B2 @ 10'	B3 @ 17.5'
SOIL TYPE:	SM	SM/ML	SP
EXPANSION INDEX UBC STANDARD 18-2	35	17	1
EXPANSION CHARACTER	LOW	VERY LOW	VERY LOW

SULFATE CONTENT

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

COMPACTION/EXPANSION DATA SHEET

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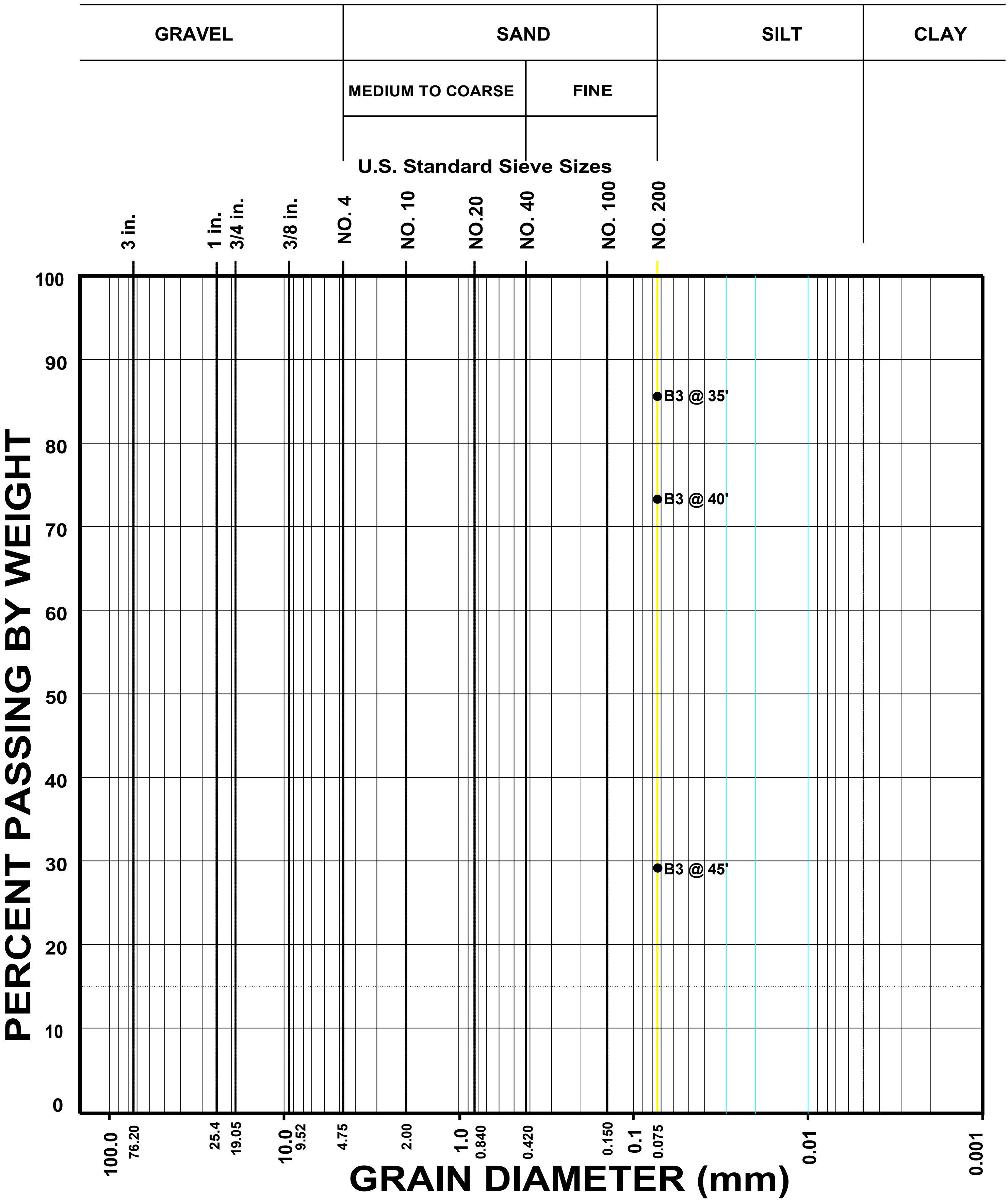
11905 WILSHIRE BLVD., LOS ANGELES

FILE NO. 22157 PLATE: D



GRAIN SIZE DISTRIBUTION

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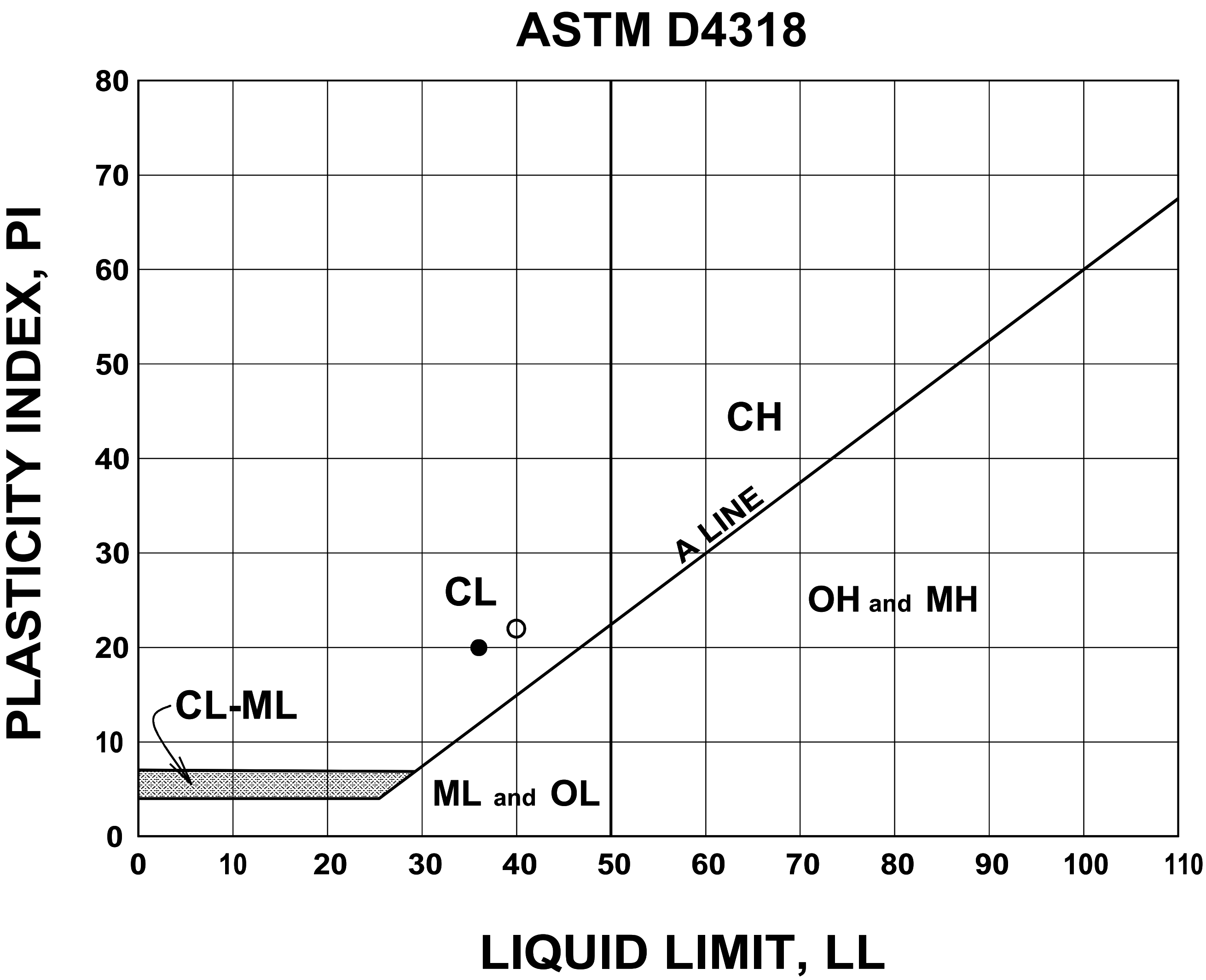
RADHA HOTEL USA, LLC
11905 WILSHIRE BLVD., LOS ANGELES

FILE NO. 22157

PLATE: E



BORING NUMBER	DEPTH (FEET)	TEST SYMBOL	LL	PL	PI	DESCRIPTION
B3	35	○	40	18	22	CL
B3	40	●	36	16	20	CL

ATTERBERG LIMITS DETERMINATION

Geotechnologies, Inc.
Consulting Geotechnical Engineers

RADHA HOTEL USA, LLC
11905 WILSHIRE BLVD., LOS ANGELES

FILE NO. 22157

PLATE: F

Geotechnologies, Inc.

Project: Radha Hotels USA, LLC
File No.: 22157
Description: Liquefaction Analysis
Boring Numbe 3

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	6.86
Peak Ground Horizontal Acceleration, PGA_M (g):	0.937
Calculated Mag.Wtg.Factor:	1.184

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	27.8
Historically Highest Groundwater Level* (ft):	20.0
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 $(N_1)_{60\text{-}cs}$	Stress Reduction Coeff, r_d	Cyclic Shear Ratio CSR	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Lateral Disp. Index ΔLDI_i (feet)	Liquefaction Settlment ΔS_i (inches)
1	133.7	Unsaturated	Unsaturated	83	5	0.0	0	133.7	133.7	197.8	1.00	0.612	2.000	Non-Liq.	0.0	0.00
2	133.7	Unsaturated	Unsaturated	83	5	0.0	0	267.4	267.4	197.8	1.00	0.610	2.000	Non-Liq.	0.0	0.00
3	133.7	Unsaturated	Unsaturated	83	5	0.0	0	401.1	401.1	180.0	1.00	0.608	2.000	Non-Liq.	0.0	0.00
4	133.7	Unsaturated	Unsaturated	83	5	0.0	0	534.8	534.8	166.9	0.99	0.606	2.000	Non-Liq.	0.0	0.00
5	133.7	Unsaturated	Unsaturated	83	5	0.0	0	668.5	668.5	167.9	0.99	0.604	2.000	Non-Liq.	0.0	0.00
6	133.7	Unsaturated	Unsaturated	83	5	0.0	0	802.2	802.2	160.0	0.99	0.601	2.000	Non-Liq.	0.0	0.00
7	133.7	Unsaturated	Unsaturated	83	5	0.0	0	935.9	935.9	153.7	0.98	0.599	2.000	Non-Liq.	0.0	0.00
8	135.6	Unsaturated	Unsaturated	52	10	0.0	0	1071.5	1071.5	92.9	0.98	0.597	2.000	Non-Liq.	0.0	0.00
9	135.6	Unsaturated	Unsaturated	52	10	0.0	0	1207.1	1207.1	95.7	0.98	0.594	2.000	Non-Liq.	0.0	0.00
10	135.6	Unsaturated	Unsaturated	52	10	0.0	0	1342.7	1342.7	93.0	0.97	0.592	2.000	Non-Liq.	0.0	0.00
11	135.6	Unsaturated	Unsaturated	52	10	0.0	0	1478.3	1478.3	90.7	0.97	0.589	2.000	Non-Liq.	0.0	0.00
12	135.6	Unsaturated	Unsaturated	52	10	0.0	0	1613.9	1613.9	88.6	0.96	0.586	2.000	Non-Liq.	0.0	0.00
13	135.0	Unsaturated	Unsaturated	52	10	0.0	0	1748.9	1748.9	86.8	0.96	0.584	2.000	Non-Liq.	0.0	0.00
14	135.0	Unsaturated	Unsaturated	52	10	0.0	0	1883.9	1883.9	85.1	0.95	0.581	2.000	Non-Liq.	0.0	0.00
15	135.0	Unsaturated	Unsaturated	66	15	0.0	0	2018.9	2018.9	118.5	0.95	0.578	2.000	Non-Liq.	0.0	0.00
16	135.0	Unsaturated	Unsaturated	66	15	0.0	0	2153.9	2153.9	116.5	0.94	0.575	2.000	Non-Liq.	0.0	0.00
17	135.0	Unsaturated	Unsaturated	66	15	0.0	0	2288.9	2288.9	114.7	0.94	0.572	2.000	Non-Liq.	0.0	0.00
18	112.3	Unsaturated	Unsaturated	66	15	0.0	0	2401.2	2401.2	113.2	0.93	0.569	2.000	Non-Liq.	0.0	0.00
19	112.3	Unsaturated	Unsaturated	66	15	0.0	0	2513.5	2513.5	111.9	0.93	0.566	2.000	Non-Liq.	0.0	0.00
20	112.3	Unsaturated	Unsaturated	82	20	0.0	0	2625.8	2625.8	137.4	0.92	0.563	2.000	Non-Liq.	0.0	0.00
21	112.3	Unsaturated	Saturated	82	20	0.0	0	2738.1	2675.7	136.7	0.92	0.573	2.000	3.5	0.0	0.00
22	112.3	Unsaturated	Saturated	82	20	0.0	0	2850.4	2725.6	136.1	0.91	0.582	2.000	3.4	0.0	0.00
23	116.3	Unsaturated	Saturated	82	20	0.0	0	2966.7	2779.5	135.4	0.91	0.591	2.000	3.4	0.0	0.00
24	116.3	Unsaturated	Saturated	82	20	0.0	0	3083.0	2833.4	134.7	0.90	0.599	2.000	3.3	0.0	0.00
25	116.3	Unsaturated	Saturated	79	25	0.0	0	3199.3	2887.3	129.1	0.90	0.606	2.000	3.3	0.0	0.00
26	116.3	Unsaturated	Saturated	79	25	0.0	0	3315.6	2941.2	128.5	0.89	0.613	2.000	3.3	0.0	0.00
27	116.3	Unsaturated	Saturated	79	25	0.0	0	3431.9	2995.1	127.9	0.89	0.619	2.000	3.2	0.0	0.00
28	134.0	Saturated	Saturated	79	25	0.0	0	3565.9	3066.7	133.8	0.88	0.624	2.000	3.2	0.0	0.00
29	134.0	Saturated	Saturated	79	25	0.0	0	3699.9	3138.3	133.0	0.88	0.629	2.000	3.2	0.0	0.00
30	134.0	Saturated	Saturated	80	30	0.0	0	3833.9	3209.9	133.9	0.87	0.633	2.000	3.2	0.0	0.00
31	134.0	Saturated	Saturated	80	30	0.0	0	3967.9	3281.5	133.1	0.86	0.636	2.000	3.1	0.0	0.00
32	134.0	Saturated	Saturated	80	30	0.0	0	4101.9	3353.1	132.3	0.86	0.639	2.000	3.1	0.0	0.00
33	130.4	Saturated	Saturated	80	30	0.0	0	4232.3	3421.1	131.6	0.85	0.642	2.000	3.1	0.0	0.00
34	130.4	Saturated	Saturated	80	30	0.0	0	4362.7	3489.1	131.0	0.85	0.645	2.000	3.1	0.0	0.00
35	130.4	Saturated	Saturated	21	35	85.6	22	4493.1	3557.1	37.3	0.84	0.647	1.899	Non-Liq.	0.0	0.00
36	130.4	Saturated	Saturated	21	35	85.6	22	4623.5	3625.1	37.1	0.83	0.648	1.773	Non-Liq.	0.0	0.00
37	130.4	Saturated	Saturated	21	35	85.6	22	4753.9	3693.1	36.8	0.83	0.650	1.663	Non-Liq.	0.0	0.00
38	133.5	Saturated	Saturated	21	35	85.6	22	4887.4	3764.2	36.6	0.82	0.651	1.562	Non-Liq.	0.0	0.00
39	133.5	Saturated	Saturated	21	35	85.6	22	5020.9	3835.3	36.4	0.82	0.652	1.471	Non-Liq.	0.0	0.00
40	133.5	Saturated	Saturated	40	40	73.3	20	5154.4	3906.4	69.1	0.81	0.652	1.937	Non-Liq.	0.0	0.00
41	133.5	Saturated	Saturated	40	40	73.3	20	5287.9	3977.5	68.8	0.81	0.652	1.924	Non-Liq.	0.0	0.00
42	133.5	Saturated	Saturated	40	40	73.3	20	5421.4	4048.6	68.5	0.80	0.652	1.912	Non-Liq.	0.0	0.00
43	134.8	Saturated	Saturated	40	40	73.3	20	5556.2	4121.0	68.2	0.79	0.652	1.900	Non-Liq.	0.0	0.00
44	134.8	Saturated	Saturated	40	40	73.3	20	5691.0	4193.4	68.0	0.79	0.651	1.888	Non-Liq.	0.0	0.00
45	134.8	Saturated	Saturated	79	45	29.2	0	5825.8	4265.8	128.0	0.78	0.650	1.876	2.9	0.0	0.00
46	134.8	Saturated	Saturated	79	45	29.2	0	5960.6	4338.2	127.4	0.78	0.650	1.864	2.9	0.0	0.00
47	134.8	Saturated	Saturated	79	45	29.2	0	6095.4	4410.6	126.9	0.77	0.648	1.852	2.9	0.0	0.00
48	130.4	Saturated	Saturated	79	45	29.2	0	6225.8	4478.6	126.4	0.76	0.647	1.842	2.8	0.0	0.00
49	130.4	Saturated	Saturated	79	45	29.2	0	6356.2	4546.6	126.0	0.76	0.646	1.831	2.8	0.0	0.00
50	130.4	Saturated	Saturated	95	50	0.0	0	6486.6	4614.6	144.5	0.75	0.645	1.821	2.8	0.0	0.00
51	130.4	Saturated	Saturated	95	50	0.0	0	6617.0	4682.6	143.9	0.75	0.643	1.810	2.8	0.0	0.00
52	130.4	Saturated	Saturated	95	50	0.0	0	6747.4	4750.6	143.4	0.74	0.642	1.800	2.8	0.0	0.00
53	132.1	Saturated	Saturated	95	50	0.0	0	6879.5	4820.3	142.8	0.74	0.640	1.790	2.8	0.0	0.00
54	132.1	Saturated	Saturated	95	50	0.0	0	7011.6	4890.0	142.3	0.73	0.638	1.780	2.8	0.0	0.00
55	132.1	Saturated	Saturated	77	55	0.0	0	7143.7	4959.7	114.9	0.73	0.636	1.770	2.8	0.0	0.00
56	132.1	Saturated	Saturated	77	55	0.0	0	7275.8	5029.4	114.5	0.72	0.634	1.761	2.8	0.0	0.00
57	132.1	Saturated	Saturated	77	55	0.0	0	7407.9	5099.1	114.1	0.71	0.632	1.751	2.8	0.0	0.00
58	129.8	Saturated	Saturated	73	60	0.0	0	7537.7	5166.5	107.8	0.71	0.630	1.742	2.8	0.0	0.00
59	129.8	Saturated	Saturated	73	60	0.0	0	7667.5	5233.9	107.4	0.70	0.628	1.733	2.8	0.0	0.00
60	129.8	Saturated	Saturated	73	60	0.0	0	7797.3	5301.3	107.0	0.70	0.626	1.724	2.8	0.0	0.00
											Total Liquefaction Settlement, S =			**0.00**		**inches**

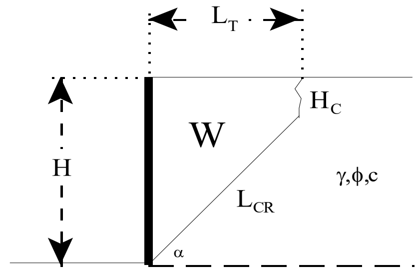
Geotechnologies, Inc.

Project: Radha Hotels USA, LLC
File No.: 22157
Description: Retaining Walls up to 15 feet

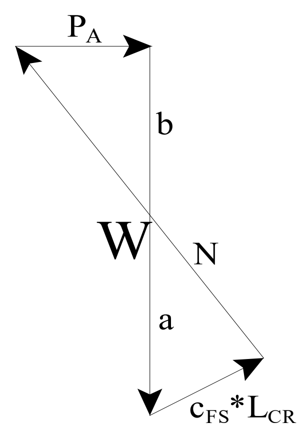
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	15.00 feet
Unit Weight of Retained Soils	(γ)	120.0 pcf
Friction Angle of Retained Soils	(ϕ)	23.0 degrees
Cohesion of Retained Soils	(c)	580.0 psf
Factor of Safety	(FS)	1.50
Factored Parameters:	(ϕ_{FS})	15.8 degrees
	(c_{FS})	386.7 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	9.0	72	8653.1	8.5	6484.4	2168.8	1212.0
46	8.9	71	8474.9	8.5	6299.9	2175.0	1265.8
47	8.8	69	8279.7	8.5	6112.3	2167.4	1312.6
48	8.7	67	8070.5	8.5	5923.3	2147.2	1352.2
49	8.6	65	7850.0	8.4	5734.3	2115.8	1384.5
50	8.6	64	7620.2	8.4	5546.1	2074.1	1409.5
51	8.5	62	7382.7	8.3	5359.5	2023.2	1427.2
52	8.5	59	7139.0	8.2	5174.8	1964.2	1437.5
53	8.5	57	6890.0	8.1	4992.2	1897.8	1440.5
54	8.5	55	6636.6	8.0	4811.7	1824.9	1436.0
55	8.6	53	6379.6	7.9	4633.3	1746.3	1424.3
56	8.6	51	6119.5	7.7	4456.7	1662.7	1405.1
57	8.6	49	5856.6	7.6	4281.8	1574.8	1378.6
58	8.7	47	5591.2	7.4	4108.0	1483.2	1344.8
59	8.8	44	5323.5	7.2	3935.1	1388.5	1303.8
60	8.9	42	5053.6	7.0	3762.4	1291.3	1255.7
61	9.0	40	4781.5	6.8	3589.3	1192.2	1200.5
62	9.2	38	4507.0	6.6	3415.3	1091.7	1138.4
63	9.3	35	4230.0	6.4	3239.5	990.5	1069.7
64	9.5	33	3950.2	6.1	3061.0	889.2	994.5
65	9.7	31	3667.3	5.9	2878.8	788.4	913.4
66	9.9	28	3380.8	5.6	2691.9	688.9	826.8
67	10.2	26	3090.1	5.2	2498.8	591.2	735.3
68	10.5	23	2794.6	4.9	2298.1	496.4	640.0
69	10.8	21	2493.4	4.5	2088.0	405.4	541.8
70	11.2	18	2185.5	4.1	1866.3	319.2	442.5



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 1440.47 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 12.8 pcf

Design Wall for an Equivalent Fluid Pressure: 30 pcf

Geotechnologies, Inc.

Project:	Radha Hotels USA, LLC
File No.:	22157

Soil Weight	γ	120	pcf
Internal Friction Angle	ϕ	23	degrees
Cohesion	c	580	psf
Height of Retaining Wall	H	15	feet

Restrained Retaining Wall Design based on At Rest Earth Pressure

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

$K_o = 1 - \sin\phi$	0.609
$\sigma'_v = \gamma H$	1800.0 psf

$\sigma'_h =$ 1096.7 psf

EFP = 73.1 pcf

$P_o =$ 8225.1 lbs/ft (based on a triangular distribution of pressure)

Design wall for an EFP of 74 pcf

Geotechnologies, Inc.

Project: Radha Hotels USA, LLC

File No.: 22157

Seismically Induced Lateral Soil Pressure on Retaining Wall

Input:

Height of Retaining Wall:	(H)	15.0	feet
Retained Soil Unit Weight:	(γ)	120.0	pcf
Peak Ground Acceleraction:	(PGA_M)	0.94	g
Horizontal Ground Acceleration:	(k_h)	0.31	g

Seismic Increment (ΔP_{AE}):

$k_h = 0.5*0.67*PGA_M$

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

$\Delta P_{AE} =$ 3178.2 lbs/ft

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

T = 2860.4 lbs/ft

$EFP = 2*T/H^2$

EFP = 25.4 pcf

triangular distribution of pressure, applied to the proposed retaining wall.

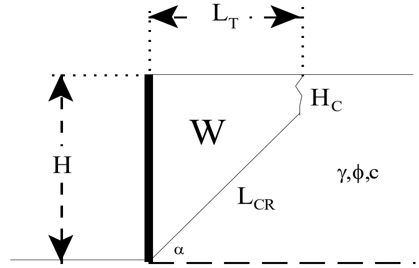
Geotechnologies, Inc.

Project: Radha Hotels USA, LLC
File No.: 22157
Description: Shoring Walls up to 20 feet

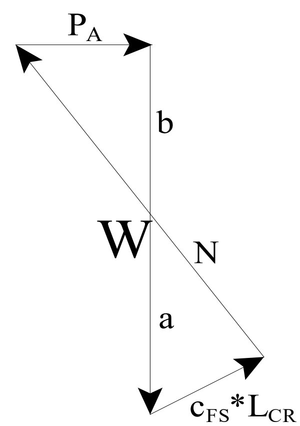
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	20.00	feet
Unit Weight of Retained Soils	(γ)	120.0	pcf
Friction Angle of Retained Soils	(ϕ)	23.0	degrees
Cohesion of Retained Soils	(c)	580.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(ϕ_{FS})	18.8	degrees
	(c_{FS})	464.0	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	11.7	131	15773.0	11.7	11649.2	4123.7	2033.0
46	11.5	129	15495.5	11.8	11322.7	4172.8	2148.5
47	11.3	126	15179.4	11.8	10987.9	4191.6	2251.6
48	11.2	124	14832.1	11.8	10649.1	4183.0	2342.0
49	11.1	120	14459.6	11.8	10309.7	4149.9	2419.5
50	11.0	117	14066.7	11.8	9972.1	4094.5	2484.0
51	10.9	114	13657.1	11.7	9637.9	4019.3	2535.3
52	10.8	110	13234.2	11.6	9308.0	3926.2	2573.5
53	10.8	107	12800.3	11.5	8983.1	3817.2	2598.4
54	10.8	103	12357.5	11.4	8663.5	3694.1	2610.1
55	10.8	99	11907.5	11.2	8349.1	3558.3	2608.5
56	10.8	95	11451.4	11.1	8039.9	3411.5	2593.6
57	10.9	92	10990.2	10.9	7735.2	3255.0	2565.4
58	10.9	88	10524.7	10.7	7434.7	3090.0	2524.1
59	11.0	84	10055.4	10.5	7137.7	2917.8	2469.5
60	11.1	80	9582.6	10.3	6843.2	2739.4	2401.8
61	11.2	76	9106.5	10.0	6550.5	2556.0	2321.2
62	11.4	72	8627.0	9.8	6258.4	2368.6	2227.7
63	11.6	68	8144.0	9.5	5965.7	2178.3	2121.5
64	11.8	64	7657.2	9.2	5671.1	1986.1	2003.1
65	12.0	60	7166.2	8.8	5373.1	1793.1	1872.7
66	12.3	56	6670.3	8.5	5069.9	1600.4	1731.0
67	12.6	51	6168.9	8.1	4759.6	1409.3	1578.6
68	12.9	47	5660.8	7.7	4439.8	1221.0	1416.8
69	13.3	43	5145.0	7.2	4107.9	1037.1	1246.7
70	13.7	38	4619.9	6.7	3760.8	859.2	1070.3



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A, max}$ 2610.1 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 13.1 pcf

Design Shoring for an Equivalent Fluid Pressure: 28 pcf