Appendix IS-4

Geotechnical Investigation

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

August 16, 2016
Revised June 21, 2017
File Number 21265

5420 Sunset Boulevard LP, LLC
22917 Pacific Coast Highway, Suite 300
Malibu, California 90265

Attention: James Smith III

Subject: Geotechnical Engineering Investigation
Proposed Multiple Use Development
5420 Sunset Boulevard, Los Angeles, California

Ladies and Gentlemen:

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

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

Should you have any questions please contact this office.

Respectfully submitted,
GEOTECHNOLOGIES, INC.

EDWARD F. HILL
G.E. 2126

EFH:km

Distribution: (5) Addressee

Email to: [jsmith@acemalibu.com]; [rwolf@vtbs.com]

TABLE OF CONTENTS

SECTION **PAGE**

Geotechnologies, Inc.
439 Western Avenue, Glendale, California 91201-2837 • Tel: 818.240.9600 • Fax: 818.240.9675
www.geoteq.com

TABLE OF CONTENTS

SECTION **PAGE**

TABLE OF CONTENTS

SECTION **PAGE**

GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED MULTIPLE USE DEVELOPMENT
5420 SUNSET BOULEVARD
LOS ANGELES, CALIFORNIA

INTRODUCTION

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

PROPOSED DEVELOPMENT

Information on the proposed development was provided by the design team. The proposed project consists of a multiple use development. The structure is proposed to be seven stories over 2 subterranean parking levels. Column loads are estimated to be between 800 and 1,000 kips. Wall loads are estimated to be between 5 and 10 kips per lineal foot. Grading will consist of excavations up to 25 feet for the proposed subterranean levels.

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 the southeast corner of Sunset Boulevard and Western Avenue in the Hollywood area of the City of Los Angeles, California. The site is relatively level with very little elevation change. Drainage across the site is by sheetflow to the streets.

The site is currently developed with a small restaurant, a larger commercial structure and paved parking. The vegetation on the site consists of small trees and shrubs in landscaped planters.

The neighboring development consists of commercial structures on most nearby properties. Residential structures exist to the east of the site.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was explored on July 8, 2016 and April 17, 2017 by excavating nine exploratory excavations. The exploratory excavations varied in depth from 35 to 65 feet. The exploration was prosecuted with the aid of a truck-mounted drilling machine using 8-inch diameter hollowstem augers. The exploration locations are shown on the Plot Plan and the geologic materials encountered are logged on Plates A-1 through A-8.

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 were encountered in each of the geotechnical excavations. The fill was found to vary from 3 to 6 feet in depth and consists of clayey silts, and sandy silts to silty clays which are dark brown, moist, and stiff. The upper native soils underlying the site consist of clayey silts, silty clays, sandy silts, with depth the native soils grade to silty sands and sands. The native soils are dark brown, medium dense to stiff, and fine grained.

The geologic materials consist of detrital sediments deposited by river and stream action typical to this area of Los Angeles County. More detailed descriptions of the earth materials encountered may be obtained from individual logs of the subsurface excavations.

Groundwater

Groundwater was encountered at a depth of 52-1/2 feet below ambient site grade in geotechnical excavation 4. The historic high groundwater level was established by review of California Geological Survey Seismic Hazard Evaluation Report 026 Plate 1.2 entitled "Historically Highest Ground Water Contours". Review of this plate indicates that the historically highest groundwater level is on the order of 42 feet below grade. The proposed structure is expected to extend 25 feet below site grade. Therefore the historic high groundwater level would be 17 feet below the base of the structure and the observed groundwater would be 27 feet below the base of the proposed structure.

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.

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

The site is underlain by unconsolidated alluvial sediments deposited by river and stream action that are deeper than 200 feet.

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.

Elysian Park Thrust

Blind or 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 sometimes not known until they produce an earthquake.

The Elysian Park anticline is thought to overlie the Elysian Park blind thrust. This fault has been estimated to cause an earthquake every 500 to 1,300 years in the magnitude range 6.2 to 6.7. The Elysian Park anticline is approximately 1.34km to the northeast of the site.

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.

Hollywood Fault

The closest fault to the site which could cause surface rupture is the Hollywood Fault which is mapped just under 1km north of the site. The Hollywood fault is part of the Transverse Ranges Southern Boundary fault system. The Hollywood fault is located approximately 4.51 miles northwest of the site. This fault trends east-west along the base of the Santa Monica Mountains from the West Beverly Hills Lineament in the West Hollywood–Beverly Hills area to the Los Feliz area of Los Angeles. The Hollywood fault is the eastern segment of the reverse oblique Santa Monica–Hollywood fault. Based on geomorphic evidence, stratigraphic correlation between exploratory borings, and fault trenching studies, this fault is classified as active.

Until recently, the approximately 9.3-mile long Hollywood fault was considered to be expressed as a series of linear ground-surface geomorphic expressions and south-facing ridges along the south margin of the eastern Santa Monica Mountains and the Hollywood Hills. Multiple recent fault rupture hazard investigations have shown that the Hollywood fault is located south of the ridges and bedrock outcroppings along portions of Sunset Boulevard. The Hollywood fault has not produced any damaging earthquakes during the historical period and has had relatively minor micro-seismic activity. It is estimated that the Hollywood fault is capable of producing a maximum 6.7 magnitude earthquake. In 2014, the California Geological Survey established an Earthquake Fault Zone for the Hollywood Fault.

SEISMIC HAZARDS AND DESIGN CONSIDERATIONS

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

Surface Rupture

In 1972, the Alquist-Priolo Special Studies Zones Act (now known as the Alquist-Priolo Earthquake Fault Zoning Act) was passed into law. 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 and results of site reconnaissance, no known active 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, and the distance to local faults 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.

The enclosed "Empirical Estimation of Liquefaction Potential" is based on Boring 4. Standard Penetration Test (SPT) data were collected at 5-foot intervals. Samples of the collected materials were conveyed to the laboratory for testing and analysis. The percent passing a Number 200 sieve, Atterberg Limits, and the plasticity index (PI) of representative samples of the soils encountered in the exploratory boring are presented on the enclosed E-Plate and F-Plate. Based on CGS Special Publication 117A (CDMG, 2008), the vast majority of liquefaction hazards are associated with sandy soils and silty soils of low plasticity. Furthermore, cohesive soils with PI between 7 and 12 and moisture content greater than 85 percent of the liquid limit are susceptible to liquefaction.

The procedure presented in the SP117A guidelines was followed in analyzing the liquefaction potential of the subject site. The SP 117A guidelines were developed based on a paper titled, "Assessment of the Liquefaction Susceptibility of Fine-Grained Soils", by Bray and Sancio

(2006). According to the SP117A, soils having a Plastic Index greater than 18 exhibit clay-like behavior, and the liquefaction potential of these soils are considered to be low. Therefore, where the results of Atterberg Limits testing showed a Plastic Index greater than 18, the soils would be considered non-liquefiable, and the analysis of these soil layers was turned off in the liquefaction susceptibility column.

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

Dynamic Dry Settlement

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

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

Tsunamis, Seiches and Flooding

Tsunamis are large ocean waves generated by sudden water displacement caused by a submarine earthquake, landslide, or volcanic eruption. Review of the City of Los Angeles Inundation and Tsunami Hazard Areas map indicates the site does not lie within the 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 City of Los Angeles Inundation and Tsunami Hazard Areas map indicates the site lies within mapped inundation boundaries due to a seiche or a breached upgradient reservoir. A determination of whether a higher site elevation would remove the site from the potential inundation zones is beyond the scope of this investigation.

Landsliding

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

CONCLUSIONS AND RECOMMENDATIONS

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

The existing fill materials are not suitable for support of the proposed foundations, floor slabs or additional fill. Excavation of the proposed subterranean levels will remove the unsuitable materials in the building area. All foundations may bear in native geologic materials found at the level of the proposed excavation.

Groundwater was encountered at a depth of 52-1/2 feet below ambient site grade in geotechnical excavation 4. The historic high groundwater level was established by review of California Geological Survey Seismic Hazard Evaluation Report 026 Plate 1.2 entitled "Historically Highest Ground Water Contours". Review of this plate indicates that the historically highest groundwater level is on the order of 42 feet below grade. The proposed structure is expected to extend 25 feet below grade. Therefore the historic high groundwater level would be 17 feet below the base of the structure and the observed groundwater would be 27 feet below the base of the proposed structure.

Excavation of the proposed subterranean level will require shoring measures to provide a stable working area due to the proposed depth, the nature of the onsite soils, the presence of and the proximity of adjacent structures.

Foundations for small outlying structures, such as property line walls, which will not be tied-in to the proposed multiple use structure may be supported on conventional foundations bearing in native geologic materials.

SEISMIC DESIGN CONSIDERATIONS

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)	2.528g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	2.528g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.685g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.882g
Site Coefficient (F_v)	1.5
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	1.323g
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.882g

FILL SOILS

The maximum depth of fill encountered on the site was six 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 upper site soils were found to be in the critical expansion range however the deeper soils which would support the proposed foundations and concrete slabs-on-grade were found to be in the low expansion range. Reinforcing beyond the minimum required by the City of Los Angeles Department of Building and Safety is not required.

WATER-SOLUBLE SULFATES

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

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

The water-soluble sulfate content of the onsite geologic materials was tested by California Test 417. The water-soluble sulfate content was determined to be less than 0.1% percentage by weight for the soils tested. Based on 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

This office has reviewed the City of Los Angeles Methane and Methane Buffer Zones map. Based on this review it appears that the subject property is not located within a Methane Zone or Methane Buffer Zone as designated by the City.

GRADING GUIDELINES

Site Preparation

- A thorough search should be made for possible underground utilities and/or structures. Any existing or abandoned utilities or structures located within the footprint of the proposed grading should be removed or relocated as appropriate.

- All vegetation, existing fill, and soft or disturbed geologic materials should be removed from the areas to receive controlled fill. All existing fill materials and any disturbed geologic materials resulting from grading operations shall be completely removed and properly recompacted prior to foundation excavation.

- Any vegetation or associated root system located within the footprint of the proposed structures should be removed during grading.

- Subsequent to the indicated removals, the exposed grade shall be scarified to a depth of six inches, moistened to optimum moisture content, and recompacted in excess of the minimum required comparative density.

- The excavated areas shall be observed by the geotechnical engineer prior to placing compacted fill.

Compaction

The City of Los Angeles Department of Building and Safety requires a minimum comparative compaction of 95 percent of the laboratory maximum density where the soils to be utilized in the fill have less than 15 percent finer than 0.005 millimeters. The soils tested by this firm would require the 95 percent compaction requirement.

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

All fill should be mechanically compacted in layers not more than 8 inches thick. The materials placed should be moisture conditions to within 3 percent of the optimum moisture content of the particular material placed. All fill shall be compacted to at least 90 or 95 percent 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 or 95 percent 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 40. 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 or 95 percent of the laboratory maximum density. Utility trench backfill should be tested by representatives of this firm in accordance with the most recent revision of ASTM D-1557.

Wet Soils

At the time of exploration the soils which will be exposed at the bottom of the excavation were locally above optimum moisture content. It is anticipated that the excavated material to be placed as compacted fill, and the materials exposed at the bottom of excavated plane may require significant drying and aeration prior to recompaction.

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

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

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.

Abandoned Seepage Pits

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

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

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

<u>Geotechnical Observations and Testing During Grading</u>

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.

LEED Considerations

The Leadership in Energy and Environmental Design (LEED) Green Building Rating System encourages adoption of sustainable green building and development practices. Credit for LEED Certification can be assigned for reuse of construction waste and diversion of materials from landfills in new construction.

In an effort to provide the design team with a viable option in this regard, demolition debris could be crushed onsite in order to use it in the ongoing grading operations. The environmental ramifications of this option, if any, should be considered by the team.

The demolition debris should be limited to concrete, asphalt and other non-deleterious materials. All deleterious materials should be removed including, but not limited to, paper, garbage, ceramic materials and wood.

For structural fill applications, the materials should be crushed to 2 inches in maximum dimension or smaller. The crushed materials should be thoroughly blended and mixed with onsite soils prior to placement as compacted fill. The amount of crushed material should not exceed 20 percent. The blended and mixed materials should be tested by this office prior to placement to insure it is suitable for compaction purposes. The blended and mixed materials should be tested by Geotechnologies, Inc. during placement to insure that it has been compacted in a suitable manner.

FOUNDATION DESIGN

Conventional

Conventional foundations may bear in native soils. All conventional foundations for a structure should bear in the same material.

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 300 pounds per square foot. The bearing capacity increase for each additional foot of depth is 800 pounds per square foot. The maximum recommended bearing capacity is 6,000 pounds per square foot.

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

Miscellaneous Foundations

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

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

Foundation Reinforcement

Based on City of Los Angeles minimum requirements 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 0.4 may be used with the dead load forces.

Passive geologic pressure for the sides of foundations poured against undisturbed or recompacted soil may be computed as an equivalent fluid having a density of 300 pounds per cubic foot with a maximum earth pressure of 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

Settlement of the foundation system is expected to occur on initial application of loading. The maximum settlement is expected to be 1 inch and occur below the heaviest loaded columns. Differential settlement is not expected to exceed 1/2 inch.

Foundation Observations

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

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

RETAINING WALL DESIGN

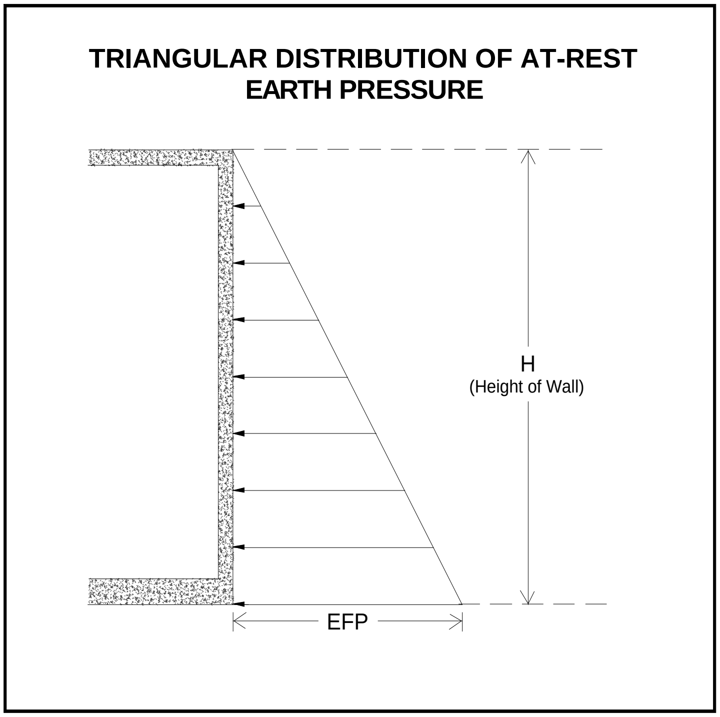
Cantilever Retaining Walls

Retaining walls supporting a level backslope may be designed utilizing a triangular distribution of pressure. Cantilever retaining walls may be designed for 30 pounds per cubic foot for walls retaining up to 25 feet of earth.

For this equivalent fluid pressure to be valid, walls which are to be restrained at the top should be backfilled prior to the upper connection being made. Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures.

Restrained Drained Retaining Walls

Restrained retaining walls may be designed to resist a triangular pressure distribution of at-rest earth pressure as indicated in the diagram below. The at-rest pressure for design purposes would be 58.2 pounds per cubic foot. Additional earth pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures.



In addition to the recommended earth pressure, 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.

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. Also, where necessary, the retaining walls should be designed to accommodate any surcharge pressures that may be imposed by existing buildings on the adjacent property.

Retaining Wall Drainage

Subdrains may consist of 4-inch diameter perforated pipes, places with perforated facing down. The pipe shall be encased in at least one-foot of gravel around the pipe. The gravel shall be wrapped in filter fabric. The gravel may consist of three-quarter inch to one inch crushed rocks. As an alternative, the use of gravel pockets and weepholes is an acceptable drainage method. Weepholes shall be a minimum of 2 inches in diameter, placed at 8 feet on center along the base of the wall. Gravel pockets shall be a minimum of 1 cubic foot in dimension, and may consist of three-quarter inch to once inch crushed rocks, wrapped in filter fabric.

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 emplacement of a standard pipe and gravel drainage system. Under these circumstances, the use of a flat drainage produce is acceptable.

Some municipalities do not allow the use of flat-drainage products. The use of such a product should be researched with the building official. As an alternative, omission of one-half of a block at the back of the wall on eight foot centers is an acceptable method of draining the walls. The resulting void should be filled with gravel. A collector is placed within the gravel which directs collected waters through the wall to a sump or standard pipe and gravel system constructed under the slab. This method should be approved by the retaining wall designer prior to implementation.

Where shoring will not allow the installation of a standard subdrainage system outside the wall rock pockets may be utilized. The rock pockets with should drain through the wall. The pockets should be a minimum of 12 inches in length, width and depth. The pocket should be filled with gravel. The rock pockets should be no more than 8 feet on center.

Sump Pump Design

The purpose of the recommended retaining wall backdrainage system is to relieve hydrostatic pressure. Groundwater was encountered during exploration to a depth of 52-1/2 feet which corresponds to 37-1/2 feet below the base of the proposed structure. Therefore the only water which could affect the proposed retaining walls would be irrigation waters and precipitation. Additionally, the proposed site grading is such that all drainage is directed to the street and the structure has been designed with adequate non-erosive drainage devices.

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

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 25.9 pounds per cubic foot. When using the load combination equations from the building code, the seismic earth pressure should be combined with the lateral active earth pressure for analyses of restrained 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 Backfill

Any required backfill should be mechanically compacted in layers not more than 8 inches thick, to at least 90 or 95 percent of the maximum density obtainable by the most recent revision of ASTM D 1557 method of compaction. Flooding should not be permitted. Compaction within 5 feet, measured horizontally, behind a retaining structure should be achieved by use of light weight, hand operated compaction equipment.

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

TEMPORARY EXCAVATIONS

Excavations on the order of 10 to 15 feet in vertical height will be required for the subterranean levels. Assuming the thickness of the concrete slab-on-grade and the foundations, excavations up to 20 feet have been addressed herein. The excavations are expected to expose fill and dense native soils, which are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic or structures. Excavations which will be surcharged by adjacent traffic or structures should be shored.

Where sufficient space is available, temporary unsurcharged embankments could be cut at a uniform 1:1 slope gradient. A uniform sloped excavation is sloped from bottom to top and does not have a vertical component.

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

Excavation Observations

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

SHORING DESIGN

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

One method of shoring would consist of steel soldier piles, placed in drilled holes and backfilled with concrete. The soldier piles may be designed as cantilevers or laterally braced utilizing drilled tied-back anchors or raker braces.

Soldier Piles Drilled and Poured

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

Casing may be required should caving be experienced in the granular geologic materials. If casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet.

The frictional resistance between the soldier piles and retained geologic material may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.4 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 500 pounds per square foot. The minimum depth of embedment for shoring piles is 5 feet below the bottom of the footing excavation or 7 feet below the bottom of excavated plane whichever is deeper.

Soldier Piles – Vibrated

The vibration method of shoring pile installation is acceptable to this firm from a geotechnical standpoint provided the recommendations presented herein are implemented. When using the vibration method of installing the soldier beams, the minimum embedment depth shall be 10 feet below the lowest excavated plane. Predrilling may be necessary by the shoring contractor in order to vibrate and install the shoring beams to the design depths.

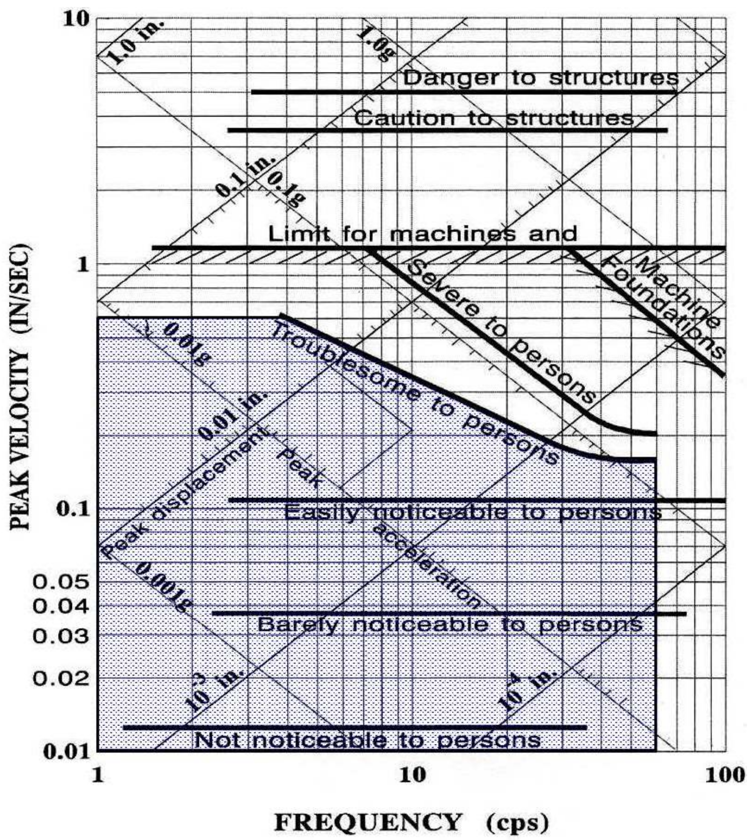
If predrilling is required, it is recommended that the diameter of the predrilled holes should not exceed 75 percent of the depth of the web of the I-beam. The depth of the predrilled holes should not exceed the planned excavation depth. In addition, when predrilling, the auger shall be backspun out of the pilot holes, leaving the soils in place. All shoring (predrilling, installation of shoring piles, tieback installation and testing, and lagging) shall be performed under the continuous inspections by a deputy grading inspector of this firm.

Monitoring of the shoring system shall be conducted on a periodic basis until the subterranean structure is completed. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all soldier piles.

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 located within a 1:1 (h:v) plane projected up from the base of the excavation. A maximum deflection of 1 inch is allowed provided there are no structures within a 1:1 (h:v) plane drawn upward from the base of the excavation.

The allowable level of vibration that results from the installation of the piles should not exceed a threshold where occupants of the nearby structures are disturbed, despite higher vibration tolerances that a building may endure without deformation. There is a relationship between particle velocity and vibration frequency that will occur due to the installation. A range of tolerable particle peak velocity and frequency of vibration is attached an "Allowable Amplitude of Vertical Vibrations". The shaded area on the graph is considered within acceptable limits to avoid damage to nearby structures. The acceptable limits should be measured at the neighboring structures.

The vibrations should be monitored with a seismograph during pile installation to detect the magnitude of vibration and oscillation experienced by the adjacent structure. The results should be recorded and provided to the owner. If, during installation, the vibrations exceed the range shown on the graph below, the shoring contractor should modify the installation procedure to reduce the values to the acceptable range.



Given Velocity = 0.2 inch/sec.
Frequency = 10 cps
Then from Graph, Displacement = 0.003 inches
Acceleration = 0.03g
Motion is easy noticeable or troublesome to persons

NOTE: Shaded area considered below threshold for structure damage

REFERENCE: Department of Defense, 1997, Soil Dynamics and Special Design Aspects, MIL-HDBK-1007/3

Lagging

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

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 750 pounds per square foot. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. This skin friction is based on 25 foot high shoring, a tied back anchor elevation 6 feet below grade and a minimum twenty foot embedment beyond the potentially active wedge yielding an overburden of 12½ feet below ground surface. Where belled anchors are utilized, the capacity of belled anchors may be designed by applying the skin friction over the surface area of the bonded anchor shaft. The diameter of the bell may be utilized as the diameter of the bonded anchor shaft when determining the surface area. This implies that in order for the belled anchor to fail, the entire parallel soil column must also fail.

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

Anchors should be placed at least 6 feet on center to be considered isolated. 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

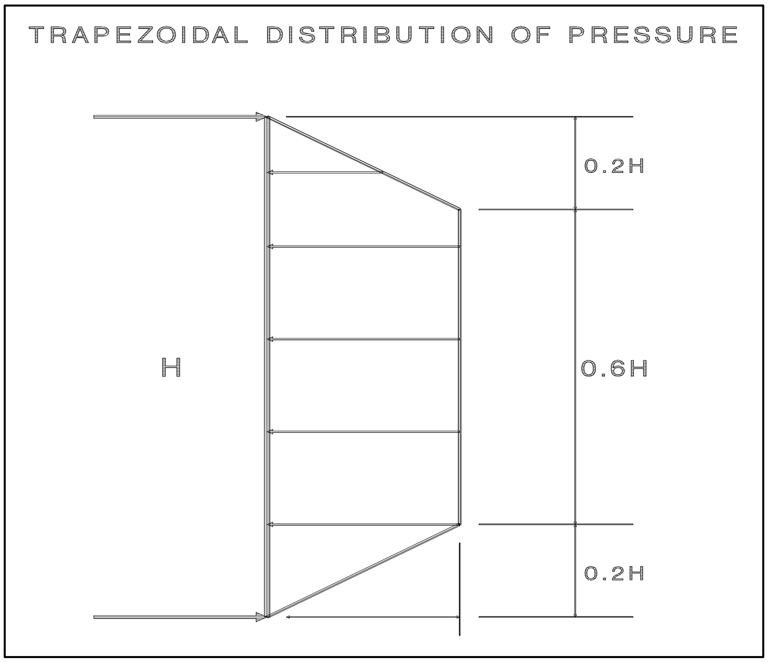
Tied-back anchors may be installed between 20 and 40 degrees below the horizontal. Caving of the anchor shafts, particularly within sand deposits, should be anticipated and the following provisions should be implemented in order to minimize such caving. The anchor shafts should be filled with concrete 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.

Lateral Pressures

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

HEIGHT OF SHORING "H" (feet)	EQUIVALENT FLUID PRESSURE (pounds per cubic foot)
Up to 25	25

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



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

HEIGHT OF SHORING "H" (feet)	DESIGN SHORING FOR (Where H is the height of the wall)
Up to 25	18H

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

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 existing structures on the adjacent properties should be made prior to, and during construction to record any movement or change due to vibration for use in the event of a dispute. In addition, adjacent structures should be monitored during shoring installation by the contractor for any changes.

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.

Raker Brace Foundations

An allowable bearing pressure of 4,000 pounds per square foot may be used for the design a raker foundations. This bearing pressure is based on a raker foundation a minimum of 4 feet in width and length as well as 4 feet in depth. The base of the raker foundations should be horizontal. Care should be employed in the positioning of raker foundations so that they do not interfere with the foundations for the proposed structure.

SLABS ON GRADE

Concrete Slabs-on Grade

Concrete floor slabs should be a minimum of 4 inches in thickness. Slabs-on-grade should be cast over undisturbed natural geologic materials or properly controlled fill materials. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 90 or 95 percent of the maximum dry density.

Outdoor concrete flatwork should be a minimum of 4 inches in thickness. Outdoor concrete flatwork should be cast over undisturbed natural geologic materials or properly controlled fill materials. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 90 or 95 percent of the maximum dry density.

Design of Slabs That Receive Moisture-Sensitive Floor Coverings

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

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

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

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

Groundwater was encountered on the subject site at a depth of 52-1/2 feet. Proposed concrete slabs-on-grade do not need to be supported on a layer of compacted aggregate to provide a capillary break.

Concrete Crack Control

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

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

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

Slab Reinforcing

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

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

PAVEMENTS

Prior to placing paving, the existing grade should be scarified to a depth of 12 inches, moistened as required to obtain optimum moisture content, and recompacted to 90 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

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 with Sections 200-2.2 or 200-2.4 of the most recent edition of “Standard Specifications for Public Works Construction”, (Green Book).

The performance of pavement is highly dependent upon providing positive surface drainage away from the edges. Ponding of water on or adjacent to pavement can result in saturation of the subgrade materials and subsequent pavement distress. If planter islands are planned, the perimeter curb should extend a minimum of 12 inches below the bottom of the aggregate base.

The management of pavement wear primarily is focused on the distress caused by vertical loads. The reduction of vertical loading from large vehicles is assisted by increasing the number of axles. Multi-axle groups reduce the peak vertical loading and, when closely spaced, reduce the magnitude of the strain cycles to which the pavement is subjected. However, where tight low-speed turns are executed, non-steering axle groups lead to transverse shear forces (scuffing) at the pavement-tire interface.

With asphaltic concrete pavements, tensile shear stresses from tires can cause surface cracking and raveling, thus, the increased use of non-steering axle groups results in increased pavement wear in the vicinity of intersections and turnarounds where tight low speed turns are executed.

When designing intersections and turnarounds the turn radius should be as large as possible. This will lead to reduced "scuffing" forces. Where tight radius turns are unavoidable, the pavement surface design should take into account the high level of "scuffing" forces that will occur and thickened pavement and subgrade and base course keyways should be considered to assist in the reduction of lateral deflection.

SITE DRAINAGE

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

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

STORMWATER DISPOSAL

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

PERCOLATION TESTING

In order to establish a percolation rate for the site soils, a boring was excavated April 17, 2017 for a percolation test. The boring was excavated to a depth of 40 feet. Casing was placed within the boring to test the area from 30 to 40 feet below site grade and seal the remaining areas of the boring. The boring was presoaked for a minimum of 2 hours prior to the test. After the presoak, the boring was refilled with water and the absorption of the soils was measured. The boring is shown on the enclosed plot plan as Boring 9.

Based on results of the percolation testing, a percolation rate of 40 inches per hour may be utilized for design of the proposed deep infiltration dry well system. This value does not include factors of safety or reduction factors. It should be noted that the recommended percolation rate is based on testing at a discrete location and the overall percolation rate of the system could vary considerably.

RECOMMENDATIONS

The location(s) for potential stormwater disposal have not been specifically addressed on this site. It is the opinion of this office that stormwater infiltration is possible on this site, however this office should review the plan once it achieves more definition.

Groundwater was encountered at a depth of 52-1/2 feet below the existing site grade. Review of Plate 1.2 of the Pasadena 7½ Minute Quadrangle Seismic Hazard Evaluation Report (CDMG, 1998 revised 2006) indicates that the historic high groundwater depth is 42 feet below grade in the vicinity of the site.

The proposed dry well system should be sealed to a minimum depth of 20 feet below the lowest foundation elevation. The use of dry well stormwater infiltration systems are acceptable to this firm provided that the recommendations presented herein are implemented. This office should review the final drainage plan and infiltration system details prior to construction to evaluate whether the intent of the recommendations presented herein are satisfied.

Based on the homogeneous nature of the alluvial soils observed during our site exploration, the stormwater should percolate in a generally vertical manner. The potential for creating a perched water condition is considered to be low. If the recommendations provided herein are followed, the proposed stormwater infiltration dry well systems should not cause any damage, settlement, or adversely affect any buildings.

The proposed stormwater infiltration dry well systems will not be located in a hillside area, and no slopes are nearby. The onsite soils are in the low to critical expansion range however the 20 foot depth below foundations should mitigate the effect of expansion of the wetted soils. The soils are not susceptible to significant hydroconsolidation. The Seismic Hazard Map for the

Hollywood Quadrangle, issued by the State of California (CDMG, 1999) does not classify the site as part of a "Liquefiable" area.

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.

The City of Los Angeles does not require corrosion testing. However, if corrosion sensitive improvements are planned, it is recommended that a comprehensive corrosion study should be commissioned. The study will develop recommendations to avoid premature corrosion of buried pipes and concrete structures in direct contact with the soils.

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. Samples from bucket-auger drilling are obtained utilizing a California Modified Sampler with successive 12-inch drops of a kelly bar, whose weight is noted on the excavation logs. 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.

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-Plate presented in the Appendix of this 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.

REFERENCES

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.

California Division of Mines and Geology, 1997, Seismic Hazard Zone Map, map scale 1:24,000.

City of Los Angeles, Department of Public Works, 2003, Methane and Methane Buffer Zones Map, Map Number A-20960.

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.

Division of Oil Gas and Geothermal Resources (DOGGR), 2001, Regional Wildcat Map, Northern Los Angeles Basin, map W1-5, map scale 1:48,000.

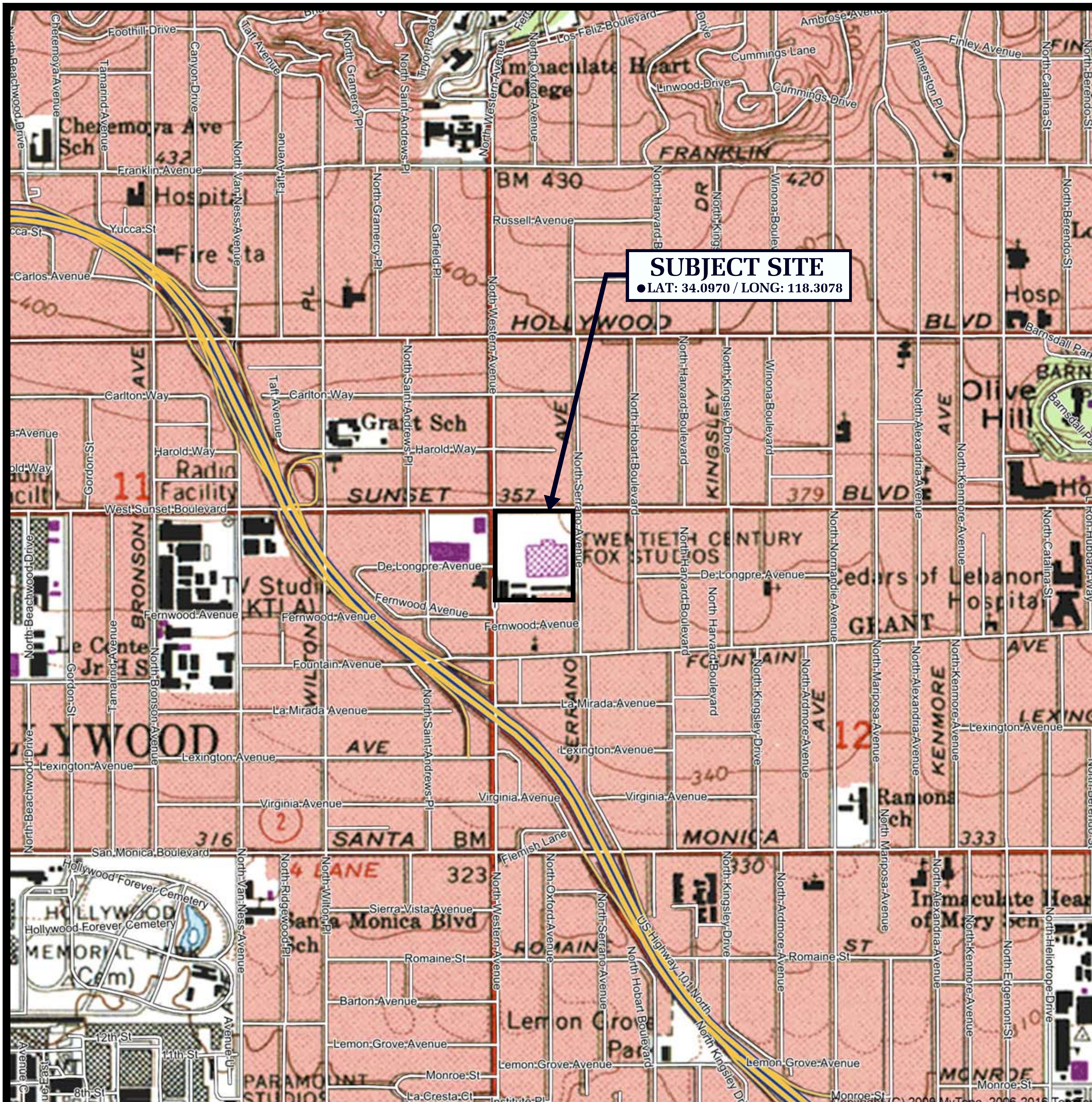
Dolan, J.F., Sieh, K., Rockwell, T.K., Guptill, P., and Miller, G., 1997, Active Tectonics, Paleoseismology, and Seismic Hazards of the Hollywood Fault, Northern Los Angeles Basin, California, GSA Bulletin, v. 109: no 12, p1595-1616.

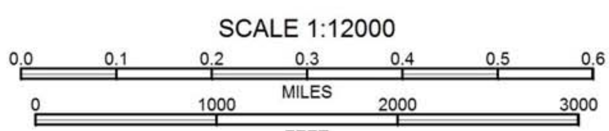
Hart, E.W. and Bryant, W.A., 1999 (updated 2005), Fault Rupture Zones in California, Division of Mines and Geology, Special Publication 42, 25pp.

Leighton and Associates, Inc. (1990), Technical Appendix to the Safety Element of the Los Angeles County General Plan: Hazard Reduction in Los Angeles County.

National Flood Insurance Rate Program, 2008, Los Angeles County and Incorporated Areas, Map #06037C1605F.

United States Geological Survey, 2011, U.S.G.S. Ground Motion Parameter Calculator (Version 5.0.9a). http://earthquake.usgs.gov/hazards/designmaps/.





N

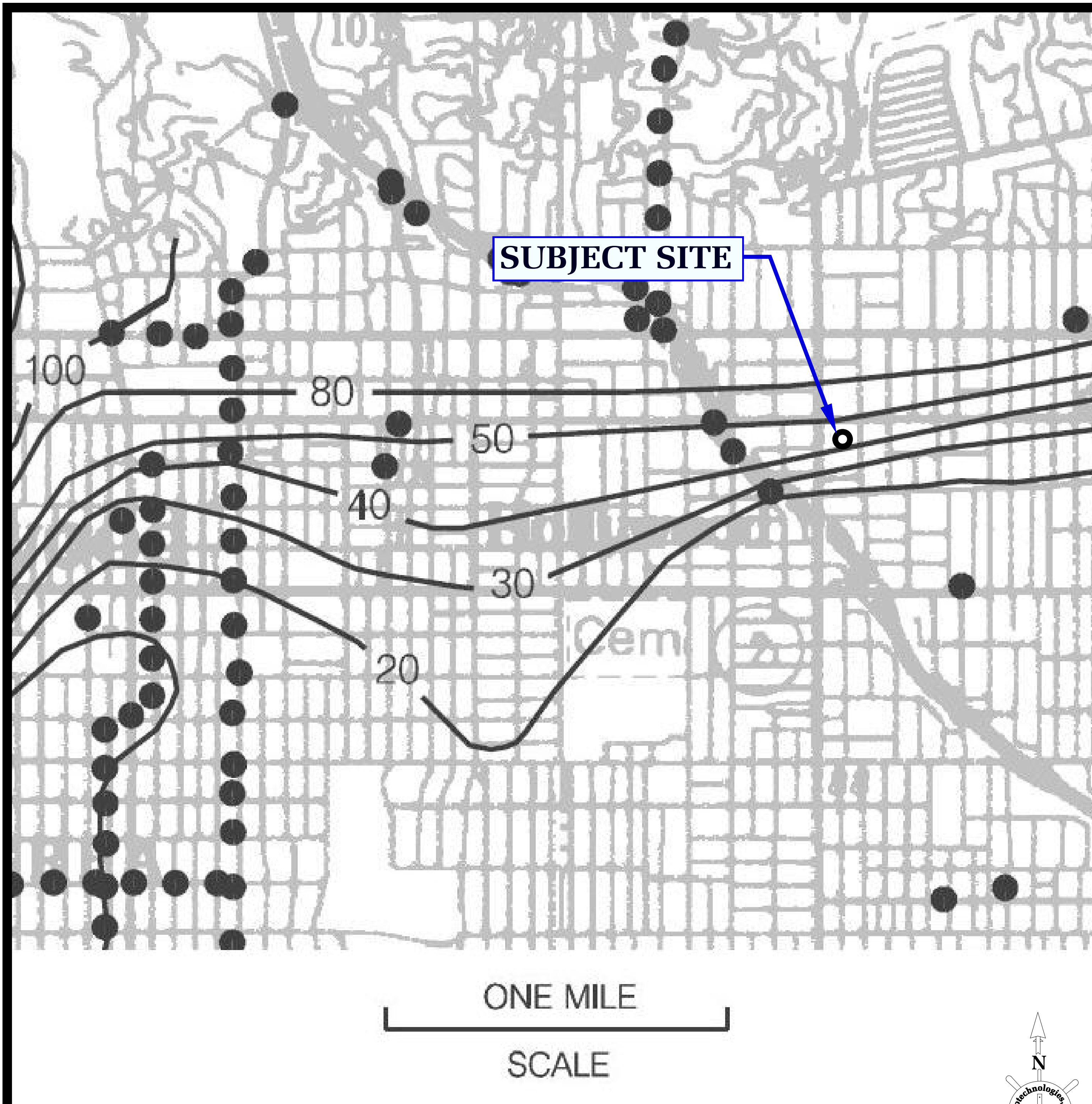
REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, HOLLYWOOD, CA QUADRANGLE

VICINITY MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

5420 SUNSET BOULEVARD

FILE NO. 21265



20 Depth to groundwater in feet

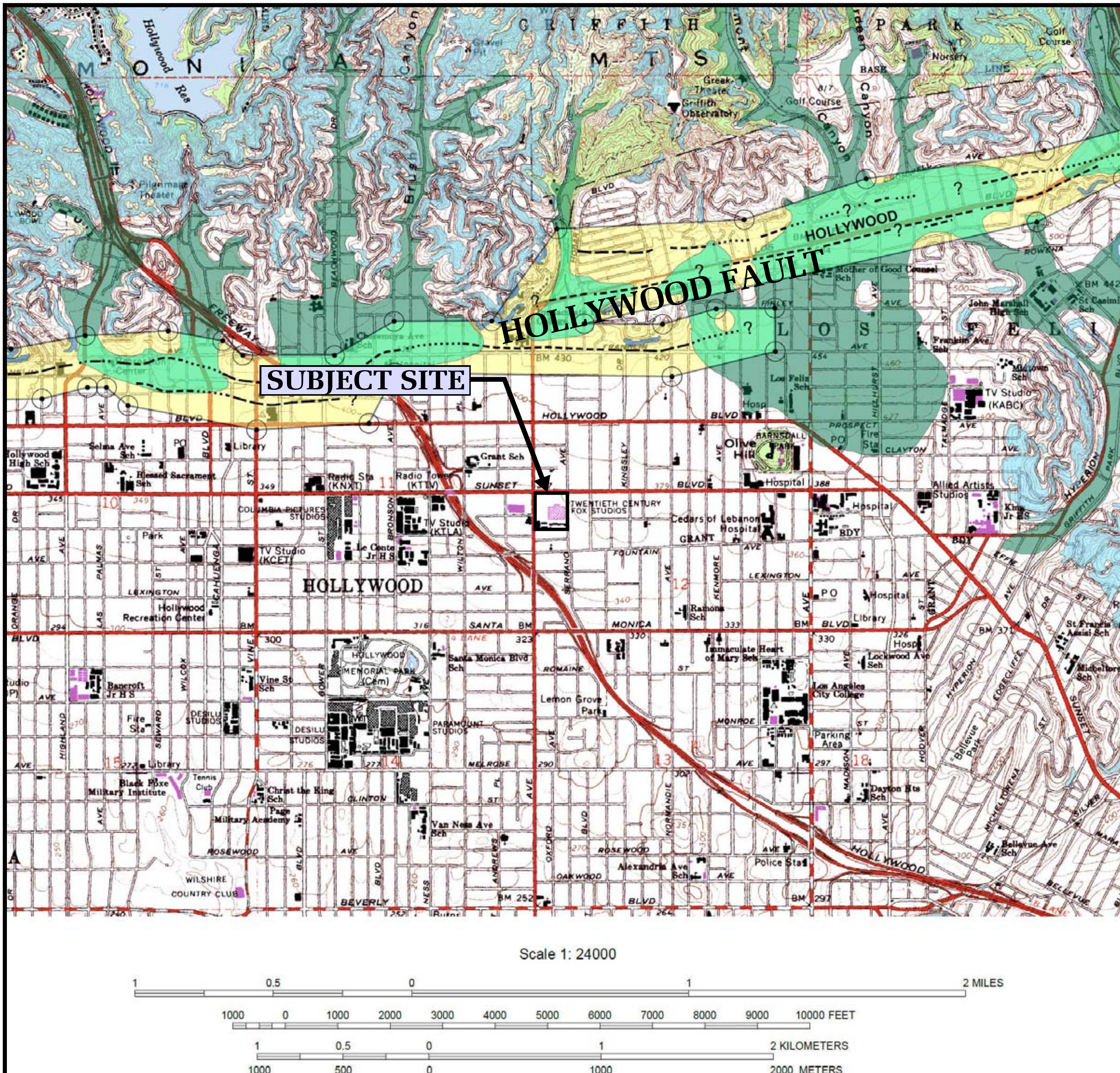
REFERENCE: CDMG, SEISMIC HAZARD ZONE REPORT, 026
HOLLYWOOD 7.5 - MINUTE QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA (1998, REVISED 2006)

HISTORICALLY HIGHEST GROUNDWATER LEVELS

Geotechnologies, Inc.
Consulting Geotechnical Engineers

5420 SUNSET BOULEVARD

FILE No. 21265



Earthquake Fault Zones
Alquist-Priolo Earthquake Fault Zone

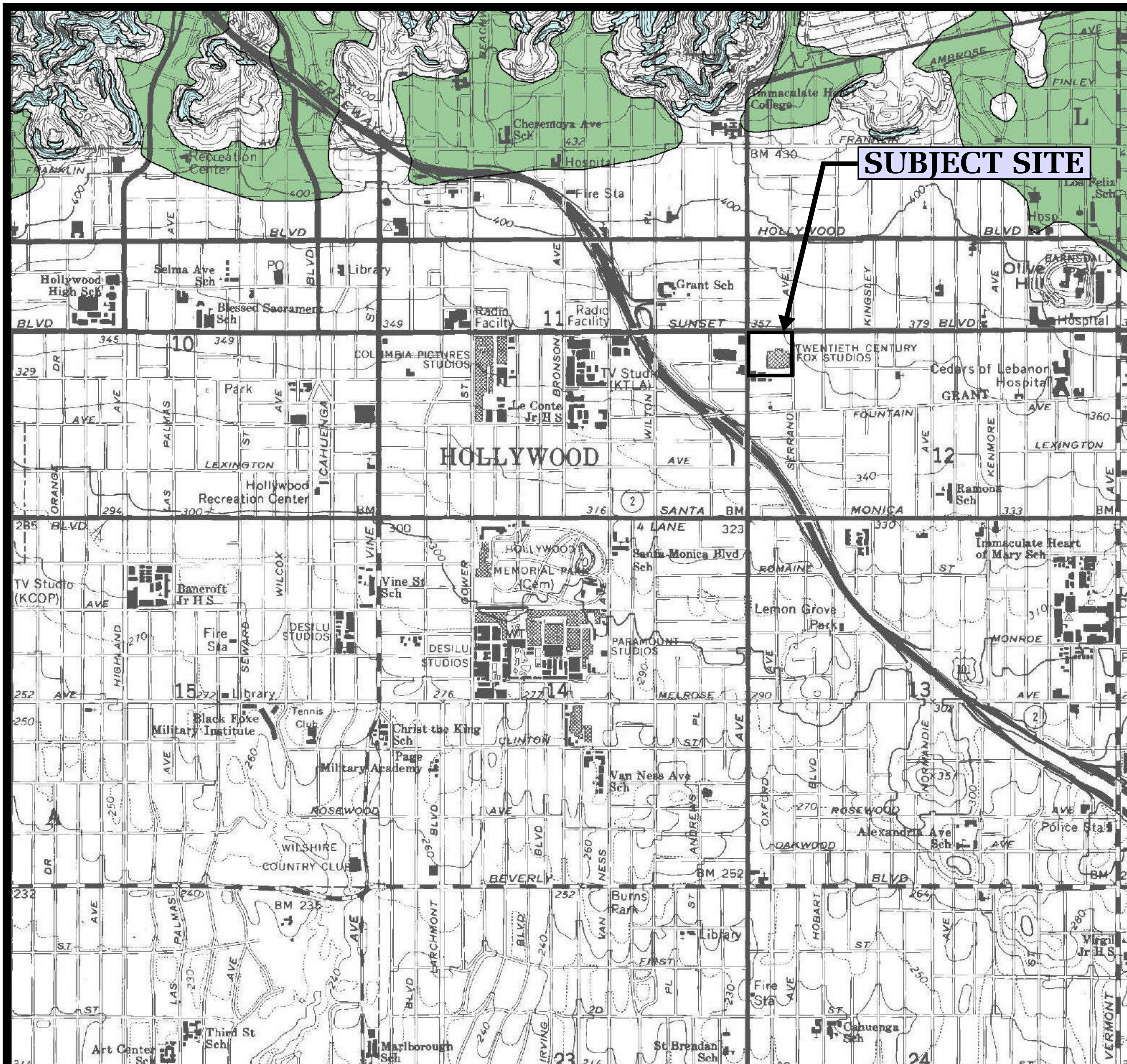
REFERENCE: EARTHQUAKE FAULT ZONES, HOLLYWOOD QUADRANGLE, CALIFORNIA GEOLOGICAL SURVEY, NOVEMBER 2014

EARTHQUAKE FAULT ZONE

Geotechnologies, Inc.
Consulting Geotechnical Engineers

5420 SUNSET BOULEVARD

FILE NO. 21265



SCALE 1:24,000

1 .5 0 1 MILE

1000 0 1000 2000 3000 4000 5000 6000 7000 FEET

1 .5 0 1 KILOMETER

LIQUEFACTION AREA

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

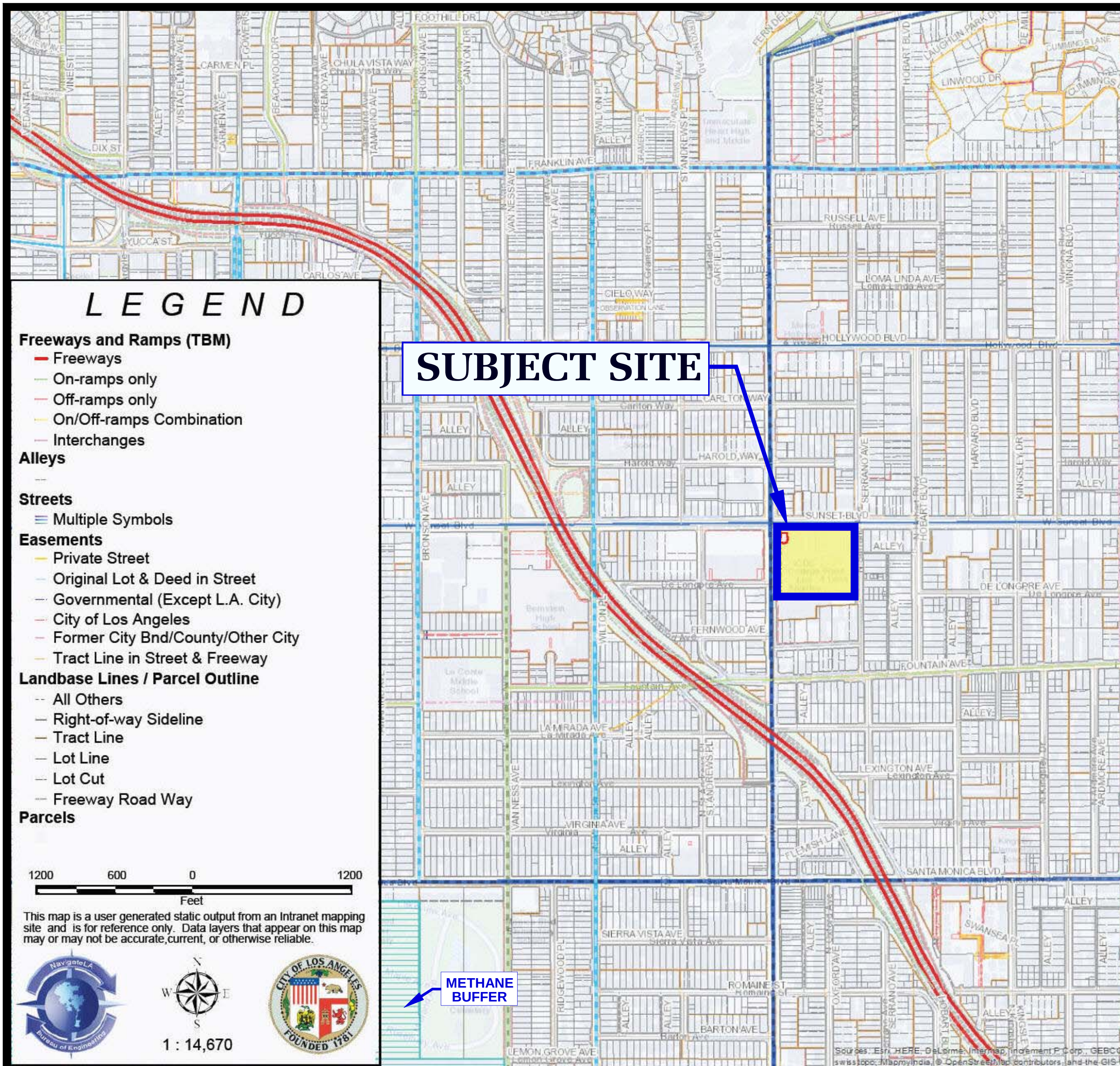
N

SEISMIC HAZARD ZONE MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

5420 SUNSET BOULEVARD

FILE NO. 21265



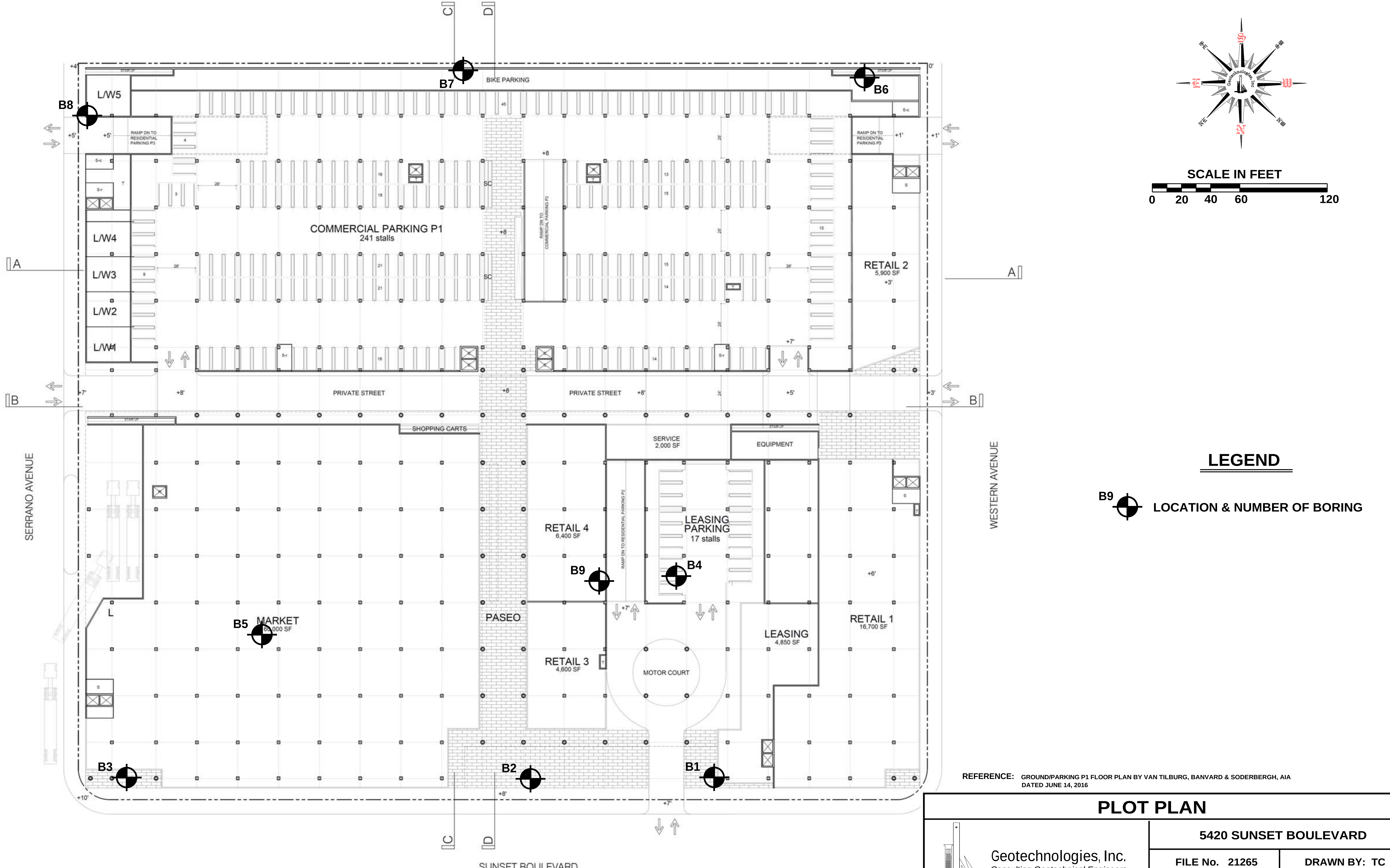
REFERENCE: **http://navigatela.lacity.org/NavigateLA/**

METHANE ZONE RISK MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

5420 SUNSET BOULEVARD

FILE NO. 21265



LEGEND

B9 LOCATION & NUMBER OF BORING

REFERENCE: GROUND/PARKING P1 FLOOR PLAN BY VAN TILBURG, BANVARD & SODERBERGH, AIA DATED JUNE 14, 2016

PLOT PLAN

Geotechnologies, Inc.
Consulting Geotechnical Engineers

5420 SUNSET BOULEVARD

FILE No. 21265	DRAWN BY: TC
DATE: April '17	

BORING LOG NUMBER 1

5420 Sunset Boulevard LP, LLC **Date: 07/08/16**

File No. 21265 **Method: 8-inch Diameter Hollow Stem Auger**

ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		5.5-inch Asphalt over 3.5-inch Base
				-		
				1 --		FILL: Sandy Silt, dark brown, moist
				-		
				2 --		Sandy to Clayey Silt, dark brown, moist, medium dense
2.5	29	19.6	102.6	-		
				3 --		
				-	ML	ALLUVIUM: Sandy to Clayey Silt, dark brown, moist, medium
				4 --		dense
				-		
5	42	19.1	108.4	5 --		
				-		
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	33	10.9	125.9	10 --	SM	Silty Sand, dark brown, moist, dense, fine grained
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	43	7.1	117.8	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	45	7.1	122.8	20 --		
				-	SM/SP	Silty Sand to Sand, dark brown, moist, dense, fine grained
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	38	11.6	116.3	25 --		
				-		

BORING LOG NUMBER 1

5420 Sunset Boulevard LP, LLC

File No. 21265

ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
30	25	17.9	108.6	30 --		
				-	SM	Silty Sand, dark brown, moist, dense, fine grained
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
35	51	16.2	113.8	35 --		
				-		Total Depth 35 feet
				36 --		No Water
				-		Fill to 3 feet
				37 --		
				-		
				38 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				39 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				40 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
				45 --		
				-		
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
				50 --		
				-		

BORING LOG NUMBER 2

5420 Sunset Boulevard LP, LLC **Date: 07/08/16**

File No. 21265 **Method: 8-inch Diameter Hollow Stem Auger**

ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		3.5-inch Asphalt over 4.5-inch Base
				-		
				1 --		FILL: Sandy to Clayey Silt, dark brown, moist, stiff
				-		
				2 --		
2.5	31	10.3	99.8	-		
				3 --		
				-		
				4 --		
				-		
5	52	16.1	112.1	5 --		
				-		
				6 --		
				-	ML	ALLUVIUM: Clayey Silt, dark brown, moist, stiff
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
				10 --		
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	50/6"	4.9	132.3	15 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark brown, moist, fine grained, dense, stiff
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
				20 --		
				-		
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	49	9.9	119.4	25 --		
				-	SM/SP	Silty Sand to Sand, dark brown, moist, fine grained, dense

BORING LOG NUMBER 2

5420 Sunset Boulevard LP, LLC

File No. 21265

ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
				30 --		
				-		
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		fine to coarse grained, very dense, minor gravel
35	100/12"	3.3	124.7	35 --		
				-		Total Depth 35 feet
				36 --		No Water
				-		Fill to 6 feet
				37 --		
				-		
				38 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				39 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				40 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
				45 --		
				-		
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
				50 --		
				-		

BORING LOG NUMBER 3

5420 Sunset Boulevard LP, LLC **Date: 07/08/16**

File No. 21265 **Method: 8-inch Diameter Hollow Stem Auger**

ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		5-inch Asphalt over 5-inch Base
				-		
				1 --		
				-		FILL: Sandy and Clayey Silt, dark brown, moist, stiff
				2 --		
2.5	35	17.3	104.5	-		
				3 --		
				-	ML	ALLUVIUM: Sandy to Clayey Silt, dark brown, moist, medium dense
				4 --		
				-		
5	67	15.6	110.8	5 --		
				-		very stiff
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
				10 --		
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	50/6"	8.4	129.0	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
				20 --		
				-		
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	78	2.9	121.0	25 --		
				-	SP	Sand, light to dark brown, moist, dense, fine to medium grained

BORING LOG NUMBER 3

5420 Sunset Boulevard LP, LLC

File No. 21265

ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
				28		
				29		
				30		
				31		
				32		
				33		
				34		
					SM/ML	Silty Sand to Sandy Silt, dark brown, moist, dense, fine grained
35	58	20.0	107.8	35		
						Total Depth 35 feet No Water Fill to 3 feet
				36		
				37		
				38		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				39		
				40		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				41		
				42		
				43		
				44		
				45		
				46		
				47		
				48		
				49		
				50		

BORING LOG NUMBER 4

5420 Sunset Boulevard LP, LLC — **Date: 07/07/16**

File No. 21265 — **Method: 8-inch Diameter Hollow Stem Auger**

ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		4-inch Asphalt over 4-inch Base
				-		
				1 --		
				-		FILL: Sandy Silt, to Clayey Silt, dark brown, moist, stiff
				2 --		
2.5	29	22.8	99.0	-		
				3 --		
				-	ML	ALLUVIUM: Clayey Silt, dark brown, moist, stiff
				4 --		
				-		
5	13	20.9	SPT	5 --		
				-	ML/CL	Sandy Silt to Silty Clay, dark brown, moist, stiff
				6 --		
				-		
				7 --		
7.5	32	18.3	110.0	-		
				8 --	ML	Sandy to Clayey Silt, dark brown, moist, stiff
				-		
				9 --		
				-		
10	7	14.1	SPT	10 --		
				-		Sandy Silt, dark brown, moist, stiff
				11 --		
				-		
				12 --		
12.5	48	12.8	123.6	-		
				13 --	SM	Silty Sand, dark brown, moist, dense, fine grained
				-		
				14 --		
				-		
15	17	14.5	SPT	15 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark brown, moist, dense, fine grained,
				16 --		stiff
				-		
				17 --		
17.5	58	10.2	129.1	-		
				18 --	SM	Silty Sand, dark brown, moist, dense, fine grained
				-		
				19 --		
				-		
20	17	8.7	SPT	20 --		
				-	SP/SM	Sand to Silty Sand, dark brown, moist, dense, fine grained
				21 --		
				-		
				22 --		
22.5	33	21.7	104.4	-		
				23 --	SM	Silty Sand, dark brown, moist, dense, fine grained
				-		
				24 --		
				-		
25	7	10.8	SPT	25 --		
				-	SM/SP	Silty Sand to Sand, dark brown, moist, dense, fine grained

BORING LOG NUMBER 4

5420 Sunset Boulevard LP, LLC

File No. 21265

ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
27.5	23	13.7	116.2	-		
				28 --	SM	Silty Sand, dark brown, moist, dense, fine grained
				-		
				29 --		
				-		
30	10	15.3	SPT	30 --		
				-	SM/SP	Silty Sand to Sand, dark brown, moist, dense, fine grained
				31 --		
				-		
				32 --		
32.5	32	20.2	102.9	-		
				33 --	SM	Silty Sand, dark brown, moist, dense, fine grained
				-		
				34 --		
				-		
35	14	10.9	SPT	35 --		
				-	SM/SP	Silty Sand to Sand, dark brown, moist, dense, fine grained
				36 --		
				-		
				37 --		
37.5	65	5.7	111.2	-		
				38 --	SP	Sand, dark yellowish brown, moist, dense
				-		
				39 --		
				-		
40	38	11.6	SPT	40 --		
				-	SM/SP	Silty Sand to Sand with gravel, dark brown to yellowish brown, dense, fine to medium grained
				41 --		
				-		
				42 --		
42.5	58	42.3	90.8	-		
				43 --	SM/ML	Silty Sand to Sandy Silt, dark brown, moist, dense, fine grained, stiff
				-		
				44 --		
				-		
45	15	24.6	SPT	45 --		
				-		
				46 --		
				-		
				47 --		
47.5	50/5"	25.6	100.3	-		
				48 --	ML/CL	Sandy Silt to Silty Clay, dark brown, moist, very stiff
				-		
				49 --		
				-		
50	43	8.7	SPT	50 --		
				-	SW/SM	Sand to Silty Sand, dark to medium brown, moist, dense, fine to coarse grained, some gravel

BORING LOG NUMBER 4

5420 Sunset Boulevard LP, LLC

File No. 21265

ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				51		
				52		
52.5	50/3"	6.5	134.1			
				53		water
				54		
55	27	17.7	SPT	55		
					SP/ML	Sand to Clayey Silt, dark brown, moist, dense, fine to medium grained, stiff
				56		
				57		
57.5	67	15.4	118.2			
				58	SM	Silty Sand, dark brown, very moist, dense, fine grained
				59		
60	25	16.6	SPT	60		
					SM/ML	Silty Sand to Clayey Silt, dark to yellowish brown, very moist, dense, fine grained, stiff
				61		
				62		
62.5	37	15.1	110.8			
				63	SM/CL	Silty Sand to Sandy Clay, dark brown, wet, dense, fine grained, stiff
				64		
					SP/SM	Sand to Silty Sand, dark brown, wet, medium dense, fine grained
65	28	15.1	SPT	65		
				66		Total Depth 65 feet Water at 52.5 feet Fill to 3 feet
				67		
				68		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				69		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				70		
				71		SPT=Standard Penetration Test
				72		
				73		
				74		
				75		

BORING LOG NUMBER 5

5420 Sunset Boulevard LP, LLC **Date: 07/08/16**

File No. 21265 **Method: 8-inch Diameter Hollow Stem Auger**

ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		5-inch Asphalt over 1-inch Base
				-		
				1 --		FILL: Clayey Silt to Silty Clay, dark brown, moist, medium dense
				-		
				2 --		
2.5	31	13.6	119.9	-		
				3 --		ALLUVIUM: Silty Sand, dark brown, moist, medium dense, fine grained
				-		
				4 --		
				-		
5	34	14.6	118.2	5 --		
				-		Sandy to Clayey Silt, dark brown, moist, dense
				6 --		
				-		
				7 --		
7.5	34	16.9	116.4	-		
				8 --		Sandy Silt to Silty Clay
				-		
				9 --		
				-		
10	50	15.9	113.5	10 --		
				-		
				11 --		
				-		
				12 --		
12.5	38	12.1	117.7	-		Silty Sand to Clayey Silt, dark brown, moist, dense
				13 --		
				-		
				14 --		
				-		
15	25	15.3	110.6	15 --		
				-		
				16 --		
				-		
				17 --		
17.5	27	16.3	113.7	-		
				18 --		
				-		
				19 --		
				-		
20	26	15.4	116.4	20 --		
				-		
				21 --		
				-		
				22 --		
22.5	41	18.5	102.8	-		Silty Sand to Sandy Clay, dark brown, moist, dense
				23 --		
				-		
				24 --		
				-		
25	38	13.9	114.4	25 --		
				-		Sandy Silt to Silty Clay, dark to greenish brown, moist, dense

BORING LOG NUMBER 5

5420 Sunset Boulevard LP, LLC

File No. 21265

ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
27.5	56	17.9	107.2			
				28		
				29		
30	41	16.1	114.5	30		
				31		
				32		
32.5	45	11.3	123.2			
				33	SM/ML	Silty Sand to Clayey Silt, dark brown, moist, dense
				34		
					SP	Sand, dark brown, moist, medium dense, fine to medium grained
35	41	4.6	117.2	35		
				36		Total Depth 35 feet No Water Fill to 2½ feet
				37		
				38		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				39		
				40		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				41		
				42		
				43		
				44		
				45		
				46		
				47		
				48		
				49		
				50		

BORING LOG NUMBER 6

5420 Sunset Boulevard LP, LLC **Date: 07/08/16**

File No. 21265 **Method: 8-inch Diameter Hollow Stem Auger**

ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		5.5-inch Asphalt over 5-inch Base
				-		
				1 --		
				-		FILL: Clayey Silt to Sandy Silt, dark brown, moist, firm
				2 --		
2.5	38	20.9	106.4	-		
				3 --		
				-	CL/ML	ALLUVIUM: Silty Clay to Clayey Silt, dark brown, moist, firm
				4 --		
				-		
5	34	17.1	111.9	5 --		
				-	ML	Sandy to Clayey Silt, minor caliche, dark brown, moist, firm
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
				10 --		
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	50/6"	11.2	128.8	15 --		
				-	SM	Silty Sand, dark brown, moist, dense, fine to medium grained
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
				20 --		
				-		
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	39	3.8	113.6	25 --		
				-	SM/SP	Silty Sand to Sand, light to dark brown, moist, dense, fine to medium grained

BORING LOG NUMBER 6

5420 Sunset Boulevard LP, LLC

File No. 21265

ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
				30 --		
				-		
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		Sand, light to dark brown, moist, very dense, fine to medium
				-	SP	grained
35	50/6"	3.7	No	35 --		
			Sample	-		Total Depth 35 feet
				36 --		No Water
				-		Fill to 3 feet
				37 --		
				-		
				38 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				39 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				40 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
				45 --		
				-		
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
				50 --		
				-		

BORING LOG NUMBER 7

5420 Sunset Boulevard LP, LLC — **Date: 07/08/16**

File No. 21265 — **Method: 8-inch Diameter Hollow Stem Auger**

ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		5-inch Asphalt over 4-inch Base
				-		
				1 --		FILL: Clayey Silt to Silty Clay, dark brown, moist
				-		
				2 --		
2.5	21	17.2	109.3	-		
				3 --		
				-	SC/ML	ALLUVIUM: Clayey Sand to Clayey Silt, dark brown, moist, medium dense, stiff
				4 --		
				-		
5	21	14.2	112.5	5 --		
				-	SM	Silty Sand, dark brown, moist, dense, fine grained
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	49	9.6	125.8	10 --		
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
				15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	43	4.2	114.5	20 --		
				-	SP/ML	Sand to Silt, dark brown, moist, dense, fine to medium grained
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
				25 --		
				-		

BORING LOG NUMBER 7

5420 Sunset Boulevard LP, LLC

File No. 21265

ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
30	73	4.9	119.6	30 --		
				-	SP	Sand with minor gravel, dark brown, moist, dense, fine to medium grained
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		Silty Sand to Sandy Silt, dark brown, moist, dense, stiff, fine grained
				-	SM/ML	
35	71	12.1	122.8	35 --		
				-		Total Depth 35 feet No Water Fill to 3 feet
				36 --		
				-		
				37 --		
				-		
				38 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				-		
				39 --		
				-		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				40 --		
				-		
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
				45 --		
				-		
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
				50 --		
				-		

BORING LOG NUMBER 8

5420 Sunset Boulevard LP, LLC **Date: 07/08/16**

File No. 21265 **Method: 8-inch Diameter Hollow Stem Auger**

ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 -- -		7-inch Asphalt over 1-inch Base
				1 -- -		FILL: Silty Clay, dark brown, moist, stiff
				2 -- -		
2.5	44	16.5	116.4			
				3 -- -	CL/ML	ALLUVIUM: Silty Clay to Clayey Silt, dark brown, moist, stiff
				4 -- -		
5	36	13.2	122.0	5 -- -	ML/SM	Sandy Silt to Silty Sand, dark brown, moist, stiff, dense, fine grained
				6 -- -		
				7 -- -		
				8 -- -		
				9 -- -		
				10 -- -		
				11 -- -		
				12 -- -		
				13 -- -		
				14 -- -		
15	50/3"	8.5	121.9	15 -- -	SP	Sand, dark yellowish brown, moist, very dense, fine to medium grained
				16 -- -		
				17 -- -		
				18 -- -		
				19 -- -		
				20 -- -		
				21 -- -		
				22 -- -		
				23 -- -		
				24 -- -		
25	68	16.7	114.3	25 -- -		

BORING LOG NUMBER 8

5420 Sunset Boulevard LP, LLC

File No. 21265

ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
				30 --		
				-		
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		Silty Sand to Sand, dark grayish brown, moist, very dense, fine
				-	SM/SP	to medium grained
35	87	6.3	122.1	35 --		
				-		Total Depth 35 feet
				36 --		No Water
				-		Fill to 5 feet
				37 --		
				-		
				38 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				39 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				40 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
				45 --		
				-		
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
				50 --		
				-		

BORING LOG NUMBER 9

5420 Sunset Boulevard LP, LLC **Date: 04/17/17**

File No. 21265 **Method: 8-inch Diameter Hollow Stem Auger**

ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		3-inch Asphalt over 4-inch Base
				-		
				1 --		FILL: Sandy Silt, dark brown, moist, stiff
				-		
				2 --		
2.5	30			-		
				3 --		Clayey Silt to Silty Clay, dark brown, moist, stiff
				-		
				4 --		
				-		
5	32			5 --		
				-		minor wood fragments
				6 --		
				-		
				7 --		
7.5	48			-		
				8 --		minor concrete fragments
				-		
				9 --		
				-		
10	40			10 --		
				-	SM	ALLUVIUM: Silty Sand, dark brown, moist, medium dense,
				11 --		fine grained
				-		
				12 --		
12.5	50/4"			-		
				13 --		
				-		
				14 --		
				-		
15	82			15 --		
				-		very dense
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	95			20 --		
				-		
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	77			25 --		
				-	SP	Sand, dark brown, moist, dense, fine to medium grained

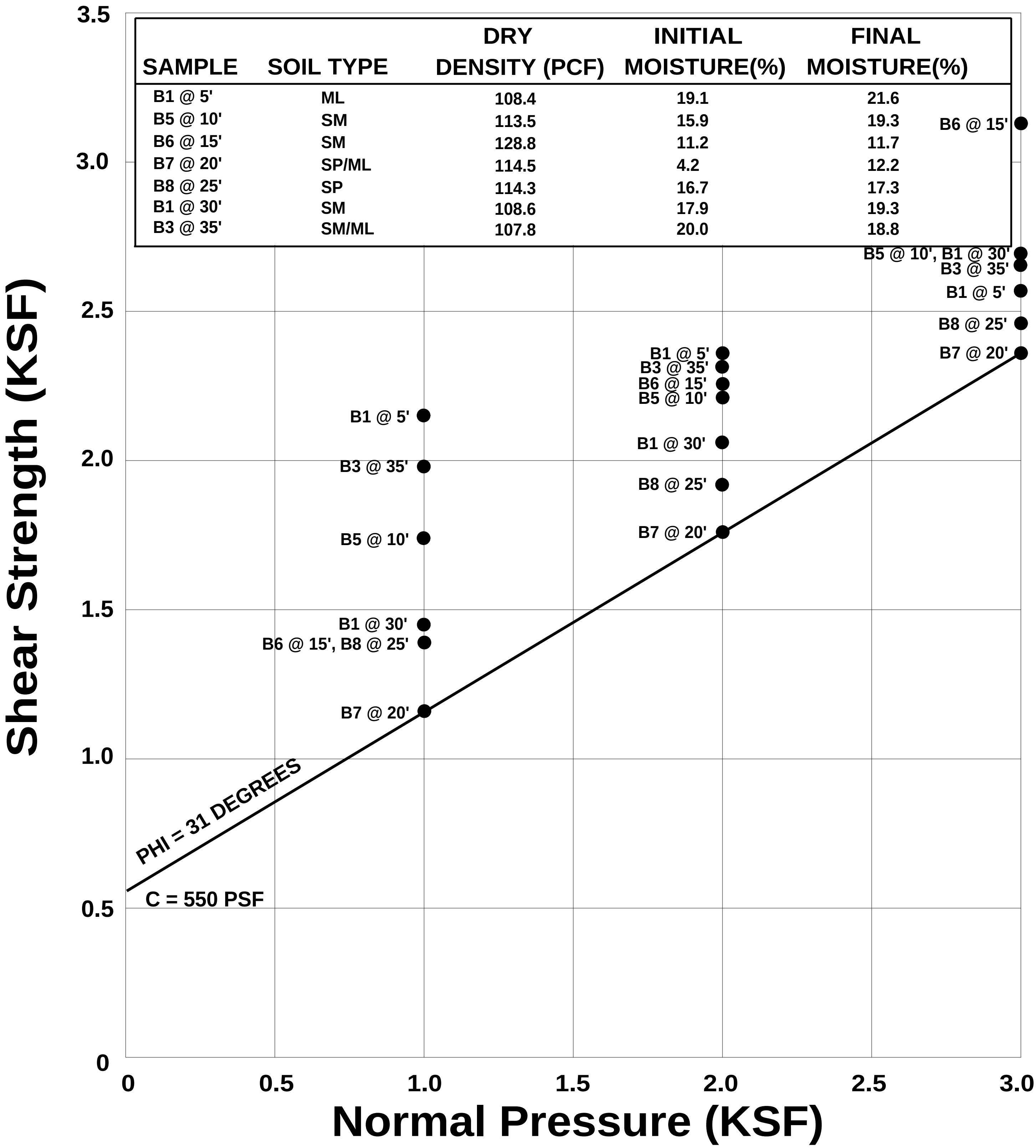
BORING LOG NUMBER 9

5420 Sunset Boulevard LP, LLC

File No. 21265

ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
30	68			30 --		
				-		more sandy
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
35	46			35 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark brown, moist, stiff
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-	ML/CL	Clayey Silt to Silty Clay, dark brown, moist, stiff
40	50			40 --		
				-		Total Depth 40 feet
				41 --		No Water
				-		Fill to 10 feet
				42 --		
				-		
				43 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				44 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				45 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
				50 --		
				-		



SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B1 @ 5'	ML	108.4	19.1	21.6
B5 @ 10'	SM	113.5	15.9	19.3
B6 @ 15'	SM	128.8	11.2	11.7
B7 @ 20'	SP/ML	114.5	4.2	12.2
B8 @ 25'	SP	114.3	16.7	17.3
B1 @ 30'	SM	108.6	17.9	19.3
B3 @ 35'	SM/ML	107.8	20.0	18.8

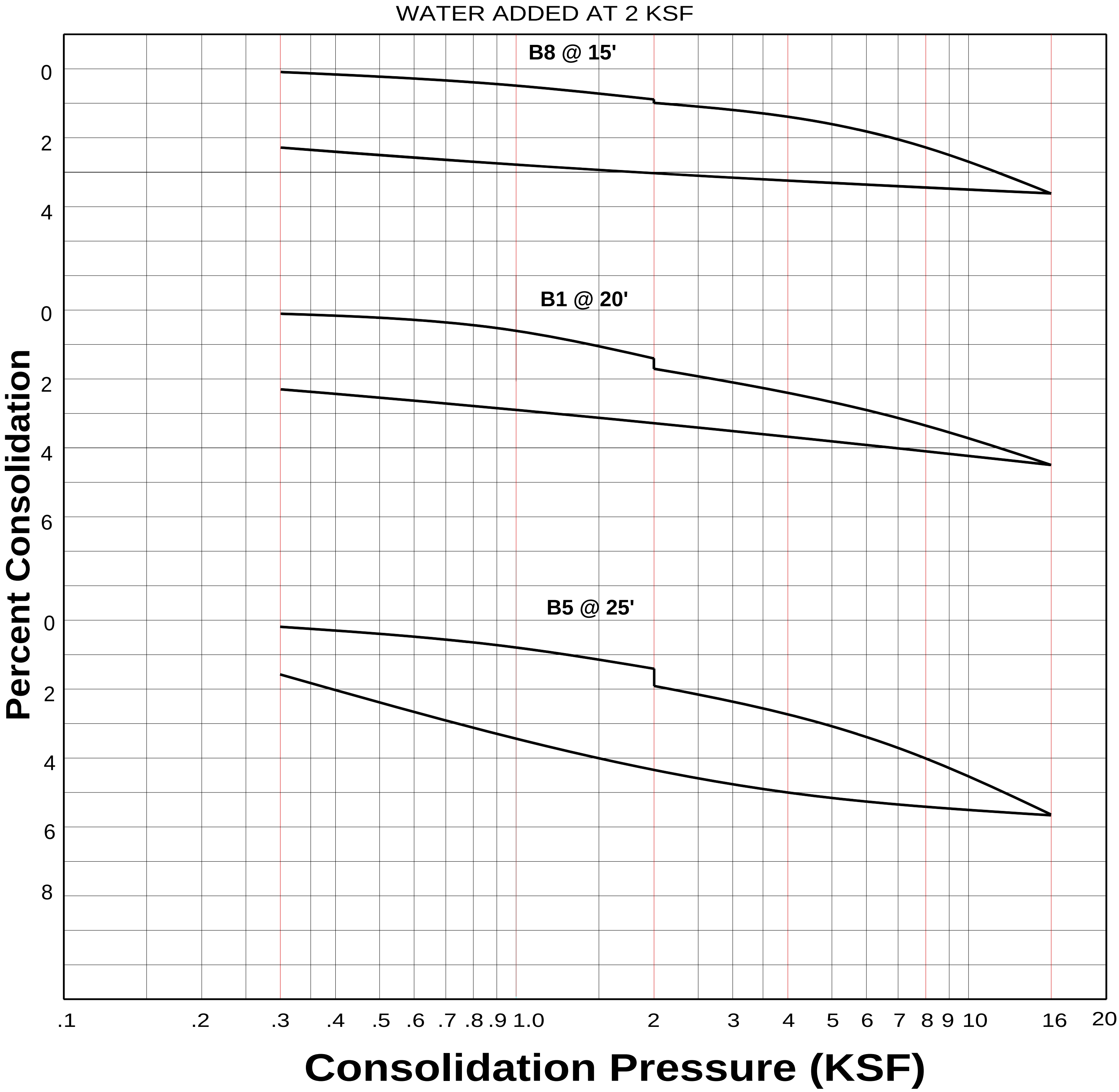
● Direct Shear, Saturated

SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

5420 SUNSET BOULEVARD, LP

FILE NO. 21265 | PLATE: B

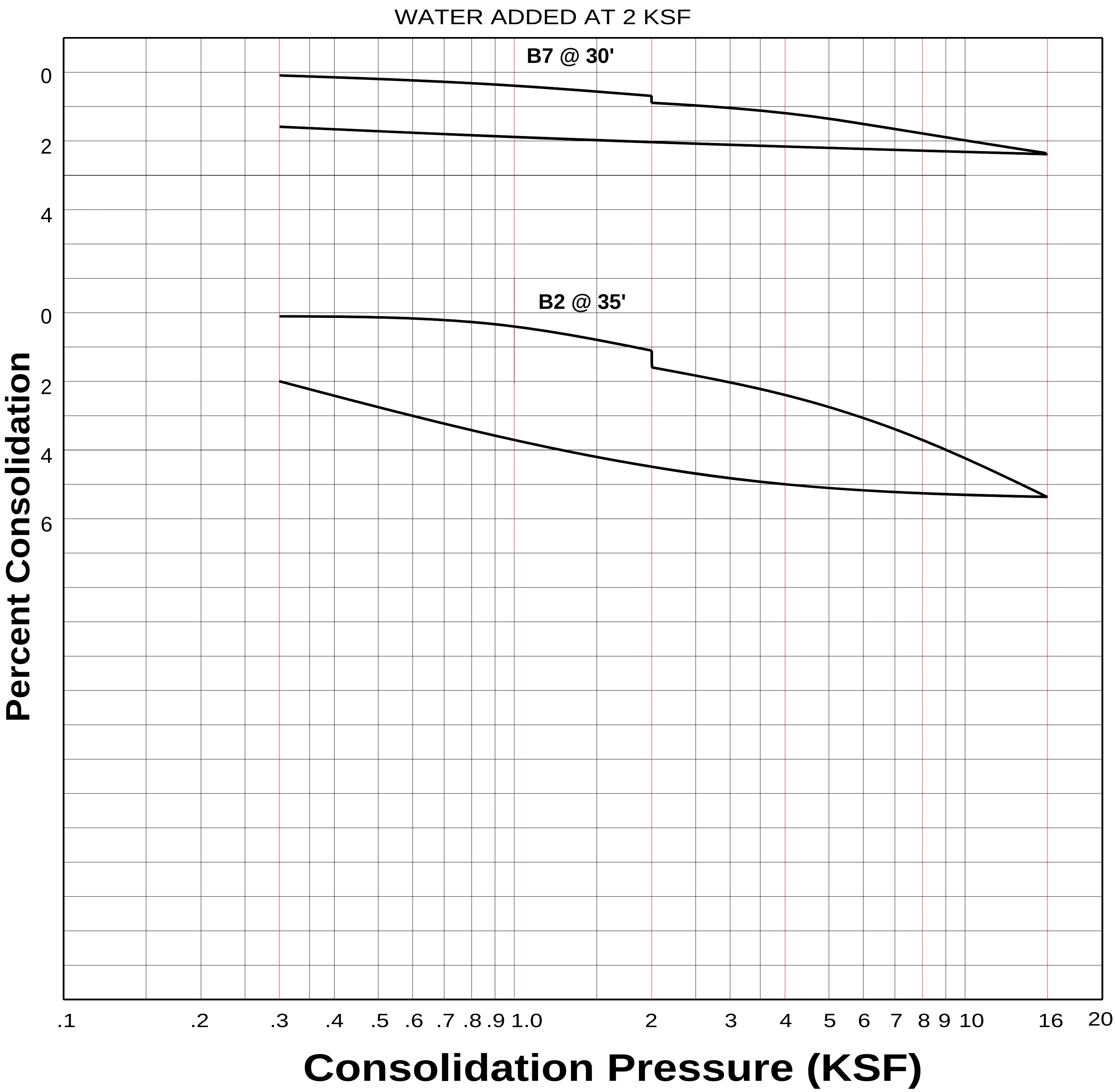


CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

5420 SUNSET BOULEVARD, LP

FILE NO. 21265 **PLATE: C-1**

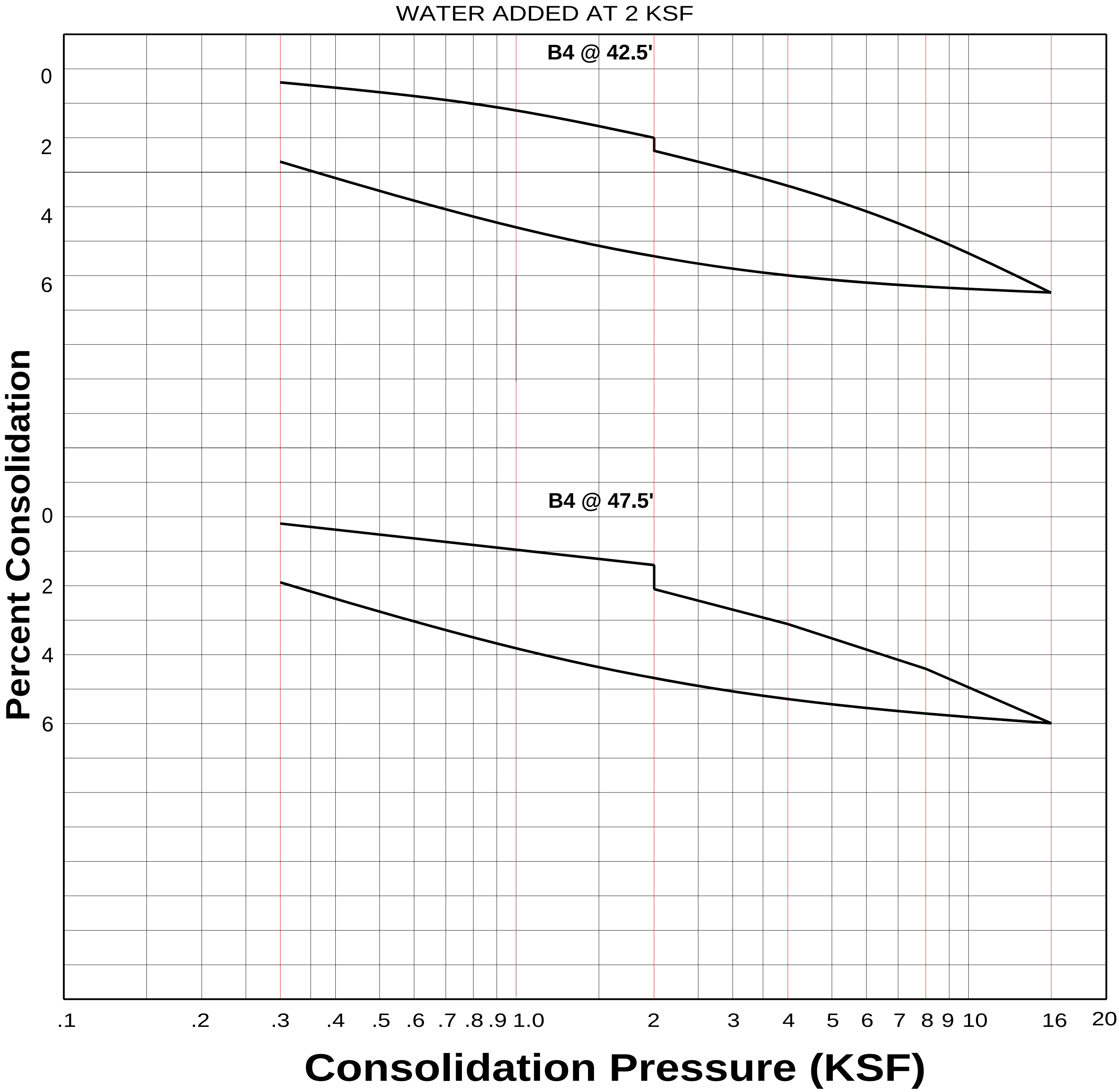


CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

5420 SUNSET BOULEVARD, LP

FILE NO. 21265 | **PLATE: C-2**



CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

5420 SUNSET BOULEVARD, LP

FILE NO. 21265 | **PLATE: C-3**

ASTM D-1557

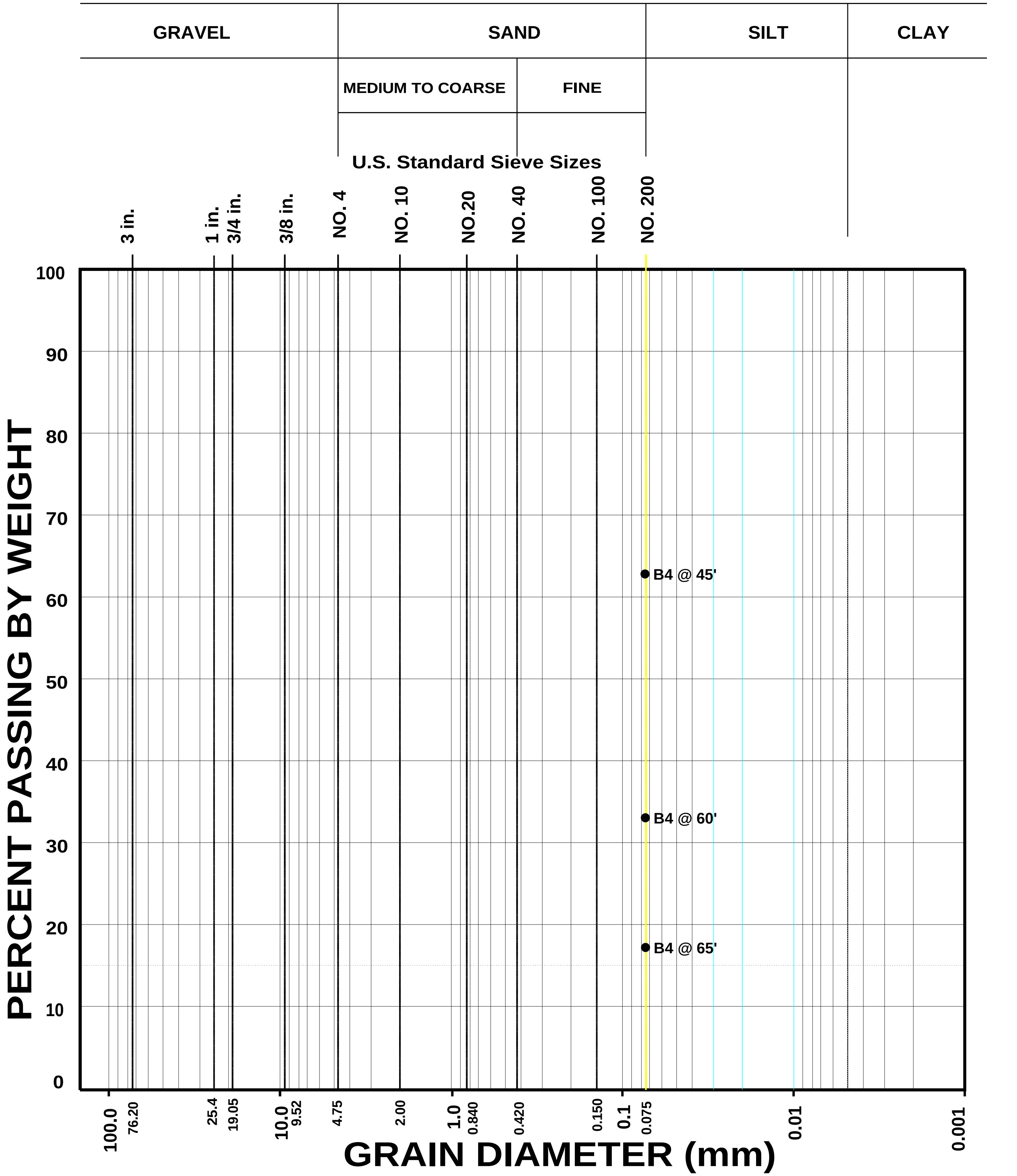
SAMPLE	B3 @ 1-5'	B6 @ 1-5'
SOIL TYPE:	ML	CL/ML
MAXIMUM DENSITY pcf.	113.6	111.5
OPTIMUM MOISTURE %	16.8	16.4

ASTM D 4829

SAMPLE	B3 @ 1- 5'	B6 @ 1-5'	B1 @ 15'	B3 @ 15'
SOIL TYPE:	ML	CL/ML	SM	ML
EXPANSION INDEX UBC STANDARD 18-2	130	156	27	21
EXPANSION CHARACTER	CRITICAL	CRITICAL	LOW	LOW

SULFATE CONTENT

SAMPLE	B3 @ 1-5'	B6 @ 1-5'
SULFATE CONTENT: (percentage by weight)	< 0.1%	< 0.1%



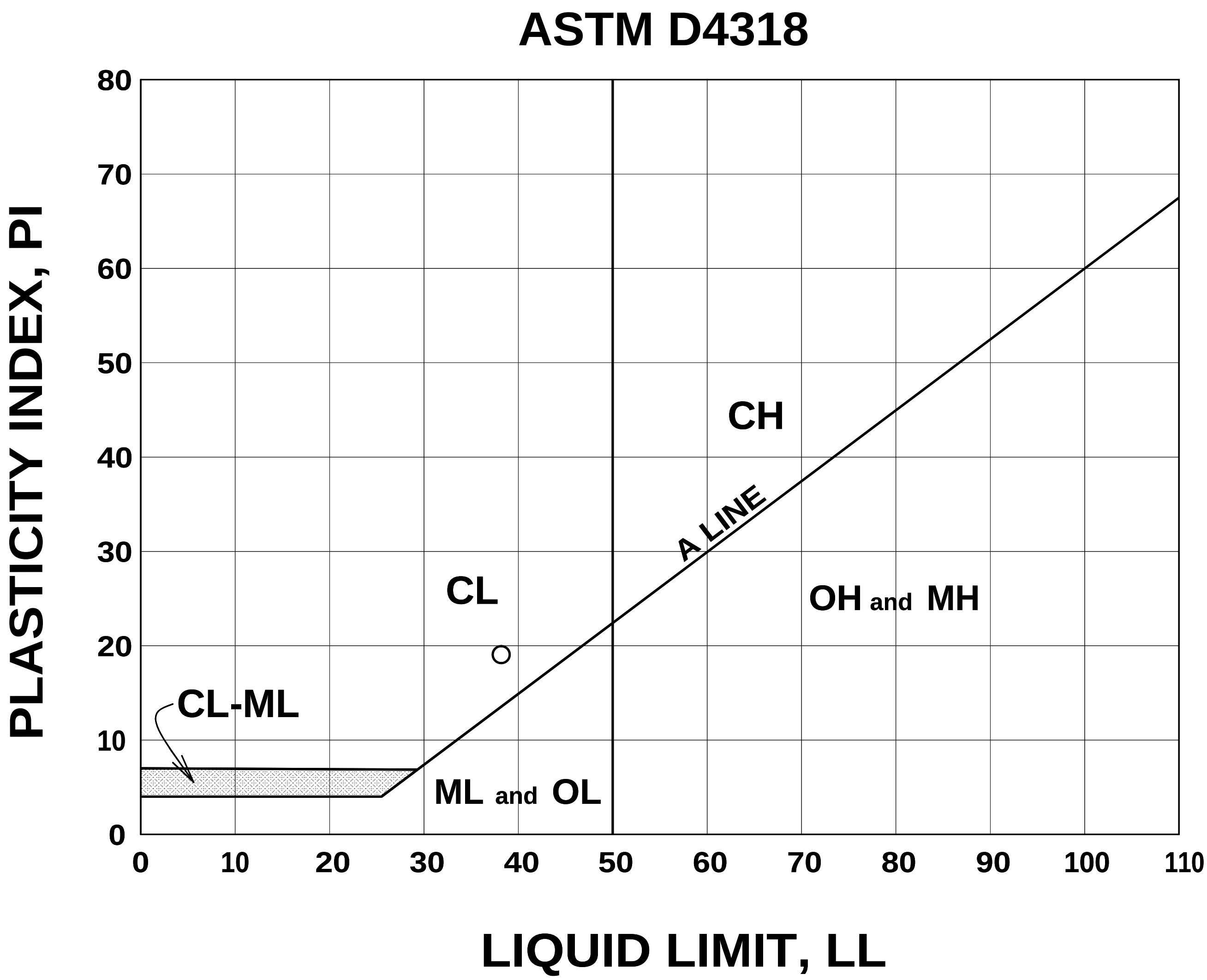
GRAIN SIZE DISTRIBUTION

Geotechnologies, Inc.
Consulting Geotechnical Engineers

5420 SUNSET BOULEVARD, LP

FILE NO. 21265

PLATE: E



BORING NUMBER	DEPTH (FEET)	TEST SYMBOL	LL	PL	PI	DESCRIPTION
B4	45	○	38	19	19	CL

ATTERBERG LIMITS DETERMINATION

Geotechnologies, Inc.
Consulting Geotechnical Engineers

5420 SUNSET BOULEVARD, LP

FILE NO. 21265 | PLATE: F

Geotechnologies, Inc.

Project: 5420 Sunset Boulevard
File No.: 21265
Description: Liquefaction Analysis (2% Exceedance in 50 Years)
Boring No: 4

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	6.6
Peak Ground Horizontal Acceleration, PGA (g):	0.95
Calculated Mag.Wtg.Factor:	1.267

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	53.0
Historically Highest Groundwater Level* (ft):	42.0
Unit Weight of Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

BOREHOLE AND SAMPLER INFORMATION:

Borehole Diameter (inches):	8
SPT Sampler with room for Liner (Y/N):	Y

LIQUEFACTION BOUNDARY:

Plastic Index Cut Off (PI):	18
Minimum Liquefaction FS:	1

Depth to Base Layer (feet)	Total Unit Weight (pcf)	Current Water Level (feet)	Historical Water Level (feet)	Field SPT Blowcount N	Depth of SPT Blowcount (feet)	Fines Content #200 Sieve (%)	Plastic Index (PI)	Vetical Stress σ_{vc}, (psf)	Effective Vert. Stress σ_{vc}', (psf)	Fines Corrected $(N_1)_{60\text{-}cs}$	Stress Reduction Coeff, r_d	Cyclic Shear Ratio CSR	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Liquefaction Settlment ΔS_i (inches)
1	121.5	Unsaturated	Unsaturated	13	5	0.0	0	121.5	121.5	29.5	1.00	0.620	0.635	Non-Liq.	0.00
2	121.5	Unsaturated	Unsaturated	13	5	0.0	0	243.0	243.0	29.5	1.00	0.618	0.635	Non-Liq.	0.00
3	121.5	Unsaturated	Unsaturated	13	5	0.0	0	364.5	364.5	29.5	1.00	0.616	0.635	Non-Liq.	0.00
4	121.5	Unsaturated	Unsaturated	13	5	0.0	0	486.0	486.0	29.5	0.99	0.613	0.635	Non-Liq.	0.00
5	121.5	Unsaturated	Unsaturated	13	5	0.0	0	607.5	607.5	29.6	0.99	0.611	0.641	Non-Liq.	0.00
6	121.5	Unsaturated	Unsaturated	13	5	0.0	0	729.0	729.0	28.0	0.99	0.608	0.535	Non-Liq.	0.00
7	121.5	Unsaturated	Unsaturated	13	5	0.0	0	850.5	850.5	26.3	0.98	0.606	0.454	Non-Liq.	0.00
8	130.1	Unsaturated	Unsaturated	7	10	0.0	0	980.6	980.6	13.3	0.98	0.603	0.195	Non-Liq.	0.00
9	130.1	Unsaturated	Unsaturated	7	10	0.0	0	1110.7	1110.7	13.3	0.97	0.600	0.192	Non-Liq.	0.00
10	130.1	Unsaturated	Unsaturated	7	10	0.0	0	1240.8	1240.8	12.5	0.97	0.598	0.182	Non-Liq.	0.00
11	130.1	Unsaturated	Unsaturated	7	10	0.0	0	1370.9	1370.9	11.9	0.96	0.595	0.174	Non-Liq.	0.00
12	130.1	Unsaturated	Unsaturated	7	10	0.0	0	1501.0	1501.0	11.3	0.96	0.592	0.167	Non-Liq.	0.00
13	139.3	Unsaturated	Unsaturated	7	10	0.0	0	1640.3	1640.3	10.8	0.95	0.588	0.160	Non-Liq.	0.00
14	139.3	Unsaturated	Unsaturated	7	10	0.0	0	1779.6	1779.6	10.3	0.95	0.585	0.155	Non-Liq.	0.00
15	139.3	Unsaturated	Unsaturated	17	15	0.0	0	1918.9	1918.9	29.9	0.94	0.582	0.615	Non-Liq.	0.00
16	139.3	Unsaturated	Unsaturated	17	15	0.0	0	2058.2	2058.2	28.9	0.94	0.579	0.542	Non-Liq.	0.00
17	139.3	Unsaturated	Unsaturated	17	15	0.0	0	2197.5	2197.5	28.1	0.93	0.575	0.486	Non-Liq.	0.00
18	142.4	Unsaturated	Unsaturated	17	15	0.0	0	2339.9	2339.9	27.2	0.93	0.572	0.441	Non-Liq.	0.00
19	142.4	Unsaturated	Unsaturated	17	15	0.0	0	2482.3	2482.3	26.5	0.92	0.569	0.406	Non-Liq.	0.00
20	142.4	Unsaturated	Unsaturated	17	20	0.0	0	2624.7	2624.7	25.8	0.92	0.565	0.378	Non-Liq.	0.00
21	142.4	Unsaturated	Unsaturated	17	20	0.0	0	2767.1	2767.1	25.1	0.91	0.561	0.354	Non-Liq.	0.00
22	142.4	Unsaturated	Unsaturated	17	20	0.0	0	2909.5	2909.5	24.5	0.90	0.558	0.334	Non-Liq.	0.00
23	127.1	Unsaturated	Unsaturated	17	20	0.0	0	3036.6	3036.6	23.9	0.90	0.554	0.319	Non-Liq.	0.00
24	127.1	Unsaturated	Unsaturated	17	20	0.0	0	3163.7	3163.7	23.4	0.89	0.550	0.305	Non-Liq.	0.00
25	127.1	Unsaturated	Unsaturated	7	25	0.0	0	3290.8	3290.8	8.2	0.89	0.547	0.129	Non-Liq.	0.00
26	127.1	Unsaturated	Unsaturated	7	25	0.0	0	3417.9	3417.9	8.0	0.88	0.543	0.127	Non-Liq.	0.00
27	127.1	Unsaturated	Unsaturated	7	25	0.0	0	3545.0	3545.0	7.8	0.87	0.539	0.125	Non-Liq.	0.00
28	132.1	Unsaturated	Unsaturated	7	25	0.0	0	3677.1	3677.1	8.1	0.87	0.535	0.127	Non-Liq.	0.00
29	132.1	Unsaturated	Unsaturated	7	25	0.0	0	3809.2	3809.2	7.9	0.86	0.531	0.125	Non-Liq.	0.00
30	132.1	Unsaturated	Unsaturated	10	30	0.0	0	3941.3	3941.3	11.4	0.85	0.528	0.153	Non-Liq.	0.00
31	132.1	Unsaturated	Unsaturated	10	30	0.0	0	4073.4	4073.4	11.2	0.85	0.524	0.150	Non-Liq.	0.00
32	132.1	Unsaturated	Unsaturated	10	30	0.0	0	4205.5	4205.5	11.0	0.84	0.520	0.148	Non-Liq.	0.00
33	123.7	Unsaturated	Unsaturated	10	30	0.0	0	4329.2	4329.2	10.8	0.84	0.516	0.146	Non-Liq.	0.00
34	123.7	Unsaturated	Unsaturated	10	30	0.0	0	4452.9	4452.9	10.6	0.83	0.512	0.144	Non-Liq.	0.00
35	123.7	Unsaturated	Unsaturated	14	35	0.0	0	4576.6	4576.6	15.8	0.82	0.508	0.188	Non-Liq.	0.00
36	123.7	Unsaturated	Unsaturated	14	35	0.0	0	4700.3	4700.3	15.5	0.82	0.504	0.185	Non-Liq.	0.00
37	123.7	Unsaturated	Unsaturated	14	35	0.0	0	4824.0	4824.0	15.3	0.81	0.500	0.182	Non-Liq.	0.00
38	117.5	Unsaturated	Unsaturated	14	35	0.0	0	4941.5	4941.5	15.1	0.80	0.496	0.180	Non-Liq.	0.00
39	117.5	Unsaturated	Unsaturated	14	35	0.0	0	5059.0	5059.0	14.8	0.80	0.492	0.177	Non-Liq.	0.00
40	117.5	Unsaturated	Unsaturated	38	40	0.0	0	5176.5	5176.5	56.1	0.79	0.488	1.863	Non-Liq.	0.00
41	117.5	Unsaturated	Unsaturated	38	40	0.0	0	5294.0	5294.0	55.7	0.78	0.484	1.846	Non-Liq.	0.00
42	117.5	Unsaturated	Unsaturated	38	40	0.0	0	5411.5	5411.5	55.4	0.78	0.480	1.830	Non-Liq.	0.00
43	129.3	Unsaturated	Saturated	15	45	62.9	19	5540.8	5478.4	21.0	0.77	0.482	0.240	Non-Liq.	0.00
44	129.3	Unsaturated	Saturated	15	45	62.9	19	5670.1	5545.3	20.7	0.77	0.483	0.237	Non-Liq.	0.00
45	129.3	Unsaturated	Saturated	15	45	62.9	19	5799.4	5612.2	20.5	0.76	0.484	0.233	Non-Liq.	0.00
46	129.3	Unsaturated	Saturated	15	45	62.9	19	5928.7	5679.1	20.3	0.75	0.485	0.230	Non-Liq.	0.00
47	129.3	Unsaturated	Saturated	15	45	62.9	19	6058.0	5746.0	20.1	0.75	0.486	0.228	Non-Liq.	0.00
48	126.0	Unsaturated	Saturated	15	45	62.9	19	6184.0	5809.6	20.0	0.74	0.487	0.225	Non-Liq.	0.00
49	126.0	Unsaturated	Saturated	15	45	62.9	19	6310.0	5873.2	19.8	0.73	0.487	0.223	Non-Liq.	0.00
50	142.9	Unsaturated	Saturated	43	50	0.0	0	6452.9	5953.7	59.9	0.73	0.487	1.758	3.6	0.00
51	142.9	Unsaturated	Saturated	43	50	0.0	0	6595.8	6034.2	59.5	0.72	0.487	1.748	3.6	0.00
52	142.9	Unsaturated	Saturated	43	50	0.0	0	6738.7	6114.7	59.2	0.72	0.487	1.738	3.6	0.00
53	142.9	Unsaturated	Saturated	43	50	0.0	0	6881.6	6195.2	58.9	0.71	0.487	1.729	3.5	0.00
54	142.9	Saturated	Saturated	43	50	0.0	0	7024.5	6275.7	58.7	0.70	0.487	1.719	3.5	0.00
55	142.9	Saturated	Saturated	27	55	0.0	0	7167.4	6356.2	31.9	0.70	0.486	0.607	1.2	0.00
56	142.9	Saturated	Saturated	27	55	0.0	0	7310.3	6436.7	31.7	0.69	0.486	0.587	1.2	0.00
57	142.9	Saturated	Saturated	27	55	0.0	0	7453.2	6517.2	31.5	0.69	0.485	0.569	1.2	0.00
58	136.4	Saturated	Saturated	27	55	0.0	0	7589.6	6591.2	31.3	0.68	0.484	0.553	1.1	0.00
59	136.4	Saturated	Saturated	27	55	0.0	0	7726.0	6665.2	31.1	0.68	0.484	0.538	1.1	0.00
60	136.4	Saturated	Saturated	25	60	33.0	0	7862.4	6739.2	32.6	0.67	0.483	0.663	1.4	0.00
61	136.4	Saturated	Saturated	25	60	33.0	0	7998.8	6813.2	32.5	0.66	0.482	0.645	1.3	0.00
62	136.4	Saturated	Saturated	25	60	33.0	0	8135.2	6887.2	32.3	0.66	0.481	0.627	1.3	0.00
63	127.5	Saturated	Saturated	25	60	33.0	0	8262.7	6952.3	32.2	0.65	0.480	0.613	1.3	0.00
64	127.5	Saturated	Saturated	25	60	33.0	0	8390.2	7017.4	32.0	0.65	0.479	0.599	1.3	0.00
65	127.5	Saturated	Saturated	28	65	17.2	0	8517.7	7082.5	35.9	0.64	0.478	1.142	2.4	0.00
											Total Liquefaction Settlement, S =			**0.00 inches**	

Geotechnologies, Inc.

Project: 5420 Sunset Boulevard
File No.: 21265
Description: Liquefaction Analysis (10% Exceedance in 50 Years)
Boring No: 4

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	6.6
Peak Ground Horizontal Acceleration, PGA (g):	0.64
Calculated Mag.Wtg.Factor:	1.267

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	53.0
Historically Highest Groundwater Level* (ft):	42.0
Unit Weight of Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

BOREHOLE AND SAMPLER INFORMATION:

Borehole Diameter (inches):	8
SPT Sampler with room for Liner (Y/N):	Y

LIQUEFACTION BOUNDARY:

Plastic Index Cut Off (PI):	18
Minimum Liquefaction FS:	1.1

Depth to Base Layer (feet)	Total Unit Weight (pcf)	Current Water Level (feet)	Historical Water Level (feet)	Field SPT Blowcount N	Depth of SPT Blowcount (feet)	Fines Content #200 Sieve (%)	Plastic Index (PI)	Vetical Stress σ_{vc}, (psf)	Effective Vert. Stress σ_{vc}', (psf)	Fines Corrected $(N_1)_{60\text{-}cs}$	Stress Reduction Coeff, r_d	Cyclic Shear Ratio CSR	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Liquefaction Settlment ΔS_i (inches)
1	121.5	Unsaturated	Unsaturated	13	5	0.0	0	121.5	121.5	29.5	1.00	0.416	0.635	Non-Liq.	0.00
2	121.5	Unsaturated	Unsaturated	13	5	0.0	0	243.0	243.0	29.5	1.00	0.415	0.635	Non-Liq.	0.00
3	121.5	Unsaturated	Unsaturated	13	5	0.0	0	364.5	364.5	29.5	1.00	0.413	0.635	Non-Liq.	0.00
4	121.5	Unsaturated	Unsaturated	13	5	0.0	0	486.0	486.0	29.5	0.99	0.412	0.635	Non-Liq.	0.00
5	121.5	Unsaturated	Unsaturated	13	5	0.0	0	607.5	607.5	29.6	0.99	0.410	0.641	Non-Liq.	0.00
6	121.5	Unsaturated	Unsaturated	13	5	0.0	0	729.0	729.0	28.0	0.99	0.409	0.535	Non-Liq.	0.00
7	121.5	Unsaturated	Unsaturated	13	5	0.0	0	850.5	850.5	26.3	0.98	0.407	0.454	Non-Liq.	0.00
8	130.1	Unsaturated	Unsaturated	7	10	0.0	0	980.6	980.6	13.3	0.98	0.405	0.195	Non-Liq.	0.00
9	130.1	Unsaturated	Unsaturated	7	10	0.0	0	1110.7	1110.7	13.3	0.97	0.403	0.192	Non-Liq.	0.00
10	130.1	Unsaturated	Unsaturated	7	10	0.0	0	1240.8	1240.8	12.5	0.97	0.401	0.182	Non-Liq.	0.00
11	130.1	Unsaturated	Unsaturated	7	10	0.0	0	1370.9	1370.9	11.9	0.96	0.399	0.174	Non-Liq.	0.00
12	130.1	Unsaturated	Unsaturated	7	10	0.0	0	1501.0	1501.0	11.3	0.96	0.397	0.167	Non-Liq.	0.00
13	139.3	Unsaturated	Unsaturated	7	10	0.0	0	1640.3	1640.3	10.8	0.95	0.395	0.160	Non-Liq.	0.00
14	139.3	Unsaturated	Unsaturated	7	10	0.0	0	1779.6	1779.6	10.3	0.95	0.393	0.155	Non-Liq.	0.00
15	139.3	Unsaturated	Unsaturated	17	15	0.0	0	1918.9	1918.9	29.9	0.94	0.391	0.615	Non-Liq.	0.00
16	139.3	Unsaturated	Unsaturated	17	15	0.0	0	2058.2	2058.2	28.9	0.94	0.389	0.542	Non-Liq.	0.00
17	139.3	Unsaturated	Unsaturated	17	15	0.0	0	2197.5	2197.5	28.1	0.93	0.386	0.486	Non-Liq.	0.00
18	142.4	Unsaturated	Unsaturated	17	15	0.0	0	2339.9	2339.9	27.2	0.93	0.384	0.441	Non-Liq.	0.00
19	142.4	Unsaturated	Unsaturated	17	15	0.0	0	2482.3	2482.3	26.5	0.92	0.382	0.406	Non-Liq.	0.00
20	142.4	Unsaturated	Unsaturated	17	20	0.0	0	2624.7	2624.7	25.8	0.92	0.379	0.378	Non-Liq.	0.00
21	142.4	Unsaturated	Unsaturated	17	20	0.0	0	2767.1	2767.1	25.1	0.91	0.377	0.354	Non-Liq.	0.00
22	142.4	Unsaturated	Unsaturated	17	20	0.0	0	2909.5	2909.5	24.5	0.90	0.375	0.334	Non-Liq.	0.00
23	127.1	Unsaturated	Unsaturated	17	20	0.0	0	3036.6	3036.6	23.9	0.90	0.372	0.319	Non-Liq.	0.00
24	127.1	Unsaturated	Unsaturated	17	20	0.0	0	3163.7	3163.7	23.4	0.89	0.370	0.305	Non-Liq.	0.00
25	127.1	Unsaturated	Unsaturated	7	25	0.0	0	3290.8	3290.8	8.2	0.89	0.367	0.129	Non-Liq.	0.00
26	127.1	Unsaturated	Unsaturated	7	25	0.0	0	3417.9	3417.9	8.0	0.88	0.365	0.127	Non-Liq.	0.00
27	127.1	Unsaturated	Unsaturated	7	25	0.0	0	3545.0	3545.0	7.8	0.87	0.362	0.125	Non-Liq.	0.00
28	132.1	Unsaturated	Unsaturated	7	25	0.0	0	3677.1	3677.1	8.1	0.87	0.360	0.127	Non-Liq.	0.00
29	132.1	Unsaturated	Unsaturated	7	25	0.0	0	3809.2	3809.2	7.9	0.86	0.357	0.125	Non-Liq.	0.00
30	132.1	Unsaturated	Unsaturated	10	30	0.0	0	3941.3	3941.3	11.4	0.85	0.354	0.153	Non-Liq.	0.00
31	132.1	Unsaturated	Unsaturated	10	30	0.0	0	4073.4	4073.4	11.2	0.85	0.352	0.150	Non-Liq.	0.00
32	132.1	Unsaturated	Unsaturated	10	30	0.0	0	4205.5	4205.5	11.0	0.84	0.349	0.148	Non-Liq.	0.00
33	123.7	Unsaturated	Unsaturated	10	30	0.0	0	4329.2	4329.2	10.8	0.84	0.346	0.146	Non-Liq.	0.00
34	123.7	Unsaturated	Unsaturated	10	30	0.0	0	4452.9	4452.9	10.6	0.83	0.344	0.144	Non-Liq.	0.00
35	123.7	Unsaturated	Unsaturated	14	35	0.0	0	4576.6	4576.6	15.8	0.82	0.341	0.188	Non-Liq.	0.00
36	123.7	Unsaturated	Unsaturated	14	35	0.0	0	4700.3	4700.3	15.5	0.82	0.339	0.185	Non-Liq.	0.00
37	123.7	Unsaturated	Unsaturated	14	35	0.0	0	4824.0	4824.0	15.3	0.81	0.336	0.182	Non-Liq.	0.00
38	117.5	Unsaturated	Unsaturated	14	35	0.0	0	4941.5	4941.5	15.1	0.80	0.333	0.180	Non-Liq.	0.00
39	117.5	Unsaturated	Unsaturated	14	35	0.0	0	5059.0	5059.0	14.8	0.80	0.331	0.177	Non-Liq.	0.00
40	117.5	Unsaturated	Unsaturated	38	40	0.0	0	5176.5	5176.5	56.1	0.79	0.328	1.863	Non-Liq.	0.00
41	117.5	Unsaturated	Unsaturated	38	40	0.0	0	5294.0	5294.0	55.7	0.78	0.325	1.846	Non-Liq.	0.00
42	117.5	Unsaturated	Unsaturated	38	40	0.0	0	5411.5	5411.5	55.4	0.78	0.323	1.830	Non-Liq.	0.00
43	129.3	Unsaturated	Saturated	15	45	62.9	19	5540.8	5478.4	21.0	0.77	0.324	0.240	Non-Liq.	0.00
44	129.3	Unsaturated	Saturated	15	45	62.9	19	5670.1	5545.3	20.7	0.77	0.325	0.237	Non-Liq.	0.00
45	129.3	Unsaturated	Saturated	15	45	62.9	19	5799.4	5612.2	20.5	0.76	0.325	0.233	Non-Liq.	0.00
46	129.3	Unsaturated	Saturated	15	45	62.9	19	5928.7	5679.1	20.3	0.75	0.326	0.230	Non-Liq.	0.00
47	129.3	Unsaturated	Saturated	15	45	62.9	19	6058.0	5746.0	20.1	0.75	0.326	0.228	Non-Liq.	0.00
48	126.0	Unsaturated	Saturated	15	45	62.9	19	6184.0	5809.6	20.0	0.74	0.327	0.225	Non-Liq.	0.00
49	126.0	Unsaturated	Saturated	15	45	62.9	19	6310.0	5873.2	19.8	0.73	0.327	0.223	Non-Liq.	0.00
50	142.9	Unsaturated	Saturated	43	50	0.0	0	6452.9	5953.7	59.9	0.73	0.327	1.758	5.4	0.00
51	142.9	Unsaturated	Saturated	43	50	0.0	0	6595.8	6034.2	59.5	0.72	0.327	1.748	5.3	0.00
52	142.9	Unsaturated	Saturated	43	50	0.0	0	6738.7	6114.7	59.2	0.72	0.327	1.738	5.3	0.00
53	142.9	Unsaturated	Saturated	43	50	0.0	0	6881.6	6195.2	58.9	0.71	0.327	1.729	5.3	0.00
54	142.9	Saturated	Saturated	43	50	0.0	0	7024.5	6275.7	58.7	0.70	0.327	1.719	5.3	0.00
55	142.9	Saturated	Saturated	27	55	0.0	0	7167.4	6356.2	31.9	0.70	0.327	0.607	1.9	0.00
56	142.9	Saturated	Saturated	27	55	0.0	0	7310.3	6436.7	31.7	0.69	0.326	0.587	1.8	0.00
57	142.9	Saturated	Saturated	27	55	0.0	0	7453.2	6517.2	31.5	0.69	0.326	0.569	1.7	0.00
58	136.4	Saturated	Saturated	27	55	0.0	0	7589.6	6591.2	31.3	0.68	0.325	0.553	1.7	0.00
59	136.4	Saturated	Saturated	27	55	0.0	0	7726.0	6665.2	31.1	0.68	0.325	0.538	1.7	0.00
60	136.4	Saturated	Saturated	25	60	33.0	0	7862.4	6739.2	32.6	0.67	0.324	0.663	2.0	0.00
61	136.4	Saturated	Saturated	25	60	33.0	0	7998.8	6813.2	32.5	0.66	0.324	0.645	2.0	0.00
62	136.4	Saturated	Saturated	25	60	33.0	0	8135.2	6887.2	32.3	0.66	0.323	0.627	1.9	0.00
63	127.5	Saturated	Saturated	25	60	33.0	0	8262.7	6952.3	32.2	0.65	0.322	0.613	1.9	0.00
64	127.5	Saturated	Saturated	25	60	33.0	0	8390.2	7017.4	32.0	0.65	0.322	0.599	1.9	0.00
65	127.5	Saturated	Saturated	28	65	17.2	0	8517.7	7082.5	35.9	0.64	0.321	1.142	3.6	0.00

Total Liquefaction Settlement, S = 0.00 inches

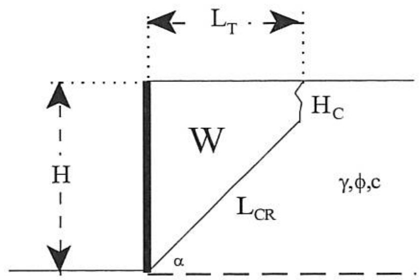
Geotechnologies, Inc.

Project: 5420 Sunset
File No.: 21265
Description: Retaining Walls

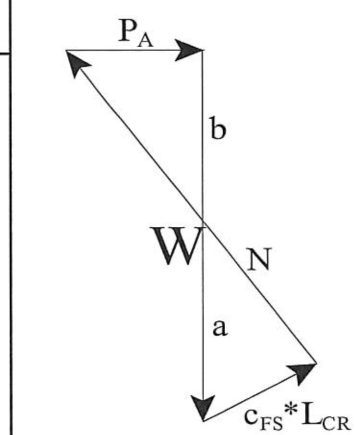
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	25.00 feet
Unit Weight of Retained Soils	(γ)	120.0 pcf
Friction Angle of Retained Soils	(ϕ)	31.0 degrees
Cohesion of Retained Soils	(c)	550.0 psf
Factor of Safety	(FS)	1.50
Factored Parameters:	(ϕ_{FS})	21.8 degrees
	(c_{FS})	366.7 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	10.2	261	31263.8	20.9	18112.4	13151.4	5628.6
46	10.0	254	30451.0	20.9	17366.4	13084.7	5872.3
47	9.8	247	29619.0	20.8	16656.0	12962.9	6091.7
48	9.6	240	28774.5	20.7	15981.0	12793.4	6286.9
49	9.5	233	27922.6	20.6	15340.2	12582.4	6458.2
50	9.3	226	27067.5	20.4	14732.2	12335.3	6605.9
51	9.2	218	26212.2	20.3	14155.3	12056.8	6730.1
52	9.2	211	25358.8	20.1	13607.7	11751.1	6831.1
53	9.1	204	24509.1	19.9	13087.6	11421.6	6909.1
54	9.1	197	23664.5	19.7	12593.0	11071.5	6964.1
55	9.0	190	22825.6	19.5	12122.1	10703.5	6996.3
56	9.0	183	21993.2	19.3	11673.1	10320.1	7005.7
57	9.0	176	21167.5	19.0	11244.2	9923.3	6992.4
58	9.1	170	20348.8	18.8	10833.8	9515.0	6956.3
59	9.1	163	19536.9	18.5	10440.1	9096.8	6897.4
60	9.2	156	18731.7	18.3	10061.4	8670.3	6815.6
61	9.3	149	17933.0	18.0	9696.2	8236.8	6710.6
62	9.4	143	17140.4	17.7	9343.0	7797.5	6582.4
63	9.5	136	16353.4	17.4	9000.0	7353.5	6430.7
64	9.6	130	15571.5	17.1	8665.6	6905.9	6255.4
65	9.8	123	14794.1	16.8	8338.3	6455.7	6056.0
66	10.0	117	14020.3	16.4	8016.3	6004.0	5832.6
67	10.2	110	13249.4	16.0	7697.8	5551.6	5584.7
68	10.5	104	12480.4	15.6	7380.8	5099.6	5312.3
69	10.8	98	11712.2	15.2	7063.1	4649.1	5015.3
70	11.1	91	10943.7	14.8	6742.5	4201.2	4693.9



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 7005.7 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 22.4 pcf Use 30pcf

Geotechnologies, Inc.

Project: 5420 Sunset
File No.: 21265

Soil Weight	γ	120	pcf
Internal Friction Angle	ϕ	31	degrees
Cohesion	c	550	psf
Height of Retaining Wall	H	25	feet

Cantilever Retaining Wall Design based on At Rest Earth Pressure

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

$K_o = 1 - \sin\phi$	0.485	
$\sigma'_v = \gamma H$	3000.0	psf

$\sigma'_h =$ 1454.9 psf

EFP = 58.2 pcf

$P_o =$ 18186.1 lbs/ft (based on a triangular distribution of pressure)

Design wall for an EFP of 58.2 pcf

Restrained Wall Design based on At Rest Earth Pressure

$P_o =$ 18186.1 lbs/ft

$\sigma'_{h,\,max} =$ 36.4 H (based on a trapezoidal distribution of pressure)

$\sigma'_{h,\,max} =$ 727.4 psf

Design restrained wall for 42.1 H

Geotechnologies, Inc.

Project: 5420 Sunset

File No.: 5420

Seismically Induced Lateral Soil Pressure on Retaining Wall

<u>Input:</u>

Height of Retaining Wall:	(H)	25.0 feet
Retained Soil Unit Weight:	(γ)	120.0 pcf
Horizontal Ground Acceleration:	(k_h)	0.32 g

Seismic Increment (ΔP_{AE}):

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

$\Delta P_{AE} =$ 9000.0 lbs/ft

Force applied at 0.6H above the base of the wall

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

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

$T =$ 8100.0 lbs/ft

$EFP = 2*T/H^2$

EFP = 25.9 pcf

Geotechnologies, Inc.

Project: 5420 Sunset

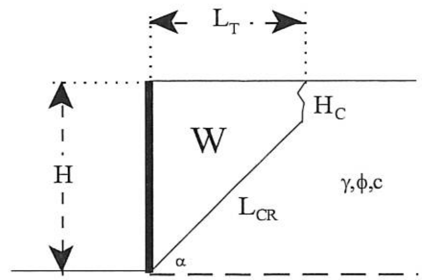
File No.: 21265

Description: Shoring

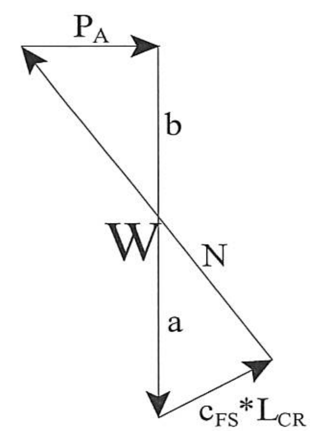
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	25.00 feet
Unit Weight of Retained Soils	(γ)	120.0 pcf
Friction Angle of Retained Soils	(ϕ)	31.0 degrees
Cohesion of Retained Soils	(c)	550.0 psf
Factor of Safety	(FS)	1.25
Factored Parameters:	(ϕ_{FS})	25.7 degrees
	(c_{FS})	440.0 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	14.1	213	25535.4	15.4	18434.6	7100.8	2490.4
46	13.7	211	25346.4	15.7	17941.2	7405.2	2743.2
47	13.3	209	25037.2	16.0	17408.8	7628.4	2978.3
48	13.0	205	24634.3	16.1	16855.5	7778.8	3194.6
49	12.7	201	24158.1	16.3	16294.0	7864.1	3391.2
50	12.5	197	23624.4	16.3	15733.1	7891.3	3567.5
51	12.3	192	23045.5	16.4	15178.5	7867.0	3723.2
52	12.1	187	22431.2	16.4	14634.3	7796.9	3858.0
53	12.0	182	21789.1	16.3	14102.9	7686.2	3971.7
54	11.8	176	21125.2	16.3	13585.7	7539.5	4064.2
55	11.8	170	20444.3	16.2	13083.3	7361.0	4135.4
56	11.7	165	19750.2	16.0	12595.8	7154.5	4185.3
57	11.7	159	19045.9	15.9	12122.8	6923.1	4213.8
58	11.7	153	18333.6	15.7	11663.6	6670.0	4221.0
59	11.7	147	17615.2	15.5	11217.5	6397.7	4206.8
60	11.7	141	16891.9	15.3	10783.2	6108.7	4171.3
61	11.8	135	16164.8	15.1	10359.6	5805.2	4114.4
62	11.9	129	15434.6	14.9	9945.2	5489.3	4036.3
63	12.0	123	14701.4	14.6	9538.6	5162.8	3936.9
64	12.2	116	13965.6	14.3	9138.1	4827.6	3816.3
65	12.3	110	13227.0	14.0	8741.8	4485.2	3674.6
66	12.6	104	12485.3	13.6	8347.9	4137.4	3512.1
67	12.8	98	11739.9	13.2	7954.1	3785.9	3329.1
68	13.1	92	10990.2	12.8	7558.0	3432.2	3126.0
69	13.4	85	10235.0	12.4	7156.8	3078.2	2903.5
70	13.8	79	9473.2	11.9	6747.4	2725.7	2662.4



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\,max}$ 4221.0 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 13.5 pcf Use 25pcf

Geotechnologies, Inc.

Tiebacks Calculations (Ref: US Army Corps of Engineers, AMF 88-3)

Project: 5420 Sunset

File No. 21265

Location: Top Row of Tiebacks

Soil Parameters:

Weight of Soil	γ	120.00	*lbs/ft^3*
Friction Angle	ϕ	31.00	*degrees*
Cohesion	c	550.00	*lbs/ft^2*
Tieback Angle	α	20.00	*degrees*
Earth Pressure Coefficient	K	0.50	

Design Assumptions:

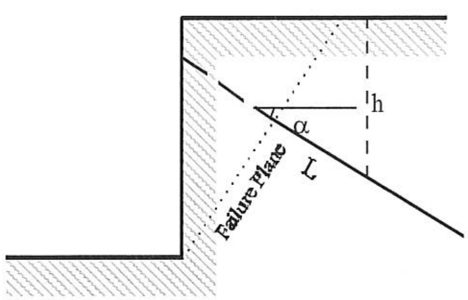
Diameter of Grout	d	1.00	*feet*
Length of Embeddment	L	20.00	*feet*
Depth to midpoint of Embeddment	h	12.00	*feet*
Factor of Safety Applied	F.S.	1.50	
Normal Stress $(\sigma v+k\sigma v)/2+(\sigma v-k\sigma v)/2*\text{Cos}2\alpha$	$\sigma n =$	1355.78	

Ultimate Resistance: R_{ult} 85.74 *kips*

Eq: pi*d*γ*L*σn*tan(ϕ)+c*pi*d*L

Allowable Resistance:	$R_{allow} = R_{ult}/F.S.$	57.16	*kips*
Allowable Skin Friction:	$R_{allow}/2/pi/r/L$	909.75	*psf*

Allowable Skin Friction Design Value **750** ***psf***



Geotechnologies, Inc.

Project: 5420 Sunset

File No.: 21265

Description: Typical Tieback Anchors at 52-68A

Pressure Grouted Tiebacks (Ref: California Trenching and Shoring Manual. Rev. 12)

Input:

H	25.0	feet	*height of shoring*
h	8.5	feet	*depth to anchor from top of shoring*
a	35.0	degrees	*active wedge failiure angle*
B	20.0	degrees	*installation angle of anchor shaft*
p_i	300.0	psi	*injection pressure during grouting*
d_s	6.0	inches	*diameter of anchor (bonded zone)*
L_s	20.0	feet	*length of anchor shaft of bonded zone*
ϕ_e	26.0	degrees	*effective friction angle between soil and grout*

$P_u = p_i * \pi * d_s * L_s * \tan\phi_e$

661.9	**kips**	**ultimate capacity**
441.3	**kips**	**allowable capacity with 1.5 factor of safety**
14.0	**ksf**	**allowable skin friction**
14046.7	**psf**	**allowable skin friction**
98.0	***kips***	***actual design load by shoring consultant***
3.1194	***ksf***	***actual skin friction***
3119.4	***psf***	***actual skin friction***