Geotechnologies, Inc.
Consulting Geotechnical Engineers

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

Celebrating 50 YEARS OF SERVICE 1971-2021

May 29, 2018
File Number 21225

GRT Beverly Hills, LLC
c/o Wald Realty Advisors, Inc.
775 Wildomar Street
Pacific Palisades, California 90272

Attention: David Wald

Subject: Preliminary Geotechnical Engineering Investigation
Proposed Mixed-Use Development
1400-1440 Vine Street, and 6263 De Longpre Avenue, Los Angeles, California

Ladies and Gentlemen:

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

This report is preliminary in nature due to the lack of a well-defined development plan and structural loading parameters. A comprehensive report should be prepared when the proposed development achieves refinement.

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

Should you have any questions please contact this office.

Respectfully submitted,
GEOTECHNOLOGIES, INC.

REGISTERED PROFESSIONAL ENGINEER
STANLEY KEI TANG
R.C.E. 56178
EXP. 12/31/22
CIVIL
STATE OF CALIFORNIA

STANLEY S. TANG
R.C.E. 56178

SST:km

Distribution: (5) Addressee

TABLE OF CONTENTS

SECTION **PAGE**

TABLE OF CONTENTS

SECTION **PAGE**

TABLE OF CONTENTS

SECTION **PAGE**

PRELIMINARY GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
1400-1440 VINE STREET, AND 6263 DE LONGPRE AVENUE
LOS ANGELES, CALIFORNIA

INTRODUCTION

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

This report is preliminary in nature due to the lack of a well-defined development plan and structural loading parameters. A comprehensive report should be prepared when the proposed development achieves refinement.

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

PROPOSED DEVELOPMENT

Preliminary design information concerning the proposed development was furnished by the client. The site is proposed to be developed with an 8 to 10-story mixed-use development. The proposed development will be constructed over two subterranean levels extending on the order of 20 feet below the existing site grade. Column loads are estimated to be between 800 and 1,200 kips. Wall loads are estimated to be between 6 and 12 kips per lineal foot. Grading will consist of excavations as deep as 25 feet in depth for the proposed subterranean parking levels and foundation elements.

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

SITE CONDITIONS

The property is located at 1400-1440 Vine Street, and 6263 De Longpre Avenue, in the Hollywood area of the City of Los Angeles, California. The site is bounded by Leland Way to the north, three single-story residential structures and a parking lot to the east, De Longpre Avenue to the south, and Vine Street to the west. The site is shown relative to nearby topographic features in the enclosed Vicinity Map.

The existing site grade is relatively level, and descends gently to the south, with approximately 5 feet of elevation change. The site is currently developed with two single-story commercial structures and a paved parking lot. Vegetation at the site consists of small planter areas containing grass lawns, shrubbery, and two mature trees. Drainage across the site appears to be by sheetflow to the city streets to the south.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was explored between March 19, 2018 and March 21, 2019, by excavating five exploratory borings. The exploratory borings varied between 50 to 120 feet in depth below the existing site grade. The borings were excavated with the aid of a truck-mounted drilling machine, equipped with an automatic hammer, and using 8-inch diameter hollowstem augers.

The location of exploratory excavations was determined by information furnished by the client. Elevations of the exploratory excavations were determined by hand level or interpolation from data provided. 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 underlying the subject site consist of silty sands to sandy silts, which are brown to grayish brown in color, slightly moist to moist, medium dense to dense, firm to stiff, fine grained. Fill thickness ranging from 3 to 4½ feet was encountered in the exploratory borings.

Native soils consist of alluvial deposits, comprising of silty sands to gravelly sands, with sandy silts and sandy clays. The native soils are grayish brown to brown in color, slightly moist to wet, medium dense to very dense, stiff, fine to coarse grained, with varying amount of gravel. The native soils consist predominantly of sediments deposited by river and stream action typical to this area of Los Angeles County. More detailed soil profiles may be obtained from individual boring logs.

Groundwater

Groundwater was encountered at depths between 46 and 49½ feet below the existing site grade in the exploratory borings. The historically highest groundwater level was established by review of California Geological Survey Seismic Hazard Zone Report of the Hollywood Quadrangle. Review of this report indicates that the historically highest groundwater level is on the order of 45 feet below the existing site grade.

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 property 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.

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 active, potentially active, or inactive. Active faults are those which show evidence of surface displacement within the last 11,000 years (Holocene-age). Potentially-active faults are those that show evidence of most recent surface displacement within the last 1.6 million years (Quaternary-age). Faults showing no evidence of surface displacement within the last 1.6 million years are considered inactive for most purposes, with the exception of design of some critical structures.

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

SEISMIC HAZARDS AND DESIGN CONSIDERATIONS

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

Surface Rupture

In 1972, the Alquist-Priolo Special Studies Zones Act (now known as the Alquist-Priolo Earthquake Fault Zoning Act) was passed into law. The Act defines "active" and "potentially 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 11,000 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 known fault trace based on the location precision, the complexity, or the regional significance of the fault. If a site lies within an Earthquake Fault Zone, a geologic fault rupture investigation must be performed that demonstrates that the proposed building site is not threatened by surface displacement from the fault before development permits may be issued.

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

Liquefaction

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

The Seismic Hazards 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.

Based on the dense nature of the underlying native soils, and the depth to historically highest groundwater level, the potential for liquefaction occurring 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. Review of the County of Los Angeles Flood and Inundation Hazards Map (Leighton, 1990) indicates the site does not lie within mapped tsunami inundation boundaries.

Seiches are oscillations generated in enclosed bodies of water which can be caused by ground shaking associated with an earthquake. Review of the County of Los Angeles Flood and Inundation Hazards Map, (Leighton, 1990), indicates the site lies within the mapped inundation boundaries of the Mulholland Dam. A determination of whether a higher site elevation would remove the site from the potential inundation zones is beyond the scope of this assessment.

Landsliding

The probability of seismically-induced landslides affecting the subject development is considered to be remote, due to the lack of significant slopes on the site and surrounding areas.

CONCLUSIONS AND RECOMMENDATIONS

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

This report is preliminary in nature due to the lack of a well-defined development plan and structural loading parameters. Additional analyses for a comprehensive report should be prepared when the proposed development achieves refinement.

Between 3 and 4½ feet of existing fill materials was encountered during exploration at the site. Due to the variable nature and the varying depths of the existing fill materials, the existing fill materials are considered to be unsuitable for support of the proposed foundations, floor slabs, or additional fill. It is anticipated that excavation of the proposed subterranean levels will remove the existing fill materials and expose the underlying dense native soils. All foundations may bear in native soils found at the level of the proposed excavation.

Due to the location of the proposed structure relative to property lines, public way, and existing structures, the excavation of the proposed subterranean levels will require shoring measures to provide a stable excavation.

Foundations for small outlying structures, such as property line walls, planters, trash enclosures, and canopies, which are not be tied-in to the proposed mixed-use development, and are to be constructed immediately adjacent to property lines or adjacent structures, such that the recommended horizontal overexcavation and recompaction cannot be achieved, should be deepened to bear in the dense 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

Seismic Velocity Measurements

Downhole seismic velocity measurements were performed by GeoPentech within Boring Number 2, which was excavated to a depth of 120 feet below the existing site grade. Results of the seismic velocity measurements are presented in the Downhole Test Results report by GeoPentech, dated April 16, 2013. According to the seismic survey report, an average shear wave velocity of 1,106 feet/second was measured between 0 and 100 feet, and an average shear wave velocity of 1,241 feet/second was measured between 20 and 120 feet.

2016 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 7-10. This information and the site coordinates were input into the USGS U.S. Seismic Design Maps tool (Version 3.1.0) to calculate the ground motions for the site.

2016 CALIFORNIA BUILDING CODE SEISMIC PARAMETERS	
Site Class	D
Mapped Spectral Acceleration at Short Periods (S_S)	2.345g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	2.345g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.563g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.868g
Site Coefficient (F_v)	1.5
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	1.302g
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.868g

FILL SOILS

The maximum depth of fill encountered on the site was 4½ feet. This material and any fill generated during demolition should be removed during the excavation of the subterranean levels and wasted from the site.

EXPANSIVE SOILS

The onsite geologic materials are in the very low to moderate expansion range. The Expansion Index was found to be between 17 and 50 for bulk samples remolded to 90 percent of the laboratory maximum density. 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 source of natural sulfate minerals in soils includes the sulfates of calcium, magnesium, sodium, and potassium. When these minerals interact and dissolve in subsurface water, a sulfate concentration is created, which will react with exposed concrete. Over time sulfate attack will destroy improperly proportioned concrete well before the end of its intended service life.

The water-soluble sulfate content of the onsite geologic materials was tested by California Test 417. The water-soluble sulfate content was determined to be less than 0.1% percentage by weight for the soils tested. Based on American Concrete Institute (ACI) Standard 318-08, the sulfate exposure is considered to be negligible for geologic materials with less than 0.1% and Type I cement may be utilized for concrete foundations in contact with the site soils.

METHANE ZONES

Based on review of the Navigate LA (*http://navigatela.lacity.org/NavigateLA/*) website, maintained by the City of Los Angeles, the subject property is not located within a Methane Zone or a Methane Buffer Zone as designated by the City.

GRADING GUIDELINES

The following grading guidelines may be utilized for any miscellaneous site grading which may be required as part of the proposed development.

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 90 percent of the maximum density, except for cohesionless soils having less than 15 percent finer than 0.005 millimeters, which shall be compacted to a minimum 95 percent of the maximum density in accordance with the most recent revision of the Los Angeles Building Code.

All fill should be mechanically compacted in layers not more than 8 inches thick. 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. using the test method described in 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. Any imported materials shall be observed and tested by the representative of the geotechnical engineer prior to use in fill areas. Imported materials should contain sufficient fines so as to be relatively impermeable and result in a stable subgrade when compacted. Any required import materials should consist of geologic materials with an expansion index of less than 50. The water-soluble sulfate content of the import materials should be less than 0.1% percentage by weight.

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

Utility Trench Backfill

Utility trenches should be backfilled with controlled fill. The utility should be bedded with clean sands at least one foot over the crown. The remainder of the backfill may be onsite soil compacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the laboratory maximum density. Utility trench backfill should be tested by representatives of this firm in accordance with the most recent revision of ASTM D-1557.

Shrinkage

Shrinkage results when a volume of soil removed at one density is compacted to a higher density. A shrinkage factor between 5 and 15 percent should be anticipated when excavating and recompacting the existing fill and underlying native geologic materials on the site to an average comparative compaction of 92 percent.

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

Depending on the structural loading and settlement tolerance, the proposed mixed-use development may be supported on conventional foundations or mat foundation bearing in the underlying native soils.

This foundation recommendations provided herein is considered to be preliminary in nature due to the lack of a well-defined development plan and structural loading parameters. Additional analyses for a comprehensive report should be prepared when the proposed development achieves refinement.

Conventional

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 250 pounds per square foot. The bearing capacity increase for each additional foot of depth is 500 pounds per square foot. The maximum recommended bearing capacity is 8,000 pounds per square foot.

A minimum factor of safety of 3 was utilized in determining the allowable bearing capacities. 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.

Mat Foundation

The proposed mixed-use structure will be constructed over 2 subterranean parking levels extending on the order of 20 to 25 feet below grade. Preliminarily, it is anticipated that the proposed structure will have an average bearing pressure of 6,000 pounds per square foot. Foundation bearing pressure will vary across the mat footings, with the highest concentrated loads located at the central cores of the mat foundations.

Given the size of the proposed mat foundation, the average bearing pressure of 6,000 pounds per square foot is well below the allowable bearing pressures, with factor of safety well exceeding 3. For design purposes, an average bearing pressure of 6,000 pounds per square foot, with locally higher pressures up to 9,000 pounds per square foot may be utilized in the mat foundation design. The mat foundation may be designed utilizing a modulus of subgrade reaction of 250 pounds per cubic inch. This value is a unit value for use with a one-foot square footing. The modulus should be reduced in accordance with the following equation when used with larger foundations.

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

where K = Reduced Subgrade Modulus
K_1 = Unit Subgrade Modulus
B = Foundation Width (feet)

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

Miscellaneous Foundations

Foundations for small miscellaneous outlying structures, such as property line fence walls, planters, exterior canopies, and trash enclosures, which will not be tied-in to the proposed structure, may be supported on conventional foundations bearing in properly compacted fill and/or the native soils. Wall footings may be designed for a bearing value of 1,500 pounds per square foot, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material. No bearing value increases are recommended.

Foundation Reinforcement

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

Lateral Design

Resistance to lateral loading may be provided by friction acting at the base of foundations and by passive earth pressure. An allowable coefficient of friction of 0.3 may be used with the dead load forces.

Passive geologic pressure for the sides of foundations poured against undisturbed or recompacted soil may be computed as an equivalent fluid having a density of 250 pounds per cubic foot with a maximum earth pressure of 3,000 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

This report is preliminary in nature due to the lack of a well-defined development plan and structural loading parameters. Additional analyses for a comprehensive report should be prepared when the proposed development achieves refinement.

Settlement of the foundation system is expected to occur on initial application of loading. Preliminarily, the settlement for a conventional footing below the heaviest loaded columns is expected to be on the order of 1½ inches. The total settlement below the more heavily loaded central core portions of the mat foundation beneath the structure will be between 2 to 3 inches. Settlement on the edges of the mat foundation is expected to be between 1 to 1½ inches.

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

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

Height of Retaining Wall (feet)	Cantilever Retaining Wall Triangular Distribution of Active Earth Pressure (pcf)	Restrained Retaining Wall Triangular Distribution of At-Rest Earth Pressure (pcf)
15 feet	35 pcf	60 pcf
25 feet	45 pcf	60 pcf

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

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. Foundations may be designed using the allowable bearing capacities, friction, and passive earth pressure found in the "Foundation Design" section above.

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 26½ pounds per cubic foot. The seismic earth pressure should be combined with the lateral active earth pressure for analyses of restrained basement walls under seismic loading condition.

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 2008-83, 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 top 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.

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.

Where retaining walls are to be constructed adjacent to property lines, there is usually not enough space for placement of a standard perforated pipe and gravel drainage system. Under these circumstances, every other head joints may be left out, or 2-inch diameter weepholes may be placed at the 8 feet on center along the base of the wall. The wall shall be backfilled with a minimum of 1 foot of gravel above the base of the retaining wall. The gravel may consist of three-quarter inch to one inch crushed rocks.

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 obtainable by the latest revision of ASTM D 1557 method of compaction. Flooding should not be permitted. Proper compaction of the backfill will be necessary to reduce settlement of 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.

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. Though groundwater was encountered during exploration between 46 and 49½ feet below grade, the proposed subterranean level is to be serviced by the backdrainage system is only on the order of 20 feet below site grade. It is considered improbable that the ambient groundwater level would rise over 20 feet during the design life of the structure to affect the retaining wall backdrainage system. Therefore the only water which could affect the proposed retaining walls would be irrigation waters and precipitation. Additionally the 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 25 feet in vertical height will be required for the proposed subterranean levels and foundation elements. The excavations are expected to expose fill and dense native soils, which are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic or structures. Excavations which will be surcharged by adjacent traffic, public way, properties, 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 should may be excavated at a uniform 1:1 (h:v) slope gradient in its entirety to a maximum height of 15 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 within seven feet of the tops of the slopes. If the temporary construction embankments are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. The soils exposed in the cut slopes should be inspected during excavation by personnel from this office so that modifications of the slopes can be made if variations in the soil conditions occur.

It is critical that the soils exposed in the cut slopes are observed by a representative of this office during excavation so that modifications of the slopes can be made if variations in the earth material conditions occur. All excavations should be stabilized within 30 days of initial excavation. Water should not be allowed to pond on top of the excavation or to flow towards it.

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 a review of the final shoring plans and specifications be made by this office prior to bidding or negotiating with a shoring contractor be made.

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 tie-back anchors or raker braces.

Soldier Piles

Drilled cast-in-place soldier piles should be placed no closer than 2 diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the earth materials. For design purposes, an allowable passive value for the earth materials below the bottom plane of excavation may be assumed to be 600 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 earth materials.

The frictional resistance between the soldier piles and retained earth material may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.30 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 450 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.

Casing may be required should caving be experienced in the saturated earth 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.

Piles placed below the water level will require the use of a tremie to place the concrete into the bottom of the hole. A tremie shall consist of a water-tight tube having a diameter of not less than 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 rapid lowering when necessary to retard or stop the flow of concrete. The discharge end shall be closed at the start of the work to prevent water entering the tube and shall be entirely sealed at all times, except when the concrete is being placed. The tremie tube shall be kept full of concrete. The flow shall be continuous until the work is completed and the resulting concrete seal shall be monolithic and homogeneous. The tip of the tremie tube shall always be kept about five feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

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

Lagging

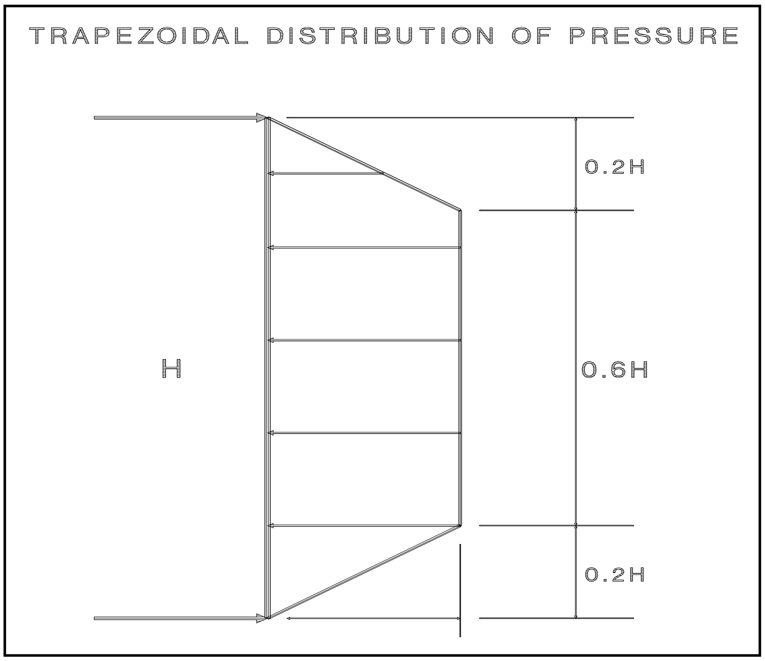
Soldier piles and anchors should be designed for the full anticipated pressures. Due to the cohesionless nature of the underlying earth materials, lagging will be required throughout the entire depth of the excavation. Due to arching in the geologic materials, the pressure on the lagging will be less.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

Lateral Pressures

A triangular distribution of lateral earth pressure should be utilized for the design of 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 cantilevered and restrained shoring are presented in the following table:

Height of Shoring (feet)	Cantilever Shoring System Equivalent Fluid Pressure (pcf) Triangular Distribution of Pressure	Restrained Shoring System Lateral Earth Pressure (psf)* Trapezoidal Distribution of Pressure
25 feet	35 pcf	24H psf

*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 pressures should be applied where the shoring will be surcharged by adjacent traffic or structures.

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. Foundations may be designed using the allowable bearing capacities, friction, and passive earth pressure found in the "Foundation Design" section above.

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.

Drilled friction anchors may be designed for a skin friction of 300 pounds per square foot. Pressure grouted anchor may be designed for a skin friction of 2,000 pounds per square foot. Where belled anchors are utilized, the capacity of belled anchors may be designed by assuming the diameter of the bonded zone is equivalent to the diameter of the bell. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads.

It is recommended that at least 3 of the initial anchors have their capacities tested to 200 percent of their design capacities for a 24-hour period to verify their design capacity. The total deflection during this test should not exceed 12 inches. The anchor deflection should not exceed 0.75 inches during the 24 hour period, measured after the 200 percent load has been applied.

All anchors should be tested to at least 150 percent of design load. The total deflection during this 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. The installation and testing of the anchors should be observed by the geotechnical engineer. Minor caving during drilling of the anchors should be anticipated.

Anchor Installation

Tied-back anchors may be installed between 20 and 40 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.

Deflection

It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized that some deflection will occur. It is estimated that the deflection could be on the order of one inch at the top of the shored embankment. 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. Where internal bracing is used, the rakers should be tightly wedged to minimize deflection. The proper installation of the raker braces and the wedging will be critical to the performance of the shoring.

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 plane projected up from the base of the excavation. A maximum deflection of 1-inch has been allowed provided there are no structures within a 1:1 plane drawn upward from the base of the excavation.

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 the existing buildings on the adjacent properties be made during construction to record any movements for use in the event of a dispute.

Shoring Observations

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

SLABS ON GRADE

Concrete Slabs-on Grade

Concrete floor slabs should be a minimum of 5 inches in thickness, and reinforced with a minimum of #4 steel bars on 16-inch centers each way. 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 reinforced with a minimum of #3 steel bars on 18-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. 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 12 feet should not be exceeded. Lesser spacing 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 95 percent of the maximum density as determined by the most recent revision of ASTM D 1557. The client should be aware that removal of all existing fill in the area of new paving is not required, however, pavement constructed in this manner will most likely have a shorter design life and increased maintenance costs. The following pavement sections are recommended:

Service	Asphalt Pavement Thickness Inches	Base Course Inches
Passenger Cars	3	4
Moderate Truck	4	6
Heavy Truck	6	9

A subgrade modulus of 100 pounds per cubic inch may be assumed for design of concrete paving. 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. Concrete paving for heavy truck traffic shall be a minimum of 7½ inches in thickness, and shall be underlain by 6 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.

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

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 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 development will be constructed over two subterranean levels extending on the order of 20 feet below the existing site grade. When considering the foundation elements, it is anticipated that excavations on the order of 25 feet in depth will be required for the proposed development.

Groundwater was encountered at depths between 46 and 49½ feet below the existing site grade in the exploratory borings. The historically highest groundwater level was established by review of California Geological Survey Seismic Hazard Zone Report of the Hollywood Quadrangle. Review of this report indicates that the historically highest groundwater level is on the order of 45 feet below the existing site grade.

Due to the depth of the proposed subterranean levels and foundation elements, and the depth of the groundwater level, it is the opinion of this firm that stormwater infiltration will not be feasible for the proposed development.

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 scope of the geotechnical services provided did not include any environmental site assessment for the presence or absence of organic substances, hazardous/toxic materials in the soil, surface water, groundwater, or atmosphere, or the presence of wetlands.

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

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 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 by 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 by 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 using 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 D4829. 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.

Laboratory Compaction Characteristics

The maximum dry unit weight and optimum moisture content of a soil are determined by use of 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.

Grain Size Distribution

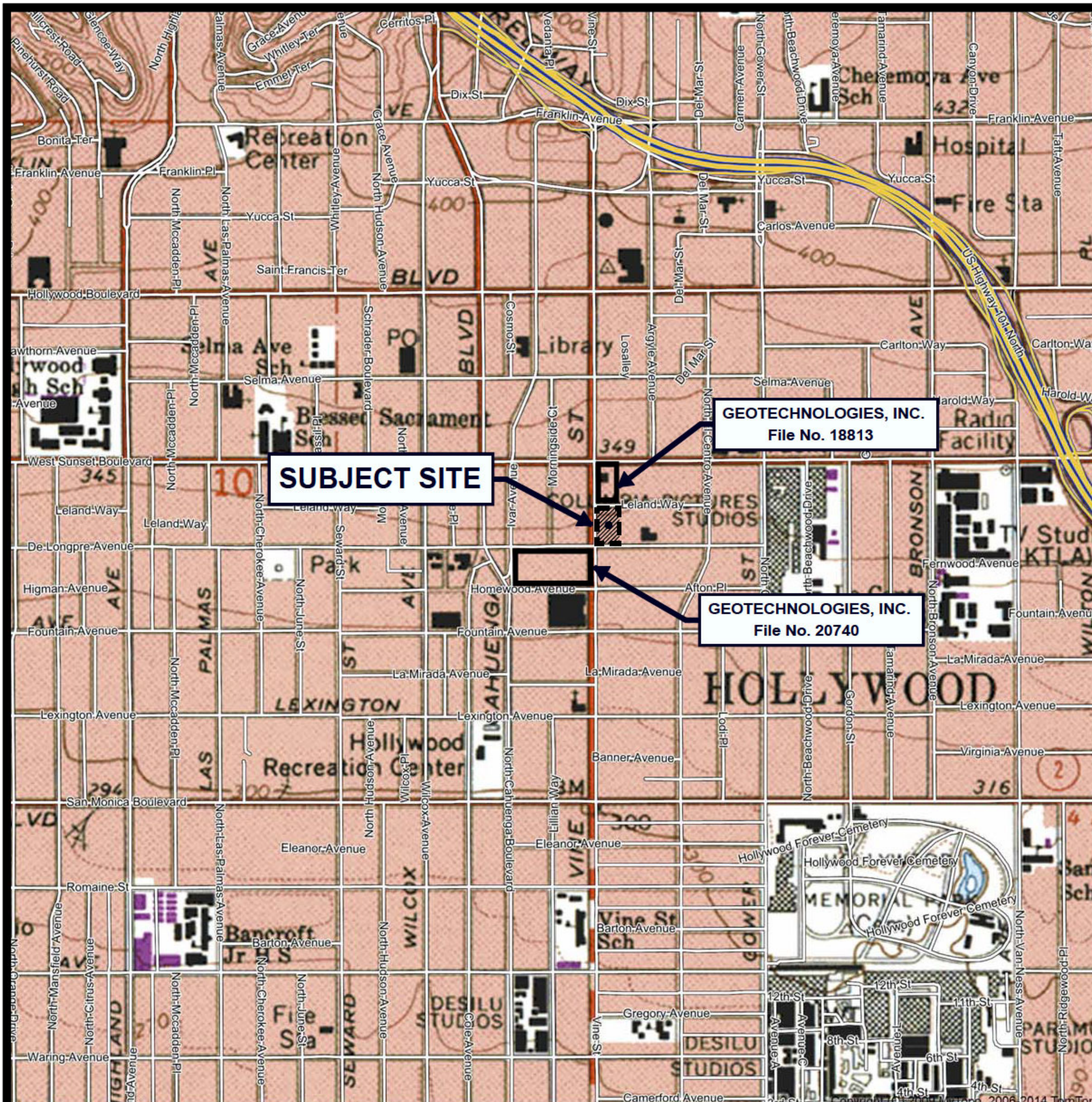
These tests cover the quantitative determination of the distribution of particle sizes in soils. Sieve analysis is used to determine the grain size distribution of the soil larger than the Number 200 sieve. The most recent revision of ASTM D 422 is used to determine particle sizes smaller than the Number 200 sieve. A hydrometer is used to determine the distribution of particle sizes by a sedimentation process. The grain size distributions are plotted on the E-Plates presented in the Appendix of this report.

REFERENCES

1. American Society of Civil Engineers, 1994, “Settlement Analysis,” Technical Engineering and Design Guides, as adapted from the U.S. Army Corps of Engineers, No. 9.
2. Bartlett, S.F. and Youd, T.L., 1992, “Empirical Analysis of Horizontal Ground Displacement Generated by Liquefaction-Induced Lateral Spreads,” Technical Report NCEER-92-0021, National Center for Earthquake Engineering Research, SUNY-Buffalo, Buffalo, NY.
3. Bartlett, S.F. and Youd, T.L., 1995, “Empirical Prediction of Liquefaction-Induced lateral Spread,” Journal of Geotechnical Engineering, Vol. 121, No.4, April.
4. Bowles, Joseph E., 1977, “Foundation Analysis and Design,” 2nd Edition, McGraw-Hill, New York.
5. California Division of Mines and Geology, 1997, Seismic Hazard Zone Map, map scale 1:24,000.
6. California Division of Mines and Geology, 1998, Seismic Hazard Evaluation Report for the Hollywood 7.5-Minute Quadrangle, Los Angeles County, California, Seismic Hazard Zone Report 026.
7. California Geological Survey, 2008, “Guidelines for Evaluation and Mitigation of Seismic Hazards in California,” CDMG Special Publication 117A.
8. City of Los Angeles, Department of Public Works, 2003, Methane and Methane Buffer Zones Map, Map Number A-20960.
9. Crook, R., Jr., Proctor, R.J., 1992, The Hollywood and Santa Monica Fault and the Southern Boundary of the Transverse Ranges Province: in Pipkin, B., and Proctor, R.J. (eds.) Engineering Geology Practice in Southern California, Star Publishing Company, Belmont, California.
10. Department of the Navy, NAVFAC Design Manual 7.1, 1982, “Soil Mechanics,” Naval Facilities Engineering Command, May.
11. Department of the Navy, NAVFAC Design Manual 7.02, 1986, “Foundations and Earth Structures,” Naval Facilities Engineering Command, September.
12. Dibblee, T.W., 1991, Geologic Map of the Hollywood and Burbank (South ½) 7.5-Minute Quadrangles, Map No DF-30, map scale 1: 24,000.
13. Dolan, J.F., Sieh, K., Rockwell, T.K., Guptill, P., and Miller, G., 1997, Active Tectonics, Paleoseismology, and Seismic Hazards of the Hollywood Fault, Northern Los Angeles Basin, California, GSA Bulletin, v. 109: no 12, p1595-1616.
14. Hart, E.W. and Bryant, W.A., 1999 (updated 2005), Fault Rupture Zones in California, Division of Mines and Geology, Special Publication 42, 25pp.

REFERENCES - continued

15. Leighton and Associates, Inc. (1990), Technical Appendix to the Safety Element of the Los Angeles County General Plan: Hazard Reduction in Los Angeles County.
16. National Flood Insurance Rate Program, 2008, Los Angeles County and Incorporated Areas, Map #06037C1605F.
17. Seed, H.B. , Idriss, I.M., and Arango, I., 1983, Evaluation of Liquefaction Potential Using Field Performance Data, Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, vol. 109, no. 3, pp. 458-482.
18. Southern California Earthquake Center, 1999, "Recommended Procedures for Implementation of DMG Special Publication 117 - Guidelines for Analyzing and Mitigating Liquefaction in California," March.
19. Tinsley, J.C., Youd, T.L, Perkins, D.M., and Chen, A.T.F., 1985, Evaluating Liquefaction Potential: in Evaluating Earthquake Hazards in the Los Angeles Region-An earth Science Perspective, U.S. Geological Survey Professional Paper 1360, edited by J.I. Ziony, U.S. Government Printing Office, pp. 263-315.
20. Tokimatsu, K., and Yoshimi, Y., 1983, Empirical Correlation of Soil Liquefaction Based on SPT N-Value and Fines Content, Soils and Foundations, Japanese Society of Soil Mechanics and Foundation Engineering, vol. 23, no. 4, pp. 56-74.
21. Tokimatsu, K. and Seed, H. B., 1987, "Evaluation of Settlements in Sands Due to Earthquake Shaking," Journal of Geotechnical Engineering, ASCE, Vol. 113, No. 8, August.
22. United States Geological Survey, 2011, U.S.G.S. Ground Motion Parameter Calculator (Version 5.0.9a). http://earthquake.usgs.gov/hazards/designmaps/.
23. Youd, T.L., Hansen, C.M., and Bartlett, S.F., 2002, "Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement", Journal of Geotechnical Engineering, Vol. 128, No. 12, December.



SCALE 1:12000

0.0 0.1 0.2 0.3 0.4 0.5 0.6

MILES

0 1000 2000 3000

FEET

N

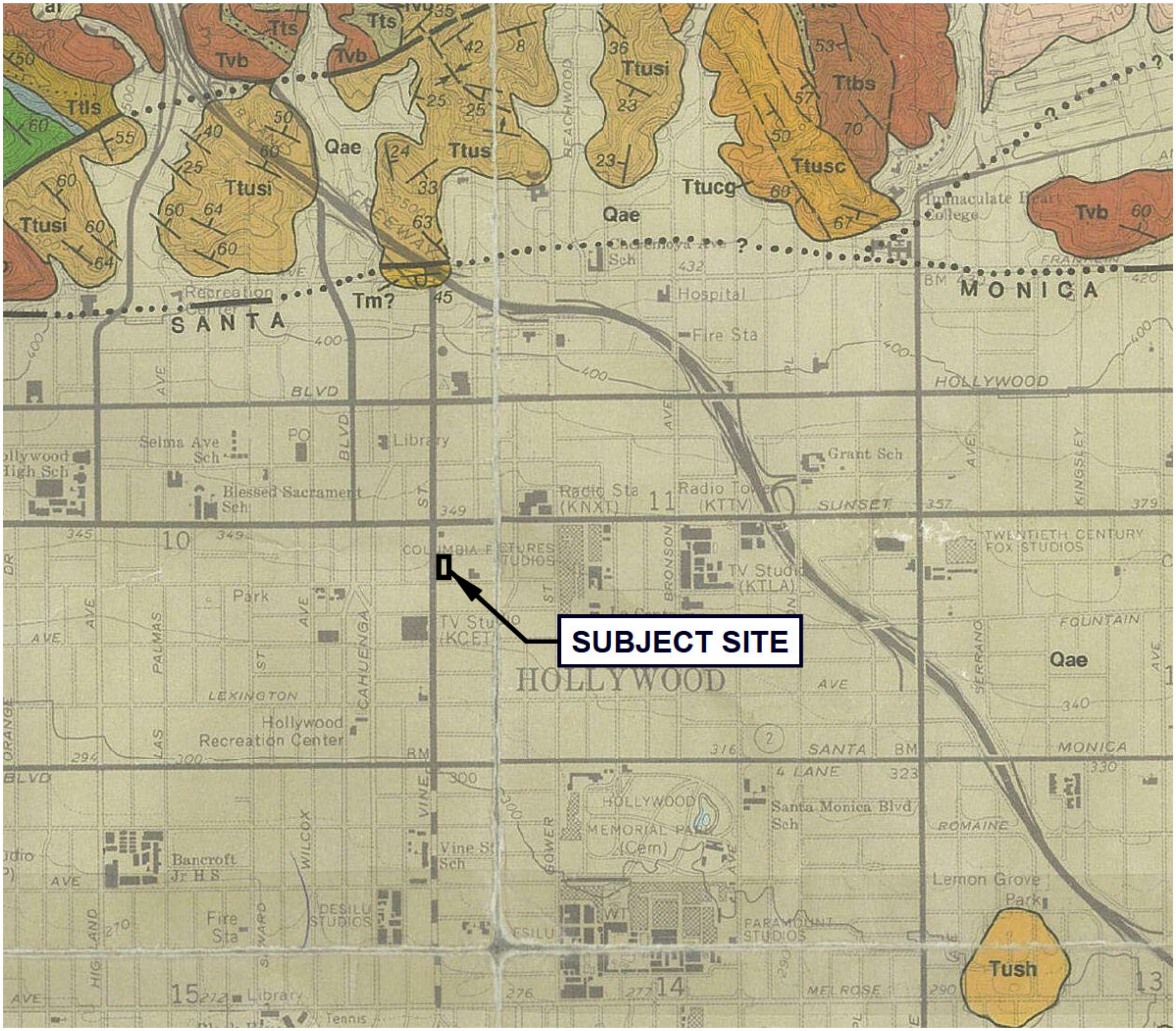
REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, HOLLYWOOD, CA QUADRANGLE

VICINITY MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

TOOLEY INTERESTS, LLC

FILE NO. 21225



SCALE IN FEET

0 500 1000 2000

LEGEND

Qae: Older Surficial Sediments - Elevated and Dissected Alluvium consisting of clay, sand and gravel

Tush: Unnamed Shale - Gray to light brown, thin-bedded silty clay shale, soft and crumbly

Tm: Monterey Formation - White weathering, thin bedded, platy silicous shale

Ttusi: Upper Topanga Formation - Mostly gray micaceous clay shale or claystone, crumbly where weathered

Ttusc: Upper Topanga Formation - Light gray massive sandstone, with pebble-cobble conglomerate

Ttucg: Upper Topanga Formation - Light to medium gray, from coarse sandstone to cobble conglomerate

—••••? Fault - dashed where indefinite or inferred, dotted where concealed, queried where existence is doubtful

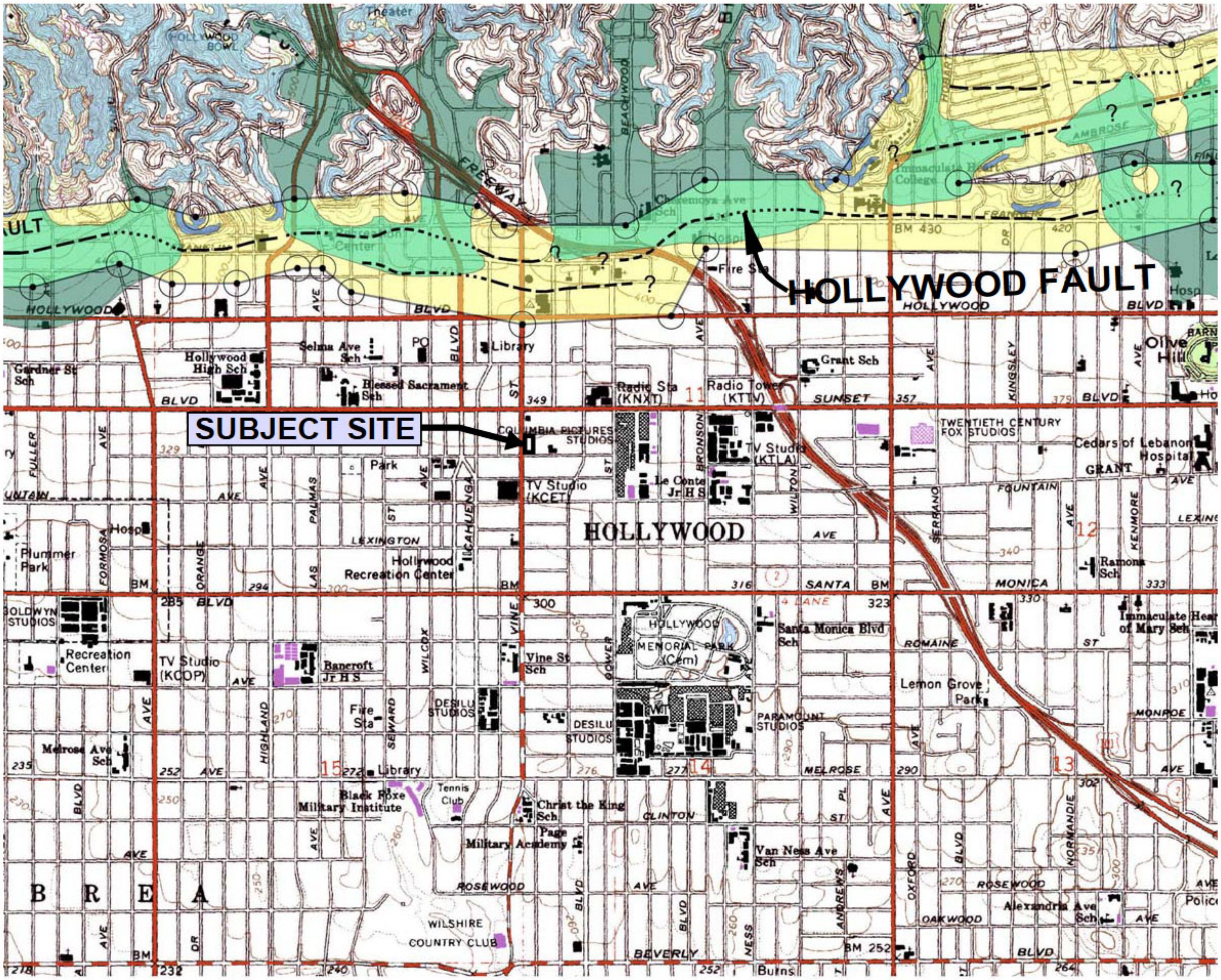
REFERENCE: DIBBLEE, T.W., (1991), GEOLOGIC MAP OF THE HOLLYWOOD AND BURBANK (SOUTH 1/2) QUADRANGLES, MAP #DF-30

LOCAL GEOLOGIC MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

TOOLEY INTERESTS, LLC

FILE NO. 21225



Scale 1: 24000

1 0.5 0 1 2 MILES

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

1 0.5 0 1 2 KILOMETERS

1000 500 0 1000 2000 METERS

Contour Interval 20 Feet

Earthquake Fault Zones
Alquist-Priolo Earthquake Fault Zone

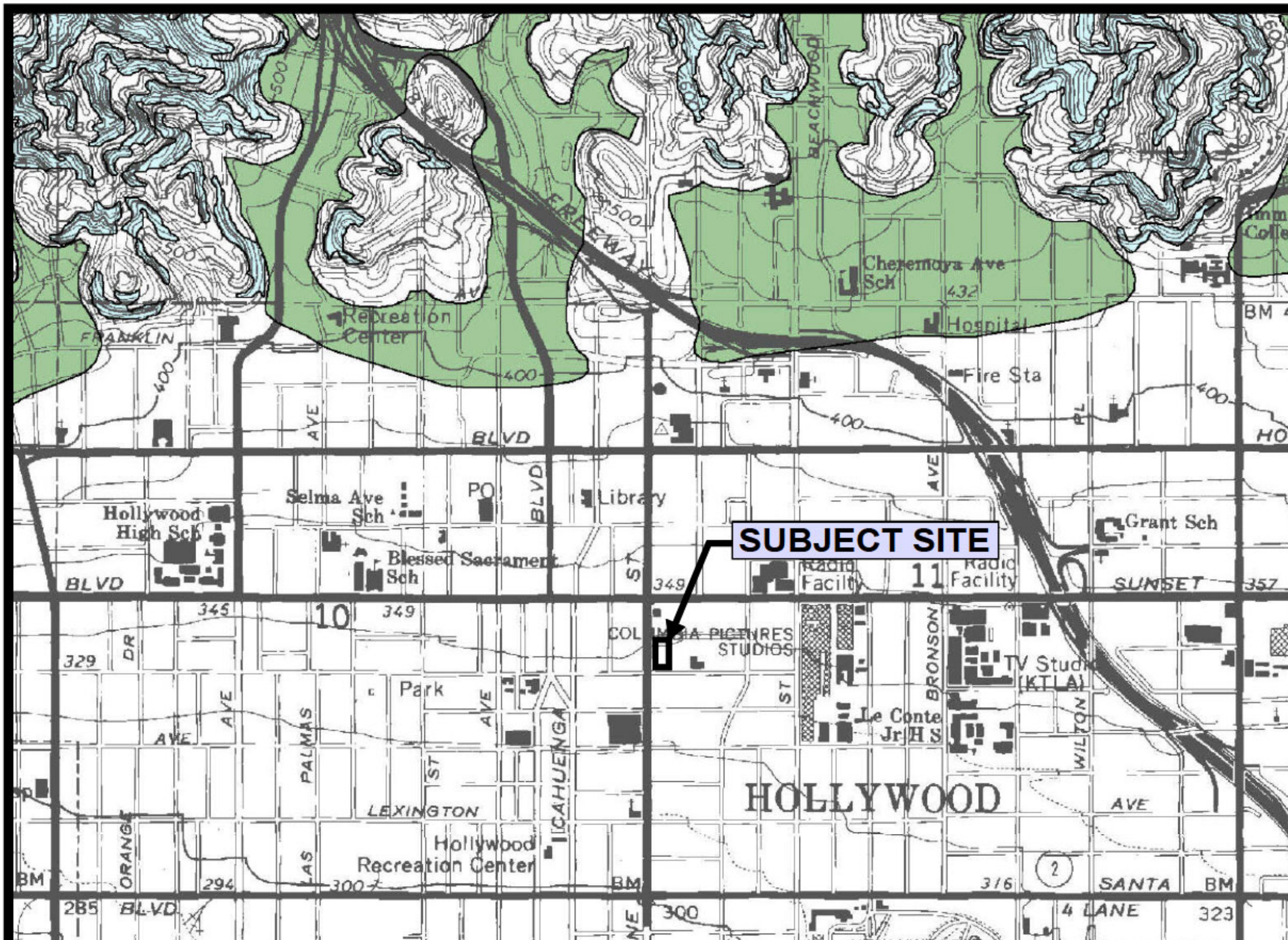
REFERENCE: EARTHQUAKE ZONES OF REQUIRED INVESTIGATION, HOLLYWOOD QUADRANGLE, CALIFORNIA GEOLOGICAAL SURVEY, NOVEMBER 6, 2014

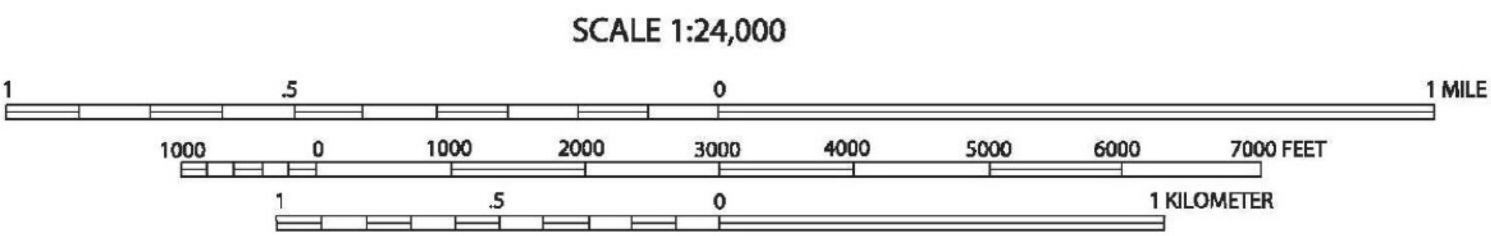
EARTHQUAKE FAULT ZONE MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

TOOLEY INTERESTS, LLC

FILE NO. 21225





LIQUEFACTION AREA

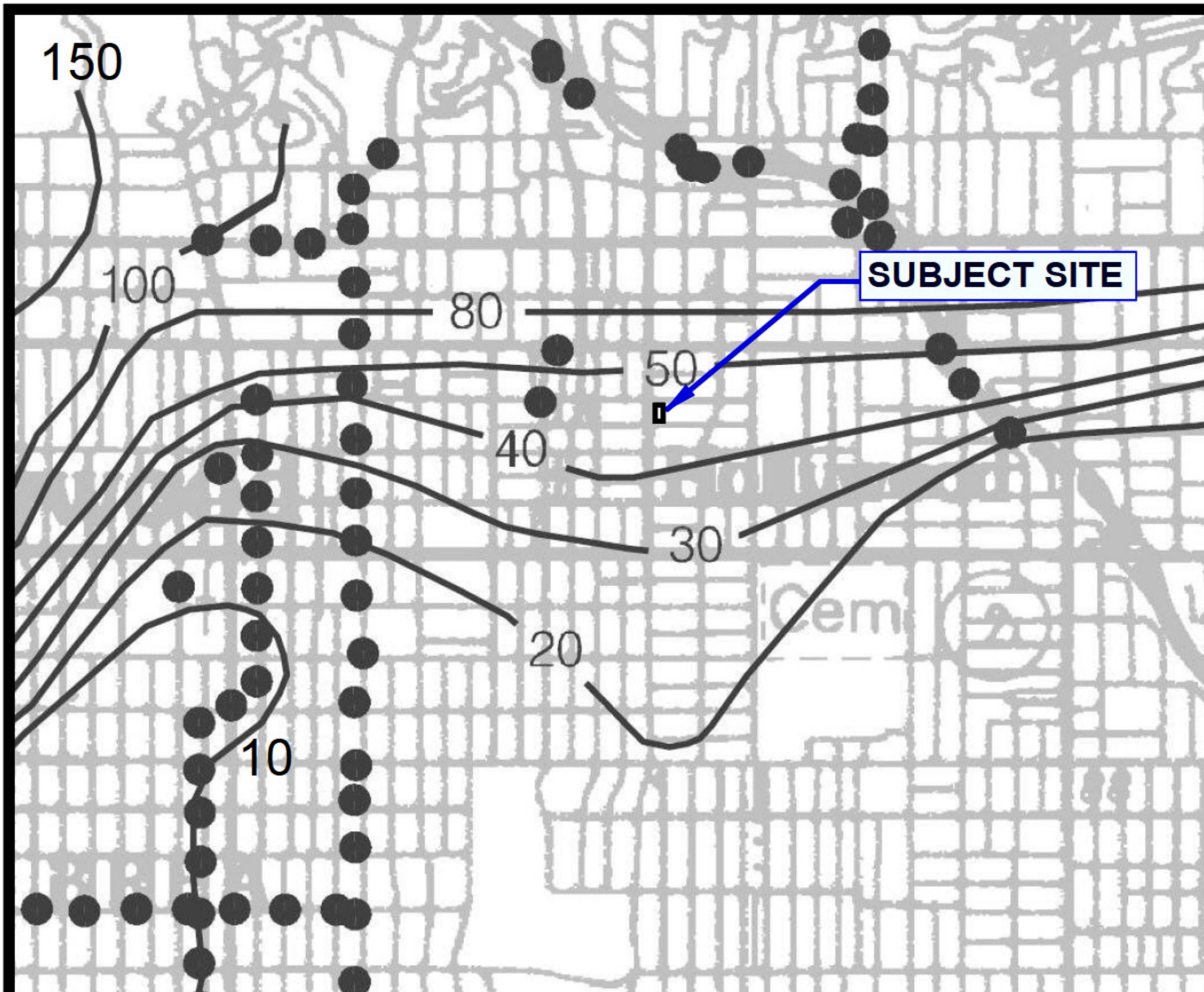
REFERENCE: SEISMIC HAZARD ZONES, HOLLYWOOD QUADRANGLE OFFICIAL MAP (CDMG, 1999)

SEISMIC HAZARD ZONE MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

TOOLEY INTERESTS, LLC

FILE NO. 21225



ONE MILE

SCALE

20 Depth to groundwater in feet

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

HISTORICALLY HIGHEST GROUNDWATER LEVELS

Geotechnologies, Inc.
Consulting Geotechnical Engineers

TOOLEY INTERESTS, LLC

FILE NO. 21225

BORING LOG NUMBER 1

GRT Beverly Hills, LLC **Date: 03/20/18** **Elevation: 344.5'**

File No. 21225 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		FILL: Silty Sand, grayish brown, slightly moist, medium dense to dense
				-		
				1 --		
				-		
				2 --		
				-		
				3 --		
				-	SM	Silty Sand, dark grayish brown, slightly moist, dense, fine grained
				4 --		
				-		
5	29	6.5	109.2	5 --		
				-		
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	31	9.9	107.6	10 --		
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	46	12.7	119.0	15 --		
	50/5"			-	SM/ML	Silty Sand to Sandy Silt, brown, moist, very dense to very stiff
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	39	10.9	119.5	20 --		
	50/6"			-	SM	Silty Sand, brown, slightly moist, very dense
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	27	7.2	120.8	25 --		
	50/6"			-		

BORING LOG NUMBER 1

GRT Beverly Hills, LLC

File No. 21225

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		NOTE: The stratification lines represent the approximate
				27 --		boundary between earth types; the transition may be gradual.
				-		
				28 --		Used 8-inch diameter Hollow-Stem Auger
				-		140-lb. Automatic Hammer, 30-inch drop
				29 --		Modified California Sampler used unless otherwise noted
				-		
30	63	7.8	125.1	30 --		
				-		
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
35	35	4.6	113.0	35 --		
	50/3"			-	SP	Sand, light brown, slightly moist, very dense
				36 --		
				-		
				37 --		
37.5	37	2.4	115.9	-		
	50/6"			38 --	SW	Gravelly Sand, brown, slightly moist, very dense, fine to coarse
				-		grained, with gravel
				39 --		
				-		
40	40	1.6	122.0	40 --		
	50/5"			-		
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
45	30	10.9	125.5	45 --		
	50/6"			-	SM	Silty Sand, dark brown, moist to very moist, very dense, with
				46 --		gravel
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-	SP	Sand, reddish brown, wet, very dense
50	37	16.1	114.5	50 --		
	50/4"			-		Total Depth 50 feet
						Water at 46 feet
						Fill to 3 feet

BORING LOG NUMBER 2

GRT Beverly Hills, LLC **Date: 03/20/18** **Elevation: 343.0'**

File No. 21225 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		8-inch Asphalt, No Base
				-		
				1 --		FILL: Sandy Silt to Silty Sand, medium to grayish brown, slightly moist, medium dense to stiff
				-		
				2 --		
				-		
				3 --		
				-		
				4 --		
				-		
5	15	6.8	SPT	5 --	SM	Silty Sand, medium brown, slightly moist, medium dense, fine grained
				-		
				6 --		
				-		
				7 --		
7.5	20	10.8	122.1	-		
				8 --		
				-		
				9 --		
				-		
10	17	8.2	SPT	10 --		
				-		
				11 --		
				-		
				12 --		
12.5	27	13.0	117.6	-		
				13 --	ML	Sandy Silt, brown, moist, medium firm to stiff medium stiff
				-		
				14 --		
				-		
15	16	10.3	SPT	15 --		
				-	SC	Clayey Sand, brown, slightly moist, dense
				16 --		
				-		
				17 --		
17.5	32	14.1	114.5	-		
				18 --	CL	Sandy Clay, brown, moist, stiff
				-		
				19 --		
				-		
20	21	14.9	SPT	20 --		
				-		
				21 --		
				-		
				22 --		
22.5	26	9.8	114.8	-		
				23 --	SM	Silty Sand, medium brown, slightly moist, dense
				-		
				24 --		
				-		
25	19	13.4	SPT	25 --		
				-	CL	Sandy Clay, medium brown, moist, stiff

BORING LOG NUMBER 2

GRT Beverly Hills, LLC

File No. 21225

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
27.5	45	24.0	98.5			- - - - - - - - - - - -
				28		Sandy Clay, medium brown, moist, stiff
				29		
30	22	14.4	SPT	30		
				31	SC	Clayey Sand, dark brown, moist, dense, medium to high plasticity
				32		
32.5	57	9.9	128.8			
				33	SM	Silty Sand, dark brown, slightly moist, very dense
				34		
35	25	9.8	SPT	35		
				36		
				37		
37.5	45	10.9	126.0			
				38		
				39		
40	28	10.7	SPT	40		
				41		
				42		
42.5	100/10"	4.8	118.9			
				43	SW	Gravelly Sand, dark brown, slightly moist, very dense
				44		
45	35	10.7	SPT	45		
				46	SC	Clayey Sand, dark brown, slightly moist, dense
				47		
47.5	18 50/6"	13.9	122.3			
				48	ML	Sandy Silt, dark brown, moist, very stiff
				49		
50	38	13.6	SPT	50		

BORING LOG NUMBER 2

GRT Beverly Hills, LLC

File No. 21225

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
52.5	21	15.5	113.2	-		
	50/6"			53 --	SP	Sand, dark brown, wet, very dense
				-		
				54 --		
				-		
55	27	14.3	SPT	55 --		
				-		
				56 --		
				-		
				57 --		
57.5	48	13.7	120.7	-		
				58 --		
				-		
				59 --		
				-		
60	29	12.9	SPT	60 --		
				-		
				61 --		
				-		
				62 --		
62.5	24	11.2	128.0	-		
	50/6"			63 --		
				-		
				64 --		
				-		
65	25	14.5	SPT	65 --		
				-	ML	Sandy Silt, dark brown, moist, very stiff
				66 --		
				-		
				67 --		
67.5	57	14.2	118.6	-		
				68 --		
				-		
				69 --		
				-		
70	43	13.8	SPT	70 --		
				-	SM	Silty Sand, dark brown, wet, very dense
				71 --		
				-		
				72 --		
72.5	40	14.7	109.1	-		
	50/5"			73 --	SP	Sand, dark brown, wet, very dense, fine grained
				-		
				74 --		
				-		
75	55	14.7	SPT	75 --		
				-		

BORING LOG NUMBER 2

GRT Beverly Hills, LLC

File No. 21225

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				76 --		
				-		
				77 --		
77.5	40	18.1	114.4	-		
	50/5"			78 --	SM	Silty Sand, brown, wet, very dense
				-		
				79 --		
				-	SP	Sand, brown, wet, very dense
80	42	13.1	SPT	80 --		
	50/5"			-		Total Depth 80 feet
				81 --		Water at 49 feet
				-		Fill to 4½ feet
				82 --		
				-		
				83 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				84 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				85 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				86 --		
				-		SPT=Standard Penetration Test
				87 --		
				-		
				88 --		
				-		
				89 --		
				-		
				90 --		
				-		
				91 --		
				-		
				92 --		
				-		
				93 --		
				-		
				94 --		
				-		
				95 --		
				-		
				96 --		
				-		
				97 --		
				-		
				98 --		
				-		
				99 --		
				-		
				100 --		
				-		

BORING LOG NUMBER 3

GRT Beverly Hills, LLC **Date: 03/19/18** **Elevation: 339.5'**

File No. 21225 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		FILL: Sandy Silt, dark grayish brown, slightly moist, stiff
				-		
				1 --		
				-		
				2 --		
				-		
				3 --		
				-	SM	Silty Sand, dark brown, slightly moist, medium dense to dense
				4 --		
				-		
5	15	7.6	110.4	5 --		
				-		
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	23	17.6	109.5	10 --		
				-	CL	Sandy Clay, dark brown, slightly moist, stiff
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	40	12.8	118.6	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	27	12.7	117.9	20 --		
				-		Sandy Clay, medium to dark brown, moist, stiff
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	56	19.2	107.1	25 --		
				-		

BORING LOG NUMBER 3

GRT Beverly Hills, LLC

File No. 21225

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		NOTE: The stratification lines represent the approximate
				27 --		boundary between earth types; the transition may be gradual.
				-		
				28 --		Used 8-inch diameter Hollow-Stem Auger
				-		140-lb. Automatic Hammer, 30-inch drop
				29 --		Modified California Sampler used unless otherwise noted
				-		
30	31	10.3	125.7	30 --		
	50/6"			-	SM	Silty Sand, medium brown, slightly moist, very dense, with
				31 --		gravel
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
35	30	9.8	130.0	35 --		
	50/6"			-		
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
40	100/10"	7.0	124.8	40 --		
				-	SP	Sand, dark brown, slightly moist, very dense
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
45	48	14.4	120.8	45 --		
				-	SM	Silty Sand, dark brown, very moist to wet, dense
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
50	110	14.5	122.5	50 --		
				-		Total Depth 50 feet
						Water at 48 feet
						Fill to 3 feet

BORING LOG NUMBER 4

GRT Beverly Hills, LLC **Date: 03/21/18** **Elevation: 343.0'**

File No. 21225 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		FILL: Silty Sand, grayish brown, slightly moist, medium dense
				-		
				1 --		
				-		
				2 --		
				-		
				3 --		
				-	SM	Silty Sand, dark brown, moist, medium dense to dense, fine grained
				4 --		
				-		
5	13	7.5	110.1	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	18	11.3	118.7	-		
				8 --		
				-		
				9 --		
				-		
10	21	7.5	115.1	10 --		
				-		Silty Sand, dark brown, moist, dense, fine grained
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	27	13.5	117.5	15 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark brown, moist, dense to stiff, fine grained
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	41	14.9	117.3	20 --		
				-	ML	Sandy Silt, dark brown, moist, very stiff, fine grained
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	43	9.7	127.4	25 --		
				-	SM	Silty Sand, dark brown, moist, dense, fine grained

BORING LOG NUMBER 4

GRT Beverly Hills, LLC

File No. 21225

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
30	38	9.9	126.2	30 --		
				-		
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
35	30	2.6	115.9	35 --		
	50/5"			-	SW	Gravelly Sand, dark brown, slightly moist, very dense, fine to coarse grained, with gravel
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
40	45	5.0	131.6	40 --		
	50/4"			-		with gravel and cobbles
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
45	78	11.5	125.9	45 --		
				-	SM	Silty Sand, dark brown, moist, very dense, fine grained
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
50	37	9.6	127.3	50 --		
	50/5"			-	SM/ML	Clayey Sand to Clayey Silt, reddish brown, moist, very dense to stiff, fine to coarse grained, with gravel

BORING LOG NUMBER 4

GRT Beverly Hills, LLC

File No. 21225

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
				-		
				53 --		
				-		
				54 --		
				-		
55	32	11.0	127.7	55 --		
	50/5"			-	SC	Clayey Sand, reddish brown, moist, very dense, fine grained
				56 --		
				-		
				57 --		
				-		
				58 --		
				-		
				59 --		
				-		
60	50	No Recovery		60 --		
	50/2"			-		
				61 --		
				-		
				62 --		
				-		
				63 --		
				-		
				64 --		
				-		
65	45	17.2	117.1	65 --		
				-	CL	Silty Clay, dark brown, moist, very stiff
				66 --		
				-		
				67 --		
				-		
				68 --		
				-		
				69 --		
				-		
70	50	13.4	125.5	70 --		
	50/3"			-	SM	Silty Sand, dark brown, moist, very dense, fine grained
				71 --		
				-		
				72 --		
				-		
				73 --		
				-		
				74 --		
				-		
75	80	14.4	115.2	75 --		
				-		

BORING LOG NUMBER 4

GRT Beverly Hills, LLC

File No. 21225

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				76 --		
				-		
				77 --		
				-		
				78 --		
				-		
				79 --		
				-		
80	74	15.4	116.2	80 --		
				-	SP	Sand, dark brown, wet, very dense, fine to medium grained
				81 --		
				-		
				82 --		
				-		
				83 --		
				-		
				84 --		
				-		
85	100/10"	8.4	128.3	85 --		
				-	SW	Sand, dark brown, wet, very dense, fine to coarse grained, minor gravel
				86 --		
				-		
				87 --		
				-		
				88 --		
				-		
				89 --		
				-		
90	40	17.4	114.5	90 --		
	50/5"			-	SM	Silty Sand, dark brown, wet, very dense, fine grained
				91 --		
				-		
				92 --		
				-		
				93 --		
				-		
				94 --		
				-		
95	75	18.7	112.6	95 --		
				-	SM	Silty Sand, dark brown, wet, very dense, fine grained
				96 --		
				-		
				97 --		
				-		
				98 --		
				-		
				99 --		
				-		
100	100/8"	17.8	111.1	100 --		
				-		

BORING LOG NUMBER 4

GRT Beverly Hills, LLC

File No. 21225

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				101 --		
				102 --		
				103 --		
				104 --		
105	100/10"	16.4	119.5	105 --		Silty Sand, dark brown, wet, very dense, fine grained, with cobbles
				106 --		
				107 --		
				108 --		
				109 --		
110	100/6"	15.7	116.9	110 --	SM/ML	Silty Sand to Sandy Silt, dark brown, moist, very dense to very stiff, fine grained
				111 --		
				112 --		
				113 --		
				114 --		
115	100/7"	10.9	126.3	115 --	SM	Silty Sand , dark brown, moist, very dense, fine to medium grained, minor gravel
				116 --		
				117 --		
				118 --		
				119 --		
120	100/11"	14.6	122.5	120 --		Total Depth 120 feet Water at 49½ feet Fill to 3 feet
				121 --		
				122 --		
				123 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				124 --		
				125 --		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted

BORING LOG NUMBER 5

GRT Beverly Hills, LLC | **Date: 03/19/18** | **Elevation: 343.5'**

File No. 21225 | **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		FILL: Silty Sand, grayish brown, slightly moist, medium dense
				-		
				1 --		
				-		
				2 --		
				-		
				3 --		
				-	SM	Silty Sand, dark brown, slightly moist, medium dense to dense, fine grained
				4 --		
				-		
5	21	8.7	116.7	5 --		
				-		
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	27	9.5	119.6	10 --		
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	54	12.5	113.6	15 --		
				-	SC	Clayey Sand, medium brown, moist, dense
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	52	13.9	116.1	20 --		
				-	CL	Sandy Clay, dark olive brown, moist, very stiff
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	65	14.4	117.2	25 --		
				-		

BORING LOG NUMBER 5

GRT Beverly Hills, LLC

File No. 21225

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				- 26 -- - 27 -- - 28 -- - 29 -- -		
30	77	11.0	127.6	30 --		
				- 31 -- - 32 -- - 33 -- - 34 -- -	SM	Silty Sand, dark brown, slightly moist, very dense
35	35	11.2	118.7	35 -- - 36 -- - 37 -- - 38 -- - 39 -- -		
40	37 50/6"	3.8	120.3	40 -- - 41 -- - 42 -- - 43 -- - 44 -- -	SW	Gravelly Sand, medium brown, slightly moist, very dense, fine to coarse grained, with gravel
45	54 50/6"	10.7	129.7	45 -- - 46 -- - 47 -- - 48 -- - 49 -- -	SM	Silty Sand, medium brown, moist, very dense, with gravel
50	38 50/5"	14.6	121.3	50 -- -	SP	Sand, dark brown, wet, very dense

BORING LOG NUMBER 5

GRT Beverly Hills, LLC

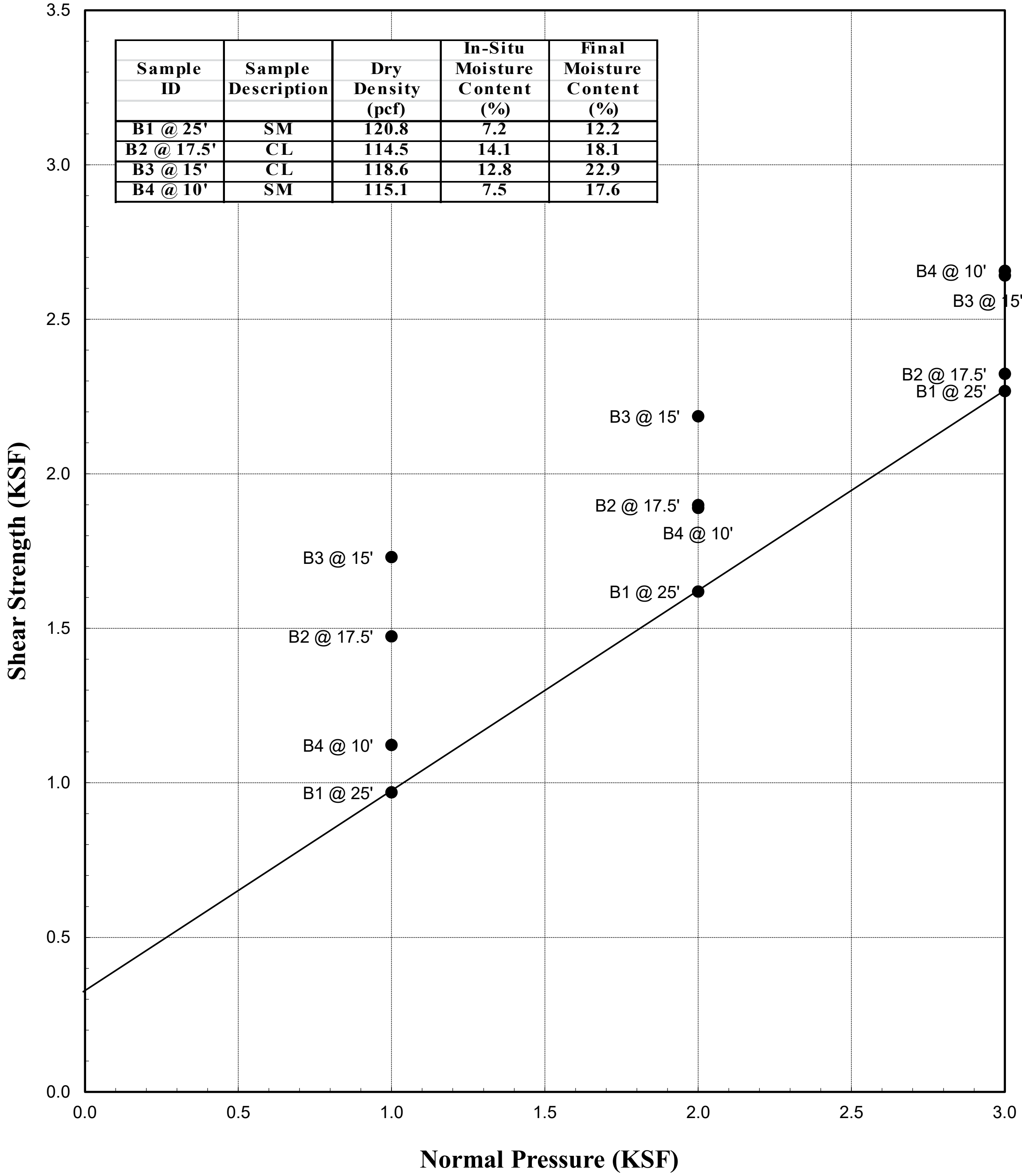
File No. 21225

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				51 --		
				52 --		
				53 --		
				54 --		
55	40 50/4"	16.6	112.2	55 --		Sand, dark brown, wet, very dense, very coarse grained, with gravel
				56 --		
				57 --		
				58 --		
				59 --	SM	Silty Sand, very dark brown, wet, very dense
60	32 50/6"	18.8	109.6	60 --		Total Depth 60 feet Water at 48½ feet Fill to 3 feet
				61 --		
				62 --		
				63 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				64 --		
				65 --		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				66 --		
				67 --		
				68 --		
				69 --		
				70 --		
				71 --		
				72 --		
				73 --		
				74 --		
				75 --		

Saturated Shear

Sample ID	Sample Description	Dry Density (pcf)	In-Situ Moisture Content (%)	Final Moisture Content (%)
B1 @ 25'	SM	120.8	7.2	12.2
B2 @ 17.5'	CL	114.5	14.1	18.1
B3 @ 15'	CL	118.6	12.8	22.9
B4 @ 10'	SM	115.1	7.5	17.6



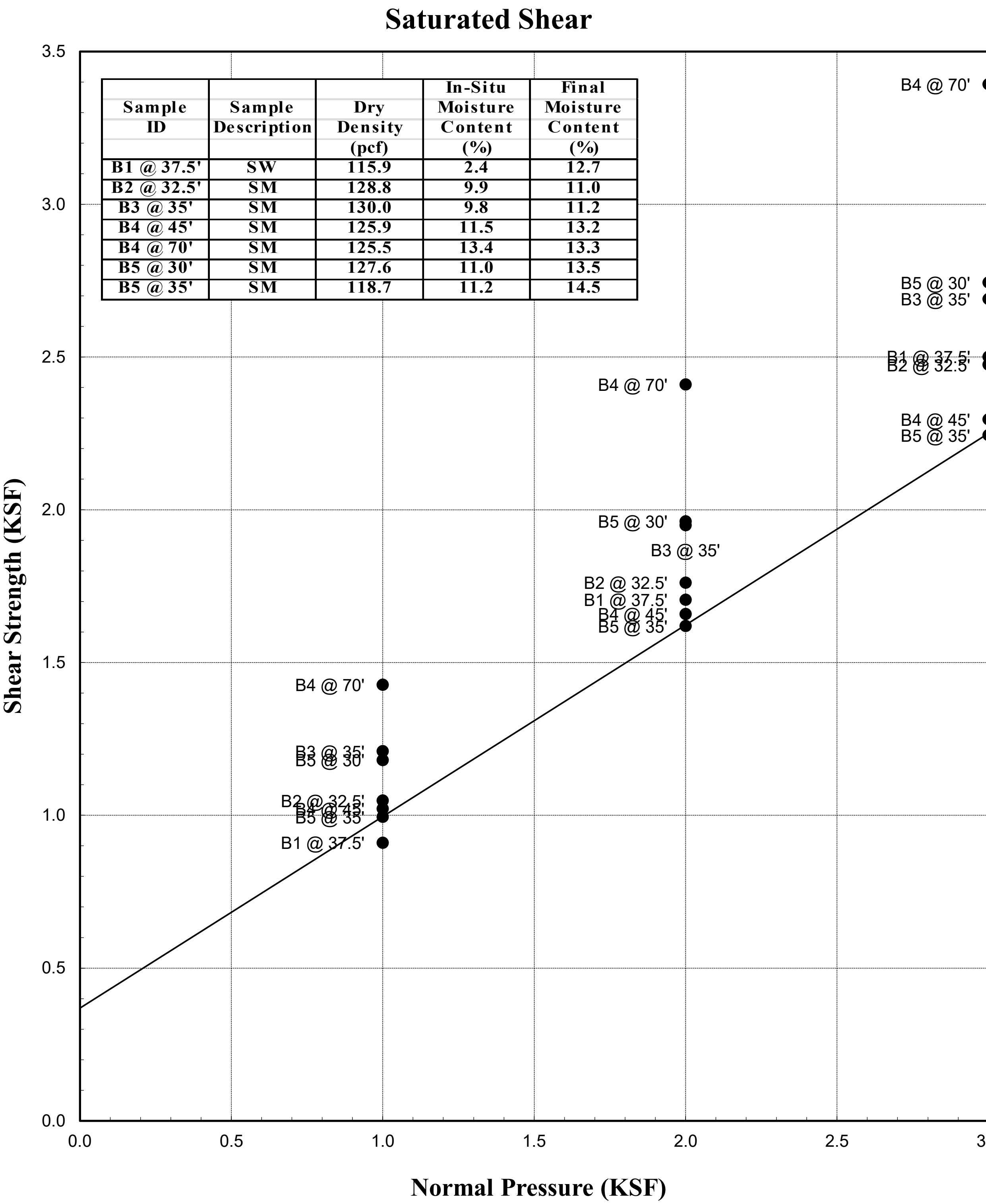
ϕ: 33.0 degrees
c: 320.0 psf

SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

PROJECT: GRT BEVERLY HILLS LLC	
FILE NO.: 21225	PLATE: B-1

SHEAR TEST DIAGRAM



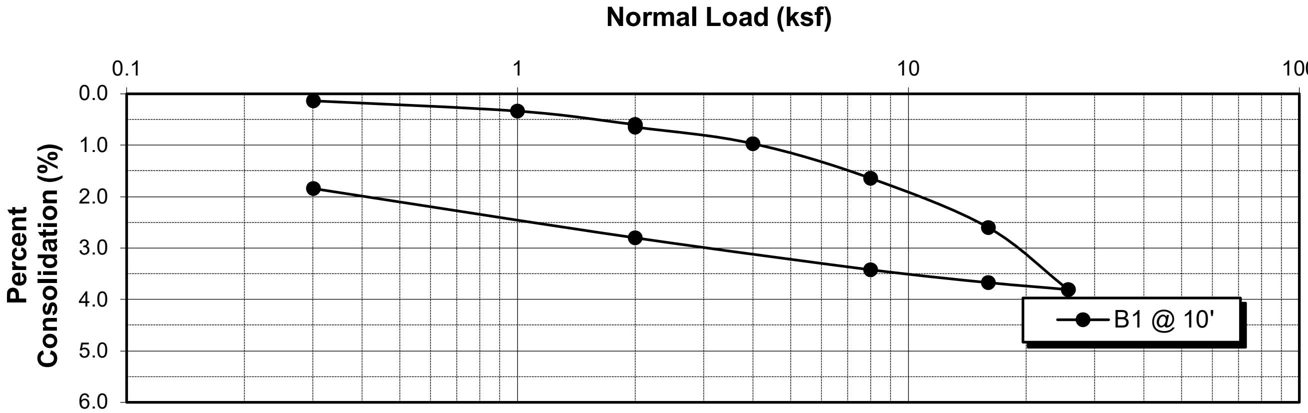
Sample ID	Sample Description	Dry Density (pcf)	In-Situ Moisture Content (%)	Final Moisture Content (%)
B1 @ 37.5'	SW	115.9	2.4	12.7
B2 @ 32.5'	SM	128.8	9.9	11.0
B3 @ 35'	SM	130.0	9.8	11.2
B4 @ 45'	SM	125.9	11.5	13.2
B4 @ 70'	SM	125.5	13.4	13.3
B5 @ 30'	SM	127.6	11.0	13.5
B5 @ 35'	SM	118.7	11.2	14.5

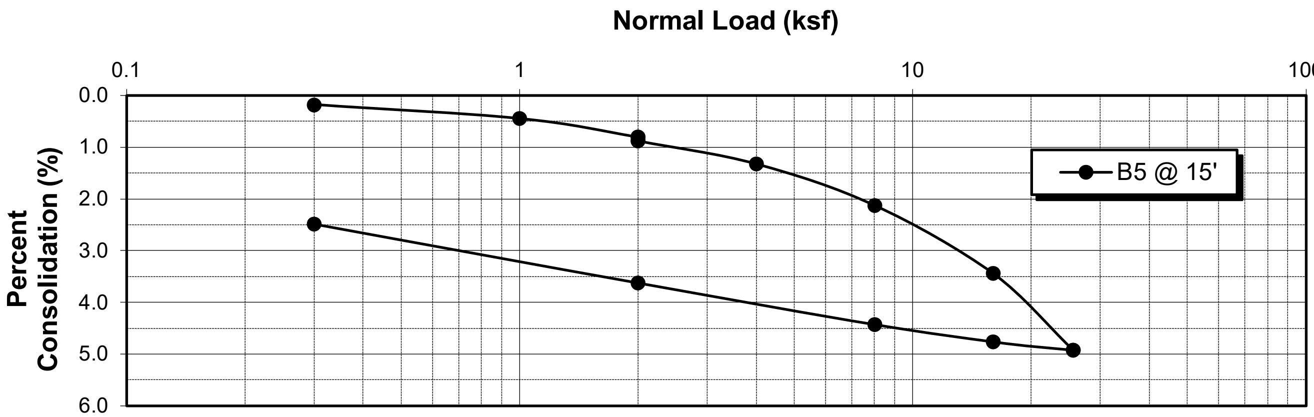
ϕ: 32.0 degrees
c: 370.0 psf

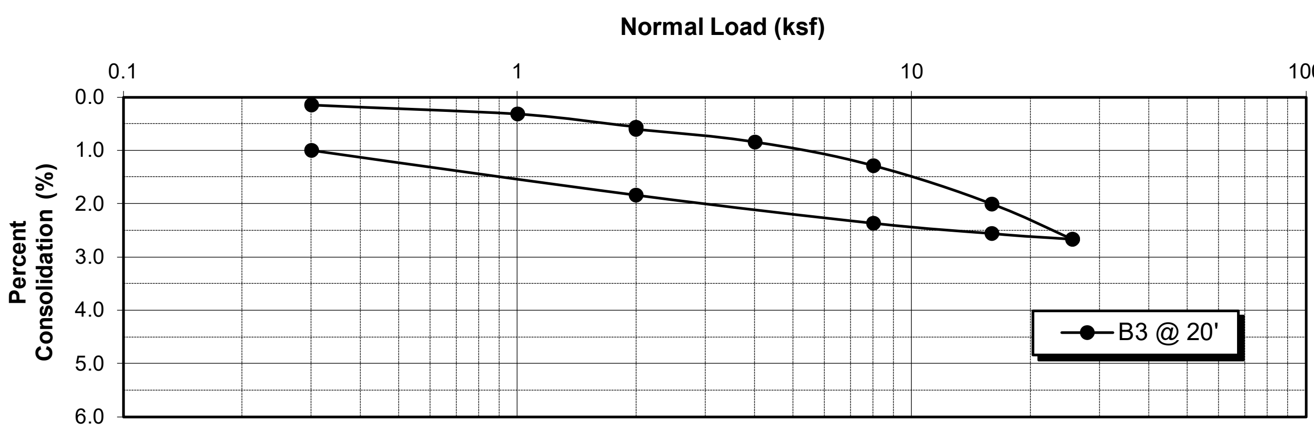
Geotechnologies, Inc.
Consulting Geotechnical Engineers

PROJECT: GRT BEVERLY HILLS LLC

FILE NO.: 21225 | PLATE: B-2







Water added at 2 KSF

CONSOLIDATION

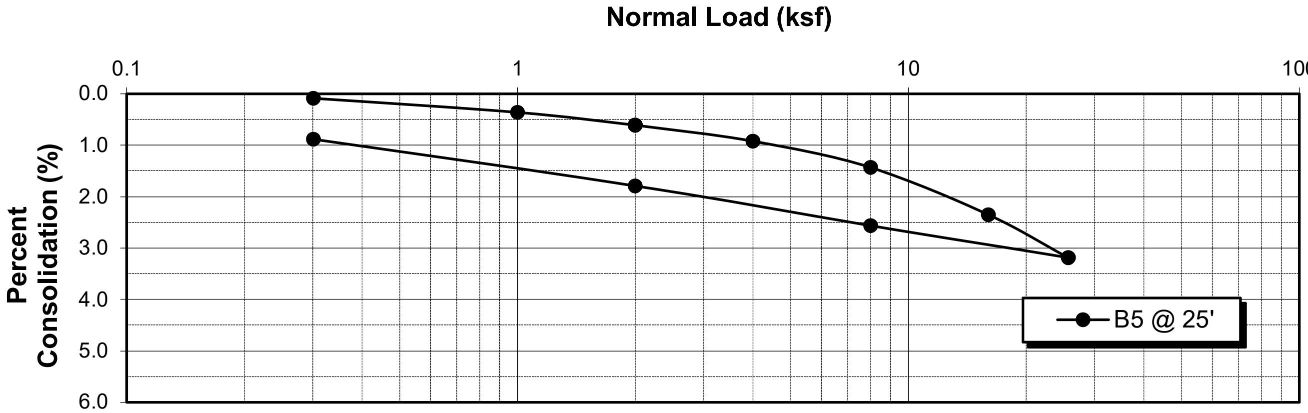
Geotechnologies, Inc.

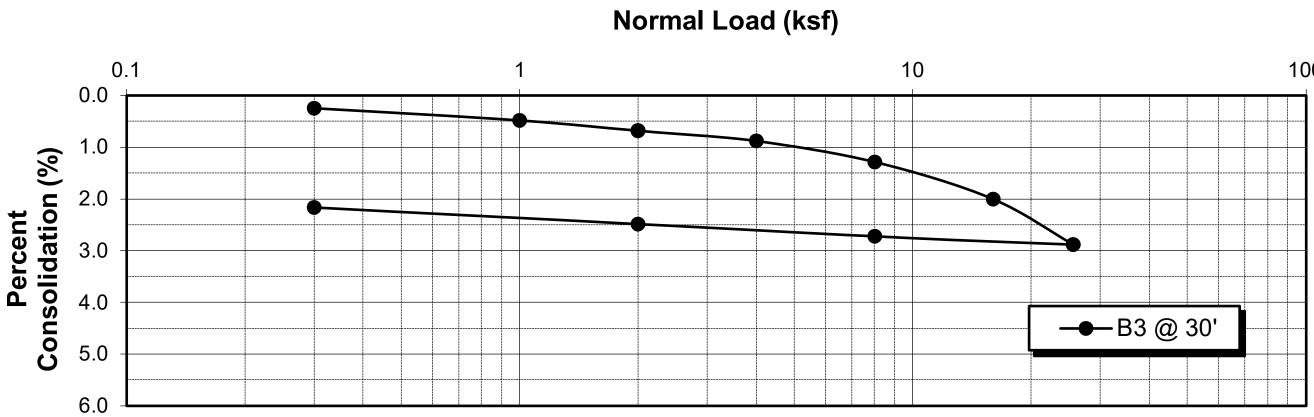
CONSULTING GEOTECHNICAL ENGINEERS

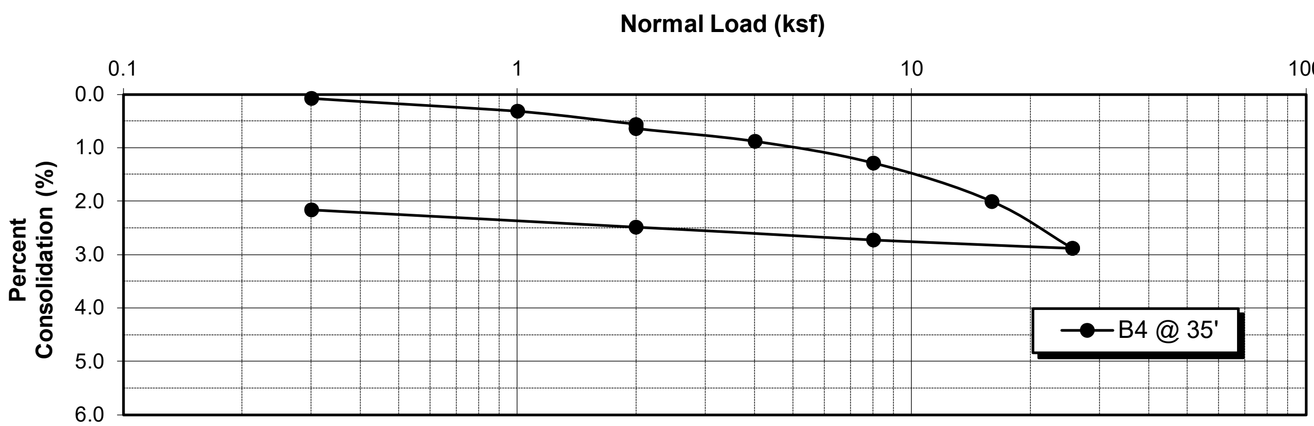
PROJECT: GRT BEVERLY HILLS, LLC

FILE NO. 21225

PLATE: C-1







Water added at 2 KSF

CONSOLIDATION

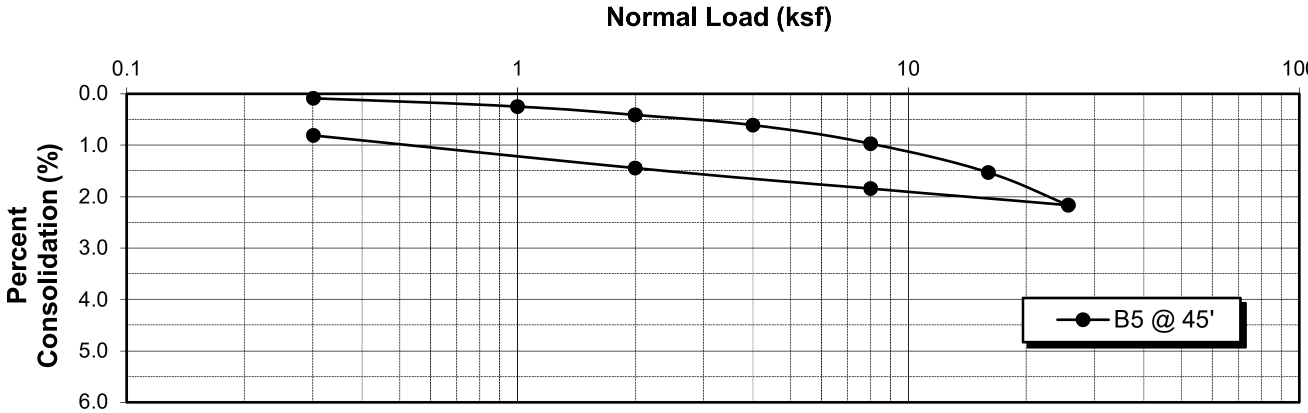
Geotechnologies, Inc.

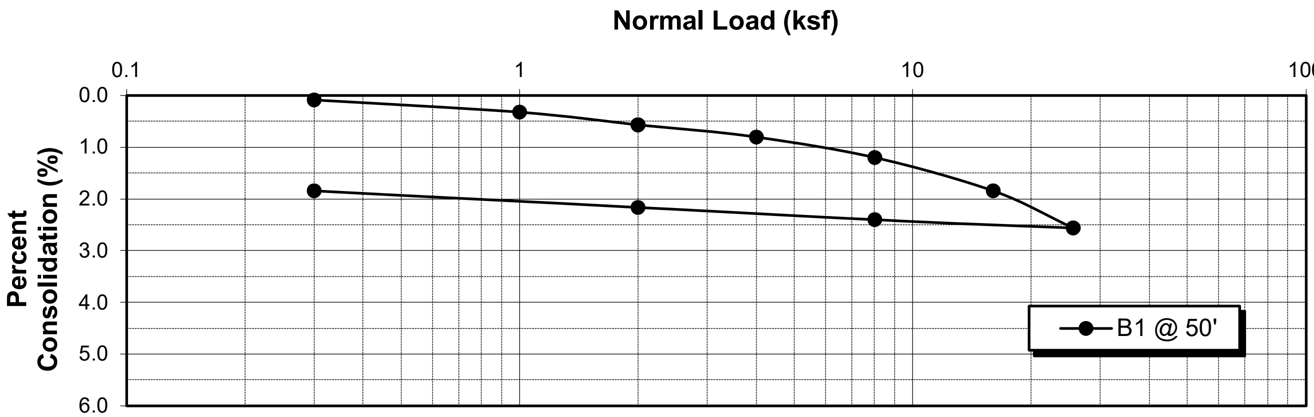
CONSULTING GEOTECHNICAL ENGINEERS

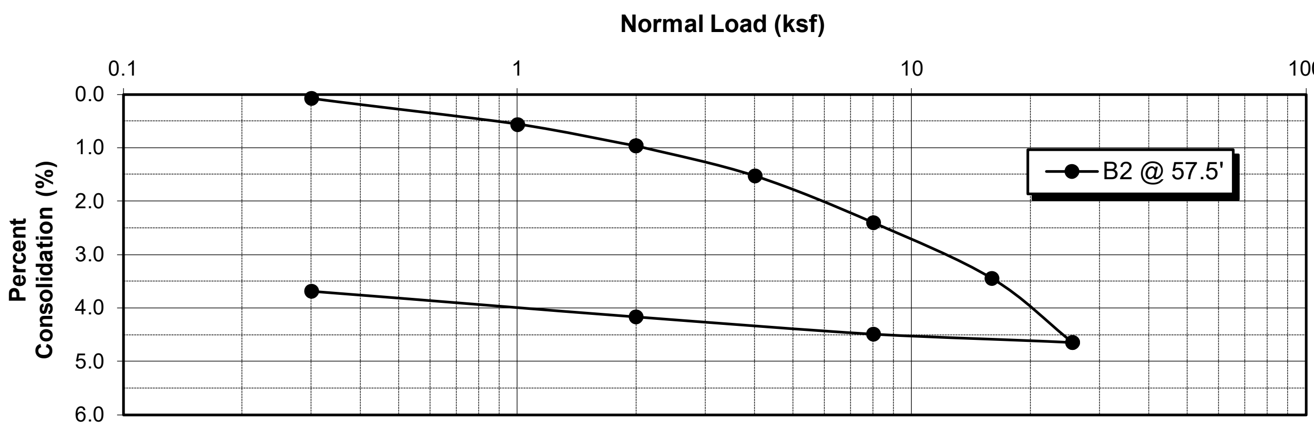
PROJECT: GRT BEVERLY HILLS, LLC

FILE NO. 21225

PLATE: C-2







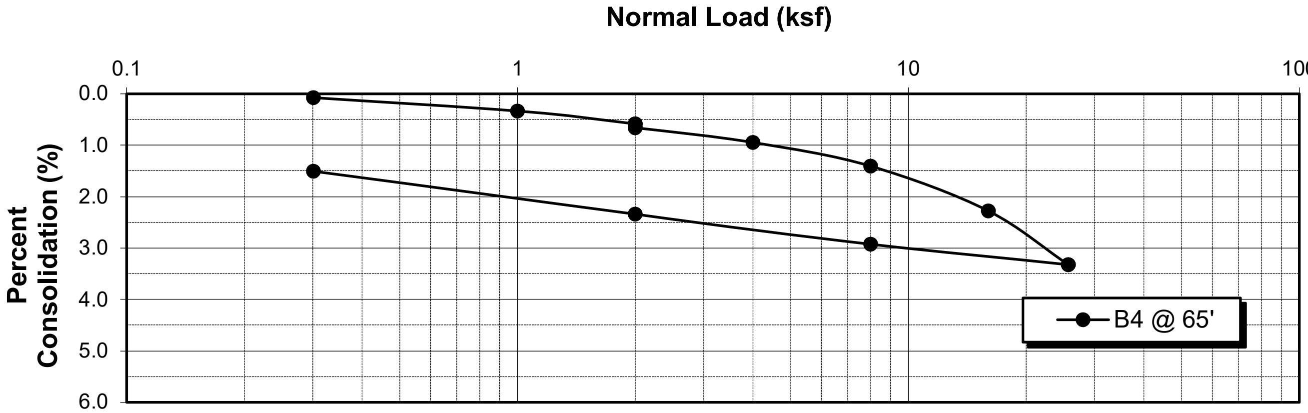
Water added at 2 KSF

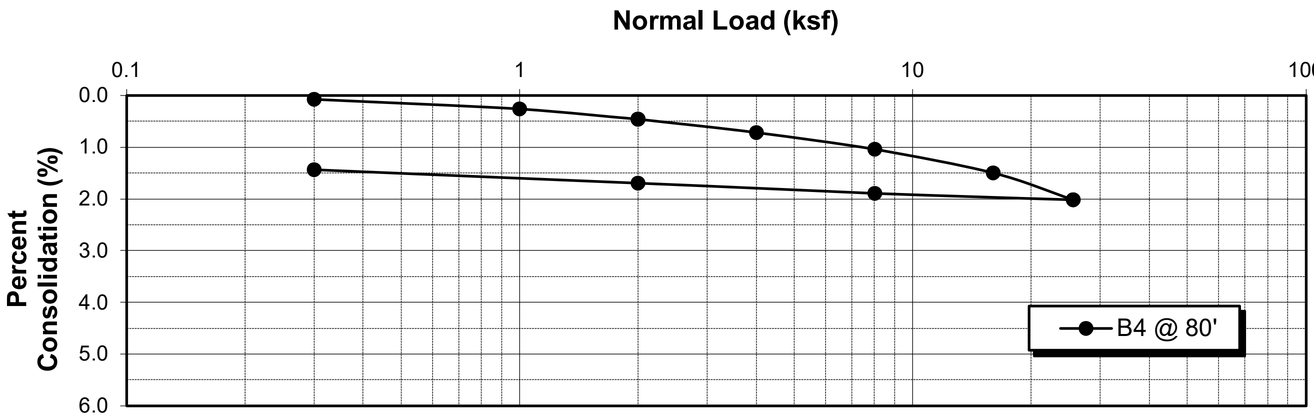
CONSOLIDATION

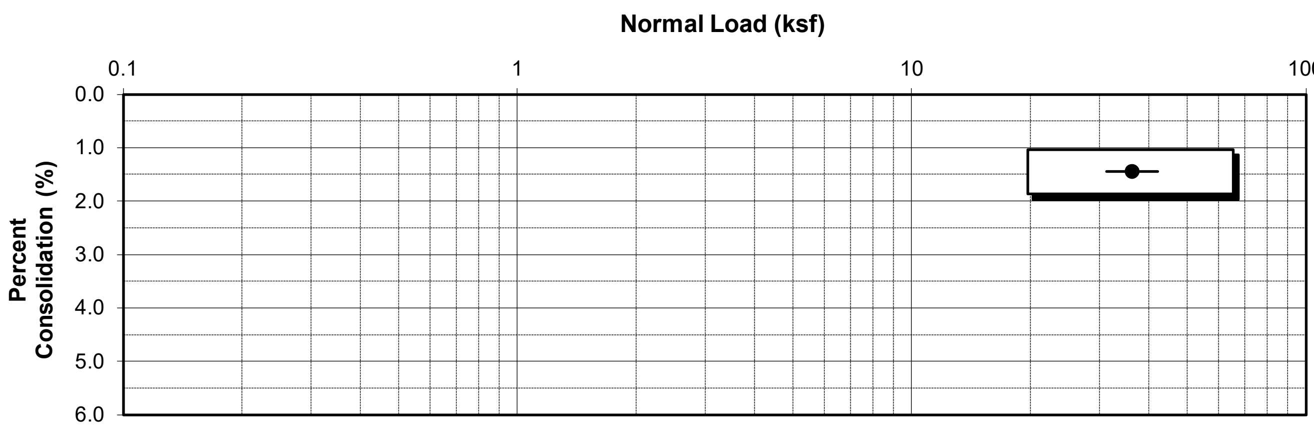
Geotechnologies, Inc.
CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: GRT BEVERLY HILLS, LLC

FILE NO. 21225 | PLATE: C-3







Water added at 2 KSF

CONSOLIDATION

Geotechnologies, Inc.

CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: GRT BEVERLY HILLS, LLC	
FILE NO. 21225	PLATE: C-4

Geotechnologies, Inc.
Consulting Geotechnical Engineers
439 Western Avenue
Glendale, California 91201-2837
818.240.9600 • Fax 818.240.9675

COMPACTION/EXPANSION/SULFATE DATA SHEET

ASTM D-1557

Sample	**B1 @ 1’ – 5’**	**B2 @ 1’ – 5’**	**B4 @ 1’ – 5’**
Soil Type	SM	SM	SM
Maximum Density (pcf)	132.0	129.0	131.0
Optimum Moisture Content (percent)	8.5	9.5	9.0
Percent finer than 0.005mm (percent)	<15%	<15%	<15%

EXPANSION INDEX

Sample	**B1 @ 1’ – 5’**	**B2 @ 1’ – 5’**	**B4 @ 1’ – 5’**
Soil Type	SM	SM	SM
Expansion Index – UBC Standard 18-2	50	43	17
Expansion Characteristic	Moderate	Low	Very Low

SULFATE CONTENT

Sample	**B1 @ 1’ – 5’**	**B2 @ 1’ – 5’**	**B4 @ 1’ – 5’**
Sulfate Content (ppm)	<250	<250	<250

PLATE D

GeoPentech

May 23, 2018

Project No. 18025A

Mr. Stan Tang
Geotechnologies, Inc.
439 Western Avenue
Glendale, California 91201

**SUBJECT: DOWNHOLE SEISMIC TEST RESULTS
BORING NUMBER 4
1400-1440 VINE STREET
HOLLYWOOD, CALIFORNIA**

Dear Mr. Tang,

Per your request and in accordance with the provisions of our proposal, dated April 12, 2018, we performed downhole seismic tests within Boring Number 4 drilled by Geotechnologies for the property located at 1400-1440 Vine Street in Hollywood, California. The log of Boring Number 4 provided by Geotechnologies, Inc. is included in Attachment 1 and indicates that the subsurface materials are composed of

1. Fill primarily consisting of silty sand (SM) from the ground surface to approximately 3 feet below ground surface; and
2. Alluvium predominantly consisting of sand (SC, SM, SP, and SW) with occasional gravel and cobbles and some clayey and sandy silt (ML) and silty clay (CL) from approximately 3 to 120 feet (bottom of the borehole).

Additionally, the groundwater surface was noted at a depth of 49½ feet during borehole drilling on March 21, 2018. Downhole seismic tests were performed within Boring Number 4 to assist Geotechnologies, Inc. with their evaluation of the site. This letter summarizes the results of the downhole seismic tests and the evaluation of V_{s30}.

Seismic Downhole Methods and Procedures

Downhole seismic tests were collected within Boring Number 4 on April 23, 2018. The downhole seismic test method makes direct measurements of in-situ vertically propagating compression (P) and horizontally polarized shear (SH) wave velocities as a function of depth within the geologic material adjacent to a borehole. Measurement procedures followed ASTM D7400-08, “Standard Test Methods for Downhole Seismic Testing”. The geophysical data were collected, processed, and interpreted by a California-licensed Professional Geophysicist (PGp).

Boring Number 4 was drilled with an 8-inch diameter bit using hollow stem auger drilling methods and a 2-inch diameter PVC casing was installed under the direction of Geotechnologies, Inc. as part of their geotechnical investigation program. The annular space between the 8-inch diameter hole and 2-inch diameter casing was backfilled with bentonite-cement grout, which was assumed to be formulated to approximate the density of the surrounding geologic material and pumped in from the base of the borehole to completely fill the annular space.

A seismic source was used to generate a seismic wave (P or SH) at the ground surface. The seismic source was offset horizontally from the borehole a distance of 5 feet. The P-wave seismic source consisted of a ground plate that was struck vertically with a sledgehammer. The SH-wave seismic source consisted of an 8-foot long by 6-inch wide by 4-inch high wood beam capped on both ends with a steel plate and loaded in place by the front end of a vehicle that was parked on top of the beam. The ends of this beam were positioned equidistant from the borehole. Initially, one end of the beam was struck horizontal with a sledgehammer to produce an SH-wave (forward hit). Next, the opposite end of the beam was struck horizontally with a sledgehammer to produce an opposite polarity SH-wave (reverse hit). The combination of the two opposite polarity SH-waves were used to determine SH travel times.

A downhole receiver positioned at a selected depth within the borehole was used to record the arrival of the seismic wave (P or SH). A three component triaxial borehole geophone (one vertical-channel and two orthogonal horizontal channels), which could be firmly pneumatically fixed against the PVC casing sidewall, was used to collect the downhole seismic measurements. Multiple downhole seismic measurements were performed at successive receiver depths within the borehole. The receiver depth was referenced to ground surface, and measurements were made at receiver intervals of 5 feet from the ground surface to the bottom of the hole (120 ft).

A Geometrics S12 signal enhancing seismograph was used to record the response of the downhole receiver. The seismic source (sledgehammer) contained a trigger that was connected to and initiated the seismograph recording, thus measuring the travel time between seismic source and downhole receiver. Downhole seismic test records were digitally recorded and stored with a 0.062 ms sample interval.

The recorded digital downhole seismic records were analyzed using the OYO Corporation program PickWin Version 5.1.1.2. The digital waveforms were analyzed to identify arrival times. The first prominent departure of the vertical receiver trace was identified as the P-wave first arrival. The SH-wave forward and reverse hits recorded on the two horizontal receiver channels were superimposed. The SH-wave first arrival was identified at the location of the first prominent relatively low-frequency departure of the forward hit and an 180° polarity change is noted to have occurred on the reverse hit. For analysis, 25 Hz low-cut and 209 Hz high-cut filters were applied to the P waveforms, and 17 Hz low-cut and 107 Hz high-cut filters were applied to the SH waveforms.

After correcting the P and SH-wave travel time for the source offset, the P and SH-wave travel-times were plotted versus depth. P and SH layer and interval velocities were calculated as the slope of lines drawn through the plotted data.

Seismic Downhole Results

The results of the seismic downhole measurements collected within Boring Number 4 are presented on Figure 1. Figure 1 shows (1) a table of the measured P and SH-wave travel-times and depths; (2) a plot of the P and SH-wave travel-times as a function of depth showing the interpreted layer velocities; (3) a table of the calculated P and SH-wave interval velocities and depth ranges; (4) a table of the interpreted P and SH-wave layer velocities and depth ranges; and (5) a plot of the layer and interval velocity models as a function of depth.

Table 1 below summarizes the interpreted P and SH layer velocities and depths shown on Figure 1 for the various geologic units within Boring Number 4, as logged by Geotechnologies, Inc. It is noted that the groundwater level was measured at a depth of 49½ feet during borehole drilling on March 21, 2018. The measured P-wave velocities suggest the material adjacent to the borehole is saturated below a depth of approximately 50 feet.

TABLE 1
SUMMARY OF SH-WAVE AND P-WAVE VELOCITY LAYERS WITHIN BORING NUMBER 4

<table>
<tr><th>PREDOMINANT LITHOLOGY</th><th>Depth Range (ft)</th><th>SH-WAVE Velocity (ft/sec)</th><th>P-WAVE Velocity (ft/sec)</th></tr>
<tr><td>Medium dense to dense, silty SAND (SM) and some stiff to very stiff, sandy SILT (ML) [Fill and Alluvium]</td><td>0 to 30</td><td>860</td><td>1,770</td></tr>
<tr><td rowspan="4">Very dense, clayey, silty, and gravelly SAND (SC, SM, and SW) Groundwater observed at 49½ feet during drilling [Alluvium]</td><td>30 to 40</td><td rowspan="4">1,350</td><td>2,250</td></tr>
<tr><td>40 to 45</td><td>2,840</td></tr>
<tr><td>45 to 50</td><td>3,740</td></tr>
<tr><td>50 to 60</td><td rowspan="3">6,270</td></tr>
<tr><td>Very dense, clayey, silty SAND (SC, SM, SP, and SW) and some very stiff, silty CLAY (CL) [Alluvium]</td><td>60 to 105</td><td>1,200</td></tr>
<tr><td>Very dense, silty SAND (SM) and some very stiff sandy SILT (ML) [Alluvium]</td><td>105 to 120</td><td>1,630</td></tr>
</table>

The V_{s30} was calculated based on the procedures outlined in the 2010 California Building Code, "2010 California Existing Building Code, Title 24, Part 10, Section 1613A.5.5 – Site Classification for Seismic Design." The V_{s30} was calculated from Equation 16A-40 of this reference which states:

$$v_s = \frac{\sum_{i=1}^{n} di}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}}$$

where:

i = distinct different soil and/or rock layer between 1 and n
v_{si} = shear wave velocity in feet per second of layer i
d_i = thickness of any layer within the 100-foot interval
$\sum_{i=1}^{n} d_i$ = 100 feet

Based on this procedure, the V_{s30} for Boring Number 4 was calculated between various depth ranges. The results are summarized on Table 2.

TABLE 2
CALCULATED V_{s30} WITHIN BORING NUMBER 4

DEPTH RANGE (ft, below ground surface)	V_{s30} (ft/sec)
0 to 100	1,106
5 to 105	1,126
10 to 110	1,162
15 to 115	1,200
20 to 120	1,241

Limitations

The above information is based on limited observations and geophysical measurements made as described above. GeoPentech does not guarantee the performance of the project, only that the information provided meets the standard of care of the profession at this time under the same scope limitations imposed by the project. In this regard, our scope of work included making the P and SH-wave velocity measurements in one borehole under the direction of Geotechnologies, Inc. personnel. We relied upon the assumption that the annular space between the PVC casing and the borehole wall was properly filled with bentonite-cement grout so that PVC casing and the borehole wall were in continuous contact and that the grout was formulated to approximate the density of the surrounding geologic material.

We trust the contents of this letter will meet your current needs. If you have questions or require additional information, please call.

Very Truly Yours,

GeoPentech

REGISTERED GEOPHYSICIST
STEVE DUKE
No. 1013
Exp. 7-31-19
STATE OF CALIFORNIA

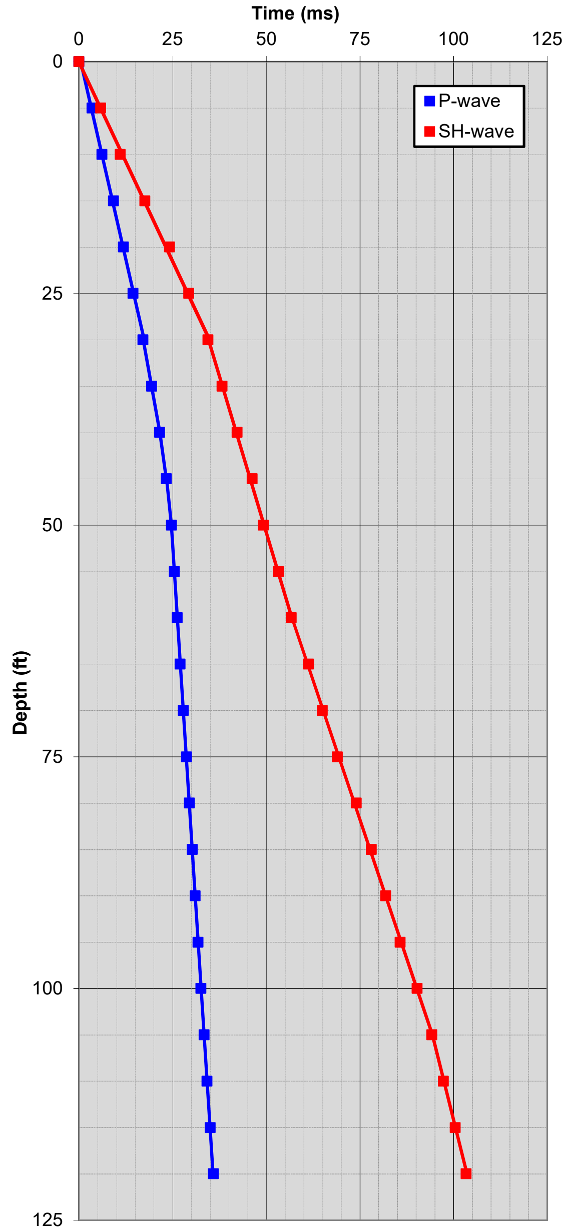
Steven K. Duke
Geophysicist
GP 1013

SEISMIC WAVE TRAVEL TIMES

Depth (ft)	P-time (ms)	P-layer	SH-wave (ms)	SH-layer
0	0	1	0	1
5	4.91	1	8.20	1
10	6.87	1	12.30	1
15	9.66	1	18.57	1
20	12.26	1	24.96	1
25	14.74	1	29.84	1
30	17.34	12	34.91	12
35	19.55	2	38.61	2
40	21.72	23	42.57	2
45	23.46	34	46.53	2
50	24.77	45	49.48	2
55	25.55	5	53.44	2
60	26.29	5	56.87	23
65	27.08	5	61.47	3
70	27.88	5	65.14	3
75	28.74	5	69.17	3
80	29.50	5	74.14	3
85	30.29	5	78.18	3
90	31.04	5	82.02	3
95	31.81	5	85.83	3
100	32.63	5	90.41	3
105	33.40	5	94.33	34
110	34.21	5	97.38	4
115	35.07	5	100.52	4
120	35.89	5	103.47	4

Source Offset (ft)
5

TRAVEL TIME PLOT



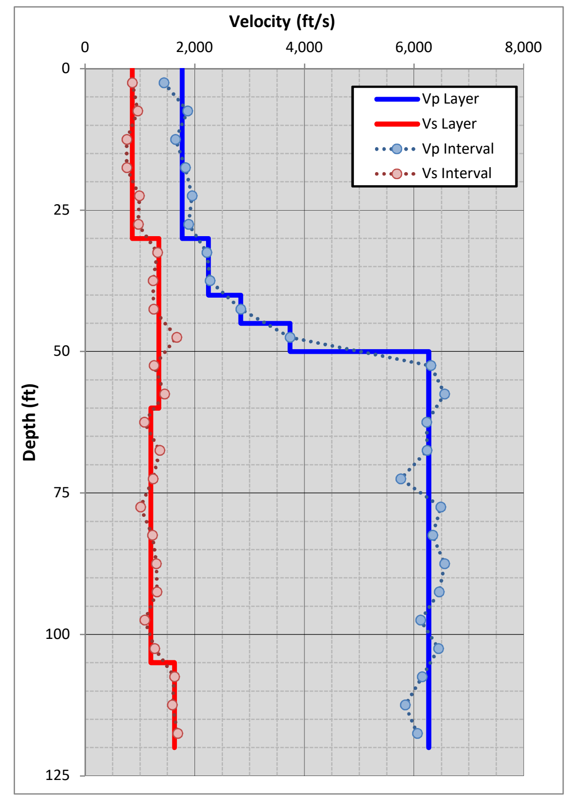
INTERVAL VELOCITES

Depth Range	P-Velocity (ft/s)	SH-Velocity (ft/s)
0 to 5	1,440	860
5 to 10	1,870	960
10 to 15	1,650	760
15 to 20	1,830	760
20 to 25	1,950	990
25 to 30	1,890	970
30 to 35	2,220	1,320
35 to 40	2,280	1,240
40 to 45	2,840	1,250
45 to 50	3,740	1,670
50 to 55	6,310	1,260
55 to 60	6,560	1,450
60 to 65	6,230	1,080
65 to 70	6,240	1,360
70 to 75	5,760	1,240
75 to 80	6,490	1,010
80 to 85	6,340	1,230
85 to 90	6,560	1,300
90 to 95	6,460	1,310
95 to 100	6,120	1,090
100 to 105	6,450	1,270
105 to 110	6,150	1,630
110 to 115	5,840	1,590
115 to 120	6,060	1,690

LAYER VELOCITES

Layer	P-Depth (ft)	P-Velocity (ft/s)	SH-Depth (ft)	SH-Velocity (ft/s)
1	0 to 30	1,770	0 to 30	860
2	30 to 40	2,250	30 to 60	1,350
3	40 to 45	2,840	60 to 105	1,200
4	45 to 50	3,740	105 to 120	1,630
5	50 to 120	6,270		
6				
7				
8				
9				
10				

VELOCITY MODEL



V_{S30} CALCULATION

Vs30 (ft/s)	Depth (ft)
1,110	0 to 100
1,240	20 to 120

GeoPentech

SEISMIC DOWNHOLE TEST RESULTS
BORING NUMBER 4

PROJECT: **1400-1440 VINE STREET**

PROJECT #: **18025A** | DATE: **MAY 2018** | FIGURE: **1**

ATTACHMENT 1

BORING LOG NUMBER 4
GEOTECHNOLOGIES, INC.

BORING LOG NUMBER 4

GRT Beverly Hills, LLC **Date: 03/21/18** **Elevation: 343.0'**

File No. 21225 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		FILL: Silty Sand, grayish brown, slightly moist, medium dense
				-		
				1 --		
				-		
				2 --		
				-		
				3 --		
				-	SM	Silty Sand, dark brown, moist, medium dense to dense, fine grained
				4 --		
				-		
5	13	7.5	110.1	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	18	11.3	118.7	-		
				8 --		
				-		
				9 --		
				-		
10	21	7.5	115.1	10 --		
				-		Silty Sand, dark brown, moist, dense, fine grained
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	27	13.5	117.5	15 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark brown, moist, dense to stiff, fine grained
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	41	14.9	117.3	20 --		
				-	ML	Sandy Silt, dark brown, moist, very stiff, fine grained
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	43	9.7	127.4	25 --		
				-	SM	Silty Sand, dark brown, moist, dense, fine grained

BORING LOG NUMBER 4

GRT Beverly Hills, LLC

File No. 21225

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
30	38	9.9	126.2	30 --		
				-		
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
35	30	2.6	115.9	35 --		
	50/5"			-	SW	Gravelly Sand, dark brown, slightly moist, very dense, fine to coarse grained, with gravel
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
40	45	5.0	131.6	40 --		
	50/4"			-		with gravel and cobbles
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
45	78	11.5	125.9	45 --		
				-	SM	Silty Sand, dark brown, moist, very dense, fine grained
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
50	37	9.6	127.3	50 --		
	50/5"			-	SM/ML	Clayey Sand to Clayey Silt, reddish brown, moist, very dense to stiff, fine to coarse grained, with gravel

BORING LOG NUMBER 4

GRT Beverly Hills, LLC

File No. 21225

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
				-		
				53 --		
				-		
				54 --		
				-		
55	32	11.0	127.7	55 --		
	50/5"			-	SC	Clayey Sand, reddish brown, moist, very dense, fine grained
				56 --		
				-		
				57 --		
				-		
				58 --		
				-		
				59 --		
				-		
60	50	No Recovery		60 --		
	50/2"			-		
				61 --		
				-		
				62 --		
				-		
				63 --		
				-		
				64 --		
				-		
65	45	17.2	117.1	65 --		
				-	CL	Silty Clay, dark brown, moist, very stiff
				66 --		
				-		
				67 --		
				-		
				68 --		
				-		
				69 --		
				-		
70	50	13.4	125.5	70 --		
	50/3"			-	SM	Silty Sand, dark brown, moist, very dense, fine grained
				71 --		
				-		
				72 --		
				-		
				73 --		
				-		
				74 --		
				-		
75	80	14.4	115.2	75 --		
				-		

BORING LOG NUMBER 4

GRT Beverly Hills, LLC

File No. 21225

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				76 --		
				-		
				77 --		
				-		
				78 --		
				-		
				79 --		
				-		
80	74	15.4	116.2	80 --		
				-	SP	Sand, dark brown, wet, very dense, fine to medium grained
				81 --		
				-		
				82 --		
				-		
				83 --		
				-		
				84 --		
				-		
85	100/10"	8.4	128.3	85 --		
				-	SW	Sand, dark brown, wet, very dense, fine to coarse grained, minor gravel
				86 --		
				-		
				87 --		
				-		
				88 --		
				-		
				89 --		
				-		
90	40	17.4	114.5	90 --		
	50/5"			-	SM	Silty Sand, dark brown, wet, very dense, fine grained
				91 --		
				-		
				92 --		
				-		
				93 --		
				-		
				94 --		
				-		
95	75	18.7	112.6	95 --		
				-	SM	Silty Sand, dark brown, wet, very dense, fine grained
				96 --		
				-		
				97 --		
				-		
				98 --		
				-		
				99 --		
				-		
100	100/8"	17.8	111.1	100 --		
				-		

BORING LOG NUMBER 4

GRT Beverly Hills, LLC

File No. 21225

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				101 --		
				-		
				102 --		
				-		
				103 --		
				-		
				104 --		
				-		
105	100/10"	16.4	119.5	105 --		
				-		Silty Sand, dark brown, wet, very dense, fine grained, with cobbles
				106 --		
				-		
				107 --		
				-		
				108 --		
				-		
				109 --		
				-		
110	100/6"	15.7	116.9	110 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark brown, moist, very dense to very stiff, fine grained
				111 --		
				-		
				112 --		
				-		
				113 --		
				-		
				114 --		
				-		
115	100/7"	10.9	126.3	115 --		
				-	SM	Silty Sand , dark brown, moist, very dense, fine to medium grained, minor gravel
				116 --		
				-		
				117 --		
				-		
				118 --		
				-		
				119 --		
				-		
120	100/11"	14.6	122.5	120 --		
				-		Total Depth 120 feet Water at 49½ feet Fill to 3 feet
				121 --		
				-		
				122 --		
				-		
				123 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				-		
				124 --		
				-		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				125 --		
				-		