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Lowe Enterprises
11777 San Vicente Boulevard, Suite 900
Los Angeles, California 90049

Attention: Gordon Howe

Subject: Geotechnical Engineering Investigation
Proposed Mixed Use Development
South side of National Boulevard between Venice Boulevard
and Washington Boulevard, Culver City and Los Angeles, California

Dear Mr. Howe:

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

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

Should you have any questions please contact this office.

Respectfully submitted,
GEOTECHNOLOGIES, INC.

REGISTERED PROFESSIONAL ENGINEER
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GEOTECHNICAL ENGINEERING INVESTIGATION
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SOUTH SIDE OF NATIONAL BOULVEARD BETWEEN
VENICE BOULEVARDAND WASHINGTON BOULEVARD
LOS ANGELES, CALIFORNIA

INTRODUCTION

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

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

PROPOSED DEVELOPMENT

Information concerning the proposed development was furnished by the client, Cuningham Group, and Englekirk. The proposed project consists of the construction mixed use development with 5 above grade levels and underground parking. In general, the ground level will consist of retail, office, restaurant, and hotel space, while the upper levels will be residential in nature with office and hotel components. The eastern portion of the development will be underlain by 2 subterranean parking levels, with the lowest finished floor levels between 21 and 26½ feet below the ground surface. The western portion will be underlain by 3 subterranean levels, with the lowest finished floor levels between 31 and 36½ feet below the ground surface. The proposed development and the approximate limits of the 2 and 3 subterranean components are shown on the enclosed Plot Plan.

The office and retail components of the proposed development are expected to consist of concrete construction, while the residential components are expected to consist of wood-frame over concrete podium. Column loads are expected to range between 650 and 1,300 kips. Wall loads are expected to range between 10 and 20 kips per lineal foot. Grading will consist of excavations on the order of 24 to 40 feet for construction of the proposed subterranean levels and foundation elements.

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

SITE CONDITIONS

The subject site is located on the south side of National Boulevard between Venice Boulevard and Washington Boulevard. The western portion of the site that fronts Venice Boulevard is situated in the City of Los Angeles, California. The eastern portion of the site is located in the City of Culver City.

At the time of exploration, the western portion of the subject site was occupied by single story retail and commercial structures fronting Venice Boulevard. The eastern portion of the site was occupied by a paved parking lot. The subject site is bounded to the north by National Boulevard, to the east by Washington Boulevard, and to the west by Venice Boulevard. It is bounded to the south by the Metropolitan Transportation Authority's (MTA) – Culver City Station. The MTA development includes elevated rail lines supported on concrete platforms and abutments.

The site is roughly level with no pronounced topographic highs or lows. The total topographic relief across the site is on the order of 3 feet, with elevations ranging between approximately 102.5 feet at the eastern end and 105.5 feet at the western end of the site. Drainage appears to occur by sheet flow along existing contours towards the city streets. Vegetation is generally non-existent on the eastern portion of the site. Some trees and plants exist in the western portion of the site. The surrounding developments predominantly consist of commercial, retail, and residential developments.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was explored on April 28, 29, 30, and May 14, 15, 2014 by excavating nine borings to depths between 50 and 80 feet. The borings were conducted with an 8-inch diameter hollowstem auger drilling machine. Soil samples were collected in the borings and transported to our office for laboratory testing. The boring locations are shown on the enclosed Plot Plan, and the geologic materials encountered are logged on Plates A-1 through A-9.

Geologic Materials

The borings encountered existing fill over natural alluvial soils and marine sediments. The fill soils generally consist of silts and clays, which are predominantly dark brown in color, slightly moist to moist, and stiff. Between 2 and 5 feet of fill was encountered in the majority of the borings during exploration. Boring B3 encountered 15 feet of fill near the eastern perimeter of the site.

Natural alluvium was encountered below the fill. The upper alluvium consists of clays and silts to a depth of approximately 15 feet. The upper alluvium is generally dark brown to grayish brown, moist, and stiff. Below approximately 15 feet, the alluvium consists of silty sands and sands, which are light brown to gray, slightly moist to moist, dense to very dense, and fine to coarse grained with varying amounts of gravel and cobbles.

Marine sediments were encountered below the alluvium at depths between approximately 22½ and 30 feet. The marine sediments consist of silty sands, sands, and silts, which are gray, light brown, and orange brown in color. They are moist to wet, dense to very dense, firm to stiff, and generally fine grained. Occasional shell fragments were observed in the sediments.

Alluvial materials consist of detrital sediments deposited by river and stream action. Marine sediments are generally deposited in ocean basins or near shorelines and lagoons. Both are typical to this area of Los Angeles County. More detailed descriptions of the earth materials encountered may be obtained from the individual boring logs.

Groundwater and Caving

Groundwater was encountered during exploration in all of the borings at depths between 27½ and 32½ feet below the ground surface.

According to the Seismic Hazard Zone Report of the Beverly Hills 7½-Minute Quadrangle (CDMG, 1998, Revised 2005), the historic high groundwater level for the subject site ranged between approximately 18 feet (at the eastern end of the site) and 23 feet (at the western end of the site). A copy of the high groundwater map is enclosed herein. For design purposes, the historic high water contours are plotted on the enclosed Survey Plan. The plotted contours are based on the published 20 foot contour, which traverses the site, and the 10 and 30 foot contours located to the east and west of the site, respectively. Intermediate contours have been interpolated between the published contours.

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 in the borings excavated with the drilling machine because the boreholes were cased during drilling, and caving was not possible. Based on the experience of this firm, large diameter excavations, excavations that encounter granular cohesionless soils (such as those underlying the site), and excavations below the groundwater table will most likely experience caving.

SEISMIC EVALUATION

REGIONAL GEOLOGIC SETTING

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

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

REGIONAL FAULTING

Based on criteria established by the California Division of Mines and Geology (CDMG) now called California Geologic Survey (CGS), faults may be categorized as 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.

The enclosed Southern California Fault Map shows the location of many mapped faults in the Southern California area. 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.

Two major buried thrust fault structures in the Los Angeles area are the Elysian Park fold and thrust belt and the Torrance-Wilmington fold and thrust belt. It is postulated that the Elysian Park structure was responsible for the magnitude 5.9, October 1, 1987 Whittier Narrows earthquake, and that the Torrance-Wilmington structure was responsible for the magnitude 5.0, January 19, 1989 Malibu earthquake. The magnitude 6.7, January 17, 1994 Northridge earthquake was caused by a buried thrust fault located beneath the San Fernando Valley.

SEISMIC 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. Design of the proposed development in accordance with the provisions of the most current California Building Code (CBC) is intended to minimize the potential effects of ground shaking. The potential for other earthquake-induced hazards was also evaluated including surface rupture, liquefaction, dynamic settlement, inundation and landsliding.

2013 CBC 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 1613.5.2 of the California Building Code (CBC). This information and the site coordinates were input into the USGS U.S. Seismic Design Maps tool to calculate the seismic ground motion parameters for the site. Ground motion parameters for the 2013 CBC (ASCE 7-10) are presented below.

2013 CALIFORNIA BUILDING CODE SEISMIC PARAMETERS	
Site Class	D
Mapped Spectral Acceleration at Short Periods (S_S)	2.029g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	2.029g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.353g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.744g
Site Coefficient (F_v)	1.5
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	1.117g
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.744g

Deaggregated Seismic Source Parameters

The peak ground acceleration (PGA) and modal magnitude were obtained from the USGS Probabilistic Seismic Hazard Deaggregation program (USGS, 2008). The results are based on a 2 percent in 50 years ground motion (2,475 year return period). A published shear wave velocity, consistent with older fine to medium grained sediment, of 300 meters per second was utilized for Vs30 (Tinsley and Fumal, 1985). The deaggregation program indicates a PGA of 0.75g and a modal magnitude of 6.59 for the site.

OTHER SEISMIC HAZARDS

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. Ground rupture is defined as surface displacement which occurs along the surface trace of the causative fault during an earthquake.

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.

Review of the Alquist-Priolo Special Studies Zones Map of the Beverly Hills Quadrangle (CDMG, 1986) indicates the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. A copy of this map is provided in the Appendix. The closest Fault Zone is the Newport Inglewood Fault Zone, which is located approximately 1,000 feet to the east of the subject site. Therefore, a fault rupture investigation is not currently required for development of the subject site.

Review of the Geologic Map by (Dibblee, 1991) and the Navigate L.A. website (Navigate L.A., 2014) indicates other fault traces have been mapped in the vicinity of the subject site. The Geologic Map by (Dibblee, 1991) indicates the Newport Inglewood Fault is located approximately 1,300 feet to the east of the site. The Navigate L.A. website indicates traces of the Newport Inglewood Fault are located approximately 1,700 feet to the southwest and 4,100 to the northwest of the site. In addition, the Navigate L.A. website indicates a trace of the Overland Avenue Fault is located approximately 2.2 miles to the southwest of the site. Copies of these maps are enclosed in the Appendix. None of these mapped fault traces currently traverse the subject site.

The geotechnical investigation of the subject site performed by this firm was not intended as a fault rupture investigation. Such an exploration is beyond the scope of this investigation. However, evidence of faulting was not observed during geotechnical exploration on the site conducted by this firm. Such evidence could include, but may not be limited to, substantial differences in stratigraphic units across the site, groundwater level variations, and repeating sequences. In addition, the subject site is not located in an Alquist-Priolo Earthquake Fault Zone, and there are no traces of faults on the subject site shown on the maps reviewed by this office. Based on these considerations, the potential for surface ground rupture at the subject site is considered low.

Liquefaction

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

Liquefaction typically occurs in areas where groundwater is less than 50 feet from the surface, and where the soils are composed of poorly consolidated, fine to medium-grained sand. In addition to the necessary soil conditions, the ground acceleration and duration of the earthquake must also be of a sufficient level to initiate liquefaction.

The Seismic Hazards Zone Map of the Beverly Hills Quadrangle by the State of California (CDMG, 1999), indicates that the eastern portion of the subject site is located within an area designated as "Liquefiable," while the western portion of the site is not. This determination is based on groundwater depth records, soil type and distance to a fault capable of producing a substantial earthquake. A copy of this map is provided in the Appendix.

Site-specific liquefaction analyses were performed following the Recommended Procedures for Implementation of CDMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California (Martin and Lew, 1999). Recommendations provided in CGS Special Publication 117A were also incorporated in to the analysis (CDMG, 2008). The enclosed liquefaction analyses were performed using the spreadsheet template LIQ2_30.WQ1 developed by Thomas F. Blake (Blake, 1996). This program utilizes the 1996 NCEER method of analysis. This semi-empirical method is based on a correlation between measured values of Standard Penetration Test (SPT) resistance and field performance data.

Groundwater was encountered during exploration at depths between 27½ and 32½ feet below the ground surface. According to the Seismic Hazard Zone Report of the Beverly Hills 7½-Minute Quadrangle (CDMG, 1998, Revised 2005), the historic high groundwater level for the subject site ranged between approximately 18 feet (at the eastern end of the site) and 23 feet (at the western end of the site). Historic high groundwater levels of 18 and 22 feet have been utilized for the enclosed liquefaction analyses of borings B3 and B7, respectively.

Section 11.8.3 of ASCE 7-10 indicates that the potential for liquefaction shall be evaluated utilizing an acceleration consistent with the MCE_G PGA. Utilizing the USGS U.S. Seismic Design Maps tool, this corresponds to a PGA of 0.75g. The USGS Probabilistic Seismic Hazard Deaggregation program (USGS, 2008) also indicates a PGA of 0.75g (2 percent in 50 years ground motion) and a modal magnitude of 6.59 for the site. Therefore, the liquefaction potential evaluations were performed by utilizing a magnitude 6.59 earthquake and a peak horizontal acceleration of 0.75g.

The enclosed "Empirical Estimation of Liquefaction Potential" calculations are based on borings B3 and B7. 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 of representative samples of the soils encountered in the exploratory borings are presented on the enclosed E Plate.

Based on the adjusted blow count data, the enclosed liquefaction analyses indicate that the soils underlying the site would not be prone to liquefaction.

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.

The proposed structure will be constructed below the groundwater level. Therefore, dynamic dry settlements of the proposed structure are not expected to occur. In addition, based on the relatively dense and / or cohesive nature of the alluvial soils underlying the site, dynamic dry settlements at the existing ground surface would be expected to be negligible.

Tsunamis, Seiches and Flooding

Tsunamis are large ocean waves generated by sudden water displacement caused by a submarine earthquake, landslide, or volcanic eruption. Review of the County of Los Angeles Flood and Inundation Hazards Map, (Leighton, 1990), indicates the site does not lie within 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 County of Los Angeles Flood and Inundation Hazards Map, (Leighton, 1990), indicates the eastern portion of the site lies within the mapped inundation boundaries of the Mulholland Dam. A determination of whether a higher site elevation would remove the site from the potential inundation zones is beyond the scope of this investigation.

Landsliding

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

CONCLUSIONS AND RECOMMENDATIONS

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

Between 2 and 5 feet of existing fill was encountered in the majority of the borings during exploration conducted on the subject site. Boring B3 encountered 15 feet of fill. The existing fill materials are considered to be unsuitable for support of new foundations, floor slabs, or additional fill. It is anticipated excavation to the proposed basement levels would remove the exiting fill soils.

Groundwater was encountered on the site at depths between 27½ and 32½ feet. The historic high groundwater level for the subject site ranged between approximately 18 feet (at the eastern end of the site) and 23 feet (at the western end of the site). The finished floor of the P2 parking level (east end of site) is expected to be between 21 and 26½ feet below the ground surface, while the finished floor of the P3 parking level (west end of site) is expected to be between 31 and 36½ feet. Foundations would be expected to extend to depths between approximately 24 and 40 feet. Therefore, the proposed structure should either be designed to resist potential hydrostatic forces, or a permanent dewatering system should be installed so that external water pressure does not develop against the proposed retaining walls and floor slabs. In either case, the design of the proposed development should be based on the historic high water levels.

Recommendations and design values for both design approaches (i.e. hydrostatic design or permanent dewatering design) are provided herein. The client should be aware that designing the proposed development to resist hydrostatic forces in lieu of installation of a permanent dewatering system eliminates the need for maintenance of the dewatering system and continuous handling, testing, and possible treatment of waters pumped from the system. In addition, it would not be necessary to comply with future changes in water quality standards for collected and released groundwater.

It is the understanding of this firm that the design could possibly incorporate the placement of a subdrain just above the historic high water levels. This would be intended to reduce the design pressures on the proposed retaining walls. Since the subdrains would be above the historic high water level, there intent would be to relieve nuisance water, not static groundwater. Therefore, the need for monitoring, testing, and treatment of released waters would be expected to be unnecessary. Should this design approach be selected, it is recommended the release, monitoring, and testing requirements be verified with the proper municipal agencies. Additional recommendations are provided in the "Retaining Wall Design" section of this report.

Due to the depth of the proposed basement excavations, it is recommended shoring be utilized to maintain a stable excavation. Soldier piles are recommended for shoring. Shoring and excavation recommendations are provided in the "Temporary Excavations" section of this report.

Excavation to the expected bottom of foundations will extend below the existing groundwater level, and temporary dewatering measures will be required to provide a dry excavation. It is recommended a qualified dewatering consultant be retained in order to develop a formal pre-construction temporary dewatering program. It will be necessary to lower the groundwater table prior to excavation of the subterranean levels. Additional recommendations for dewatering are provided in the "Temporary Dewatering" section of this report.

Although temporary dewatering will lower the groundwater elevation prior to construction, the soils at the proposed subgrade level should be expected to be well above their optimum moisture level. These soils could be wet, soft, and susceptible to disturbance from construction activities. The placement of a mat of gravel over the bottom excavation will most likely be necessary to protect the subgrade soils from disturbance, create a firm working surface, and provide a firm bottom that is suitable for support of the proposed structure. Placement of gravel and wet subgrade soils are discussed in the "Temporary Dewatering" section below.

HYDROSTATIC DESIGN APPROACH

Due to the depth of the proposed basement below the historic high water level, it is anticipated a conventional floor slab on grade would not resist the expected hydrostatic uplift pressure. Therefore, it is recommended the proposed structure be supported on a mat foundation bearing in competent native soils at or below the basement depth of 21 feet below the ground surface. The mat foundation should be designed to resist hydrostatic uplift based on the historic high water level. In addition, the proposed retaining walls should be designed to resist hydrostatic pressures. Hydrostatic forces are addressed in the "Foundation Design" and "Retaining Wall Design" sections of this report.

It is recommended that the mat foundation system and retaining walls be completely watertight in order to prevent water seepage through normal shrinkage cracks or construction joints. It is recommended care be taken in the design and installation of waterproofing to avoid moisture problems, and to prevent water seepage into the structure. The design and inspection of waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method which would provide protection to subterranean walls, floors, and foundations.

DESIGN APPROACH INCORPORATING PERMANENT DEWATERING

If a permanent dewatering system is installed during construction, the proposed structure may be supported on conventional spread footings bearing in competent native soils at or below the bottom of the proposed basement level. A concrete floor slab on grade could also be utilized.

The permanent dewatering system shall be installed below the bottom of the slab on grade, as discussed in "Slabs on Grade" section of this report. The proposed retaining walls shall also be equipped with drainage systems so that hydrostatic forces do not develop on the basement walls. Recommendations for retaining wall drainage are provided in the "Retaining Walls" section of this report.

TEMPORARY DEWATERING

It is recommended that a qualified dewatering consultant be retained during the design phase of the project. Temporary dewatering on this project will be necessary to lower the water table beneath the site and allow for the proposed excavations and construction to proceed. The expected number and depths of well-points, expected flow rates, and expected pre-pumping time frames should be determined during a dewatering test program conducted by a qualified dewatering consultant.

It is anticipated that the well points will collect the majority of the water, however, even after pre-pumping, some free water may be encountered during excavation due to entrapment within cohesive lenses. Such water may be collected and removed from the excavation through the use of french drains and sump pumps.

Wet Subgrade Soils

Soils at the proposed subgrade level should be expected to be well above their optimum moisture level. A representative of this office should observe the subgrade as it becomes exposed so that the recommendations provided herein may be revised or reaffirmed as necessary. At this time, pumping, rutting, and disturbance of the high-moisture content soils should be expected to occur during operation of heavy equipment. In order to minimize disturbance of the subgrade bearing soils, provide a firm working surface, and provide a subgrade suitable for support of the proposed foundations, it is recommended the subgrade be protected and/or stabilized as it becomes exposed.

Protection or stabilization of the subgrade may be accomplished by placement of a minimum one-foot thick layer of angular 1-inch gravel. The gravel should be placed and vibrated to a dense state as the subgrade becomes exposed. The elevation at the bottom of excavation will require adjustment to provide space for the gravel mat. It is not recommended that rubber tire construction equipment attempt to operate directly on the subgrade soils prior to placing the gravel. Direct operation of rubber tire equipment on soft subgrade soils will likely result in excessive disturbance to the soils, which in turn could result in a delay to the construction schedule. Extreme care should be utilized to place gravel as the subgrade becomes exposed.

FILL SOILS

Between 2 and 15 feet of fill was encountered during exploration on the site. It is anticipated that this material will be removed during excavation of the proposed basement levels. Any fill remaining at the proposed subgrade should be removed and recompacted as controlled fill.

EXPANSIVE SOILS

The site soils are in the very low and high expansion ranges. The Expansion Index was found to range between 3 and 13 for representative samples of the site soils below 20 feet. The expansion index of the upper alluvial soils was found to range between 104 and 116. Recommended reinforcing is provided in the “Foundation Design” and “Slabs on Grade” sections of this report.

WATER-SOLUBLE SULFATES

The portland cement portion of concrete is subject to attack when exposed to water-soluble sulfates. Usually the two most common sources of exposure are from soil and marine environments. The source of natural sulfate minerals in soils 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 the 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 materials was determined for six bulk samples collected on the site. The sulfate content was found to range from less than 0.10 percentage by weight to greater than 0.20 percentage by weight. The results are shown on the enclosed D-Plates. Based on the CBC and American Concrete Institute - (ACI 318), the sulfate exposure is considered to be severe for soils with sulfate contents in excess of 0.20 percentage by weight. Therefore, it is recommended structural concrete in contact with the site soils consist of Type V cement, with a maximum water to cementitious materials ratio of 0.45, and a minimum compressive strength of 4,500 psi.

GRADING GUIDELINES

The following guidelines may be used in preparation of the grading plan and job specifications for any areas where fill or recompaction may be required, such as the driveway and sidewalk areas.

Site Preparation

- All vegetation, existing fill, and soft or disturbed earth materials should be removed from the areas to receive controlled fill. The excavated areas shall be observed by the geotechnical engineer prior to placing compacted fill.

- Where compacted fill is utilized for support of miscellaneous foundations, all existing fill should be completely removed and recompacted. The newly placed fill should extend beyond the edge of foundations for a distance equal to the depth of compacted fill beneath the foundation.

- It is very important that the positions of the proposed improvements are accurately located so that the limits of the graded areas are accurate and the grading operation proceeds efficiently.

- Any vegetation or associated root system located within the area to be graded should be removed during grading. Any existing or abandoned utilities located within the area to be graded should be removed or relocated as appropriate. All fill materials and disturbed earth materials resulting from grading operations should be removed and properly recompacted.

- 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.

Compaction

Fill, consisting of soil approved by a representative of this firm shall be placed in loose lifts not more than 8 inches in thickness. The loose materials shall be compacted with suitable compaction equipment. Once a layer has been adequately compacted, the next loose lift may be placed.

Fill materials shall be moisture conditioned to within 3 percent of optimum moisture content and sufficiently blended prior to placement as controlled fill. Materials larger than 6 inches in maximum dimension shall not be used in the fill.

All fill shall be compacted to at least 90 percent of the maximum laboratory density, except for cohesionless soils having less than 15 percent finer than 0.005 millimeters, which shall be compacted to a minimum 95 percent of the maximum density, in accordance with the April 15, 1998 amendment to the Los Angeles Municipal Code.

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 soils with an expansion index of less than 50. The water-soluble sulfate content of the import materials should be less than 0.10 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.

Over Optimum Subgrade Soils

At the time of exploration, the site soils were above their optimum moisture level. Some drying, aeration, and processing of the onsite soils should be anticipated prior to placement as compacted fill. If necessary, wet subgrades should be stabilized as indicated in the “Temporary Dewatering” section of this report.

A representative of this office should observe subgrades as they becomes exposed so that the recommendations provided herein may be revised or reaffirmed as necessary.

Shrinkage

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

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 ASTM D-1556 or ASTM D-6938.

Weather Related Grading Considerations

When rain is forecast all fill that has been spread and awaits compaction shall be properly compacted prior to stopping work for the day or prior to stopping due to inclement weather.

These fills, once compacted, shall have the surface sloped to drain to an area where water can be removed.

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

Work may start again, after a period of rainfall, once the site has been reviewed by a representative of this office. Any soils saturated by the rain shall be removed and aerated so that the moisture content will fall within three percent of the optimum moisture content.

Surface materials previously compacted before the rain shall be scarified, brought to the proper moisture content and recompacted prior to placing additional fill, if considered necessary by a representative of this firm.

Geotechnical Observations and Testing During Grading

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

FOUNDATION DESIGN

The proposed development should either be designed to resist hydrostatic forces, or a permanent dewatering system shall be installed so that hydrostatic forces do not develop against the floor slabs and retaining walls. A mat foundation is recommended to resist hydrostatic uplift forces. Conventional foundations may be utilized if a permanent dewatering system is installed.

MAT FOUNDATIONS - (Hydrostatic Design Approach)

For the hydrostatic design approach, it is recommended the proposed structure be supported on a mat foundation bearing in competent native soils at or below the minimum basement depth of 21 feet below the ground surface. Based on information provided by Englekirk, it is anticipated the proposed mat foundation would impart bearing pressures ranging between approximately 2,000 and 3,000 pounds per square foot under static loading conditions. Should the actual bearing stresses exceed these values, the foundation recommendations contained herein should be reviewed, reconfirmed, and revised if necessary. In addition, the subgrade modulus provided below should be reviewed, reconfirmed, and revised once the distribution of bearing stresses below the mat foundation has been analyzed by the structural engineer.

The anticipated bearing pressures are well below the allowable bearing pressures given the size of the proposed mat foundation. An allowable bearing value of 4,000 pounds per square foot may be utilized in the design of the proposed mat foundation. For initial design purposes, the mat foundation may be designed utilizing a modulus of subgrade reaction of 60 pounds per cubic inch.

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

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

Hydrostatic Considerations for Mat Foundations

The proposed mat foundation shall be waterproofed and designed to withstand the hydrostatic uplift pressure based on the historic high water levels between 18 and 23 feet below the ground surface. The uplift pressure to be used in design should be 62.4(H) pounds per square foot, where "H" is the height of the height of the historic high water level above the bottom of the mat foundation in feet.

Mat Foundation Settlement

Settlement of the mat foundation system is expected to occur on application of loading. The maximum settlement is expected to be 1½ inch. Differential settlement is not expected exceed 1/2 inch.

CONVENTIONAL FOUNDATIONS - (Permanent Dewatering Design Approach)

If a permanent dewatering system is installed behind the proposed basement walls and below the proposed floor slab, it is recommended the proposed structure be supported on conventional spread footings bearing in competent native soils at or below the minimum proposed basement depth of 21 feet below the ground surface.

Continuous wall foundations may be designed for a bearing value 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 native soils. Isolated pad foundations may be designed for a bearing value 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 native soils.

The bearing value increase for each additional foot of width is 100 pounds per square foot. The bearing value increase for each additional foot of depth is 300 pounds per square foot. The maximum recommended bearing value is 6,000 pounds per square foot.

If depth increases are utilized, this office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary.

Miscellaneous Conventional Foundations

Miscellaneous conventional foundations for minor at-grade structures such as planter walls and trash enclosures, which will not be rigidly connected to the proposed structure, may bear in native soils and/or properly compacted fill. These footings may be designed for a bearing value of 1,500 pounds per square foot, and should be a minimum of 12 inches in width, 24 inches in depth below the lowest adjacent grade and at least 12 inches into the recommended bearing material. No bearing value increases are recommended.

Conventional Foundations General

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

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

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

Conventional Foundation Settlement

The maximum settlement of conventional foundations is not expected to exceed 1 inch, and is expected to occur below the heaviest loaded elements. Differential settlement is not expected to exceed 1/2 inches.

LATERAL FOUNDATION DESIGN - (Mat and Conventional Foundations)

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 0.27 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 200 pounds per cubic foot with a maximum earth pressure of 1,500 pounds per square foot. Passive resistance values for design of soldier piles associated with shoring systems are provided in the "Shoring Design" section of this report.

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 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 earth materials, if necessary.

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

RETAINING WALL DESIGN

Retaining walls on the order of 36½ feet in height will be required for the proposed subterranean levels. It is anticipated these walls will be restrained. The proposed structure will either be designed to resist hydrostatic forces, or a permanent dewatering system will be installed so that hydrostatic forces do not develop on the basement walls. Retaining wall parameters for both design approaches are provided below.

Additional active pressure should be added for any additional surcharge conditions, such as sloping ground, or adjacent traffic and structures. Foundations may be designed in accordance with the "Foundation Design" section above.

Restrained Retaining Walls

Restrained basement retaining walls up to 37 feet in height and supporting a level back slope may be designed to resist a triangular distribution of earth pressure. It is recommended the walls be designed to resist the greater of the at-rest pressure, or the active pressure plus the seismic pressure, as discussed in the "Dynamic (Seismic) Earth Pressure" section below. Wall pressures are provided in the following tables for both hydrostatic and permanent dewatering design approaches.

RESTRAINED BASEMENT WALLS – HYDROSTATIC DESIGN		
	AT-REST EARTH PRESSURE (Pounds per Cubic Foot) Includes Hydrostatic Pressure of 62.4 pcf	**ACTIVE EARTH PRESSURE *(To be Combined with Dynamic Seismic Earth Pressure) Includes Hydrostatic Pressure of 62.4 pcf**
Height of Wall (Feet)	Triangular Distribution of Pressure (Pounds per Cubic Foot)	Triangular Distribution of Pressure (Pounds per Cubic Foot)*
Up to 37 feet	110	93

RESTRAINED BASEMENT WALLS – PERMANENT DEWATERING DESIGN		
	AT-REST EARTH PRESSURE (Pounds per Cubic Foot)	**ACTIVE EARTH PRESSURE *(To be Combined with Dynamic Seismic Earth Pressure)**
Height of Wall (Feet)	Triangular Distribution of Pressure (Pounds per Cubic Foot)	Triangular Distribution of Pressure (Pounds per Cubic Foot)*
Up to 23 feet	81	42
23 – 37 feet	81	47

<u>Dynamic (Seismic) Earth Pressure</u>

Retaining wall design shall consider the additional earth pressure caused by seismic ground shaking. A normal triangular pressure distribution should be utilized for the additional seismic loads, with an equivalent fluid pressure of 23 pounds per cubic foot. The seismic earth pressure should be combined with the lateral active earth pressure for analyses of restrained basement walls under seismic loading condition when using the load combination equations provided in the building code.

<u>Partially Drained Walls (Subdrain Above the Historic High Water Level)</u>

It is the understanding of this firm that the design could possibly incorporate the placement of a subdrain just above the historic high water levels. This would be intended to reduce the design pressures on the proposed retaining walls. This approach is acceptable to this firm, provided that hydrostatic design values are utilized below the level of the subdrain and the historic high water level. Design values provided for permanent dewatering approach would be appropriate for design above the subdrain system. All collected sub-drainage should outlet to an acceptable location.

Traffic Surcharge

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.

Surcharge from Adjacent Structures

The proposed basement walls should be designed to resist any potential surcharge from adjacent existing structures. It is anticipated this would include the MTA metro rail station and rail line. Columns supporting the rail line are reportedly supported on pile foundations. However, this office has not been provided with foundation plans of any existing adjacent structures. In either case, design of the proposed basement walls (and shoring systems) shall consider surcharge from adjacent structures.

Waterproofing

Moisture affecting retaining walls is one of the most common post- construction complaints. Poorly applied or omitted waterproofing can lead to efflorescence or standing water inside the building. Efflorescence is a process in which a powdery substance is produced on the surface of the concrete by the evaporation of water. The white powder usually consists of soluble salts such as gypsum, calcite, or common salt.

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

Retaining Wall Drainage

If the proposed development be designed to resist hydrostatic forces, retaining wall back drains may be omitted from the design.

If the development incorporates permanent dewatering, retaining walls should be provided with a subdrain covered with a minimum of 12 inches of gravel, and a compacted fill blanket or other seal at the surface. 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.

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

Sump Pump Design

Sump pumps will be required if a permanent dewatering system is installed. It is recommended that a de-watering specialist be retained to establish design flow rates, and to provide recommendations regarding the handling of groundwater. The flow rates should be based on the historic high groundwater levels of 18 to 23 feet below the ground surface.

It is anticipated that sump pumps would not be necessary if the development is designed to resist hydrostatic forces.

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 ASTM Designation D 1557 method of compaction. Flooding should not be permitted. Proper compaction of the backfill will be necessary to reduce settlement of the backfill and to reduce settlement of overlying walks and paving. Some settlement of required backfill should be anticipated, and any utilities supported therein should be designed to accept differential settlement.

TEMPORARY EXCAVATIONS

It is anticipated that excavations up to approximately 40 feet in vertical height will be required for construction of the proposed subterranean levels and foundation elements. The excavations are expected to expose fill and dense native soils, which are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic or structures.

Due to the presence of groundwater, the depth of the excavation, and the proximity of property lines, adjacent structures and public ways, excavation of the proposed subterranean levels will require shoring and dewatering measures to provide a stable and dry excavation. Soldier piles are recommended for shoring. Shoring recommendations are provided in the following section.

Where sufficient space is available, temporary unsurcharged embankments could be cut at a uniform 1:1 (h:v) slope gradient in their entirety, up to a maximum height of 10 feet. A uniform sloped excavation does not have a vertical component. Sloped excavations with vertical cuts at the toe of the slope are not recommended.

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 suggested along the tops of the slopes where necessary 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 this office during excavation so that modifications of the slopes can be made if variations in the earth material conditions occur. Many building officials require that temporary excavations should be made during the continuous observations of the geotechnical engineer.

SHORING

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

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

Soldier Piles

Drilled cast-in-place soldier piles should be placed no closer than 2½ diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the earth materials. For soldier pile design purposes, an allowable passive value for the earth materials below the bottom plane of excavation may be assumed to be 300 pounds per square foot per foot of depth, up to a maximum of 4,000 pounds per square foot. This assumes a saturated condition. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed earth materials.

The frictional resistance between the soldier piles and retained earth material may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.27 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 400 pounds per square foot. The minimum depth of embedment for shoring piles is 5 feet below the bottom of the footing excavation, or 7 feet below the bottom of excavated plane, whichever is deeper.

Groundwater was encountered during exploration at depths between 27½ and 32½ feet below the existing site grade. Caving of the saturated earth materials below the groundwater level may occur during drilling of piles. Casing or polymer drilling fluid will most likely be required during drilling in order to maintain open shafts. If casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet.

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

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

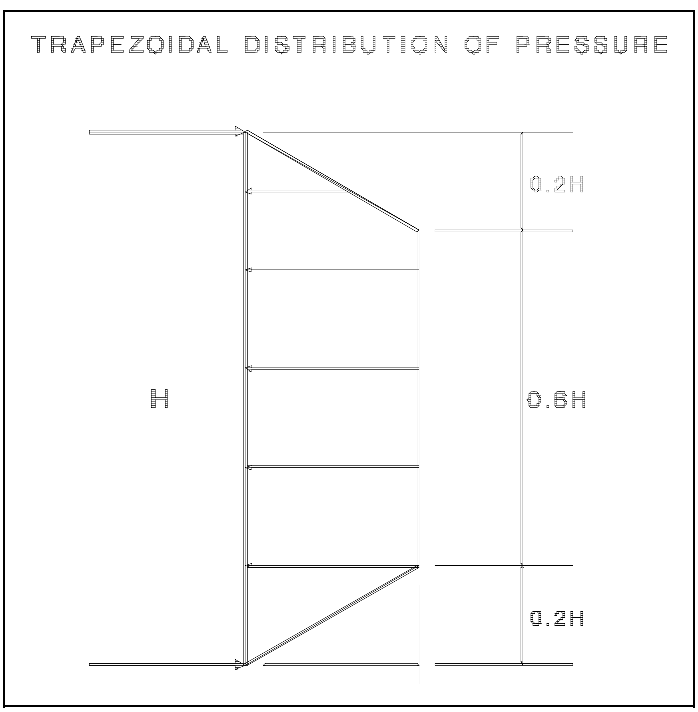
Lagging

At this time, it is anticipated that most or all of the excavation will require continuous lagging. It is recommended that the exposed soils be observed by a representative of the geotechnical engineer to verify the cohesive nature of the earth materials, and determine whether any lagging may be omitted.

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

Lateral Pressures

A triangular distribution of lateral earth pressure should be utilized for the design of a cantilever shoring system. A trapezoidal distribution of lateral earth pressure (as shown in the diagram below) would be appropriate where shoring is to be restrained at the top by tie backs or raker braces. The lateral pressures provided below assume temporary dewatering will be maintained during the use of the shoring system, and hydrostatic forces will not develop on the shoring.



Pressures for the design of cantilevered and restrained shoring supporting level back slopes are presented in the following table.

Height of Shoring (feet)	**Cantilever Shoring System Equivalent Fluid Pressure (pcf) Triangular Distribution of Pressure**	**Restrained Shoring System Lateral Earth Pressure (psf)* Trapezoidal Distribution of Pressure**
Up to 24 feet	34 pcf	22H psf
24 to 30 feet	36 pcf	23H psf
30 to 40 feet	39 pcf	25H psf

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

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

Surcharge from Adjacent Traffic or Structures

Additional active pressures should be applied where the shoring will be surcharged by adjacent traffic or structures. Traffic and/or structure surcharge pressures should be determined in accordance with the "Retaining Wall Design" section of this report.

Tieback Anchor Design and Installation

Tieback anchors may be used to resist lateral loads. Friction anchors are recommended. For design purposes, it may be assumed that the active wedge 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.

Tieback anchors may be installed between 20 and 40 degrees below the horizontal. Caving may occur within granular materials. Where caving occurs 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.

Drilled friction anchors constructed without utilizing pressure-grouting techniques may be designed for a skin friction of 400 pounds per square foot. Depending on the techniques utilized, and the experience of the contractor performing the installation, it is anticipated that a skin friction of 2,000 pounds per square foot could be utilized for post-grouted anchors, provided the design does not rely on end-bearing plates to provide the necessary capacity. Only the frictional resistance developed beyond the active wedge should be utilized in resisting lateral loads. Anchors should be placed at least 6 feet on center to be considered isolated.

Tieback Anchor Testing

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

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

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

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

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

Deflection

It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized that some deflection will occur. Where there are structures within a 1:1 plane drawn upward from the bottom of the excavation, it is recommended that the shoring be designed for a maximum deflection of ½-inch at the top of the shored embankment. Where there are not structures within a 1:1 projection from the bottom of the excavation, it is recommended the shoring be designed for a maximum deflection of 1 inch. If greater deflection occurs during construction, additional bracing may be necessary to minimize settlement of adjacent buildings and streets.

Pre-Construction Survey

Prior to shoring installation and excavation, it is recommended the adjacent improvements be surveyed to provide a documented record of their condition. Such a survey would aid in the resolution of any disputes that may arise concerning damage to adjacent facilities caused by the proposed construction.

Monitoring

Because of the depth of the excavations, some means of monitoring the performance of the shoring system is suggested. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all soldier piles and the lateral movement along the entire lengths of selected soldier piles.

Shoring Observations

It is critical that the installation of shoring is observed by a representative of this office. Many local agencies require that shoring installation be performed under the continuous observation of the geotechnical engineer. The observations are made so that modifications of the recommendations can be made if variations in the earth material or groundwater conditions occur. Also, the observations will allow for a report to be prepared on the installation of shoring for the use of the local building official.

SLABS ON GRADE

Interior Building Floor Slab

If a permanent dewatering system (including under slab drainage) is incorporated into the proposed design, a concrete slab on grade could be utilized in the basements of the proposed development. The underslab drainage system should consist of a minimum 1-foot thick layer of gravel underlying the entire floor slab. Subdrain pipes should be placed in gravel-filled drainage trenches leading to the sump pump. As a minimum, the subdrain pipes should consist of 4-inch perforated pipe, perforations down, placed in trenches approximately 1 foot wide and 1 foot in depth below the bottom of the gravel blanket. The pipes would then be covered with gravel, and the entire gravel and pipe system within the trenches would be wrapped in filter fabric. The gravel filled drainage trenches are typically spaced on approximate 40-foot centers, although there is flexibility in the spacing, depending on the column grid line spacing. In either case, it is recommended a qualified dewatering consultant be retained in order to establish design flow rates and ensure adequate sizing of subdrainage system.

The under slab drainage system should be placed above competent native soils and/or properly controlled fill materials. Any soils loosened or over-excavated should be wasted from the site or properly compacted to 90 or 95 percent of the maximum dry density.

Building floor slabs cast above the permanent dewatering system should be a minimum of 5 inches thick and reinforced with a minimum of #4 steel bars on 16-inch centers each way.

Outdoor Concrete Flatwork

Outdoor concrete flatwork, such as sidewalks and patio areas, should be a minimum of 4 inches in thickness and reinforced with a minimum of #4 steel bars on 16-inch centers each way. The slabs may be cast over undisturbed natural earth materials and/or properly controlled fill materials. Any earth materials loosened or over-excavated should be wasted from the site or properly compacted to 90 or 95 percent of the maximum dry density.

Exterior Concrete Pavements

Exterior concrete pavement subject to passenger vehicle and truck traffic should be a minimum of 6 inches in thickness and reinforced with a minimum of #4 steel bars on 16-inch centers each way. The concrete pavement should be underlain by 4 inches of base. A subgrade modulus of 150 pounds per cubic inch may be assumed for design of concrete paving.

Base materials may consist of aggregate base or crushed miscellaneous base and should be compacted to a minimum of 95 percent of the ASTM D 1557 laboratory maximum dry density. Base materials should conform with Sections 200-2.2 or 200-2.4 of the “Standard Specifications for Public Works Construction”, (Green Book), current edition.

Design Of Slabs That Receive Moisture-Sensitive Floor Coverings

In areas where dampness or vapor transmission through concrete floor slabs would be undesirable, it is recommended the slab be underlain by a vapor barrier. It is recommended a qualified waterproofing consultant should be retained in order to recommend a product or method which would provide protection for concrete slabs-on-grade.

As a minimum, it is recommended the vapor barrier consist of a minimum 15 mil extruded polyolefin plastic (no recycled content or woven materials). The barrier should have a permeance of less than 0.01 perms [grains / (ft^2 x hr x inHg)], as tested before and after mandatory conditioning (ASTM E1745 Section 7.1 and Sub-paragraphs 7.1.1-7.1.5). The barrier should comply with the ASTM E 1745 Class A requirements. The barrier should be installed according to ASTM E1643, including proper perimeter seal.

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 crack control maximum expansion joint spacing of 8 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 and exterior concrete pavements 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 or 95 percent relative compaction.

SITE DRAINAGE

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

All site drainage should be collected and transferred to an acceptable location in non-erosive drainage devices. The proposed structure should be provided with roof drainage. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. Planters located adjacent to a structure should be sealed to prevent moisture intrusion into the underlying soils. Irrigation in the planter areas around the proposed development should be properly controlled. Excessive irrigation may saturate the underlying soils and adversely affect the proposed development.

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. This requirement goes against prudent engineering practice. 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.

The proposed structure will be constructed below the current and historic high water levels and is expected to occupy the majority of the site. In addition, the upper site soils are highly expansive in nature. Based on these considerations, it is the opinion of this firm that stormwater infiltration is not feasible as part of the proposed development.

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

It is recommended that the design team (including the structural engineer, waterproofing consultant, plumbing engineer, and landscape architect) be consulted in regards to the design and construction of filtration systems. Please be advised that stormwater infiltration and treatment is a relatively new requirement by the various cities and has been subject to change without notice.

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 is satisfied.

CONSTRUCTION MONITORING

Geotechnical observations and testing during construction is considered to be a continuation of the geotechnical investigation. Therefore, it is critical that the geotechnical aspects of the project be reviewed by this firm 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 this office at least twenty-four hours prior to any required site visit.

If conditions encountered during construction appear to differ from those disclosed herein, notify this office 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. 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.

Corrosion testing was not conducted as part of this investigation. However, if corrosion sensitive improvements are planned, it is recommended that a comprehensive corrosion study should be commissioned. The study would 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 boring logs.

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

Moisture and Density Relationships

The field moisture content and dry unit weight are determined for each of the undisturbed soil samples, and the moisture content is determined for SPT samples by 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 “Boring 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 ASTM D 3080 with a strain controlled, direct shear machine 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 Diagrams," B-Plates.

Consolidation Testing

Settlement predictions of the soil's behavior under load are made on the basis of the consolidation tests 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 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 hours 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 as mold of given dimensions, with each layer compacted by 25 blows of a 10 pound hammer dropped from a distance of 18 inches subjecting the soil to a total compactive effort of about 56,000 pounds per cubic foot. The resulting dry unit weight is determined. The procedure is repeated for a sufficient number of moisture contents to establish a relationship between the dry unit weight and the water content of the soil. The data when plotted represent a curvilinear relationship known as the compaction curve. The values of optimum moisture content and modified maximum dry unit weight are determined from the compaction curve.

Grain Size Distribution

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

200 sieve. ASTM D 422-63 (Reapproved 2007) 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. Hydrometer testing was not performed as part of this investigation. Particle size determination for this investigation utilized the Number 200 sieve. The results are plotted on Plate E presented in the Appendix of this report.

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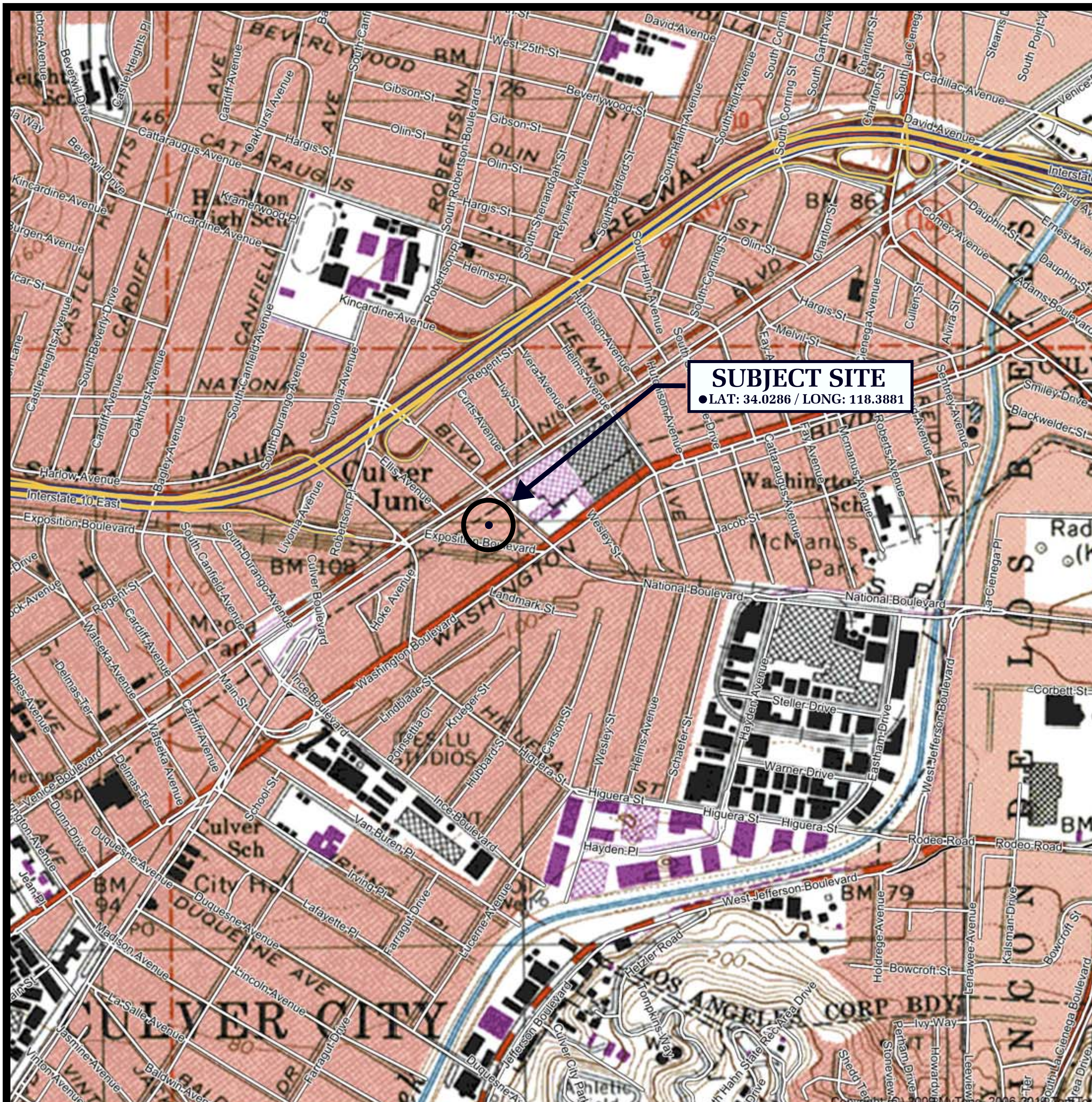


SCALE 1:12000

0.0 0.1 0.2 0.3 0.4 0.5 0.6
MILES

0 1000 2000 3000
FEET

N

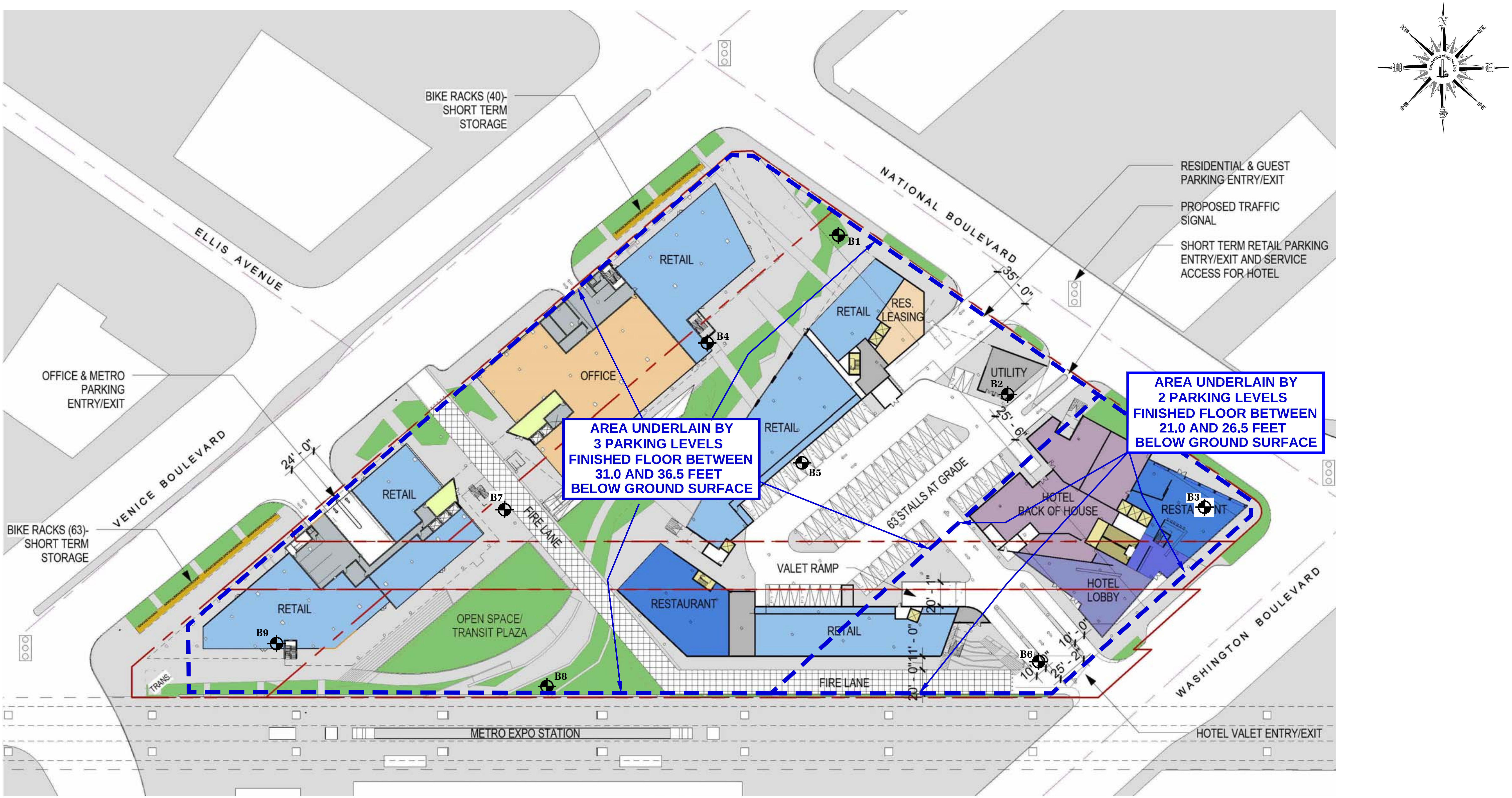
REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, BEVERLY HILLS, CA QUADRANGLE

VICINITY MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

LOWE ENTERPRISES

FILE NO. 20760



SCALE IN FEET

0 80 160

LEGEND

B9 LOCATION & NUMBER OF BORING

LIMITS OF SUBTERRANEAN PARKING LEVELS

REFERENCE: GROUND LEVEL PLAN PROVIDED BY CLIENT DATED 11/25/14

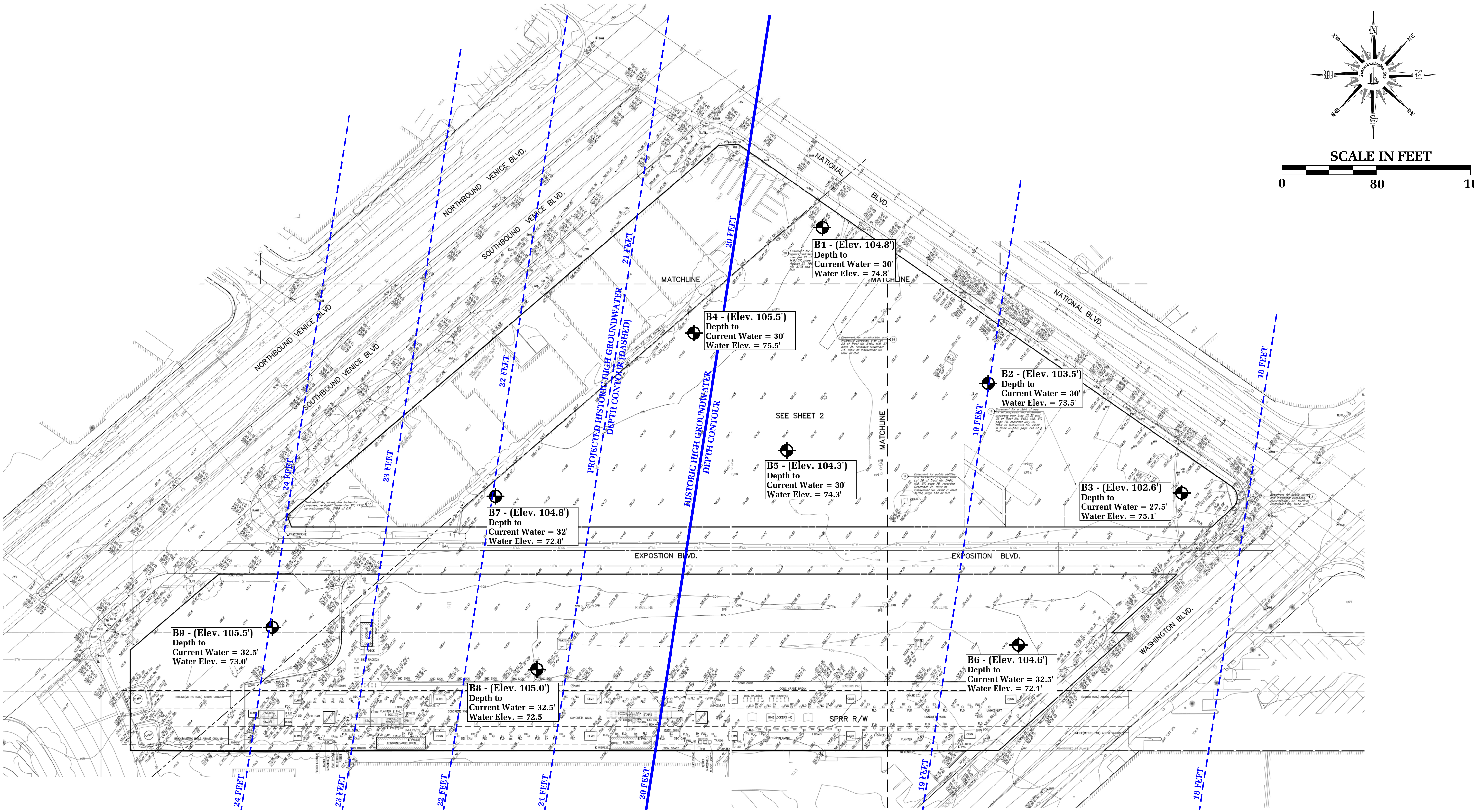
Geotechnologies, Inc.
Consulting Geotechnical Engineers

PLOT PLAN

LOWE ENTERPRISES

File No.: 20760

Date: September '15



LEGEND

B9 — LOCATION & NUMBER OF BORING

HISTORIC HIGH GROUNDWATER DEPTH CONTOUR
(DEPTH TO HIST. HIGH WATER FROM GROUND SURFACE)
FROM SEISMIC HAZARD ZONE REPORT 023
BEVERLY HILLS QUADRANGLE, CDMG, 1998, REVISED 2005)
(DASHED WHERE PROJECTED BETWEEN CONTOURS)

REFERENCE: BOUNDARY, AERIAL TOPOGRAPHIC AND LIMITED DESIGN SURVEY, DATED MAY 14, 2014, BY PSOMAS.

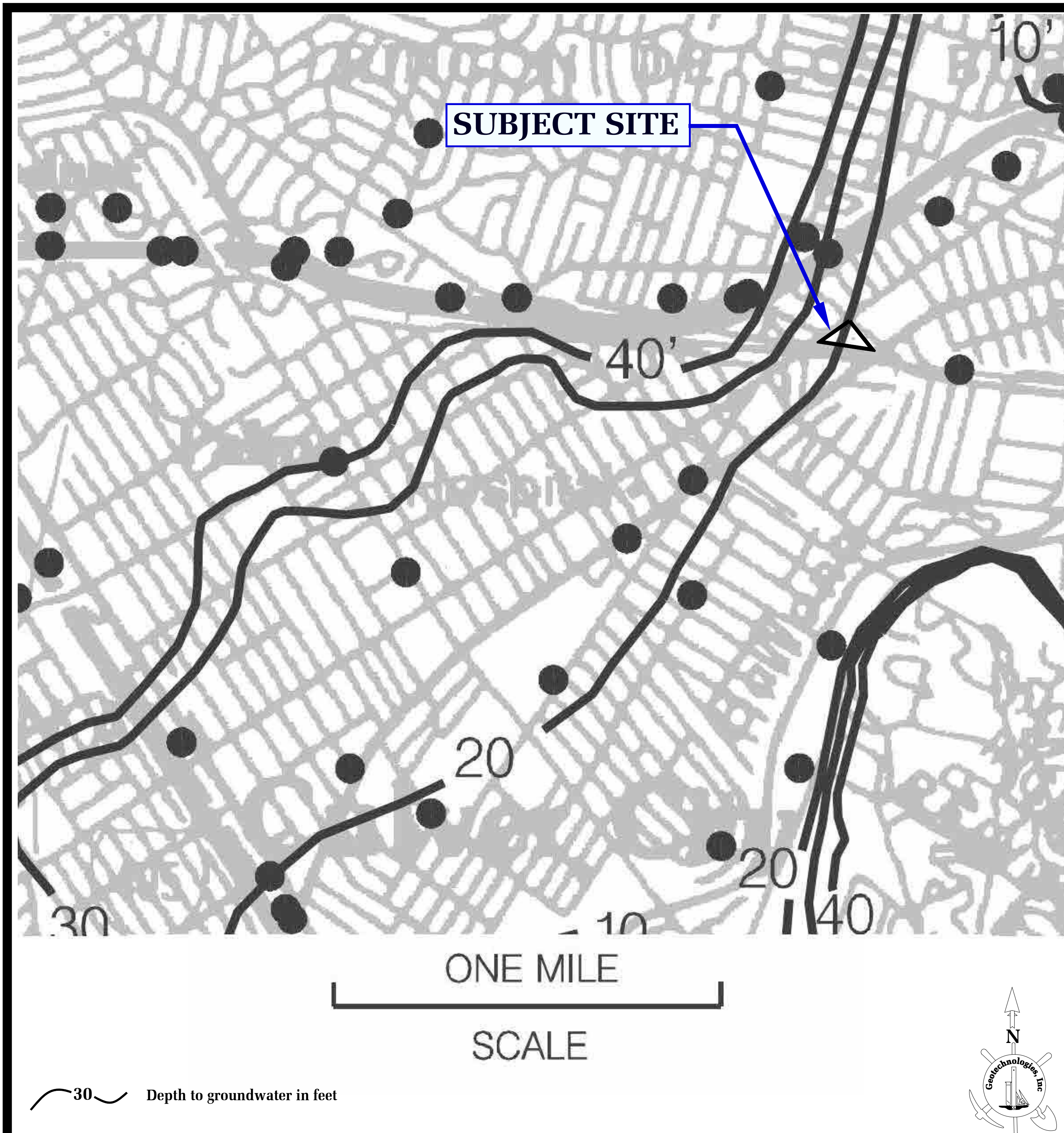
Geotechnologies, Inc.
Consulting Geotechnical Engineers

SURVEY PLAN

LOWE ENTERPRISES

File No.: 20760

Date: September '14



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

HISTORICALLY HIGHEST GROUNDWATER LEVELS

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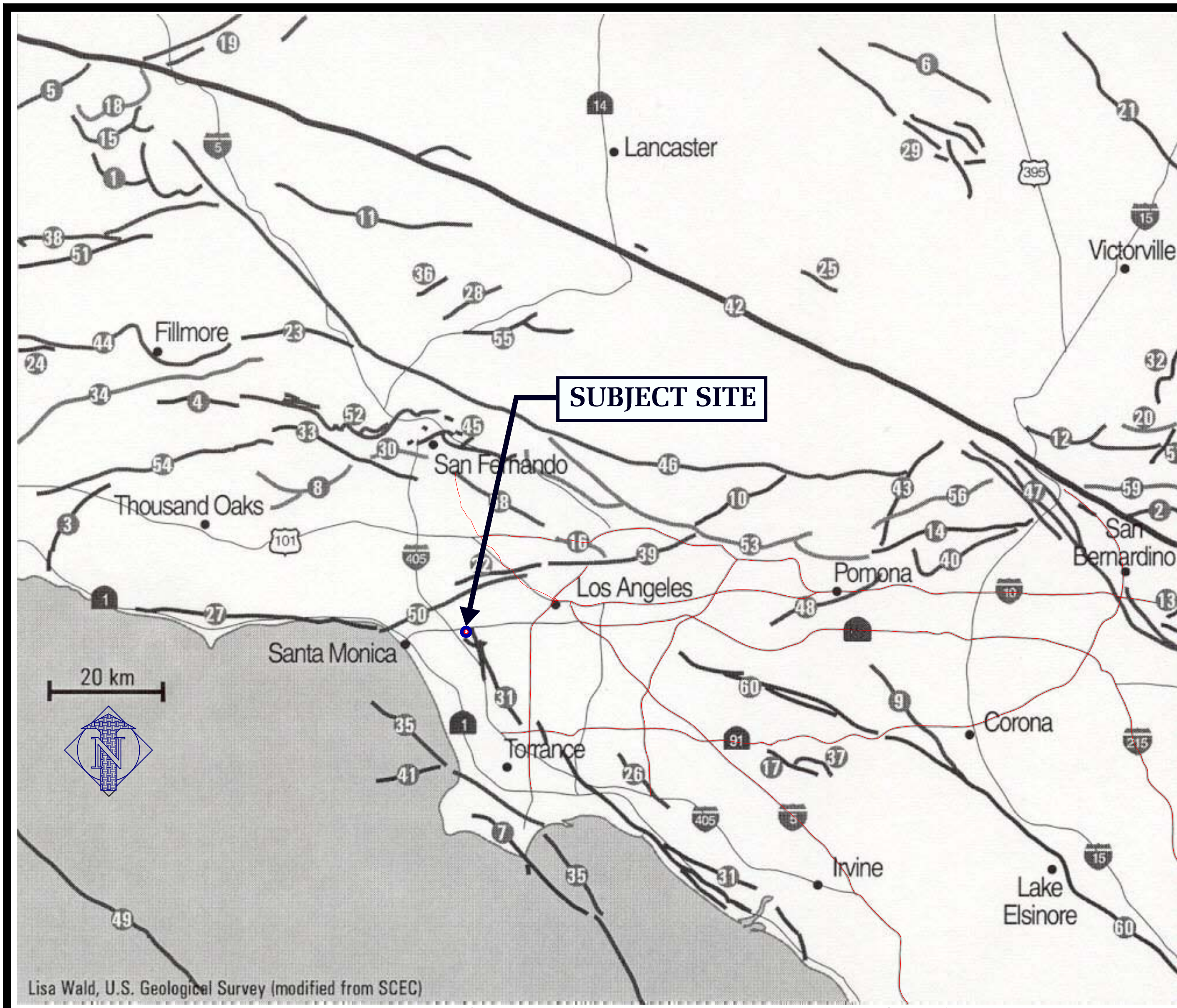
LOWE ENTERPRISES

FILE NO. 20760



1 Alamo thrust
2 Arrowhead fault
3 Bailey fault
4 Big Mountain fault
5 Big Pine fault
6 Blake Ranch fault
7 Cabrillo fault
8 Chatsworth fault
9 Chino fault
10 Clamshell-Sawpit fault
11 Clearwater fault
12 Cleghorn fault
13 Crafton Hills fault zone
14 Cucamonga fault zone
15 Dry Creek fault
16 Eagle Rock fault
17 El Modeno fault
18 Frazier Mountain thrust
19 Garlock fault zone
20 Grass Valley fault
21 Helendale fault
22 Hollywood fault
23 Holser fault
24 Lion Canyon fault
25 Llano fault
26 Los Alamitos fault
27 Malibu Coast fault
28 Mint Canyon fault
29 Mirage Valley fault zone
30 Mission Hills fault
31 Newport Inglewood fault zone
32 North Frontal fault zone
33 Northridge Hills fault
34 Oak Ridge fault
35 Palos Verdes fault zone
36 Pelona fault
37 Peralta Hills fault
38 Pine Mountain fault
39 Raymond fault
40 Red Hill (Etiwanda Ave) fault
41 Redondo Canyon fault
42 San Andreas Fault
43 San Antonio fault
44 San Cayetano fault
45 San Fernando fault zone
46 San Gabriel fault zone
47 San Jacinto fault
48 San Jose fault
49 Santa Cruz-Santa Catalina Ridge f.z.
50 Santa Monica fault
51 Santa Ynez fault
52 Santa Susana fault zone
53 Sierra Madre fault zone
54 Simi fault
55 Soledad Canyon fault
56 Stoddard Canyon fault
57 Tunnel Ridge fault
58 Verdugo fault
59 Waterman Canyon fault
60 Whittier fault

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

SOUTHERN CALIFORNIA FAULT MAP

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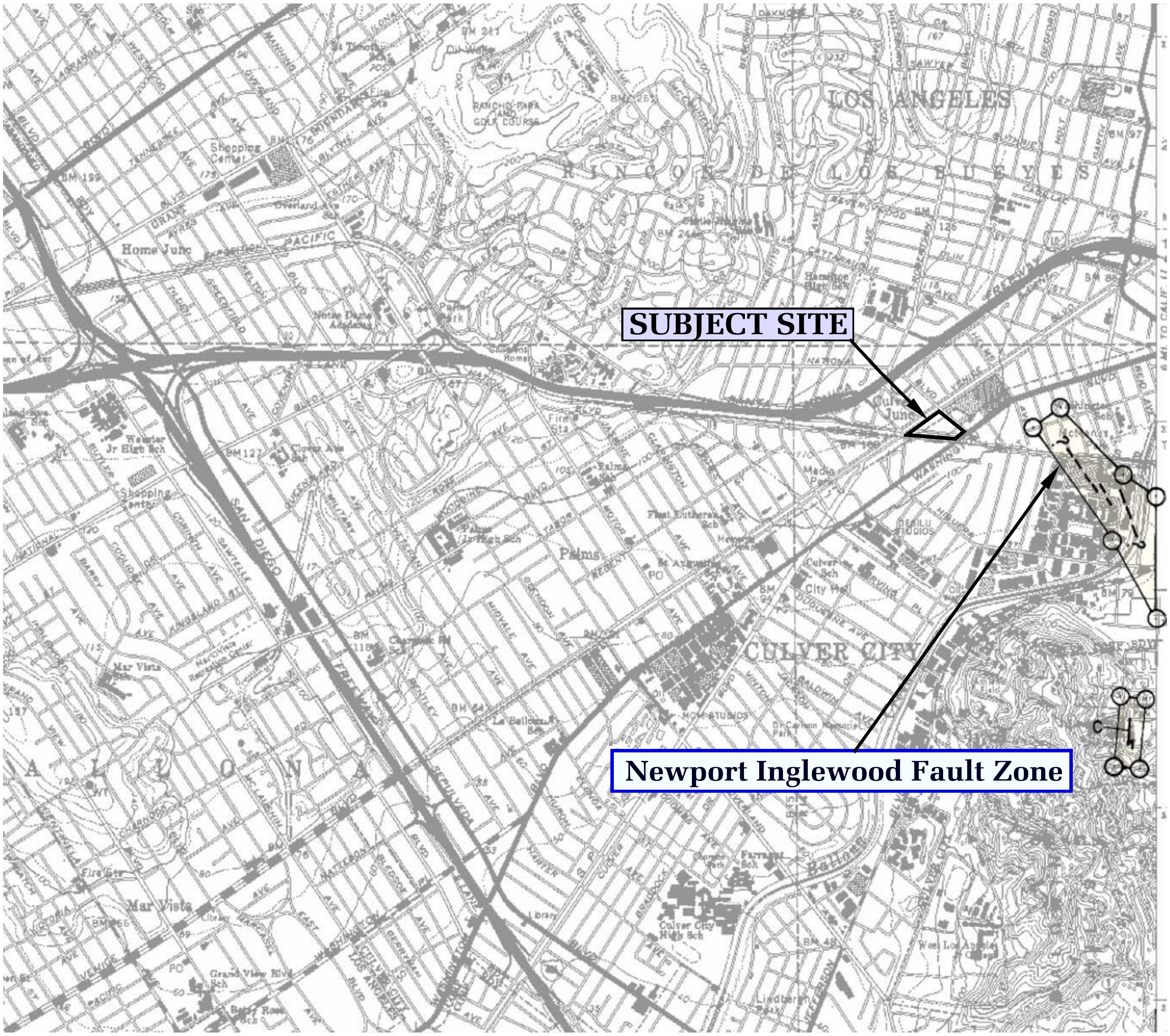
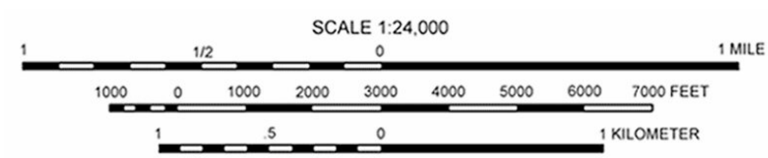
LOWE ENTERPRISES

FILE No. 20760 | **FIGURE I**





N

Earthquake Fault Zones
Alquist-Priolo Earthquake Fault Zone

REFERENCE: SPECIAL STUDIES ZONES, BEVERLY HILLS QUADRANGLE, CALIFORNIA, DMG, 1986.

EARTHQUAKE FAULT ZONE

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Consulting Geotechnical Engineers

LOWE ENTERPRISES

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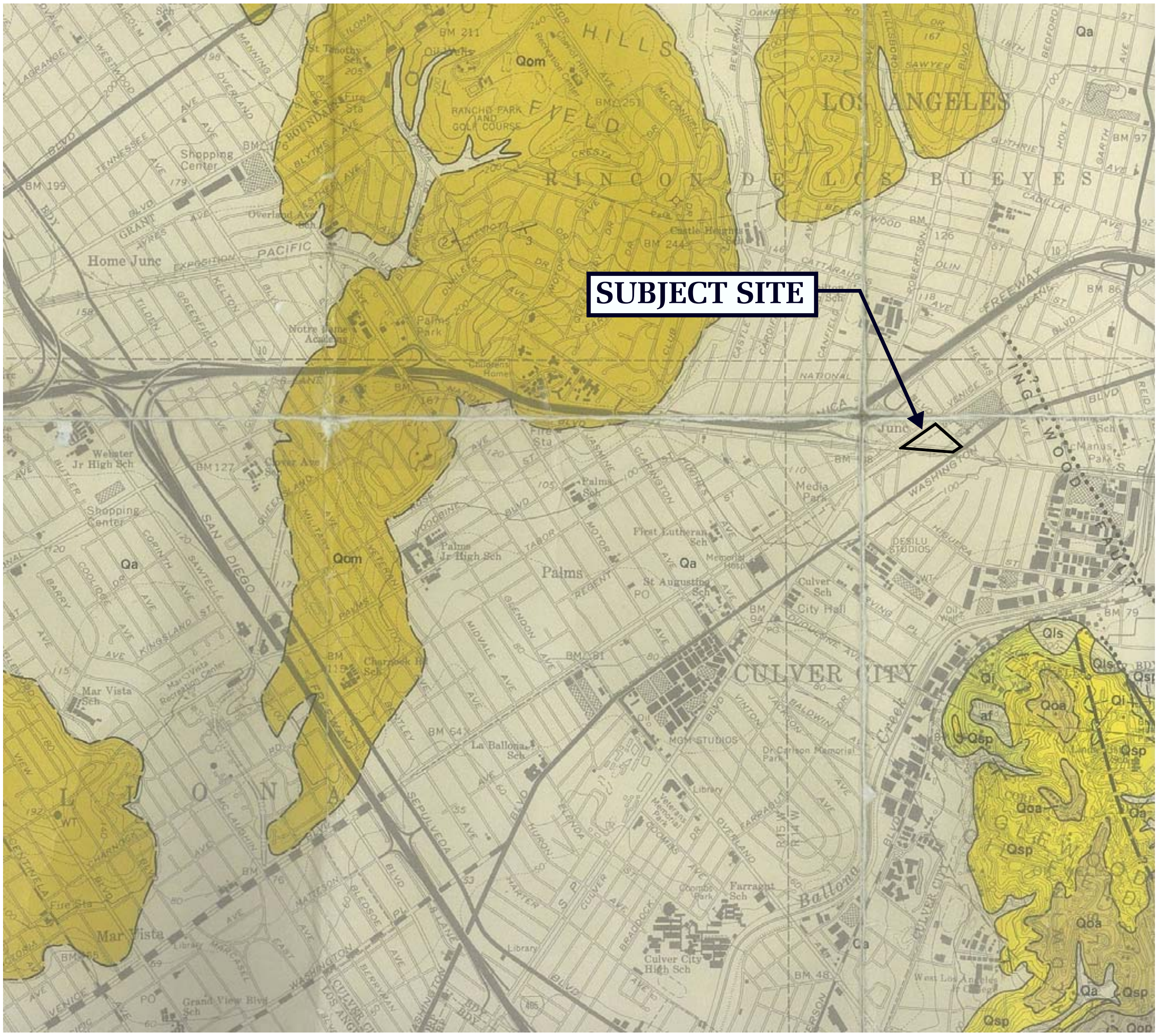

LEGEND

af: Surficial Sediments - Artificial cut and fill

Qa: Surficial Sediments - alluvial gravel, sand, and silt-clay, derived mostly from Santa Monica Mountains; includes gravel and sand of stream channels

Ql: Shallow Marine Sediments - Inglewood Formation: light gray, friable fine grained sandstone and interbedded soft gray siltstone; base not exposed

Qls: Surficial Sediments - landslide debris

Qoa: Older Surficial Sediments - Older alluvium of gray to light brown pebble-gravel, sand and silt-clay derived from Santa Monica Mountains; slightly consolidated

Qsp: Shallow Marine Sediments - San Pedro Sand; light gray to light brown sand, fine to coarse grained or pebbly, locally contains shell fragments

N

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REFERENCE: DIBBLEE, T.W., (1991) GEOLOGIC MAP OF THE BEVERLY HILLS AND VAN NUYS (SOUTH 1/2) QUADRANGLES

LOCAL GEOLOGIC MAP

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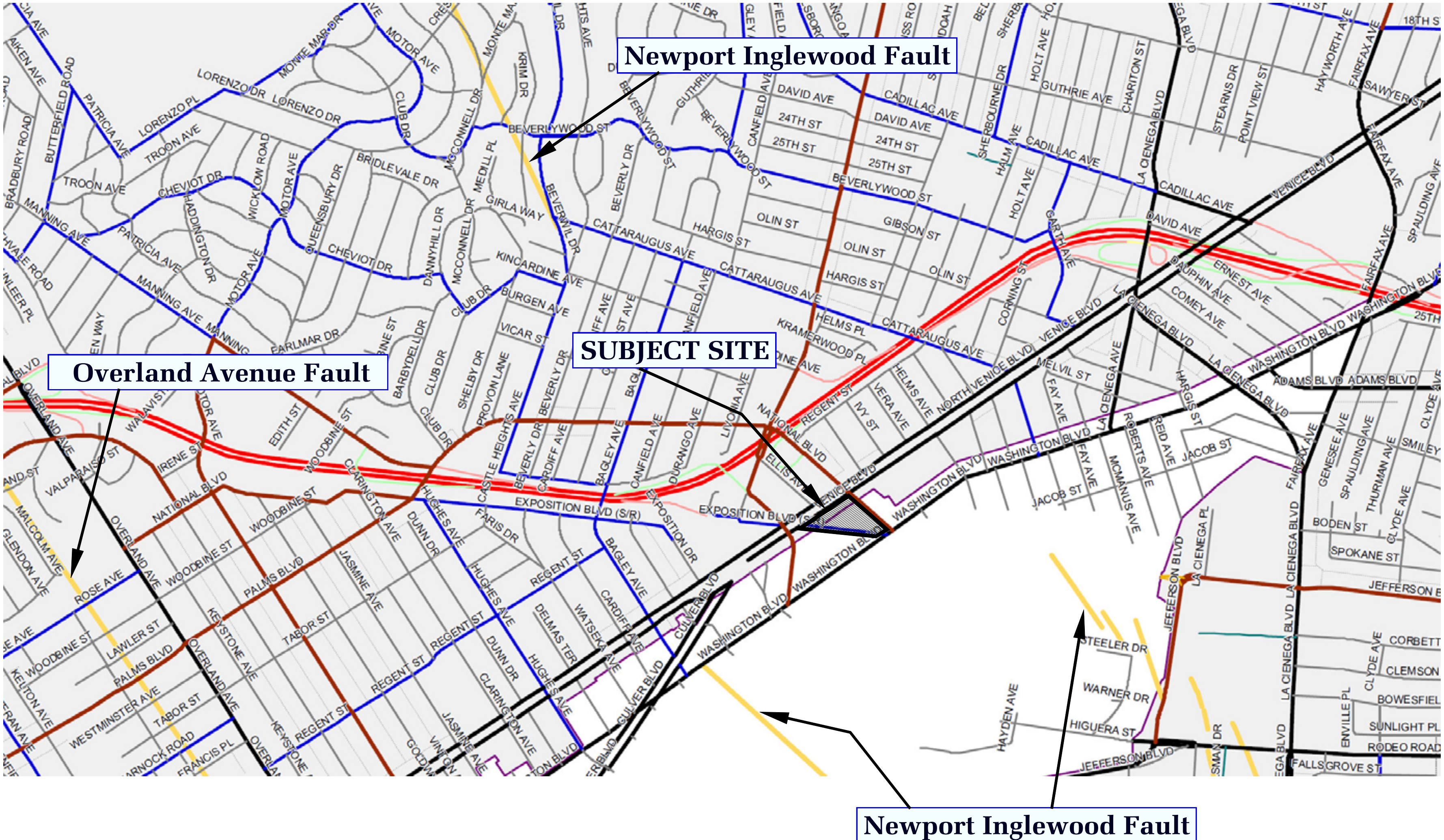
Consulting Geotechnical Engineers

LOWE ENTERPRISES

FILE NO. 20760



N

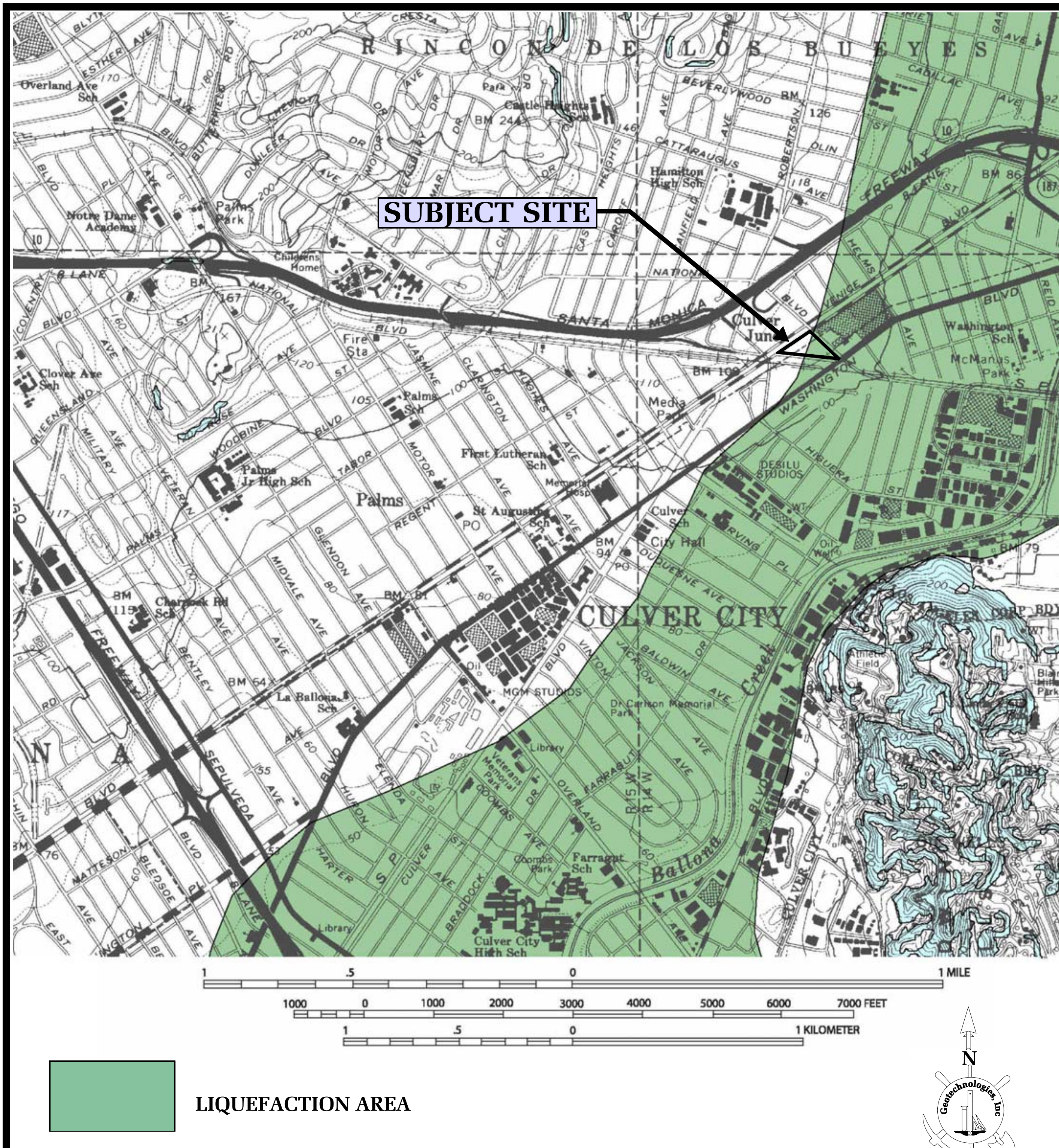
Not to Scale

LEGEND

Quaternary Faults
Based on 2010 Fault Activity Map of California,
by California Geological Survey.

Reference: Map from Navigate L.A. Webstie (http://navigatela.lacity.org/).

NAVIGATE L.A. MAP	
Geotechnologies, Inc. Consulting Geotechnical Engineers	LOWE ENTERPRISES
	FILE No. 20760



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

SEISMIC HAZARD ZONE MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

LOWE ENTERPRISES

FILE NO. 20760

BORING LOG NUMBER 1

LOWE ENTERPRISES **Date: 05/15/14** **Elevation: 104.8 feet***

File No. 20760 **Method: 8-inch Diameter Hollow Stem Auger**

sa *Reference: Survey, dated May 14, 2014, by Psomas

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, No Base
				-		
				1 --		FILL: Silty Clay, dark brown, moist, stiff
				-		
2	25	19.7	104.9	2 --		
				-	CL	ALLUVIUM: Silty Clay, dark brown, moist, stiff
				3 --		
				-		
4	22	22.4	100.2	4 --		
				-		
				5 --		
				-		
				6 --		
				-		
7	34	21.8	103.9	7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	30	25.4	99.2	10 --		
				-		brown to dark brown
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	37	6.3	112.4	15 --		
				-	SW	Sand, light gray and light brown, slightly moist, dense, fine to coarse grained, some gravel
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	50/6"	6.0	112.4	20 --		
				-		
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	24 50/5"	5.1	116.6	25 --		
				-		

BORING LOG NUMBER 1

LOWE ENTERPRISES

File No. 20760

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
				28		
				29		
30	34 50/5"	15.3	109.1	30	SP	MARINE SEDIMENTS: Sand, light brown and orange brown, wet, dense, fine grained
				31		
				32		
				33		
				34		
35	26 50/5"	18.2	105.9	35	SM	Silty Sand, light brown to light gray and orange brown, very moist to wet, fine grained
				36		
				37		
				38		
				39		
40	50/6"	14.7	110.2	40		
				41		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				42		
				43		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				44		
45	29 50/5"	19.8	106.6	45		some interbedded gray silts
				46		
				47		
				48		
				49		
50	29 50/3"	18.5	108.6	50		Total Depth 50 feet Water at 30 feet Fill to 2 feet

BORING LOG NUMBER 2

LOWE ENTERPRISES | **Date: 05/14/14** | **Elevation: 103.5 feet***

File No. 20760 | **Method: 8-inch Diameter Hollow Stem Auger**

sa | *Reference: Survey, dated May 14, 2014, by Psomas

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		3½-inch Asphalt, No Base
				-		
				1 --		FILL: Sandy Clay, dark brown, slightly moist, stiff
				-		
2	41	17.5	110.7	2 --		
				-	CL	ALLUVIUM: Silty Clay, dark brown, slightly moist, stiff
				3 --		
				-		
4	34	17.2	110.4	4 --		
				-		
				5 --		
				-		
				6 --		
				-		
7	39	15.6	114.0	7 --		
				-	ML	Sandy Silt, grayish brown, moist, stiff
				8 --		
				-		
				9 --		
				-		
10	21	12.9	110.6	10 --		
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	75	8.8	127.2	15 --		
				-	SW	Sand, gray, slightly moist, very dense, fine to coarse grained,
				16 --		abundant gravel and cobbles
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	62	6.8	127.2	20 --		
				-		
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	31	16.9	108.1	25 --		
	50/5"			-	ML	MARINE SEDIMENTS: Sandy Silt, gray, light brown and orange brown, moist, firm to stiff

BORING LOG NUMBER 2

LOWE ENTERPRISES

File No. 20760

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
30	37	18.6	104.1	30 --		
	50/4"			-	SP	Sand, light gray to gray, wet, very dense, fine grained
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
35	50/6"	20.6	103.4	35 --		
				-		
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
40	28	23.0	101.8	40 --		
	50/5"			-	SM	Silty Sand, gray and light gray, wet, dense, fine grained
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
45	44	12.7	118.9	45 --		
	50/3"			-	SP	Sand, light gray to gray, wet, very dense, fine to medium grained, with occasional gravel and cobbles
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
50	34	24.1	102.5	50 --		
				-	SM	Silty Sand, light gray to gray, dense, wet, fine grained

BORING LOG NUMBER 2

LOWE ENTERPRISES

File No. 20760

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
				-		
				53 --		
				-		
				54 --		
				-		
55	50/6"	21.0	107.3	55 --		
				-	SM/ML	Silty Sand to Sandy Silt, gray to light gray and orange brown,
				56 --		moist, dense to stiff, fine grained
				-		
				57 --		
				-		
				58 --		
				-		
				59 --		
				-		
60	30	24.2	100.2	60 --		
	50/4"			-		Total Depth 60 feet
				61 --		Water at 30 feet
				-		Fill to 2 feet
				62 --		
				-		
				63 --		
				-		
				64 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				65 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				66 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				67 --		
				-		
				68 --		
				-		
				69 --		
				-		
				70 --		
				-		
				71 --		
				-		
				72 --		
				-		
				73 --		
				-		
				74 --		
				-		
				75 --		
				-		

BORING LOG NUMBER 3

LOWE ENTERPRISES **Date: 04/30/14** **Elevation: 102.6 feet***

File No. 20760 **Method: 8-inch Diameter Hollow Stem Auger**

sa *Reference: Survey, dated May 14, 2014, by Psomas

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0 --		FILL: Clayey Silt to Silty Clay, dark brown, slightly moist, stiff
				-		
				1 --		
				-		
				2 --		
2.5	37	14.5	112.0	-		
				3 --		- - - - - - - - - - - - - -
				-		Silty Clay, dark brown and dark gray, moist, stiff, some brick
				4 --		and cement debris
				-		
5	18	13.3	SPT	5 --		- - - - - - - - - - - - - -
				-		Sandy Silt, dark grayish brown and brown mottling, slightly
				6 --		moist, stiff
				-		
				7 --		
7.5	35	10.2	113.4	-		- - - - - - - - - - - - - -
				8 --		Sandy Clay, dark brown, slightly moist, stiff
				-		
				9 --		
				-		
10	10	13.0	SPT	10 --		- - - - - - - - - - - - - -
				-		Silty Sand to Sandy Silt, mottled brown and gray, moist, medium
				11 --		dense to stiff, fine grained, minor pebbles
				-		
				12 --		
12.5	12	12.5	117.0	-		
				13 --		
				-		
				14 --		
				-		
15	18	5.3	SPT	15 --		
				-	SM/SW	ALLUVIUM: Silty Sand to Sand, gray, slightly moist, dense
				16 --		fine to coarse grained, abundant gravel
				-		
				17 --		
17.5	100/8"	2.4	111.5	-		
				18 --	SW	Sand, light brown and orange brown, slightly moist, very
				-		dense, fine to coarse grained, abundant gravel
				19 --		
				-		
20	38 50/5"	2.0	SPT	20 --		
				-		
				21 --		
				-		
				22 --		
22.5	100/9"	6.2	120.6	-		
				23 --		
				-		
				24 --		
				-		
25	23	13.1	SPT	25 --		
				-	SM	MARINE SEDIMENTS: Silty Sand, brown and gray, moist,
						medium dense, fine grained

BORING LOG NUMBER 3

LOWE ENTERPRISES

File No. 20760

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
27.5	30	18.5	106.2	-		
	50/6"			28 --	ML	Sandy Silt, gray to dark gray, wet, firm
				-		
				29 --		
				-		
30	33	27.3	SPT	30 --		
				-		
				31 --		
				-		
				32 --		
32.5	28	25.5	97.8	-		
	50/6"			33 --		
				-		
				34 --		
				-		
35	33	29.8	SPT	35 --		
				-	CL	Clay, dark gray, moist, firm
				36 --		
				-		
				37 --		
37.5	40	22.7	102.9	-		
	50/4"			38 --	ML	Sandy Silt, gray to dark gray, moist, stiff
				-		
				39 --		
				-		
40	36	24.4	SPT	40 --		
				-		
				41 --		
				-		
				42 --		
42.5	70	24.8	99.4	-		
				43 --		trace shell fragments
				-		
				44 --		
				-		
45	27	30.5	SPT	45 --		
				-	CH	Clay, dark gray, moist, stiff
				46 --		
				-		
				47 --		
47.5	50/6"	25.8	98.4	-		
				48 --		
				-		
				49 --		
				-		
50	29	29.5	SPT	50 --		
				-		

BORING LOG NUMBER 3

LOWE ENTERPRISES

File No. 20760

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
52.5	50/6"	23.9	100.5	-		
				53 --	ML	Sandy Silt, gray to dark gray, moist, stiff
				-		
				54 --		
				-		
55	34	24.1	SPT	55 --		
				-		
				56 --		
				-		
				57 --		
57.5	80	No Recovery		-		
				58 --		
				-		
				59 --		
				-		
60	62	26.5	SPT	60 --		
				-		
				61 --		
				-		
				62 --		
62.5	50/6"	25.3	98.9	-		
				63 --		
				-		
				64 --		
				-		
65	35	26.6	SPT	65 --		
				-	CL	Clay, gray, moist, stiff
				66 --		
				-		
				67 --		
67.5	50/6"	25.6	98.0	-		
				68 --		
				-		
				69 --		
				-		
70	43	25.2	SPT	70 --		
				-		brown to gray
				71 --		
				-		
				72 --		
72.5	45	24.6	101.3	-		
	50/5"			73 --		
				-		
				74 --		
				-		
75	32	28.0	SPT	75 --		
				-	CH	Clay, light gray to gray, moist, stiff

BORING LOG NUMBER 3

LOWE ENTERPRISES

File No. 20760

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				76		
				77		
77.5	72	25.5	98.7	78	CL	Clay, light brown to light gray, slightly moist to moist, stiff
				79		
80	34	26.7	SPT	80		
				81		Total Depth 80 feet Water at 27½ feet Fill to 15 feet
				82		
				83		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				84		
				85		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				86		SPT=Standard Penetration Test
				87		
				88		
				89		
				90		
				91		
				92		
				93		
				94		
				95		
				96		
				97		
				98		
				99		
				100		

BORING LOG NUMBER 4

LOWE ENTERPRISES **Date: 05/15/14** **Elevation: 105.5 feet***

File No. 20760 **Method: 8-inch Diameter Hollow Stem Auger**

sa *Reference: Survey, dated May 14, 2014, by Psomas

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, 2-inch Base
				-		
1	39	9.6	100.5	1 --		FILL: Sandy Clay to Clayey Sand, medium to orange brown, moist, stiff to dense, fine grained, minor gravel
				-		
				2 --		
				-	CL	ALLUVIUM: Silty Clay, dark brown, moist, stiff
				3 --		
				-		
4	20	19.6	105.5	4 --		
				-		
				5 --		
				-		
				6 --		
				-		
7	25	21.5	102.3	7 --		- - - - - - - - - -
				-		grayish brown and dark brown
				8 --		
				-		
				9 --		
				-		
10	47	17.7	110.9	10 --	ML	Sandy Silt, grayish brown and orange brown, moist, stiff
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	84	2.6	119.7	15 --		
				-	SW	Sand, grayish brown, slightly moist, very dense, fine to coarse grained, some gravel and cobbles
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	67	7.7	117.8	20 --		
				-	SM	Silty Sand, light brown and gray, moist, dense, fine to coarse grained, some gravel
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	43	32.3	87.8	25 --		
				-	SM/ML	MARINE SEDIMENTS: Sandy Silt, gray, moist, stiff

BORING LOG NUMBER 4

LOWE ENTERPRISES

File No. 20760

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
				28		
				29		
30	50/6"	17.6	107.5	30	SP	Sand, light gray to light brown and orange brown, wet, very dense, fine grained
				31		
				32		
				33		
				34		
35	36 50/6"	21.1	105.2	35	SM	Silty Sand, light brown and gray, moist, very dense, fine grained
				36		
				37		
				38		
				39		
40	30 50/5"	24.5	100.1	40		slightly more silty
				41		
				42		
				43		
				44		
45	50/6"	28.0	95.5	45		less silty, orange brown and gray, very moist
				46		
				47		
				48		
				49		
50	28 50/4"	26.1	98.8	50		slightly more silty

BORING LOG NUMBER 4

LOWE ENTERPRISES

File No. 20760

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
				-		
				53 --		
				-		
				54 --		
				-		
55	33	18.3	109.3	55 --		
	50/4"			-	SM/SP	Silty Sand to Sand, gray and orange brown, wet, very dense,
				56 --		fine grained
				-		
				57 --		
				-		
				58 --		
				-		
				59 --		
				-		
60	30	19.3	107.6	60 --		
	50/5"			-		Total Depth 60 feet
				61 --		Water at 30 feet
				-		Fill to 2 feet
				62 --		
				-		
				63 --		
				-		
				64 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				65 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				66 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				67 --		
				-		
				68 --		
				-		
				69 --		
				-		
				70 --		
				-		
				71 --		
				-		
				72 --		
				-		
				73 --		
				-		
				74 --		
				-		
				75 --		
				-		

BORING LOG NUMBER 5

LOWE ENTERPRISES **Date: 05/14/14** **Elevation: 104.3 feet***

File No. 20760 **Method: 8-inch Diameter Hollow Stem Auger**

sa *Reference: Survey, dated May 14, 2014, by Psomas

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		3½-inch Asphalt, 6½-inch Base
				-		
				1 --		FILL: Silty Clay, dark brown, moist, stiff
				-		
2	27	17.8	100.5	2 --		
				-	CL	ALLUVIUM: Silty Clay, dark brown, moist, stiff
				3 --		
				-		
4	38	20.1	106.4	4 --		
				-		
				5 --		
				-		
				6 --		
				-		
7	23	22.4	97.0	7 --		
				-		grayish brown to dark brown
				8 --		
				-		
				9 --		
				-		
10	43	17.8	111.1	10 --	ML	Sandy Silt, grayish brown, moist, stiff
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	50/6"	3.3	113.4	15 --		
				-	SW	Sand, gray, slightly moist, very dense, fine to coarse grained, some gravel
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	81	5.0	128.1	20 --		
				-		
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	96	4.5	106.6	25 --		
				-		light brown

BORING LOG NUMBER 5

LOWE ENTERPRISES

File No. 20760

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
30	50/6"	13.7	111.2	30 --		
				-	SP	MARINE SEDIMENTS: Sand, light brown to light gray, wet, very dense, fine grained
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
35	30 50/5"	17.1	109.4	35 --		
				-		
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
40	50/4"	21.6	104.9	40 --		
				-	ML	Sandy Silt, light grayish brown, moist, stiff
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
45	66	24.1	101.2	45 --		
				-		
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
50	50/6"	28.0	94.7	50 --		
				-	SM/ML	Silty Sand to Sandy Silt, gray and light gray, moist, very dense to stiff, fine grained

BORING LOG NUMBER 5

LOWE ENTERPRISES

File No. 20760

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
				-		
				53 --		
				-		
				54 --		
				-		
55	30	23.3	102.7	55 --		
	50/5"			-	ML	Sandy Silt, gray and light gray, moist, stiff
				56 --		
				-		
				57 --		
				-		
				58 --		
				-		
				59 --		
				-		
60	50/6"	25.6	98.5	60 --		
				-		Total Depth 60 feet
				61 --		Water at 30 feet
				-		Fill to 2 feet
				62 --		
				-		
				63 --		
				-		
				64 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				65 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				66 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				67 --		
				-		
				68 --		
				-		
				69 --		
				-		
				70 --		
				-		
				71 --		
				-		
				72 --		
				-		
				73 --		
				-		
				74 --		
				-		
				75 --		
				-		

BORING LOG NUMBER 6

LOWE ENTERPRISES **Date: 04/29/14** **Elevation: 104.6 feet***

File No. 20760 **Method: 8-inch Diameter Hollow Stem Auger**

sa *Reference: Survey, dated May 14, 2014, by Psomas

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, No Base
				1 --		FILL: Sandy Clay, mottled dark brown, light brown and orange brown, moist, stiff
				2 --		
2.5	52	16.9	113.6			
				3 --		
				4 --		
5	42	16.3	110.1	5 --		
				-	CL	ALLUVIUM: Silty Clay, dark brown, moist, stiff
				6 --		
				7 --		
				8 --		
				9 --		
10	56	18.0	110.9	10 --		
				-		medium brown to grayish brown
				11 --		
				12 --		
				13 --		
				14 --		
15	48	9.1	118.1	15 --		
				-	SM//SW	Silty Sand to Sand, dark to grayish brown, moist, dense, fine to coarse grained, some gravel and cobbles
				16 --		
				17 --		
				18 --		
				19 --		
20	68	8.9	125.2	20 --		
				21 --		
				22 --		
				23 --		
				24 --		
25	40 50/5"	1.4	119.0	25 --		
				-	SP	Sand, light gray, slightly moist, very dense, fine grained, some gravel

BORING LOG NUMBER 6

LOWE ENTERPRISES

File No. 20760

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
				28		
				29		
30	50/6"	29.4	95.9	30		
				31	SM/ML	MARINE SEDIMENTS: Silty Sand to Sandy Silt, gray, moist, very dense to stiff, fine grained
				32		
				33		
				34		
35	38 50/5"	23.6	100.6	35		
				36		trace shell fragments
				37		
				38		
				39		
40	50/6"	23.7	102.1	40		
				41	ML	Sandy Silt, gray, moist, stiff, minor shell fragments
				42		
				43		
				44		
45	50/6"	27.6	96.6	45		
				46		gray and orange brown
				47		
				48		
				49		
50	86	26.5	97.8	50		
					CL	Clay, grayish brown, moist, stiff

BORING LOG NUMBER 6

LOWE ENTERPRISES

File No. 20760
sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				51		
				52		
				53		
				54		
55	50/6"	25.5	98.1	55		
					ML	Sandy Silt, grayish brown and brown, moist, stiff
				56		
				57		
				58		
				59		
60	89	18.5	108.0	60		
						Total Depth 60 feet
				61		Water at 32½ feet
						Fill to 3 feet
				62		
				63		
						NOTE: The stratification lines represent the approximate
				64		boundary between earth types; the transition may be gradual.
				65		Used 8-inch diameter Hollow-Stem Auger
						140-lb. Automatic Hammer, 30-inch drop
				66		Modified California Sampler used unless otherwise noted
				67		
				68		
				69		
				70		
				71		
				72		
				73		
				74		
				75		

BORING LOG NUMBER 7

LOWE ENTERPRISES **Date: 04/28/14** **Elevation: 104.8 feet***

File No. 20760 **Method: 8-inch Diameter Hollow Stem Auger**

sa *Reference: Survey, dated May 14, 2014, by Psomas

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, No Base
				-		
				1 --		FILL: Clayey Silt to Silty Clay, dark brown, moist, stiff
				-		
				2 --		
2.5	63	18.2	109.6	-		
				3 --		
				-	CL	ALLUVIUM: Silty Clay, dark brown, moist, stiff
				4 --		
				-		
5	19	15.6	SPT	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	57	18.0	111.3	-		
				8 --	CL/ML	Silty Clay to Clayey Silt, medium to dark brown and grayish brown, moist, stiff
				-		
				9 --		
				-		
10	30	9.1	SPT	10 --		
				-	ML	Sandy Silt, grayish brown and orange brown, moist, stiff
				11 --		
				-		
				12 --		
12.5	53	11.1	111.5	-		
				13 --		
				-		
				14 --		
				-		
15	15	18.0	SPT	15 --		
				-		
				16 --		
				-		
				17 --		
17.5	42	2.6	115.4	-		
	50/4"			18 --	SM/SW	Silty Sand to Sand, light gray, slightly moist, dense to very dense, fine to coarse grained, some pebbles and gravel
				-		
				19 --		
				-		
20	31	7.9	SPT	20 --		
				-		
				21 --		
				-		
				22 --		
22.5	50/6"	8.6	108.3	-		
				23 --	SM/SP	MARINE SEDIMENTS: Silty Sand to Sand, light brown and orange brown, moist, very dense, fine grained
				-		
				24 --		
				-		
25	41	3.7	SPT	25 --		
				-		

BORING LOG NUMBER 7

LOWE ENTERPRISES

File No. 20760

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
27.5	40 50/5"	12.9	104.7			
				28		wet to saturated
				29		
30	36	18.2	SPT	30		
				31	SP	Sand, gray and orange brown, wet to saturated, dense, fine grained
				32		
32.5	50/6"	15.5	111.6			
				33		
				34		
35	35	24.2	SPT	35		
				36		gray
				37		
37.5	52	18.5	105.3			
				38		
				39		
40	48	22.2	SPT	40		
				41		
				42		
42.5	85	14.7	110.6			
				43		
				44		
45	46	24.3	SPT	45		
				46	SM/ML	Silty Sand to Sandy Silt, gray to dark gray, very moist, dense to firm, fine grained
				47		
47.5	40 50/5"	16.2	108.0			
				48	SP	Sand, gray, wet to saturated, dense, fine grained
				49		
50	40	19.1	SPT	50		

BORING LOG NUMBER 7

LOWE ENTERPRISES

File No. 20760

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
52.5	88	25.7	98.5	-		
				53 --	SM/ML	Silty Sand to Sandy Silt, gray to dark gray, wet, dense to
				-		firm, fine grained
				54 --		
				-		
55	50	26.8	SPT	55 --		
				-		
				56 --		
				-		
				57 --		
57.5	90	21.5	104.9	-		
				58 --		
				-		
				59 --		
				-		
60	44	16.4	SPT	60 --		
				-	SP	Sand, gray and orange brown, wet, dense, fine grained
				61 --		
				-		
				62 --		
62.5	50/6"	18.6	108.9	-		
				63 --	SM/ML	Silty Sand to Sandy Silt, gray, wet, very dense to stiff, fine
				-		grained
				64 --		
				-		
65	62	25.5	SPT	65 --		
				-		
				66 --		
				-		
				67 --		
67.5	50/6"	20.5	103.2	-		
				68 --	SP	Sand, gray, wet, very dense, fine grained
				-		
				69 --		
				-		
70	60	21.3	SPT	70 --		
				-		Total Depth 70 feet
				71 --		Water at 32 feet
				-		Fill to 3 feet
				72 --		
				-		NOTE: The stratification lines represent the approximate
				73 --		boundary between earth types; the transition may be gradual.
				-		
				74 --		Used 8-inch diameter Hollow-Stem Auger
				-		140-lb. Automatic Hammer, 30-inch drop
				75 --		Modified California Sampler used unless otherwise noted
				-		
						SPT=Standard Penetration Test

BORING LOG NUMBER 8

LOWE ENTERPRISES | **Date: 04/28/14** | **Elevation: 105.0 feet***

File No. 20760 | **Method: 8-inch Diameter Hollow Stem Auger**

sa | *Reference: Survey, dated May 14, 2014, by Psomas

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, No Base
				-		
				1 --		FILL: Silty Clay to Clayey Sand, dark brown, moist, stiff to medium dense, fine grained
				-		
				2 --		
2.5	30	20.4	103.8	-		
				3 --		
				-	CL	ALLUVIUM: Silty Clay, dark brown, moist, stiff
				4 --		
				-		
5	33	19.6	105.3	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	39	20.1	104.9	-		
				8 --		brown to dark brown
				-		
				9 --		
				-		
10	45	18.5	110.0	10 --		
				-	ML	Sandy Silt, grayish brown and orange brown, moist, stiff
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	33	21.9	102.2	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	40	3.4	120.9	20 --		
	50/4"			-	SW	Sand, light gray, slightly moist, dense, fine to coarse grained, abundant gravel
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	90	13.5	103.0	25 --		
				-	SP	MARINE SEDIMENTS: Silty Sand to Sand, light gray and orange brown, moist, very dense, fine grained

BORING LOG NUMBER 8

LOWE ENTERPRISES

File No. 20760

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26 --		
				27 --		
				28 --		
				29 --		
30	45 50/4"	15.5	102.3	30 --	SP	Sand, light gray, wet, very dense, fine grained
				31 --		
				32 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual. Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				33 --		
				34 --		
35	88	15.9	106.1	35 --		light gray and orange brown
				36 --		
				37 --		
				38 --		
				39 --		
40	50/6"	25.7	97.5	40 --	ML/SM	Sandy Silt to Silty Sand, gray to dark gray, moist, stiff to very dense, fine grained
				41 --		
				42 --		
				43 --		
				44 --		
45	40 50/5"	21.1	104.0	45 --	SM	Silty Sand, grayish brown, moist to wet, very dense, fine grained
				46 --		
				47 --		
				48 --		
				49 --		
50	45 50/4"	22.8	102.1	50 --		Total Depth 50 feet Water at 32½ feet Fill to 3 feet

BORING LOG NUMBER 9

LOWE ENTERPRISES **Date: 04/29/14** **Elevation: 105.5 feet***

File No. 20760 **Method: 8-inch Diameter Hollow Stem Auger**

sa *Reference: Survey, dated May 14, 2014, by Psomas

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground with Gravel
				0 --		FILL: Silty Sand to Sandy Silt, dark brown, moist, dense to stiff, fine grained
				-		
				1 --		
				-		
				2 --		
2.5	25	22.1	101.3	-		
				3 --		
				-	CL	ALLUVIUM: Silty Clay, dark brown, moist, stiff
				4 --		
				-		
5	24	21.4	103.5	5 --		
				-		
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	36	19.6	107.0	10 --		
				-	ML/CL	Clayey Silt to Silty Clay, medium brown and reddish brown, moist, stiff
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
				15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	50/6"	10.3	124.1	20 --		
				-	SM/SW	Silty Sand to Sand, dark to grayish brown, moist, very dense, fine to coarse grained, abundant gravel
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
				25 --		
				-		

BORING LOG NUMBER 9

LOWE ENTERPRISES

File No. 20760

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				- 26 -- - 27 -- - 28 -- - 29 -- -		
30	42 50/5"	9.4	110.3	30 --		
				- 31 -- - 32 -- - 33 -- - 34 -- - 35 -- - 36 -- - 37 -- - 38 -- - 39 -- -	SM/SP	MARINE SEDIMENTS: Silty Sand to Sand, gray to dark brown, moist to very moist, very dense, fine grained
40	50/6"	21.0	104.8	40 --		
				- 41 -- - 42 -- - 43 -- - 44 -- - 45 -- - 46 -- - 47 -- - 48 -- - 49 -- -	SM/ML	Silty Sand to Sandy Silt, gray, very moist, very dense to stiff, fine grained
50	45 50/5"	21.8	103.3	50 -- -		

BORING LOG NUMBER 9

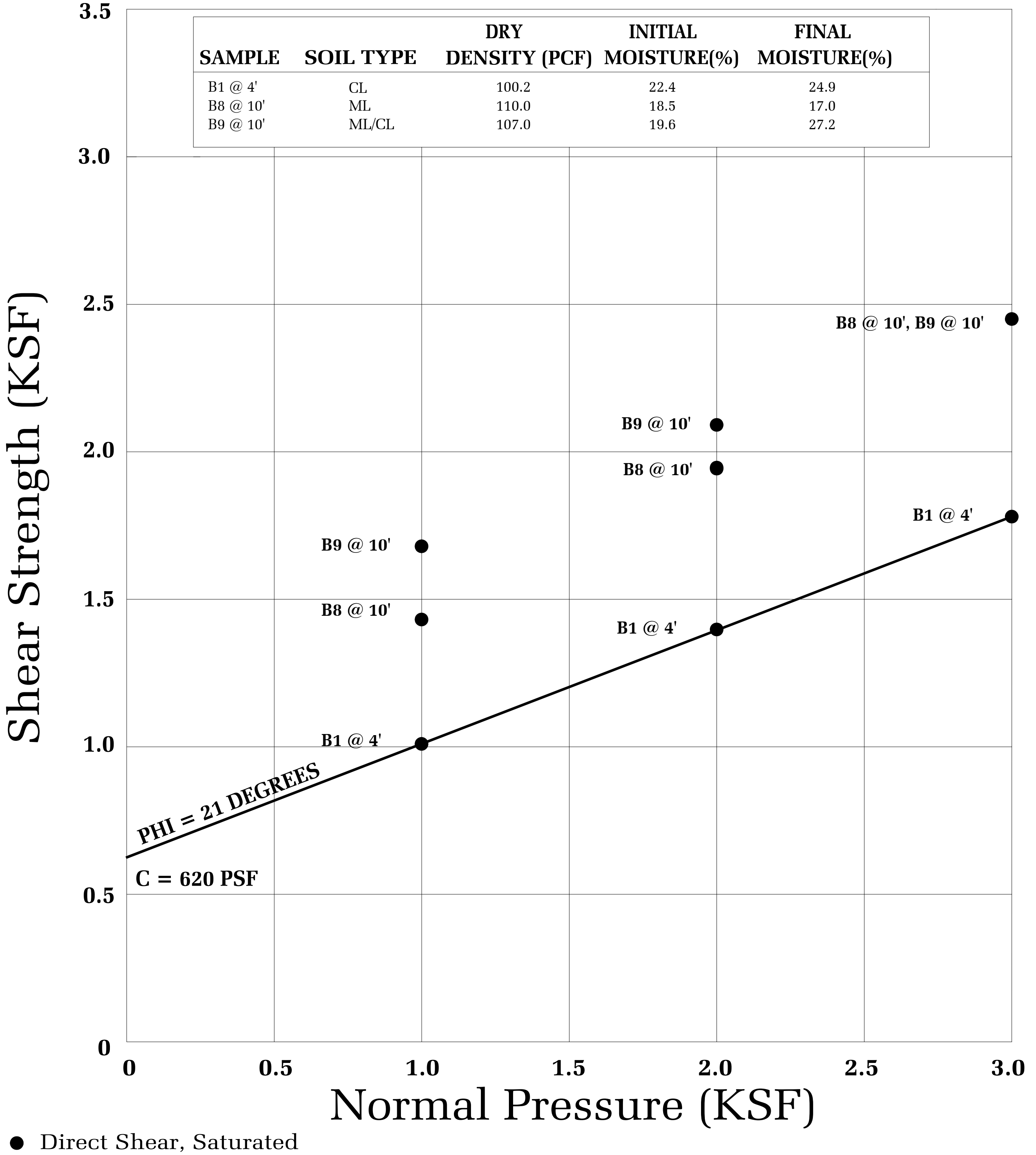
LOWE ENTERPRISES

File No. 20760

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-	SP	Sand, gray, wet, very dense, fine grained
				52 --		
				-		
				53 --		
				-		
				54 --		
				-		
				55 --		
				-		
				56 --		
				-		
				57 --		
				-		
				58 --		
				-		
				59 --		
				-		
60	40	17.3	97.1	60 --		
	50/4"			-		Total Depth 60 feet
				61 --		Water at 32½ feet
				-		Fill to 3 feet
				62 --		
				-		
				63 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				64 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				65 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				66 --		
				-		
				67 --		
				-		
				68 --		
				-		
				69 --		
				-		
				70 --		
				-		
				71 --		
				-		
				72 --		
				-		
				73 --		
				-		
				74 --		
				-		
				75 --		
				-		

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B1 @ 4'	CL	100.2	22.4	24.9
B8 @ 10'	ML	110.0	18.5	17.0
B9 @ 10'	ML/CL	107.0	19.6	27.2

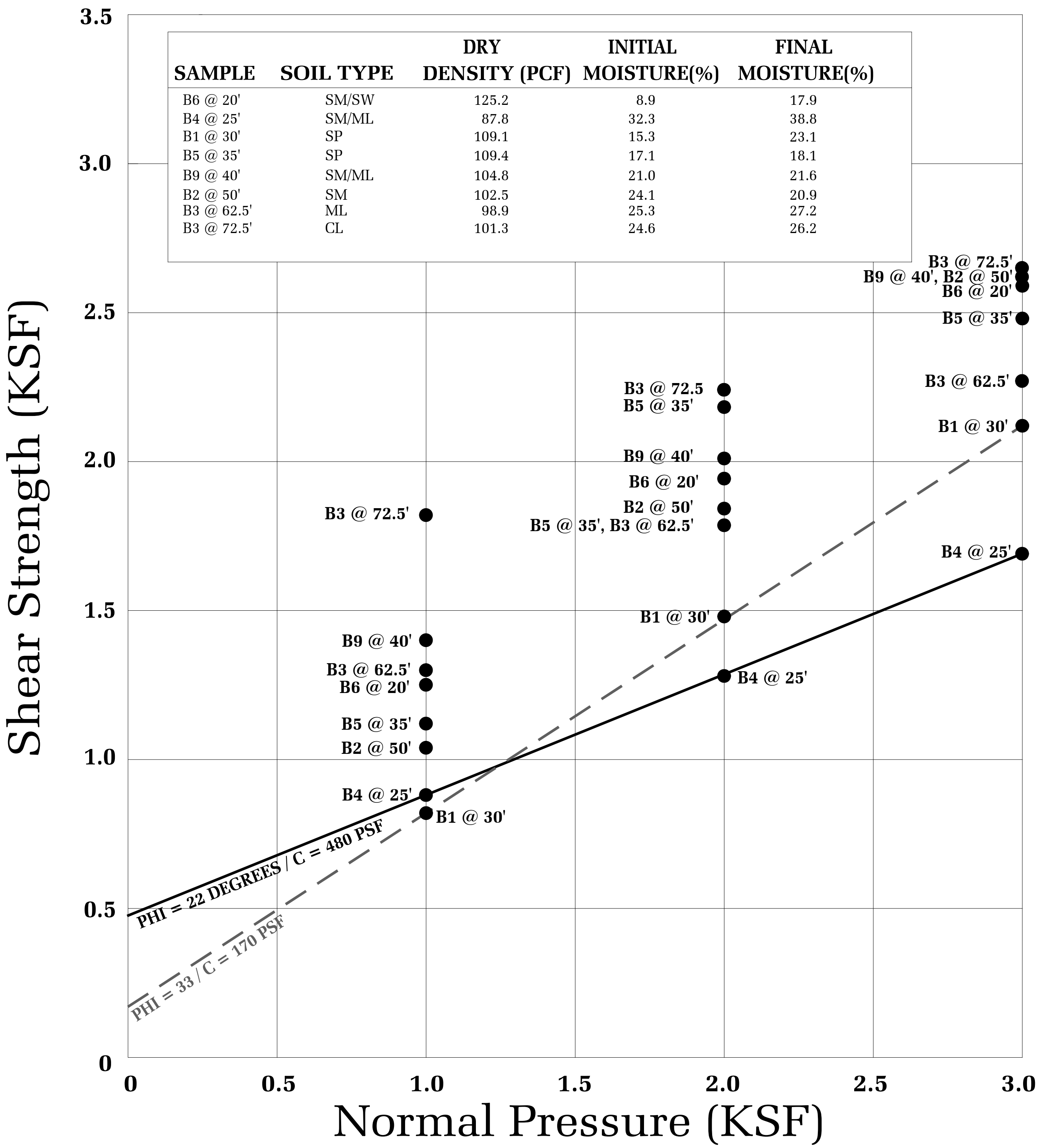


SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

LOWE ENTERPRISES

FILE NO. 20760 | PLATE: B-1



SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B6 @ 20'	SM/SW	125.2	8.9	17.9
B4 @ 25'	SM/ML	87.8	32.3	38.8
B1 @ 30'	SP	109.1	15.3	23.1
B5 @ 35'	SP	109.4	17.1	18.1
B9 @ 40'	SM/ML	104.8	21.0	21.6
B2 @ 50'	SM	102.5	24.1	20.9
B3 @ 62.5'	ML	98.9	25.3	27.2
B3 @ 72.5'	CL	101.3	24.6	26.2

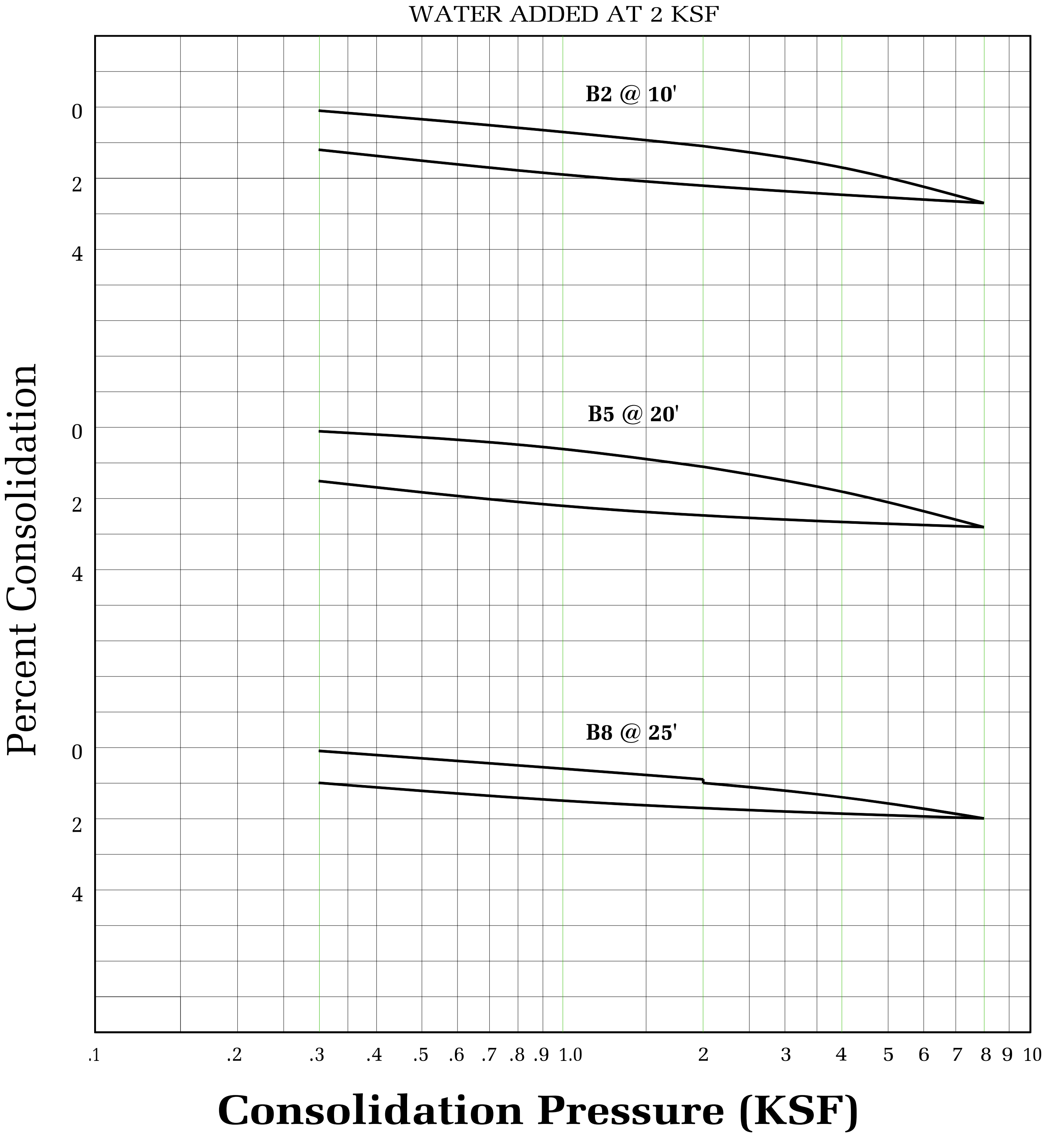
● Direct Shear, Saturated

SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

LOWE ENTERPRISES

FILE NO. 20760 | PLATE: B-2



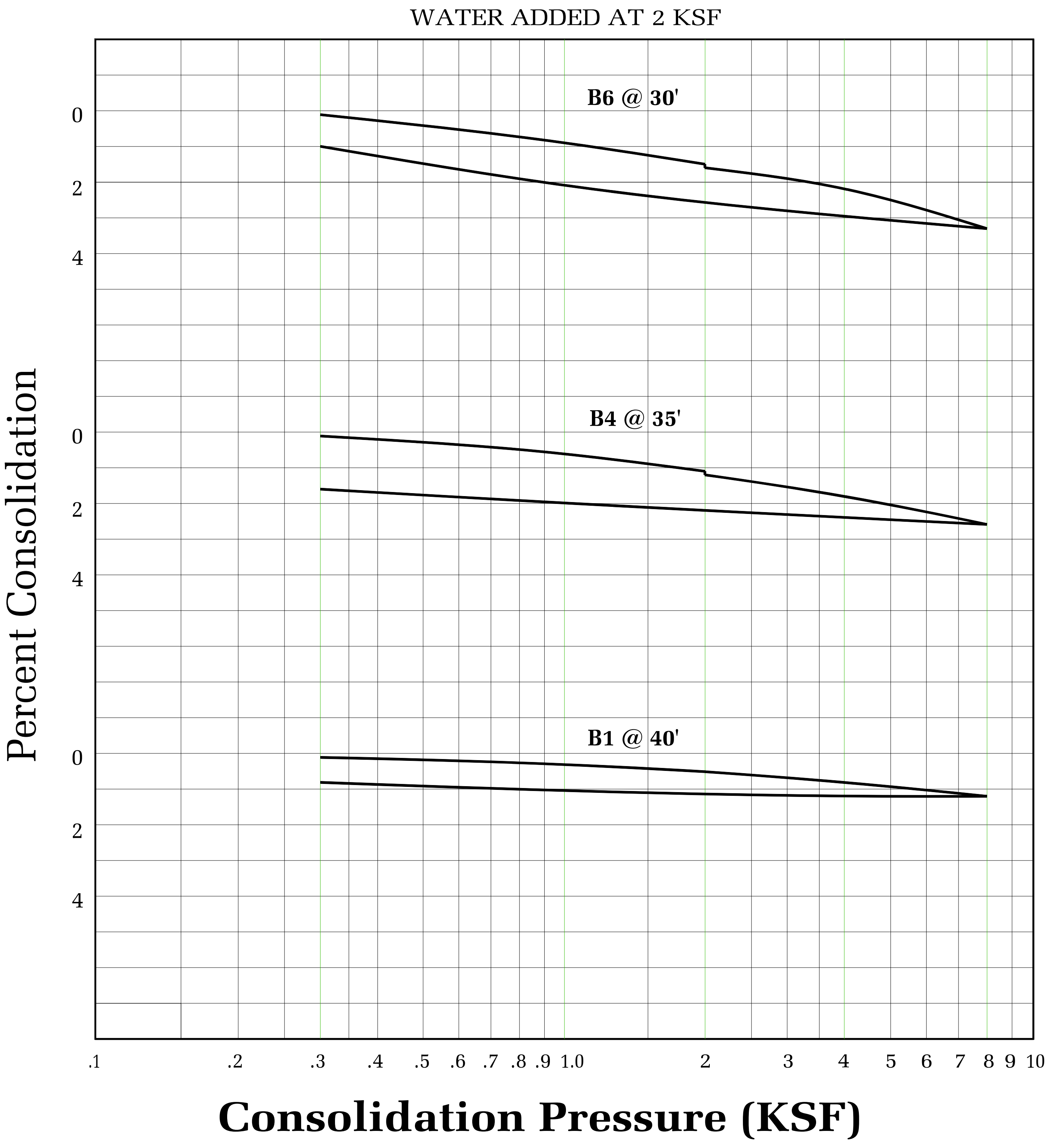
CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

LOWE ENTERPRISES

FILE NO. 20760

PLATE: C-1



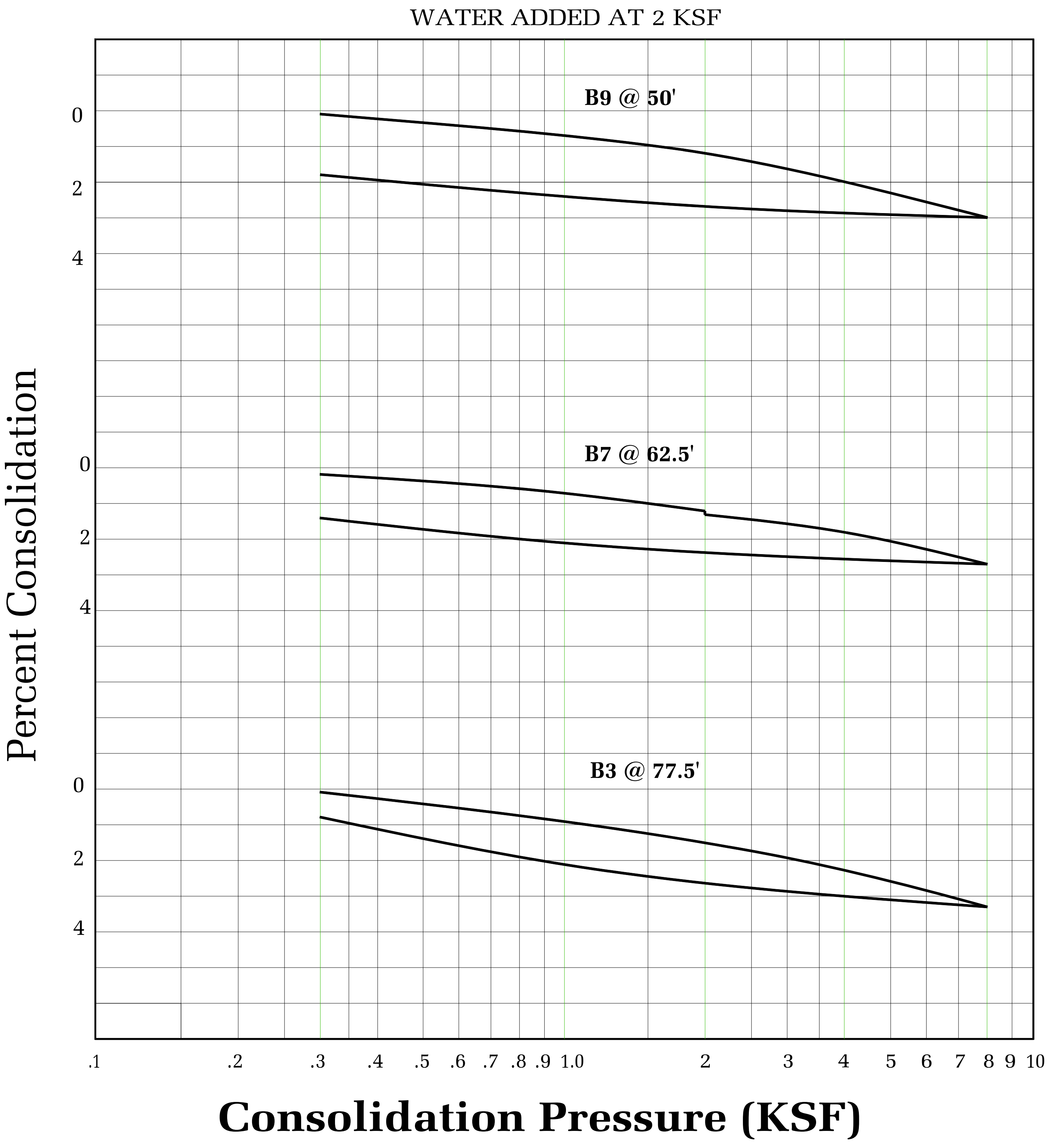
CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

LOWE ENTERPRISES

FILE NO. 20760

PLATE: C-2



CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

LOWE ENTERPRISES

FILE NO. 20760 | PLATE: C-3

ASTM D-1557

SAMPLE	B5 @ 1- 3'	B6 @ 1-5'
SOIL TYPE:	CL	CL
MAXIMUM DENSITY pcf.	119.4	127.5
OPTIMUM MOISTURE %	12.5	9.6

ASTM D 4829-03

SAMPLE	B5 @ 1- 3'	B6 @ 1-5'	B5 @ 20'	B6 @ 30'	B4 @ 35'
SOIL TYPE:	CL	CL	SW	SM/ML	SM
EXPANSION INDEX UBC STANDARD 18-2	116	104	3	13	4
EXPANSION CHARACTER	HIGH	HIGH	VERY LOW	VERY LOW	VERY LOW

SULFATE CONTENT

SAMPLE	B3 @ 1- 3'	B6 @ 1- 5'	B5 @ 20'	B6 @ 30'	B4 @ 35'
SULFATE CONTENT: (percentage by weight)	< 0.1 %	< 0.1 %	< 0.1 %	≥ 0.2 %	≥ 0.2 %

COMPACTION/EXPANSION/SULFATE DATA SHEET

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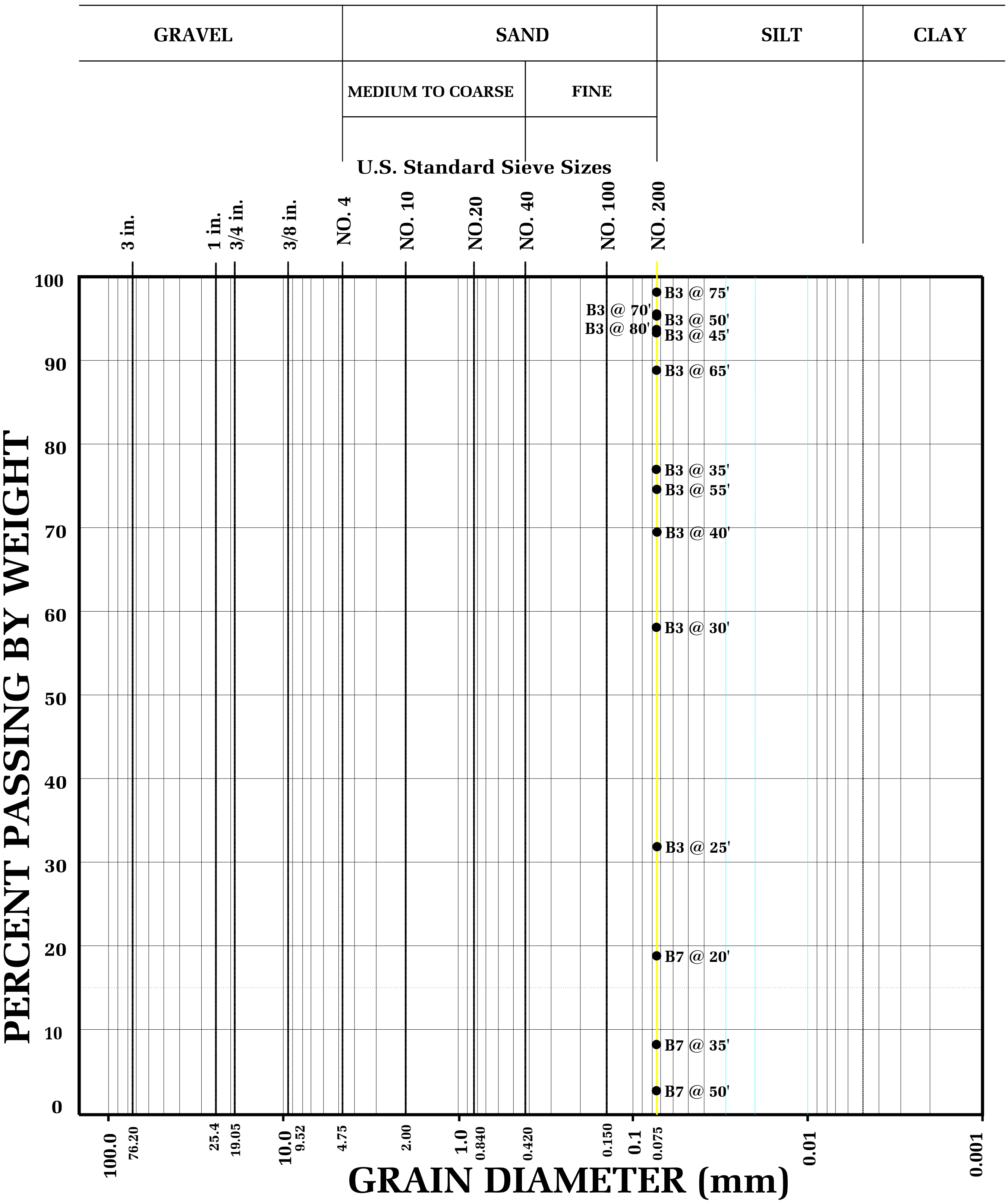
LOWE ENTERPRISES

FILE NO. 20760 | PLATE: D



GRAIN SIZE DISTRIBUTION

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LOWE ENTERPRISES

FILE NO. 20760

PLATE: E

Geotechnologies, Inc.

Project: Lowe Enterprises
File No.: 20760
Description: Liquefaction Analysis
Boring Number: 3

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL

NCEER (1996) METHOD | By Thomas F. Blake (1994-1996) | LIQ2_30.WQ1

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.6
Peak Horiz. Acceleration (g):	0.75
Calculated Mag.Wtg.Factor:	0.722

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	27.5
Historic Highest Groundwater Level* (ft):	18.0
Unit Wt. Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.30
Rod Len.Corr.(CR)(0-no or 1-yes):	1.0
Bore Dia. Corr. (CB):	1.00
Sampler Corr. (CS):	1.20
Use Ksigma (0 or 1):	1.0

LIQUEFACTION CALCULATIONS:

Depth to Base (ft)	Total Unit Wt. (pcf)	Current Water Level (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected $(N_1)_{60}$	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	128.2	0	NA	1.0	0	0.0		2.000	0.0	~	0.998	0.351	~
2.0	128.2	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.993	0.349	~
3.0	128.2	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.989	0.348	~
4.0	128.2	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.984	0.346	~
5.0	128.2	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.979	0.345	~
6.0	128.2	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.975	0.343	~
7.0	128.2	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.970	0.341	~
8.0	125.0	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.966	0.340	~
9.0	125.0	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.961	0.338	~
10.0	125.0	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.957	0.337	~
11.0	125.0	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.952	0.335	~
12.0	125.0	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.947	0.333	~
13.0	131.6	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.943	0.332	~
14.0	131.6	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.938	0.330	~
15.0	131.6	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.934	0.328	~
16.0	114.2	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.929	0.327	~
17.0	114.2	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.925	0.325	~
18.0	114.2	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.920	0.324	~
19.0	114.2	0	98.0	20.0	1	0.0	167	0.933	127.6	Infin.	0.915	0.322	Non-Liq.
20.0	114.2	0	98.0	20.0	1	0.0	167	0.933	127.6	Infin.	0.911	0.320	Non-Liq.
21.0	114.2	0	98.0	20.0	1	0.0	167	0.933	127.6	Infin.	0.906	0.319	Non-Liq.
22.0	114.2	0	98.0	20.0	1	0.0	167	0.933	127.6	Infin.	0.902	0.317	Non-Liq.
23.0	128.1	0	98.0	20.0	1	0.0	167	0.933	127.6	Infin.	0.897	0.316	Non-Liq.
24.0	128.1	0	98.0	20.0	1	0.0	167	0.933	127.6	Infin.	0.893	0.314	Non-Liq.
25.0	128.1	0	98.0	20.0	1	0.0	167	0.933	127.6	Infin.	0.888	0.312	Non-Liq.
26.0	128.1	0	23.0	25.0	1	31.8	76	0.835	34.9	Infin.	0.883	0.311	Non-Liq.
27.0	128.1	0	23.0	25.0	1	31.8	76	0.835	34.9	Infin.	0.879	0.309	Non-Liq.
28.0	125.8	1	33.0	30.0	1	58.0	87	0.776	46.9	Infin.	0.874	0.310	Non-Liq.
29.0	125.8	1	33.0	30.0	1	58.0	87	0.776	46.9	Infin.	0.870	0.314	Non-Liq.
30.0	125.8	1	33.0	30.0	1	58.0	87	0.776	46.9	Infin.	0.865	0.318	Non-Liq.
31.0	125.8	1	33.0	30.0	1	58.0	87	0.776	46.9	Infin.	0.861	0.321	Non-Liq.
32.0	125.8	1	33.0	30.0	1	58.0	87	0.776	46.9	Infin.	0.856	0.324	Non-Liq.
33.0	122.7	1	33.0	30.0	1	58.0	87	0.776	46.9	Infin.	0.851	0.327	Non-Liq.
34.0	122.7	1	33.0	30.0	1	58.0	87	0.776	46.9	Infin.	0.847	0.330	Non-Liq.
35.0	122.7	1	33.0	30.0	1	58.0	87	0.776	46.9	Infin.	0.842	0.333	Non-Liq.
36.0	122.7	1	33.0	35.0	1	77.0	84	0.744	45.3	Infin.	0.838	0.335	Non-Liq.
37.0	122.7	1	33.0	35.0	1	77.0	84	0.744	45.3	Infin.	0.833	0.337	Non-Liq.
38.0	126.2	1	36.0	40.0	1	69.5	86	0.715	47.2	Infin.	0.829	0.339	Non-Liq.
39.0	126.2	1	36.0	40.0	1	69.5	86	0.715	47.2	Infin.	0.824	0.341	Non-Liq.
40.0	126.2	1	36.0	40.0	1	69.5	86	0.715	47.2	Infin.	0.819	0.343	Non-Liq.
41.0	126.2	1	36.0	40.0	1	69.5	86	0.715	47.2	Infin.	0.815	0.344	Non-Liq.
42.0	126.2	1	36.0	40.0	1	69.5	86	0.715	47.2	Infin.	0.810	0.346	Non-Liq.
43.0	124.0	1	36.0	40.0	1	69.5	86	0.715	47.2	Infin.	0.806	0.347	Non-Liq.
44.0	124.0	1	36.0	40.0	1	69.5	86	0.715	47.2	Infin.	0.801	0.348	Non-Liq.
45.0	124.0	1	36.0	40.0	1	69.5	86	0.715	47.2	Infin.	0.797	0.349	Non-Liq.
46.0	123.8	1	27.0	45.0	1	93.3	73	0.690	36.0	Infin.	0.792	0.350	Non-Liq.
47.0	123.8	1	27.0	45.0	1	93.3	73	0.690	36.0	Infin.	0.787	0.351	Non-Liq.
48.0	123.8	1	27.0	45.0	1	93.3	73	0.690	36.0	Infin.	0.783	0.351	Non-Liq.
49.0	123.8	1	27.0	45.0	1	93.3	73	0.690	36.0	Infin.	0.778	0.352	Non-Liq.
50.0	123.8	1	27.0	45.0	1	93.3	73	0.690	36.0	Infin.	0.774	0.353	Non-Liq.
51.0	123.8	1	29.0	50.0	1	95.3	73	0.667	37.2	Infin.	0.769	0.353	Non-Liq.
52.0	123.8	1	29.0	50.0	1	95.3	73	0.667	37.2	Infin.	0.765	0.353	Non-Liq.
53.0	124.5	1	35.0	55.0	1	74.6	79	0.646	42.3	Infin.	0.760	0.353	Non-Liq.
54.0	124.5	1	35.0	55.0	1	74.6	79	0.646	42.3	Infin.	0.755	0.354	Non-Liq.
55.0	124.5	1	35.0	55.0	1	74.6	79	0.646	42.3	Infin.	0.751	0.354	Non-Liq.

56.0	124.5	1	35.0	55.0	1	74.6	79	0.646	42.3	Infin.	0.746	0.354	Non-Liq.
57.0	124.5	1	35.0	55.0	1	74.6	79	0.646	42.3	Infin.	0.742	0.353	Non-Liq.
58.0	124.5	1	35.0	55.0	1	74.6	79	0.646	42.3	Infin.	0.737	0.353	Non-Liq.
59.0	124.5	1	35.0	55.0	1	74.6	79	0.646	42.3	Infin.	0.733	0.353	Non-Liq.
60.0	124.5	1	35.0	55.0	1	74.6	79	0.646	42.3	Infin.	0.728	0.353	Non-Liq.
61.0	124.5	1	62.0	60.0	1	0.0	103	0.627	60.7	Infin.	0.723	0.352	Non-Liq.
62.0	124.5	1	62.0	60.0	1	0.0	103	0.627	60.7	Infin.	0.719	0.352	Non-Liq.
63.0	123.9	1	62.0	60.0	1	0.0	103	0.627	60.7	Infin.	0.714	0.351	Non-Liq.
64.0	123.9	1	62.0	60.0	1	0.0	103	0.627	60.7	Infin.	0.710	0.351	Non-Liq.
65.0	123.9	1	62.0	60.0	1	0.0	103	0.627	60.7	Infin.	0.705	0.350	Non-Liq.
66.0	123.1	1	35.0	65.0	1	88.8	76	0.610	40.3	Infin.	0.701	0.349	Non-Liq.
67.0	123.1	1	35.0	65.0	1	88.8	76	0.610	40.3	Infin.	0.696	0.349	Non-Liq.
68.0	123.1	1	35.0	65.0	1	88.8	76	0.610	40.3	Infin.	0.691	0.348	Non-Liq.
69.0	123.1	1	35.0	65.0	1	88.8	76	0.610	40.3	Infin.	0.687	0.347	Non-Liq.
70.0	123.1	1	35.0	65.0	1	88.8	76	0.610	40.3	Infin.	0.682	0.346	Non-Liq.
71.0	123.1	1	43.0	70.0	1	95.5	82	0.600	47.2	Infin.	0.678	0.345	Non-Liq.
72.0	123.1	1	43.0	70.0	1	95.5	82	0.600	47.2	Infin.	0.673	0.344	Non-Liq.
73.0	126.3	1	43.0	70.0	1	95.5	82	0.600	47.2	Infin.	0.669	0.343	Non-Liq.
74.0	126.3	1	43.0	70.0	1	95.5	82	0.600	47.2	Infin.	0.664	0.342	Non-Liq.
75.0	126.3	1	43.0	70.0	1	95.5	82	0.600	47.2	Infin.	0.659	0.341	Non-Liq.
76.0	126.3	1	32.0	75.0	1	98.2	70	0.600	37.0	Infin.	0.655	0.340	Non-Liq.
77.0	126.3	1	32.0	75.0	1	98.2	70	0.600	37.0	Infin.	0.650	0.339	Non-Liq.
78.0	123.9	1	34.0	80.0	1	93.7	70	0.600	38.8	Infin.	0.646	0.337	Non-Liq.
79.0	123.9	1	34.0	80.0	1	93.7	70	0.600	38.8	Infin.	0.641	0.336	Non-Liq.
80.0	123.9	1	34.0	80.0	1	93.7	70	0.600	38.8	Infin.	0.637	0.335	Non-Liq.

Geotechnologies, Inc.

Project: Lowe Enterprises
File No.: 20760
Description: Liquefaction Analysis
Boring Number: 7

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL

NCEER (1996) METHOD | By Thomas F. Blake (1994-1996) | LIQ2_30.WQ1

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.6
Peak Horiz. Acceleration (g):	0.75
Calculated Mag.Wtg.Factor:	0.722

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	32.0
Historic Highest Groundwater Level* (ft):	22.0
Unit Wt. Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

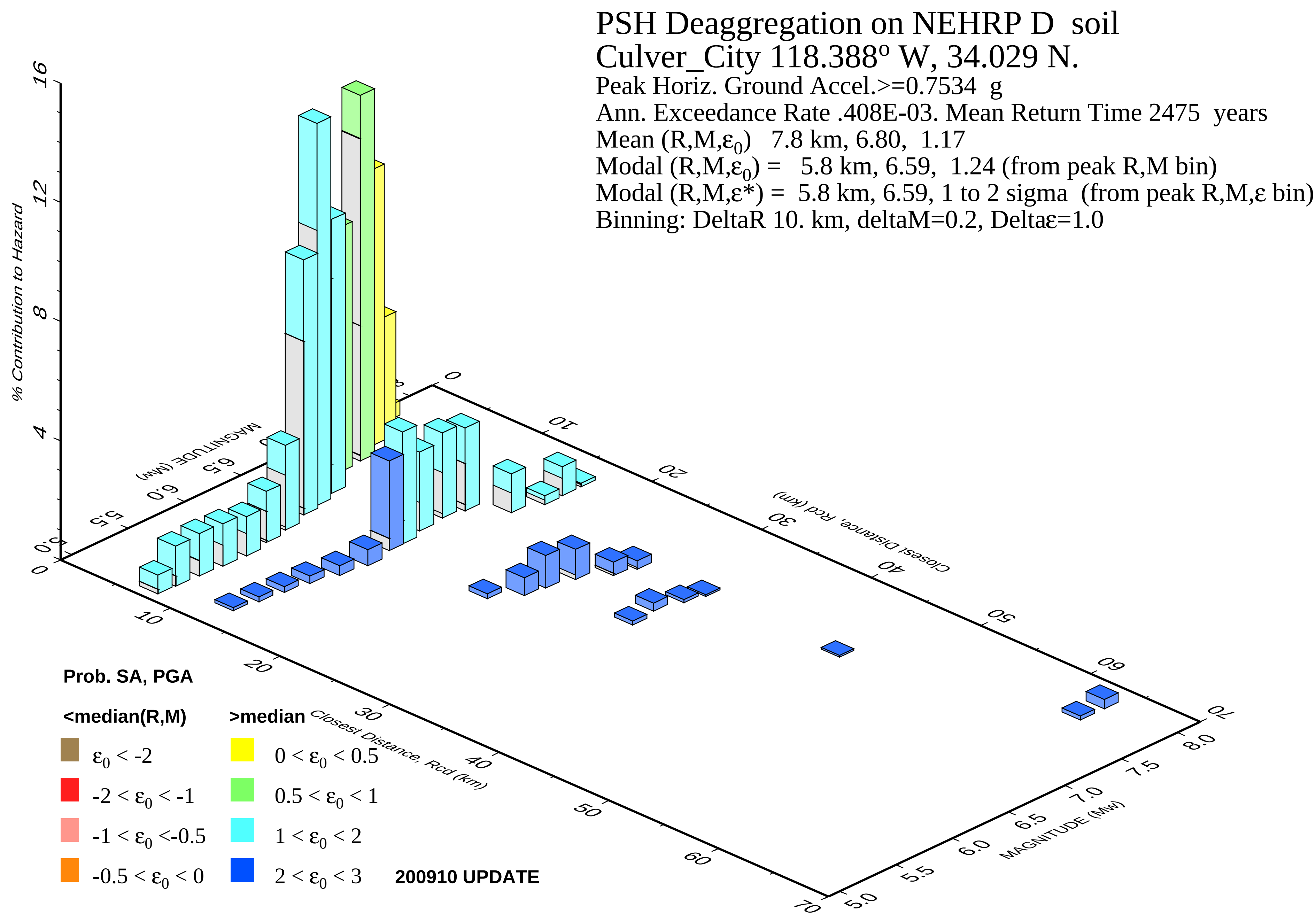
ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.30
Rod Len.Corr.(CR)(0-no or 1-yes):	1.0
Bore Dia. Corr. (CB):	1.00
Sampler Corr. (CS):	1.20
Use Ksigma (0 or 1):	1.0

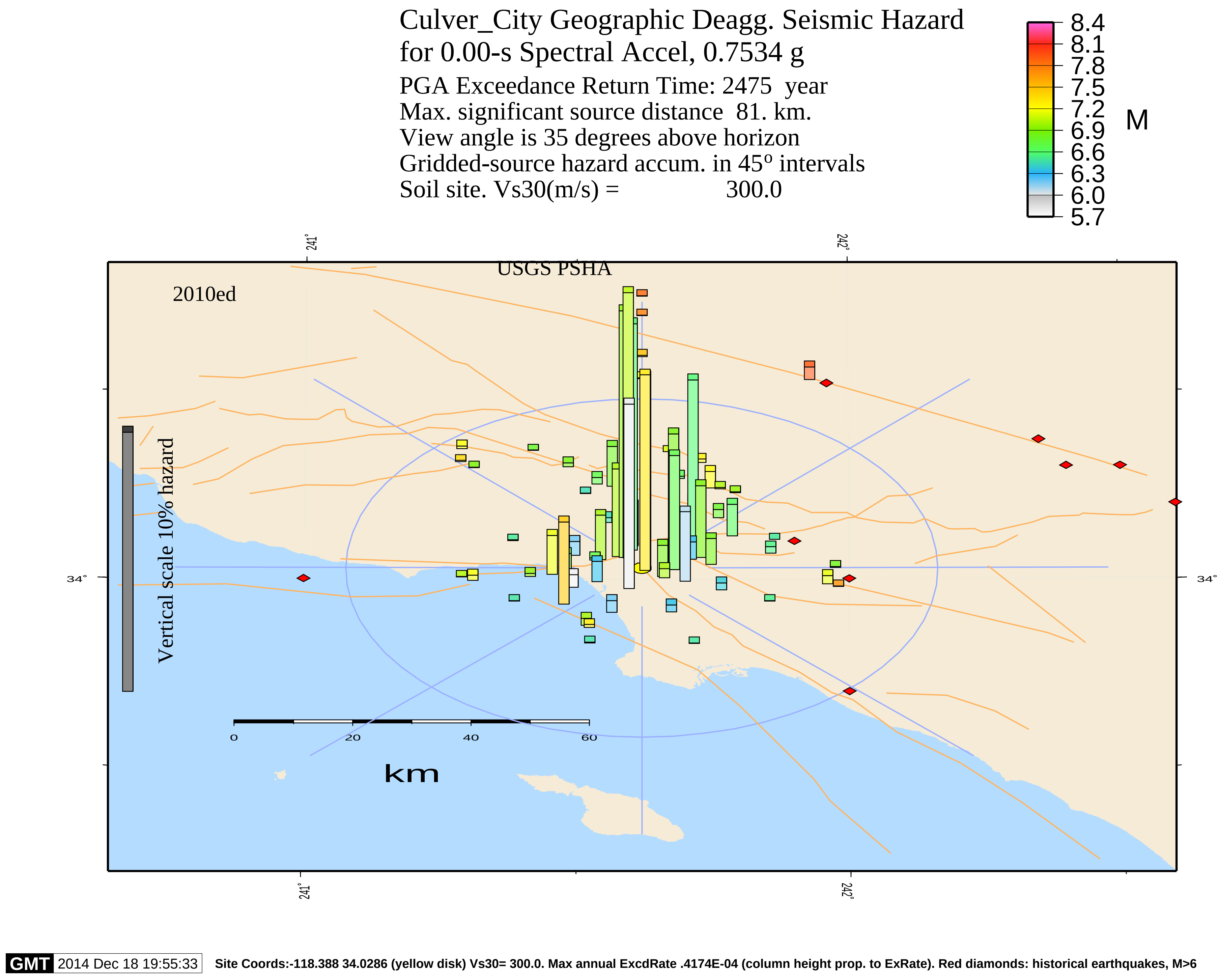
LIQUEFACTION CALCULATIONS:

Depth to Base (ft)	Total Unit Wt. (pcf)	Current Water Level (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected $(N_1)_{60}$	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	129.5	0	NA	1.0	0	0.0		2.000	0.0	~	0.998	0.351	~
2.0	129.5	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.993	0.349	~
3.0	129.5	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.989	0.348	~
4.0	129.5	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.984	0.346	~
5.0	129.5	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.979	0.345	~
6.0	129.5	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.975	0.343	~
7.0	129.5	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.970	0.341	~
8.0	131.3	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.966	0.340	~
9.0	131.3	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.961	0.338	~
10.0	131.3	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.957	0.337	~
11.0	123.8	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.952	0.335	~
12.0	123.8	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.947	0.333	~
13.0	123.8	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.943	0.332	~
14.0	123.8	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.938	0.330	~
15.0	123.8	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.934	0.328	~
16.0	123.8	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.929	0.327	~
17.0	123.8	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.925	0.325	~
18.0	118.4	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.920	0.324	~
19.0	118.4	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.915	0.322	~
20.0	118.4	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.911	0.320	~
21.0	118.4	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.906	0.319	~
22.0	118.4	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.902	0.317	~
23.0	117.7	0	41.0	25.0	1	0.0	101	0.832	50.9	Infin.	0.897	0.316	Non-Liq.
24.0	117.7	0	41.0	25.0	1	0.0	101	0.832	50.9	Infin.	0.893	0.314	Non-Liq.
25.0	117.7	0	41.0	25.0	1	0.0	101	0.832	50.9	Infin.	0.888	0.312	Non-Liq.
26.0	117.7	0	41.0	25.0	1	0.0	101	0.832	50.9	Infin.	0.883	0.311	Non-Liq.
27.0	117.7	0	41.0	25.0	1	0.0	101	0.832	50.9	Infin.	0.879	0.309	Non-Liq.
28.0	118.2	0	41.0	25.0	1	0.0	101	0.832	50.9	Infin.	0.874	0.308	Non-Liq.
29.0	118.2	0	41.0	25.0	1	0.0	101	0.832	50.9	Infin.	0.870	0.306	Non-Liq.
30.0	118.2	0	41.0	25.0	1	0.0	101	0.832	50.9	Infin.	0.865	0.304	Non-Liq.
31.0	128.9	0	36.0	30.0	1	0.0	90	0.762	42.8	Infin.	0.861	0.303	Non-Liq.
32.0	128.9	0	36.0	30.0	1	0.0	90	0.762	42.8	Infin.	0.856	0.301	Non-Liq.
33.0	128.9	1	36.0	30.0	1	0.0	90	0.762	42.8	Infin.	0.851	0.302	Non-Liq.
34.0	128.9	1	36.0	30.0	1	0.0	90	0.762	42.8	Infin.	0.847	0.305	Non-Liq.
35.0	128.9	1	36.0	30.0	1	0.0	90	0.762	42.8	Infin.	0.842	0.308	Non-Liq.
36.0	128.9	1	35.0	35.0	1	8.2	85	0.716	39.8	Infin.	0.838	0.310	Non-Liq.
37.0	128.9	1	35.0	35.0	1	8.2	85	0.716	39.8	Infin.	0.833	0.312	Non-Liq.
38.0	124.8	1	35.0	35.0	1	8.2	85	0.716	39.8	Infin.	0.829	0.315	Non-Liq.
39.0	124.8	1	35.0	35.0	1	8.2	85	0.716	39.8	Infin.	0.824	0.317	Non-Liq.
40.0	124.8	1	35.0	35.0	1	8.2	85	0.716	39.8	Infin.	0.819	0.319	Non-Liq.
41.0	124.8	1	48.0	40.0	1	0.0	97	0.690	51.6	Infin.	0.815	0.320	Non-Liq.
42.0	124.8	1	48.0	40.0	1	0.0	97	0.690	51.6	Infin.	0.810	0.322	Non-Liq.
43.0	126.9	1	48.0	40.0	1	0.0	97	0.690	51.6	Infin.	0.806	0.323	Non-Liq.
44.0	126.9	1	48.0	40.0	1	0.0	97	0.690	51.6	Infin.	0.801	0.325	Non-Liq.
45.0	126.9	1	48.0	40.0	1	0.0	97	0.690	51.6	Infin.	0.797	0.326	Non-Liq.
46.0	126.9	1	46.0	45.0	1	0.0	92	0.666	47.8	Infin.	0.792	0.327	Non-Liq.
47.0	126.9	1	46.0	45.0	1	0.0	92	0.666	47.8	Infin.	0.787	0.328	Non-Liq.
48.0	125.5	1	40.0	50.0	1	2.7	84	0.645	40.3	Infin.	0.783	0.329	Non-Liq.
49.0	125.5	1	40.0	50.0	1	2.7	84	0.645	40.3	Infin.	0.778	0.330	Non-Liq.
50.0	125.5	1	40.0	50.0	1	2.7	84	0.645	40.3	Infin.	0.774	0.331	Non-Liq.
51.0	125.5	1	40.0	50.0	1	2.7	84	0.645	40.3	Infin.	0.769	0.331	Non-Liq.
52.0	125.5	1	40.0	50.0	1	2.7	84	0.645	40.3	Infin.	0.765	0.332	Non-Liq.
53.0	123.8	1	50.0	55.0	1	0.0	92	0.626	48.8	Infin.	0.760	0.332	Non-Liq.
54.0	123.8	1	50.0	55.0	1	0.0	92	0.626	48.8	Infin.	0.755	0.333	Non-Liq.
55.0	123.8	1	50.0	55.0	1	0.0	92	0.626	48.8	Infin.	0.751	0.333	Non-Liq.

56.0	123.8	1	50.0	55.0	1	0.0	92	0.626	48.8	Infin.	0.746	0.333	Non-Liq.
57.0	123.8	1	50.0	55.0	1	0.0	92	0.626	48.8	Infin.	0.742	0.333	Non-Liq.
58.0	127.5	1	50.0	55.0	1	0.0	92	0.626	48.8	Infin.	0.737	0.333	Non-Liq.
59.0	127.5	1	50.0	55.0	1	0.0	92	0.626	48.8	Infin.	0.733	0.333	Non-Liq.
60.0	127.5	1	50.0	55.0	1	0.0	92	0.626	48.8	Infin.	0.728	0.333	Non-Liq.
61.0	129.2	1	44.0	60.0	1	0.0	85	0.609	41.8	Infin.	0.723	0.333	Non-Liq.
62.0	129.2	1	44.0	60.0	1	0.0	85	0.609	41.8	Infin.	0.719	0.333	Non-Liq.
63.0	129.2	1	62.0	65.0	1	0.0	98	0.600	58.0	Infin.	0.714	0.332	Non-Liq.
64.0	129.2	1	62.0	65.0	1	0.0	98	0.600	58.0	Infin.	0.710	0.332	Non-Liq.
65.0	129.2	1	62.0	65.0	1	0.0	98	0.600	58.0	Infin.	0.705	0.331	Non-Liq.
66.0	129.2	1	62.0	65.0	1	0.0	98	0.600	58.0	Infin.	0.701	0.331	Non-Liq.
67.0	129.2	1	62.0	65.0	1	0.0	98	0.600	58.0	Infin.	0.696	0.330	Non-Liq.
68.0	124.4	1	60.0	70.0	1	0.0	95	0.600	56.2	Infin.	0.691	0.330	Non-Liq.
69.0	124.4	1	60.0	70.0	1	0.0	95	0.600	56.2	Infin.	0.687	0.329	Non-Liq.
70.0	124.4	1	60.0	70.0	1	0.0	95	0.600	56.2	Infin.	0.682	0.328	Non-Liq.



GMT 2014 Dec 18 19:55:32 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 300. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted


Culver_City Geographic Deagg. Seismic Hazard
for 0.00-s Spectral Accel, 0.7534 g
PGA Exceedance Return Time: 2475 year
Max. significant source distance 81. km.
View angle is 35 degrees above horizon
Gridded-source hazard accum. in 45° intervals
Soil site. Vs30(m/s) = 300.0
8.4
8.1
7.8
7.5
7.2
6.9
6.6
6.3
6.0
5.7
M
241°
242°
USGS PSHA
2010ed
Vertical scale 10% hazard
34°
0
20
40
60
km
GMT 2014 Dec 18 19:55:33
Site Coords:-118.388 34.0286 (yellow disk) Vs30= 300.0. Max annual ExcdRate .4174E-04 (column height prop. to ExRate). Red diamonds: historical earthquakes, M>6

*** Deaggregation of Seismic Hazard at One Period of Spectral Accel. ***
*** Data from U.S.G.S. National Seismic Hazards Mapping Project, 2008 version ***
PSHA Deaggregation. %contributions. site: Culver_City long: 118.388 W., lat: 34.029 N.
Vs30(m/s)= 300.0 (some WUS atten. models use Site Class not Vs30).
NSHMP 2007-08 See USGS OFR 2008-1128. dM=0.2 below
Return period: 2475 yrs. Exceedance PGA =0.7534 g. Weight * Computed_Rate_Ex 0.408E-03
#Pr[at least one eq with median motion>=PGA in 50 yrs]=0.00131
#This deaggregation corresponds to Mean Hazard w/all GMPEs

DIST(KM)	MAG(MW)	ALL_EPS	EPSILON>2	1<EPS<2	0<EPS<1	-1<EPS<0	-2<EPS<-1	EPS<-2
7.1	5.05	0.628	0.488	0.140	0.000	0.000	0.000	0.000
7.2	5.20	1.347	1.008	0.339	0.000	0.000	0.000	0.000
12.4	5.21	0.107	0.107	0.000	0.000	0.000	0.000	0.000
7.3	5.40	1.425	0.909	0.516	0.000	0.000	0.000	0.000
12.7	5.40	0.155	0.155	0.000	0.000	0.000	0.000	0.000
7.4	5.60	1.415	0.716	0.699	0.000	0.000	0.000	0.000
13.0	5.60	0.201	0.201	0.000	0.000	0.000	0.000	0.000
7.5	5.80	1.314	0.574	0.716	0.025	0.000	0.000	0.000
13.3	5.80	0.243	0.243	0.000	0.000	0.000	0.000	0.000
7.1	6.01	1.714	0.664	0.975	0.075	0.000	0.000	0.000
13.9	6.01	0.322	0.312	0.010	0.000	0.000	0.000	0.000
6.8	6.22	2.847	1.002	1.742	0.103	0.000	0.000	0.000
14.2	6.22	0.535	0.497	0.038	0.000	0.000	0.000	0.000
6.1	6.44	8.562	2.738	5.595	0.229	0.000	0.000	0.000
13.7	6.46	3.002	2.600	0.402	0.000	0.000	0.000	0.000
23.2	6.41	0.172	0.172	0.000	0.000	0.000	0.000	0.000
5.8	6.59	12.824	3.584	8.356	0.884	0.000	0.000	0.000
13.7	6.59	3.761	2.978	0.784	0.000	0.000	0.000	0.000
24.6	6.60	0.601	0.601	0.000	0.000	0.000	0.000	0.000
5.3	6.77	9.194	1.970	6.238	0.986	0.000	0.000	0.000
13.2	6.78	2.652	1.691	0.946	0.016	0.000	0.000	0.000
24.9	6.76	1.092	1.045	0.046	0.000	0.000	0.000	0.000
32.7	6.78	0.139	0.139	0.000	0.000	0.000	0.000	0.000
3.5	7.00	8.226	1.135	4.388	2.460	0.243	0.000	0.000
13.0	7.01	2.859	1.334	1.381	0.144	0.000	0.000	0.000
25.4	6.98	1.012	0.857	0.155	0.000	0.000	0.000	0.000
32.3	7.00	0.280	0.280	0.000	0.000	0.000	0.000	0.000
3.4	7.21	12.276	1.460	6.278	4.347	0.191	0.000	0.000
13.3	7.17	2.786	1.197	1.374	0.215	0.000	0.000	0.000
26.8	7.18	0.460	0.384	0.076	0.000	0.000	0.000	0.000
32.9	7.22	0.108	0.108	0.000	0.000	0.000	0.000	0.000
2.4	7.40	9.261	0.982	4.143	3.857	0.278	0.000	0.000
15.7	7.36	1.299	0.639	0.642	0.018	0.000	0.000	0.000
27.3	7.34	0.291	0.230	0.061	0.000	0.000	0.000	0.000
33.3	7.37	0.061	0.059	0.002	0.000	0.000	0.000	0.000
45.7	7.35	0.061	0.061	0.000	0.000	0.000	0.000	0.000
1.9	7.55	3.955	0.341	1.585	1.681	0.348	0.000	0.000
16.4	7.58	0.298	0.152	0.146	0.000	0.000	0.000	0.000
0.6	7.71	0.482	0.038	0.178	0.232	0.035	0.000	0.000
16.4	7.74	0.978	0.401	0.577	0.000	0.000	0.000	0.000
63.3	7.77	0.135	0.135	0.000	0.000	0.000	0.000	0.000
16.4	7.91	0.121	0.042	0.077	0.002	0.000	0.000	0.000
63.3	7.98	0.303	0.303	0.000	0.000	0.000	0.000	0.000

Summary statistics for above PSHA PGA deaggregation, R=distance, e=epsilon:
Contribution from this GMPE(%): 100.0
Mean src-site R= 7.8 km; M= 6.80; eps0= 1.17. Mean calculated for all sources.
Modal src-site R= 5.8 km; M= 6.59; eps0= 1.24 from peak (R,M) bin
MODE R*= 5.8km; M*= 6.59; EPS.INTERVAL: 1 to 2 sigma % CONTRIB.= 8.356

Principal sources (faults, subduction, random seismicity having > 3% contribution)

Source Category:	% contr.	R(km)	M	epsilon0 (mean values).
California B-faults Char	59.35	7.1	7.04	1.03
California B-faults GR	23.81	7.7	6.74	1.28
CA Compr. crustal gridded	15.94	8.2	5.94	1.48

Individual fault hazard details if its contribution to mean hazard > 2%:

Fault ID	% contr.	Rcd(km)	M	epsilon0	Site-to-src azimuth(d)
Hollywood Char	6.56	6.4	6.59	1.44	-15.2
Newport-Inglewood, alt 1 Char	4.02	0.6	7.14	0.30	65.7
Newport-Inglewood, alt 2 Char	3.98	0.7	7.14	0.32	61.0
Elysian Park (Upper) Char	4.04	12.9	6.60	1.88	42.1
Puente Hills Char	2.40	10.3	7.06	1.10	68.3
Puente Hills (LA) Char	5.22	5.3	6.88	0.93	84.8
Newport Inglewood Connected alt	3.51	0.6	7.50	0.25	65.7
Newport Inglewood Connected alt	3.46	0.7	7.50	0.27	61.0
Santa Monica, alt 2 Char	2.99	4.2	6.68	1.12	-30.6
Santa Monica, alt 1 Char	2.66	4.9	6.49	1.30	-36.5
Santa Monica Connected alt 1 Cha	5.39	4.9	7.30	0.76	-36.5
Santa Monica Connected alt 2 Cha	5.72	4.3	7.35	0.62	-30.6
Hollywood, nominally GR	2.17	6.4	6.50	1.49	-15.2
Puente Hills (LA) GR	4.40	5.5	6.69	1.06	85.9
Santa Monica Connected alt 2 GR	3.38	6.0	7.01	0.94	-43.6

#*********End of deaggregation corresponding to Mean Hazard w/all GMPEs *********#

PSHA Deaggregation. %contributions. site: Culver_City long: 118.388 W., lat: 34.029 N.
Vs30(m/s)= 300.0 (some WUS atten. models use Site Class not Vs30).
NSHMP 2007-08 See USGS OFR 2008-1128. dM=0.2 below
Return period: 2475 yrs. Exceedance PGA =0.7534 g. Weight * Computed_Rate_Ex 0.155E-03
#Pr[at least one eq with median motion>=PGA in 50 yrs]=0.00142
#This deaggregation corresponds to Boore-Atkinson 2008

DIST(KM)	MAG(MW)	ALL_EPS	EPSILON>2	1<EPS<2	0<EPS<1	-1<EPS<0	-2<EPS<-1	EPS<-2
6.3	5.05	0.049	0.049	0.000	0.000	0.000	0.000	0.000
6.5	5.20	0.130	0.130	0.000	0.000	0.000	0.000	0.000
6.6	5.40	0.168	0.168	0.000	0.000	0.000	0.000	0.000
6.9	5.60	0.206	0.192	0.014	0.000	0.000	0.000	0.000
7.1	5.80	0.235	0.199	0.037	0.000	0.000	0.000	0.000
13.0	5.81	0.021	0.021	0.000	0.000	0.000	0.000	0.000
6.6	6.02	0.409	0.298	0.110	0.000	0.000	0.000	0.000
14.0	6.01	0.049	0.049	0.000	0.000	0.000	0.000	0.000
6.3	6.22	0.803	0.506	0.297	0.000	0.000	0.000	0.000
14.6	6.23	0.124	0.124	0.000	0.000	0.000	0.000	0.000
6.0	6.45	2.956	1.274	1.682	0.000	0.000	0.000	0.000
14.6	6.46	0.815	0.756	0.059	0.000	0.000	0.000	0.000
23.7	6.42	0.096	0.096	0.000	0.000	0.000	0.000	0.000
5.8	6.58	4.944	1.725	3.092	0.127	0.000	0.000	0.000
14.3	6.58	1.465	1.341	0.124	0.000	0.000	0.000	0.000
24.6	6.60	0.361	0.361	0.000	0.000	0.000	0.000	0.000
32.1	6.61	0.026	0.026	0.000	0.000	0.000	0.000	0.000
5.0	6.77	3.566	0.826	2.431	0.309	0.000	0.000	0.000
13.9	6.79	1.027	0.771	0.255	0.000	0.000	0.000	0.000
24.6	6.77	0.575	0.563	0.012	0.000	0.000	0.000	0.000
32.7	6.78	0.138	0.138	0.000	0.000	0.000	0.000	0.000
2.5	7.02	3.650	0.457	1.853	1.242	0.098	0.000	0.000
13.7	7.01	1.128	0.594	0.520	0.015	0.000	0.000	0.000
25.5	6.99	0.610	0.527	0.083	0.000	0.000	0.000	0.000
32.3	7.00	0.276	0.276	0.000	0.000	0.000	0.000	0.000
46.1	7.05	0.044	0.044	0.000	0.000	0.000	0.000	0.000
3.6	7.23	4.261	0.548	2.353	1.290	0.070	0.000	0.000
14.3	7.18	1.225	0.637	0.559	0.030	0.000	0.000	0.000
27.0	7.18	0.350	0.295	0.055	0.000	0.000	0.000	0.000
32.9	7.22	0.103	0.102	0.000	0.000	0.000	0.000	0.000
46.9	7.21	0.041	0.041	0.000	0.000	0.000	0.000	0.000

56.8	7.17	0.022	0.022	0.000	0.000	0.000	0.000	0.000
1.6	7.42	4.096	0.349	1.816	1.758	0.174	0.000	0.000
16.0	7.36	0.802	0.387	0.412	0.003	0.000	0.000	0.000
27.3	7.34	0.219	0.158	0.061	0.000	0.000	0.000	0.000
33.3	7.37	0.056	0.054	0.002	0.000	0.000	0.000	0.000
45.7	7.35	0.057	0.057	0.000	0.000	0.000	0.000	0.000
2.1	7.58	1.074	0.097	0.507	0.430	0.039	0.000	0.000
16.5	7.59	0.174	0.072	0.102	0.000	0.000	0.000	0.000
32.2	7.59	0.031	0.026	0.006	0.000	0.000	0.000	0.000
63.3	7.56	0.050	0.050	0.000	0.000	0.000	0.000	0.000
0.6	7.71	0.228	0.015	0.085	0.115	0.014	0.000	0.000
16.4	7.74	0.581	0.204	0.376	0.000	0.000	0.000	0.000
32.1	7.77	0.027	0.024	0.003	0.000	0.000	0.000	0.000
63.3	7.77	0.135	0.135	0.000	0.000	0.000	0.000	0.000
16.4	7.91	0.070	0.021	0.046	0.002	0.000	0.000	0.000
63.3	7.98	0.293	0.293	0.000	0.000	0.000	0.000	0.000
63.3	8.20	0.031	0.031	0.000	0.000	0.000	0.000	0.000

Summary statistics for above PSHA PGA deaggregation, R=distance, e=epsilon:
Contribution from this GMPE(%): 37.9
Mean src-site R= 9.1 km; M= 6.92; eps0= 1.25. Mean calculated for all sources.
Modal src-site R= 5.8 km; M= 6.58; eps0= 1.38 from peak (R,M) bin
MODE R*= 5.7km; M*= 6.59; EPS.INTERVAL: 1 to 2 sigma % CONTRIB.= 3.092

Principal sources (faults, subduction, random seismicity having > 3% contribution)

Source Category:	% contr.	R(km)	M	epsilon0 (mean values).
California B-faults Char	24.98	8.0	7.05	1.11
California B-faults GR	8.90	8.3	6.75	1.41
CA Compr. crustal gridded	3.19	8.2	6.13	1.59

Individual fault hazard details if its contribution to mean hazard > 2%:

Fault ID	% contr.	Rcd(km)	M	epsilon0	Site-to-src azimuth(d)
Hollywood Char	2.95	6.4	6.60	1.47	-15.2
Newport-Inglewood, alt 1 Char	1.93	0.6	7.14	0.28	65.7
Newport-Inglewood, alt 2 Char	1.92	0.7	7.14	0.29	61.0
Elysian Park (Upper) Char	1.19	12.9	6.61	1.96	42.1
Puente Hills Char	0.65	10.3	7.06	1.31	68.3
Puente Hills (LA) Char	1.39	5.3	6.89	1.11	84.8
Newport Inglewood Connected alt	1.67	0.6	7.50	0.24	65.7
Newport Inglewood Connected alt	1.65	0.7	7.50	0.26	61.0
Santa Monica, alt 2 Char	1.33	4.2	6.69	1.15	-30.6
Santa Monica, alt 1 Char	1.09	4.9	6.50	1.37	-36.5
Santa Monica Connected alt 1 Cha	1.83	4.9	7.30	0.86	-36.5
Santa Monica Connected alt 2 Cha	1.69	4.3	7.35	0.78	-30.6
Hollywood, nominally GR	0.94	6.4	6.50	1.54	-15.2
Puente Hills (LA) GR	1.11	5.5	6.70	1.26	85.9
Santa Monica Connected alt 2 GR	0.99	6.3	7.02	1.11	-43.6

#*********End of deaggregation corresponding to Boore-Atkinson 2008 *********#

PSHA Deaggregation. %contributions. site: Culver_City long: 118.388 W., lat: 34.029 N.
Vs30(m/s)= 300.0 (some WUS atten. models use Site Class not Vs30).
NSHMP 2007-08 See USGS OFR 2008-1128. dM=0.2 below
Return period: 2475 yrs. Exceedance PGA =0.7534 g. Weight * Computed_Rate_Ex 0.502E-04
#Pr[at least one eq with median motion>=PGA in 50 yrs]=0.00000
#This deaggregation corresponds to Campbell-Bozorgnia 2008

DIST(KM)	MAG(MW)	ALL_EPS	EPSILON>2	1<EPS<2	0<EPS<1	-1<EPS<0	-2<EPS<-1	EPS<-2
6.9	5.05	0.068	0.068	0.000	0.000	0.000	0.000	0.000
7.0	5.20	0.186	0.186	0.000	0.000	0.000	0.000	0.000
7.2	5.40	0.267	0.254	0.013	0.000	0.000	0.000	0.000
12.1	5.42	0.009	0.009	0.000	0.000	0.000	0.000	0.000
7.4	5.60	0.294	0.252	0.042	0.000	0.000	0.000	0.000
12.4	5.60	0.021	0.021	0.000	0.000	0.000	0.000	0.000

7.5	5.80	0.260	0.216	0.044	0.000	0.000	0.000	0.000
12.6	5.80	0.027	0.027	0.000	0.000	0.000	0.000	0.000
7.2	6.01	0.295	0.261	0.034	0.000	0.000	0.000	0.000
13.2	6.01	0.037	0.037	0.000	0.000	0.000	0.000	0.000
7.1	6.20	0.405	0.362	0.043	0.000	0.000	0.000	0.000
13.6	6.24	0.078	0.076	0.002	0.000	0.000	0.000	0.000
6.9	6.42	0.590	0.503	0.087	0.000	0.000	0.000	0.000
13.2	6.46	0.504	0.428	0.075	0.000	0.000	0.000	0.000
22.6	6.41	0.015	0.015	0.000	0.000	0.000	0.000	0.000
5.6	6.61	1.405	0.466	0.843	0.097	0.000	0.000	0.000
13.1	6.56	0.698	0.596	0.102	0.000	0.000	0.000	0.000
23.2	6.58	0.016	0.016	0.000	0.000	0.000	0.000	0.000
5.4	6.79	1.285	0.389	0.784	0.112	0.000	0.000	0.000
12.5	6.77	0.445	0.339	0.106	0.000	0.000	0.000	0.000
24.3	6.76	0.042	0.042	0.000	0.000	0.000	0.000	0.000
4.7	6.99	1.207	0.319	0.724	0.165	0.000	0.000	0.000
12.4	7.01	0.443	0.246	0.187	0.010	0.000	0.000	0.000
25.3	6.99	0.028	0.028	0.000	0.000	0.000	0.000	0.000
4.4	7.22	1.534	0.307	0.850	0.377	0.000	0.000	0.000
11.8	7.16	0.340	0.145	0.176	0.018	0.000	0.000	0.000
26.1	7.16	0.013	0.013	0.000	0.000	0.000	0.000	0.000
3.9	7.37	1.366	0.345	0.761	0.259	0.000	0.000	0.000
14.1	7.33	0.070	0.036	0.032	0.002	0.000	0.000	0.000
27.4	7.34	0.007	0.007	0.000	0.000	0.000	0.000	0.000
3.8	7.54	0.330	0.071	0.184	0.075	0.000	0.000	0.000
0.6	7.71	0.018	0.008	0.010	0.000	0.000	0.000	0.000

Summary statistics for above PSHA PGA deaggregation, R=distance, e=epsilon:
Contribution from this GMPE(%): 12.3
Mean src-site R= 7.1 km; M= 6.74; eps0= 1.42. Mean calculated for all sources.
Modal src-site R= 4.4 km; M= 7.22; eps0= 0.92 from peak (R,M) bin
MODE R*= 4.4km; M*= 7.22; EPS.INTERVAL: 1 to 2 sigma % CONTRIB.= 0.850

Principal sources (faults, subduction, random seismicity having > 3% contribution)

Source Category:	% contr.	R(km)	M	epsilon0 (mean values).
California B-faults Char	6.42	6.9	7.05	1.33
California B-faults GR	3.11	7.1	6.78	1.37

Individual fault hazard details if its contribution to mean hazard > 2%:

Fault ID	% contr.	Rcd(km)	M	epsilon0	Site-to-src azimuth(d)
Hollywood Char	0.11	6.4	6.59	2.42	-15.2
Newport-Inglewood, alt 1 Char	0.16	0.6	7.15	1.52	65.7
Newport-Inglewood, alt 2 Char	0.15	0.7	7.15	1.55	61.0
Elysian Park (Upper) Char	0.92	12.9	6.59	2.02	42.1
Puente Hills Char	0.48	10.3	7.06	1.35	68.3
Puente Hills (LA) Char	1.45	5.3	6.88	1.04	84.8
Newport Inglewood Connected alt	0.13	0.6	7.50	1.51	65.7
Newport Inglewood Connected alt	0.13	0.7	7.50	1.53	61.0
Santa Monica, alt 2 Char	0.08	4.2	6.68	2.19	-30.6
Santa Monica, alt 1 Char	0.07	4.9	6.49	2.31	-36.5
Santa Monica Connected alt 1 Cha	0.90	4.9	7.30	1.13	-36.5
Santa Monica Connected alt 2 Cha	1.41	4.3	7.34	0.78	-30.6
Hollywood, nominally GR	0.04	6.4	6.50	2.43	-15.2
Puente Hills (LA) GR	1.28	5.5	6.69	1.14	85.9
Santa Monica Connected alt 2 GR	0.83	5.6	7.00	1.06	-43.6

#*********End of deaggregation corresponding to Campbell-Bozorgnia 2008 *********#

PSHA Deaggregation. %contributions. site: Culver_City long: 118.388 W., lat: 34.029 N.
Vs30(m/s)= 300.0 (some WUS atten. models use Site Class not Vs30).
NSHMP 2007-08 See USGS OFR 2008-1128. dM=0.2 below
Return period: 2475 yrs. Exceedance PGA =0.7534 g. Weight * Computed_Rate_Ex 0.203E-03
#Pr[at least one eq with median motion>=PGA in 50 yrs]=0.00259

#This deaggregation corresponds to Chiou-Youngs 2008

DIST(KM)	MAG(MW)	ALL_EPS	EPSILON>2	1<EPS<2	0<EPS<1	-1<EPS<0	-2<EPS<-1	EPS<-2
7.3	5.05	0.512	0.447	0.065	0.000	0.000	0.000	0.000
12.2	5.05	0.038	0.038	0.000	0.000	0.000	0.000	0.000
7.4	5.20	1.031	0.870	0.161	0.000	0.000	0.000	0.000
12.4	5.20	0.106	0.106	0.000	0.000	0.000	0.000	0.000
7.5	5.40	0.990	0.815	0.175	0.000	0.000	0.000	0.000
12.8	5.40	0.145	0.145	0.000	0.000	0.000	0.000	0.000
7.6	5.60	0.915	0.678	0.236	0.000	0.000	0.000	0.000
13.1	5.60	0.174	0.174	0.000	0.000	0.000	0.000	0.000
7.6	5.80	0.819	0.521	0.298	0.000	0.000	0.000	0.000
13.4	5.80	0.196	0.196	0.000	0.000	0.000	0.000	0.000
7.3	6.01	1.010	0.600	0.410	0.000	0.000	0.000	0.000
14.0	6.01	0.235	0.235	0.000	0.000	0.000	0.000	0.000
6.9	6.22	1.638	0.824	0.812	0.003	0.000	0.000	0.000
14.2	6.22	0.329	0.314	0.015	0.000	0.000	0.000	0.000
6.2	6.44	4.759	1.560	3.120	0.079	0.000	0.000	0.000
13.6	6.45	1.354	1.140	0.214	0.000	0.000	0.000	0.000
22.6	6.41	0.063	0.063	0.000	0.000	0.000	0.000	0.000
5.8	6.60	7.419	1.881	5.013	0.524	0.000	0.000	0.000
13.4	6.58	1.850	1.312	0.537	0.000	0.000	0.000	0.000
25.1	6.59	0.202	0.202	0.000	0.000	0.000	0.000	0.000
5.3	6.78	3.665	0.647	2.497	0.521	0.000	0.000	0.000
12.8	6.78	1.247	0.689	0.542	0.016	0.000	0.000	0.000
25.1	6.76	0.493	0.459	0.034	0.000	0.000	0.000	0.000
3.5	7.00	4.255	0.461	2.180	1.469	0.145	0.000	0.000
12.5	7.01	1.294	0.524	0.650	0.119	0.000	0.000	0.000
25.3	6.96	0.376	0.306	0.070	0.000	0.000	0.000	0.000
3.2	7.21	4.988	0.479	2.517	1.871	0.121	0.000	0.000
12.8	7.18	1.229	0.424	0.639	0.167	0.000	0.000	0.000
26.2	7.17	0.094	0.072	0.022	0.000	0.000	0.000	0.000
2.6	7.39	5.006	0.389	1.952	2.471	0.194	0.000	0.000
15.3	7.36	0.427	0.216	0.198	0.013	0.000	0.000	0.000
27.3	7.34	0.065	0.065	0.000	0.000	0.000	0.000	0.000
1.9	7.55	1.942	0.135	0.695	0.893	0.219	0.000	0.000
16.4	7.57	0.123	0.079	0.044	0.000	0.000	0.000	0.000
0.6	7.71	0.236	0.015	0.082	0.118	0.021	0.000	0.000
16.4	7.74	0.393	0.192	0.201	0.000	0.000	0.000	0.000
16.4	7.91	0.050	0.020	0.030	0.000	0.000	0.000	0.000

Summary statistics for above PSHA PGA deaggregation, R=distance, e=epsilon:
Contribution from this GMPE(%): 49.7
Mean src-site R= 7.0 km; M= 6.72; eps0= 1.06. Mean calculated for all sources.
Modal src-site R= 5.8 km; M= 6.60; eps0= 1.18 from peak (R,M) bin
MODE R*= 5.9km; M*= 6.60; EPS.INTERVAL: 1 to 2 sigma % CONTRIB.= 5.013

Principal sources (faults, subduction, random seismicity having > 3% contribution)

Source Category:	% contr.	R(km)	M	epsilon0 (mean values).
California B-faults Char	27.95	6.4	7.02	0.89
California B-faults GR	11.79	7.3	6.72	1.16
CA Compr. crustal gridded	9.97	8.4	5.87	1.39

Individual fault hazard details if its contribution to mean hazard > 2%:

Fault ID	% contr.	Rcd(km)	M	epsilon0	Site-to-src azimuth(d)
Hollywood Char	3.50	6.4	6.58	1.38	-15.2
Newport-Inglewood, alt 1 Char	1.93	0.6	7.14	0.23	65.7
Newport-Inglewood, alt 2 Char	1.91	0.7	7.14	0.24	61.0
Elysian Park (Upper) Char	1.93	12.9	6.59	1.77	42.1
Puente Hills Char	1.27	10.3	7.06	0.90	68.3
Puente Hills (LA) Char	2.38	5.3	6.88	0.76	84.8
Newport Inglewood Connected alt	1.71	0.6	7.50	0.17	65.7
Newport Inglewood Connected alt	1.69	0.7	7.50	0.18	61.0

Santa Monica, alt 2 Char	1.58	4.2	6.68	1.04	-30.6
Santa Monica, alt 1 Char	1.50	4.9	6.48	1.21	-36.5
Santa Monica Connected alt 1 Cha	2.66	4.9	7.31	0.56	-36.5
Santa Monica Connected alt 2 Cha	2.62	4.3	7.35	0.42	-30.6
Hollywood, nominally GR	1.19	6.4	6.50	1.41	-15.2
Puente Hills (LA) GR	2.01	5.5	6.69	0.90	85.9
Santa Monica Connected alt 2 GR	1.56	5.9	7.00	0.78	-43.6

#*********End of deaggregation corresponding to Chiou-Youngs 2008 *********#

******************** Southern California **