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December 14, 2020
File Number 22055

The John Buck Company
151 N. Franklin, Suite 300
Chicago, Illinois 60606

Attention: Jaqui Braver

Subject: Geotechnical Engineering Investigation
Proposed Office Development
9160 – 9174 West Sunset Boulevard, West Hollywood, California

Dear Ms. Braver:

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

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

Should you have any questions please contact this office.

Respectfully submitted,
GEOTECHNOLOGIES, INC.

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GEOTECHNICAL ENGINEERING INVESTIGATION
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9160 – 9174 WEST SUNSET BOULEVARD
WEST HOLLYWOOD, 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 the excavation of 3 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 exploratory excavation locations are shown on the enclosed Plot Plan. The results of the exploration and the laboratory testing are presented in the Appendix of this report.

SITE CONDITIONS

The project site is located at 9160 - 9174 West Sunset Boulevard, in the City of West Hollywood, California. The site encompasses an area of approximately 18,500 square feet and is bounded by West Sunset Boulevard to the north, by an asphalt-paved parking lot to the east, by an alleyway and a commercial property owned by Southern California Edison to the south, and by Cory Avenue to the west. The site is shown relative to nearby topographic features in the enclosed Vicinity Map.

The site is currently developed with an automobile dealership comprising of a 2-story, at-grade building, an asphalt-paved parking lot, and an elevated stone ramp along the northeast corner of the site. Based on the available survey prepared by Christensen & Plouff Land Surveying (dated July 10, 2019), the project site descends towards the southwest with ground surface elevations ranging from a high elevation of approximately 104.0 feet near the northeast corner of the site to a low elevation of approximately 91.0 feet near the southwest corner of the site. This corresponds to approximately 13 feet of elevation change across the subject site.

Drainage across the site occurs by sheet-flow along the existing topographic contours towards the adjacent alleyway and city streets. Vegetation on the site is present within isolated planters and landscaped areas consisting of bushes, shrubs, and grass. The neighboring developments consist primarily of commercial and residential structures.

PROPOSED DEVELOPMENT

Information concerning the proposed development was furnished by the client. According to the entitlement plans, the project consists of a new 5-story office building development which will be constructed over 3 subterranean levels of parking garage. The lowest subterranean level will extend up to 36 feet below the proposed ground floor level, which corresponds to the highest ground surface elevation of the site. Based on the available survey plan prepared by Christensen & Plouff Land Surveying (dated July 10, 2019), the proposed finished floor elevation of the lowest subterranean level will have an approximate elevation of 68.0 feet.

The proposed structure will provide retail establishments on the first floor with office units on the upper four floors. Additional improvements including outdoor recreational areas and landscaping are anticipated as part of the proposed development.

Column loads are estimated to be between 800 and 1,000 kips. Wall loads are estimated to be between 10 and 20 kips per lineal foot. Grading is anticipated to consist of excavations on the order 35 to 40 feet in depth for the proposed subterranean levels and foundation elements. The enclosed Plot Plan shows the proposed development site and its location relative to surrounding structures.

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

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was explored between November 2, 2020, and November 4, 2020, by excavating three exploratory borings. The exploratory borings were excavated to depths of 50 to 80 feet below the existing site grade with the aid of a drilling machine using 8-inch diameter hollow-stem augers. The exploration locations are shown on the enclosed Plot Plan and the geologic materials encountered are logged on the A-Plates.

The locations of the borings were determined by measurements relative to hardscape features onsite. The elevations of the borings were determined by interpolation of the topographic data shown on the site survey provided by the client. The boring locations and elevations and should be considered accurate only to the degree implied by the method used.

Geologic Materials

The geologic materials underlying the subject site consist of fill and older alluvial fan deposits. These materials which were encountered during exploration are described below.

Fill

The existing fill consists of silty sands which are light brown to brown in color, slightly moist to moist, medium dense, and fine to coarse grained with varying amounts of clays, gravels, and debris fragments intermixed. Fill was encountered in all of the borings to depths ranging from 3 to 7.5 feet below the existing grade.

Older Alluvial Fan

The fill is underlain by older alluvial fan deposits comprising of stratified layers of sandy clays and clayey to silty sands. The native soils range from brown, dark reddish and bluish gray with varying degrees of mottling in color, moist to wet, medium dense to very dense, stiff to very stiff, and fine to coarse grained with occasional gravels intermixed.

The distribution of geologic materials in the vicinity of the site is shown on the enclosed Local Geologic Map (Dibblee, 1991). More detailed descriptions of the geologic materials encountered may be obtained from individual logs of the subsurface excavations enclosed in the Appendix of this report.

Groundwater

Groundwater was encountered during exploration at depths of 50 to 53 feet below the existing site grade. Review of the Seismic Hazard Zone Report for the Beverly Hills 7.5-Minute Quadrangle (CDMG 1998, 2005) indicates that the historically highest groundwater level is on the order of 29 feet below the ground surface.

For design purposes, it is recommended that the highest site elevation (which corresponds to the proposed First Floor Level of the project) be utilized as reference for determining the historically highest groundwater elevation. According to the available survey by Christensen & Plouff, the highest site elevation is located at the northeast corner of the project site, and is recorded at approximately 104.0 feet. Therefore, a historically highest groundwater elevation of 75.0 feet may be utilized for the proposed building design. 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.

Caving

Caving could not be directly observed during exploration due to the type of excavation equipment utilized. Based on the experience of this firm, large diameter excavations, excavations that encounter granular, cohesionless soils and excavations below the groundwater table will most likely experience caving.

SEISMIC EVALUATION

REGIONAL GEOLOGIC SETTING

The subject property is located in the Transverse Ranges Geomorphic Province. The Transverse Ranges are characterized by roughly east-west trending mountains and the northern and southern boundaries are formed by reverse fault scarps. The convergent deformational features of the Transverse Ranges are a result of north-south shortening due to plate tectonics. This has resulted in local folding and uplift of the mountains along with the propagation of thrust faults (including blind thrusts). The intervening valleys have been filled with sediments derived from the bordering mountains.

REGIONAL FAULTING

Based on criteria established by the California Division of Mines and Geology (CDMG) now called California Geologic Survey (CGS), faults may be categorized as active, potentially active, or inactive. Active faults are those which show evidence of surface displacement within the last 11,000 years (Holocene-age). Potentially-active faults are those that show evidence of most recent surface displacement within the last 1.6 million years (Quaternary-age). Faults showing no evidence of surface displacement within the last 1.6 million years are considered inactive for most purposes, with the exception of design of some critical structures.

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

Hollywood Fault

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

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

SEISMIC HAZARDS AND DESIGN CONSIDERATIONS

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

Surface Rupture

In 1972, the Alquist-Priolo Special Studies Zones Act (now known as the Alquist-Priolo Earthquake Fault Zoning Act) was passed into law. The Act defines “active” and “potentially active” faults utilizing the same aging criteria as that used by California Geological Survey (CGS). However, established state policy has been to zone only those faults which have direct evidence of movement within the last 11,000 years. It is this recency of fault movement that the CGS considers as a characteristic for faults that have a relatively high potential for ground rupture in the future.

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

Based on review of the Earthquake Zones of Required Investigation for the Beverly Hills Quadrangle, which was recently updated by CGS in 2018, the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. The southern boundary of the project site is located approximately 120 feet north of the northern edge of the Hollywood Fault Zone.

As part of the City of West Hollywood Seismic Safety Element, the City of West Hollywood has also identified fault zones requiring additional fault studies. These zones were created based on geologic evidence of active fault movement (within the last 11,000 years) along the Hollywood Fault. Based on review of the City of West Hollywood Fault Location and Precaution Zone Map (City of West Hollywood Seismic Safety Element, 2010), the site is not located within a Fault Precaution Zone (FP-1 or FP-2). A copy of the map showing the location of the site relative to the Fault Precaution Zones has been enclosed in the Appendix.

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. When the saturated sediments are shaken, a sudden increase in pore water pressure causes the soils to lose strength and behave as a liquid. Liquefaction-related effects include loss of bearing strength, amplified ground oscillations, lateral spreading, and flow failures.

According to the State of California Earthquake Zones of Required Investigation Map for the Beverly Hills Quadrangle (CGS, 2018), the site is not located within a potentially liquefiable area. This determination is based on groundwater depth records, soil type and distance to a fault capable of producing a substantial earthquake.

A site-specific liquefaction analysis was performed following Guidelines for Evaluating and Mitigating Seismic Hazards in California Special Publication 117A (CGS, 2008). In addition, recommendations provided in EERI Monograph (MNO-12) (Idriss and Boulanger, 2008) were also incorporated into the analysis.

Groundwater was encountered during exploration at depths between 50 and 53 feet below the existing site grade. According to the Seismic Hazard Zone Report of the Beverly Hills 7.5-Minute

Quadrangle (CDMG, 1998, Revised 2005), the historic high groundwater level for the subject site is approximately 29 feet below the ground surface. The historically highest groundwater level was conservatively utilized for the enclosed liquefaction analysis.

Section 11.8.3 of ASCE 7-16 indicates that the potential for liquefaction shall be evaluated utilizing a site-modified peak ground acceleration (PGA_M) corresponding to the Maximum Considered Earthquake (MCE_G). The OSHPD Seismic Design Maps Tool yielded a site modified peak ground acceleration (PGA_M) of 1.00g. The USGS Probabilistic Seismic Hazard Analysis Deaggregation program (USGS, 2014) was utilized to determine the magnitude of the Maximum Considered Earthquake (MCE_G). The deaggregation program yielded a modal magnitude (M_w) of 6.9 for the site. Therefore, the liquefaction potential evaluation was performed using a magnitude 6.9 earthquake and a peak ground acceleration of 0.934g.

The enclosed liquefaction analysis is based upon the Standard Penetration Test (SPT) performed within Boring B1. SPT data were collected at 5-foot intervals. Samples of the collected materials were conveyed to the laboratory for testing and analysis. Fines content, as defined by percentage passing the #200 sieve, were utilized for the fines correction factor in computing the corrected blowcount. The results of these laboratory tests are presented in Plate E.

Based on criteria set forth in CGS Special Publication SP117A, a factor of safety against the occurrence of liquefaction greater than about 1.3 can be considered an acceptable level of risk where high-quality, site-specific penetration resistance and geotechnical laboratory data is collected. The results of the site-specific liquefaction analysis indicate that the underlying soils would not be prone to liquefaction during the seismic design ground motions.

Dynamic Dry Settlement

Seismically-induced settlement of dry to 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 office building development is anticipated to be underlain by 3 subterranean levels extending up to 36 feet below the existing ground surface. The alluvial soil layers above the current groundwater level and underlying the proposed structure comprised of dense to very dense clayey sands and very stiff sandy clays. Due to the dense nature and high clay content of the underlying soils, the effects of seismically-induced dry settlements on the proposed structure will 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. No major water-retaining structures are located immediately up gradient from the project site. Review of the County of Los Angeles Flood and Inundation Hazards Map, Leighton (1990), indicates that the site does not lie within the mapped inundation boundaries of a breached up-gradient reservoir.

Landsliding

The County of Los Angeles Landslide Inventory Map (Leighton, 1990) indicates that the site is not located in an area highly susceptible to landslides. Additionally, the Earthquake Zones of Required Investigation of the Beverly Hills Quadrangle shows that the site is not located within an Earthquake-Induced Landslide Zone (CGS, 2014). Based on these considerations, the potential of landslides negatively affecting the proposed development is considered to be low.

CONCLUSIONS AND RECOMMENDATIONS

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

The geologic materials underlying the site consist of fill and older alluvial fan deposits. Fill was encountered in all of the borings to depths ranging from 3 to 7.5 feet below the existing grade. The fill is underlain by older alluvial fan deposits comprising of stratified layers of sandy clays and clayey to silty sands.

The existing fill materials are considered to be unsuitable for the support of new foundations, floor slabs, or additional fill. The proposed office building development will be constructed over 3 subterranean parking levels. The lowest subterranean parking level will extend 36 feet below the existing grade, with an approximate lowest finished floor elevation of 68.0 feet. The planned excavations are expected to remove the existing fill materials and expose the underlying native soils. The proposed structure may be supported on a mat foundation bearing in the underlying native soils.

Groundwater was encountered during exploration at depths of 50 to 53 feet below the existing grade. Based on data provided by the Seismic Hazard Zone Report of the Beverly Hills Quadrangle, the historically highest groundwater level for the site is approximately 29 feet below the ground surface. For design purposes, it is recommended that the highest site elevation (which corresponds to the proposed First Floor Level of the project) be utilized as reference for determining the historically highest groundwater elevation. According to the available survey by Christensen & Plouff, the highest site elevation is located at the northeast corner of the project site, and is recorded at approximately 104.0 feet. Therefore, a historically highest groundwater elevation of 75.0 feet may be utilized for the proposed building design.

It is recommended that the proposed structure be designed for hydrostatic pressure based on the historically highest groundwater level such that the permanent dewatering system below the base of structure and the code required wall subdrain system may be eliminated. The basement walls shall be designed for hydrostatic pressure based on the existing ground surface. In addition, the project structural engineer shall evaluate and design the proposed foundations for hydrostatic uplift pressures based upon the historically highest groundwater elevation of 75.0 feet. Hydrostatic uplift pressure would be equivalent to 62.4(H), where “H” is the elevation difference between the bottom of the foundations and the historically highest groundwater level. In any case, it is recommended that the proposed subterranean structure be waterproofed.

Excavations for the proposed subterranean levels will require shoring measures due to the anticipated depths and proximity of adjacent private property lines and public right-of-ways in order to provide stable working conditions. Shoring and excavation recommendations are provided in the “Temporary Excavations” and “Shoring” sections of this report.

Foundations for small outlying structures, such as property line walls, planters, trash enclosures, and canopies, which will not be rigidly connected to the proposed office building may be supported on conventional foundations bearing in properly compacted fill or undisturbed native soils.

The validity of the conclusions and design recommendations presented herein is dependent upon review of the geotechnical aspects of the proposed construction by this firm. The subsurface conditions described herein have been projected from borings on the site as indicated and should in no way be construed to reflect any variations which may occur between these borings or which may result from changes in subsurface conditions. Any changes in the design or location of any structure, as outlined in this report, should be reviewed by this office. The recommendations contained herein should not be considered valid until reviewed and modified or reaffirmed subsequent to such review.

SEISMIC DESIGN CONSIDERATIONS

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 ASCE 7-16 standard. This information and the site coordinates were input into the OSHPD Seismic Design Maps Tool to calculate the ground motions associated with the risk-targeted maximum considered earthquake (MCE_R).

2019 CALIFORNIA BUILDING CODE SEISMIC PARAMETERS	
Site Class	D
Mapped Spectral Acceleration at Short Periods (S_S)	2.127g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	2.127g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.418g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.761g
Site Coefficient (F_v)	1.7*
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	1.294g*
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.863g*

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

FILL SOILS

The depth of fill encountered onsite during exploration ranged from approximately 3 to 7.5 feet below the existing grade. The existing fill materials are not suitable for the support of new foundations, floor slabs or additional fill. This material and any fill generated during demolition will be removed during the excavation of the proposed subterranean levels and wasted from the site.

EXPANSIVE SOILS

The surficial geologic materials are in the very low expansion range. The Expansion Index was measured to range from 7 to 10 for representative bulk samples of the upper 5 feet of the site soils remolded to 90 percent of the laboratory maximum dry density. Recommended reinforcing is provided in the "Foundation Design" and "Slabs-on-Grade" sections of this report.

SOIL CORROSION POTENTIAL

Soil corrosivity testing was performed by HDR Engineering, Inc. on representative samples of onsite soils from various depths. The results indicate that the electrical resistivities of the soils are in the mildly to moderately corrosive category with as-received moisture. Saturated soil resistivities yielded values corresponding to the moderately corrosive category. Soil pH values ranged from 7.8 to 7.9 making it neutral to mildly alkaline. These values do not significantly increase soil corrosivity. The soluble salt content was low. Chloride, nitrate, and sulfate were also found in low concentrations. Ammonium was not detected. Overall, the soil is classified as moderately corrosive to ferrous metals and sulfate attack on concrete is not applicable. Therefore, there are no restrictions on the cement types which may be utilized for concrete foundations in contact with the site soils.

Detailed results, discussion of results, and recommended mitigating measures are provided within the report by HDR Engineering, Inc. enclosed in the Appendix. Any questions regarding the results of the soil corrosion report should be addressed to HDR Engineering, Inc.

HYDROCONSOLIDATION

Hydroconsolidation is a phenomenon wherein soils lose volume when they are saturated with water. This can result in settlement of structures bearing thereon. The hydroconsolidation potential of the site soils was considered by the evaluation of seven consolidation tests. The test samples were of native soils.

The samples showed very minor degree of hydroconsolidation strains, on the order of 0 to 0.1 percent. The property owner shall maintain proper drainage of the subject site throughout the life of the structure. All utility and irrigation lines and drainage devices should be checked periodically and maintained. In addition, landscape irrigation should be properly controlled, in order to reduce the amount of water infiltration into the underlying soils, which provide support to the proposed structure. The Site Drainage section below should be followed and implemented into the final construction documents.

GRADING GUIDELINES

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

Site Preparation

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

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

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

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

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

Compaction

All fill, consisting of soil approved by a representative of this firm, should be mechanically compacted in layers not more than 8 inches thick. All fill shall be compacted to at least 90 percent of the maximum laboratory dry density for the materials used.

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 six inches in maximum dimension shall not be used in the fill. The maximum dry density shall be determined by the laboratory operated by Geotechnologies, Inc. using test method described in the most recent revision of ASTM D 1557.

Field observation and testing shall be performed by a representative of the geotechnical engineer during grading to assist the contractor in obtaining the required degree of compaction and the proper moisture content. Where compaction is less than required, additional compactive effort shall be made with adjustment of the moisture content, as necessary, until a minimum of 90 percent is obtained.

Acceptable Materials

The excavated onsite materials are considered satisfactory for reuse in the controlled fills as long as any debris and/or organic matter is removed. Any imported materials shall be observed and tested by the representative of the geotechnical engineer prior to use in fill areas. Imported materials should contain sufficient fines so as to be relatively impermeable and result in a stable subgrade when compacted. Any required import materials should consist of geologic materials with an expansion index of less than 50. The water-soluble sulfate content of the import materials should be less than 0.1% percentage by weight.

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

Utility Trench Backfill

Utility trenches should be backfilled with controlled fill. The utility should be bedded with clean sands at least one foot over the crown. The remainder of the backfill may be onsite soil compacted to 90 percent of the laboratory maximum density. Utility trench backfill should be tested by representatives of this firm in accordance with the most recent revision of ASTM D-1557.

Shrinkage

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

Weather Related Grading Considerations

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

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

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

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

Geotechnical Observations and Testing During Grading

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

FOUNDATION DESIGN

Mat Foundations

The proposed development may be supported on a mat foundation bearing in the underlying dense native soils below the lowest subterranean level. Preliminarily, it is anticipated that the average bearing pressure across the mat foundation will be between 2,000 and 4,000 pounds per square foot. Given the size of the proposed mat foundation, the average bearing pressure of 4,000 pounds per square foot is well below the allowable bearing pressures, with factor of safety well exceeding 3. For design purposes, an average bearing pressure (contact pressure) of 4,000 pounds per square foot, with locally higher pressures up to 8,000 pounds per square foot may be utilized in the mat foundation design. The mat foundation may be designed utilizing a modulus of subgrade reaction of 300 pounds per cubic inch. This value is a unit value for use with a one-foot square footing. The modulus should be reduced in accordance with the following equation when used with larger foundations.

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

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

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

Hydrostatic Uplift Pressure on Mat Foundation

Based on data provided by the Seismic Hazard Zone Report of the Beverly Hills Quadrangle, the historically highest groundwater level for the site is approximately 29 feet below the ground surface. It is recommended that the historically highest groundwater elevation of 75.0 feet be utilized for design purposes.

The proposed mat foundation for the subterranean structure shall be designed to withstand the potential hydrostatic uplift pressure based on the historically highest groundwater level. The proposed mat foundation uplift pressure to be used in design would be 62.4(H) psf, where "H" is the depth to the bottom of footing from the recommended historically highest groundwater elevation of 75.0 feet.

Miscellaneous Foundations

Foundations for small outlying structures, such as property line walls, planters, or trash enclosures which will not be rigidly connected to the proposed residential complex may be supported on conventional foundations bearing in undisturbed native soils or properly compacted fill. These footings may be designed for a bearing value of 2,000 pounds per square foot, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material. No bearing value increases are recommended. All continuous foundations should be reinforced with a minimum of four #4 steel bars. Two should be placed near the top of the foundation, and two should be placed near the bottom.

Lateral Design

Resistance to lateral loading may be provided by friction acting at the base of foundations and by passive earth pressure. An allowable coefficient of friction of 0.35 may be used with the dead load forces. Where a waterproofing membrane is used below the base of the structure, an allowable coefficient of friction of 0.15 may be utilized.

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 3,000 pounds per square foot. The passive and friction components may be combined for lateral resistance without reduction. A one-third increase in the passive value may be used for short duration loading such as wind or seismic forces.

Foundation Settlement

Settlement of the foundation system is expected to occur on initial application of loading. The maximum settlement is expected to be 1 inch and occur below the heaviest loaded columns. Differential settlement is not expected to exceed 0.5 inches within 30 feet.

Foundation Observations

It is critical that all foundation excavations are observed by a representative of this firm to verify penetration into the recommended bearing materials. The observation should be performed prior to the placement of reinforcement. Foundations should be deepened to extend into satisfactory geologic materials, if necessary. Foundation excavations should be cleaned of all loose soils prior to placing steel and concrete. Any required foundation backfill should be mechanically compacted, flooding is not permitted.

RETAINING WALL DESIGN

The proposed development will be constructed over 3 subterranean levels, extending up to 36 feet below the existing site grade. Due to the historically highest groundwater level, it is recommended that the proposed subterranean level be designed for full hydrostatic pressure. Restrained retaining walls may be designed utilizing a triangular distribution of at-rest earth pressure. Retaining walls may be designed utilizing the following table:

Height of Retaining Wall (feet)	Restrained Retaining Wall Triangular Distribution of At-Rest Earth Pressure With Hydrostatic Pressure (pcf)
40 feet	95 pcf

The lateral earth pressures recommended above for retaining walls assume full hydrostatic design based on the existing ground surface, such that the code required wall subdrain may be eliminated.

Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures. Surcharge from adjacent buildings may be determined following the procedures presented in NAVFAC 7.02.

The upper ten feet of the retaining wall adjacent to streets, driveways or parking areas should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the walls due to normal street traffic. If the traffic is kept back at least ten feet from the retaining walls, the traffic surcharge may be neglected. Foundations may be designed using the allowable bearing capacities, friction, and passive earth pressure found in the "Foundation Design" section above.

Dynamic (Seismic) Earth Pressure

Section 1803.5.12 of the 2019 CBC states that dynamic seismic lateral earth pressures on foundation walls and retaining walls are required, when supporting more than 6 feet of backfill height due to design earthquake ground motions.

In accordance with the City of West Hollywood requirements, a free field ground acceleration equivalent to $S_{DS}/2.5$ shall be utilized in the seismic wall pressure. This corresponds to a ground acceleration of 0.57g. The procedure prescribed by Mikola and Sitar (2013), was utilized to determine the mean seismic wall pressure. A triangular pressure distribution should be utilized for the additional seismic loads, with an equivalent fluid pressure of 42 pounds per cubic foot. The point of application should be at 1/3(H) from the base of the retaining wall, where H is the height of the retaining wall.

When using the load combination equations in the Building Code, the seismic earth pressure shall be combined with the lateral at-rest earth pressure for design of the non-yielding basement walls under seismic loading condition, as required by the City of West Hollywood.

Waterproofing

Moisture effecting retaining walls is one of the most common post construction complaints. Poorly applied or omitted waterproofing can lead to efflorescence or standing water inside the building.

Efflorescence is a process in which a powdery substance is produced on the surface of the concrete by the evaporation of water. The white powder usually consists of soluble salts such as gypsum, calcite, or common salt. Efflorescence is common to retaining walls and does not affect their strength or integrity.

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

Retaining Wall Backfill

Any required backfill should be mechanically compacted in layers not more than 8 inches thick, to at least 90 percent of the maximum density obtainable by the most recent revision of ASTM D 1557. Flooding should not be permitted. Proper compaction of the backfill will be necessary to reduce settlement of overlying walks and paving. Some settlement of required backfill should be anticipated, and any utilities supported therein should be designed to accept differential settlement, particularly at the points of entry to the structure.

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TEMPORARY EXCAVATIONS

It is anticipated that excavations on the order of 40 feet in vertical height will be required for the proposed subterranean levels and foundation elements. The excavations are expected to expose fill and native soils, which are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic or structures. Excavations which will be surcharged by adjacent traffic or structures or undermine private property lines and/or public right-of-ways should be shored.

Where sufficient space is available, temporary unsurcharged embankments could be sloped back without shoring. Excavations over 5 feet in height should may be excavated at a uniform 1:1 (h:v) slope gradient in its entirety to a maximum height of 15 feet. A uniform sloped excavation does not have a vertical component.

Where sloped embankments are utilized, the tops of the slopes should be barricaded to prevent vehicles and storage loads within seven feet of the tops of the slopes. If the temporary construction embankments are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. The soils exposed in the cut slopes should be inspected during excavation by personnel from this office so that modifications of the slopes can be made if variations in the soil conditions occur.

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

<u>Excavation Observations</u>

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

SHORING DESIGN

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

One method of shoring would consist of steel soldier piles, placed in drilled holes and backfilled with concrete. Due to the depth of the proposed excavation, it is anticipated that the soldier piles will need to be designed as laterally braced utilizing drilled tie-back anchors or raker braces.

Soldier Piles

Drilled cast-in-place soldier piles should be placed no closer than 3 diameters on center. The minimum diameter of the piles is 24 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the earth materials. For design purposes, an allowable passive value for the earth materials below the bottom plane of excavation may be assumed to be 400 pounds per square foot per foot, with increases per additional foot up to a maximum allowable pressure of 4,000 pounds per square foot. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed earth materials.

The frictional resistance between the soldier piles and retained earth material may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.3 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 500 pounds per square foot. The minimum depth of embedment for shoring piles is 5 feet below the bottom of the footing excavation, or 7 feet below the bottom of excavated plane, whichever is deeper.

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

Piles placed below the water level will require the use of a tremie to place the concrete into the bottom of the hole. A tremie shall consist of a water-tight tube having a diameter of not less than 10 inches with a hopper at the top. The tube shall be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie shall be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end shall be closed at the start of the work to prevent water entering the tube and shall be entirely sealed at all times, except when the concrete is being placed. The tremie tube shall be kept full of concrete. The flow shall be continuous until the work is completed and the resulting concrete seal shall be monolithic and homogeneous. The tip of the tremie tube shall always be kept about five feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

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

Lagging

Soldier piles and anchors should be designed for the full anticipated pressures. Lagging will be required throughout the entire depth of the excavation. Due to arching in the geologic materials, the pressure on the lagging will be less. It is recommended that the lagging should be designed for the full design pressure but be limited to a maximum of 400 pounds per square foot for earth pressure. In addition to the recommended earth pressure, additional surcharge loads from adjacent buildings and vehicular traffic, where appropriate, shall be incorporated into the design of the lagging system. It is recommended that a representative of this firm observe the installation of lagging to insure uniform support of the excavated embankment.

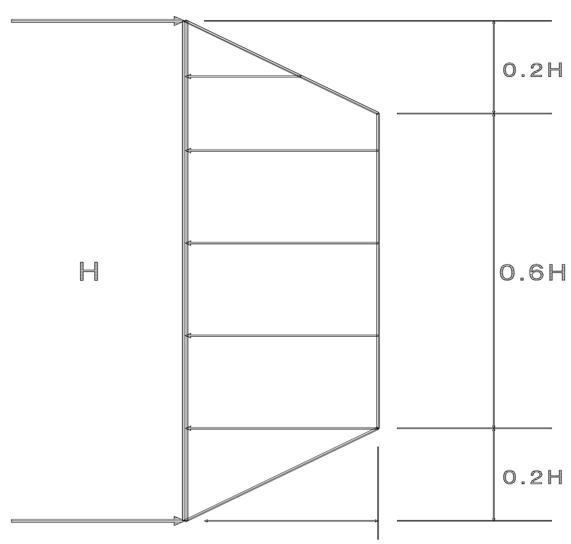
Lateral Pressures

A trapezoidal distribution of lateral earth pressure would be appropriate where shoring is to be laterally restrained by way of bracing or tie backs. The design of trapezoidal distribution of pressure is shown in the diagram below. Equivalent fluid pressures for the design of restrained shoring are presented in the following table:

Height of Shoring (feet)	Restrained Shoring System Lateral Earth Pressure (psf)* Trapezoidal Distribution of Pressure
40 feet	25H psf

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

TRAPEZOIDAL DISTRIBUTION OF PRESSURE



Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination. Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures. Surcharge from adjacent buildings may be determined following the procedures presented in NAVFAC 7.02.

The upper ten feet of the retaining wall adjacent to streets, driveways or parking areas should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the walls due to normal street traffic. If the traffic is kept back at least ten feet from the retaining walls, the traffic surcharge may be neglected. Foundations may be designed using the allowable bearing capacities, friction, and passive earth pressure found in the "Foundation Design" section above.

Tied-Back Anchors

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

Due to the granular nature of the upper strata, caving of the anchor shafts within the sandy deposits may occur. It is recommended that pressure grouted tieback anchors be utilized as part of the proposed shoring system such that any vids created as a result of drilling the anchor shafts would be filled by grouting. Pressure grouted anchor may be designed for a skin friction of 2,000 pounds per square foot. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads.

All tieback anchors shall be tested to a minimum of 150 percent of the design load. Testing shall be performed in accordance with PTI DC35.1-14, and with the City of Los Angeles Research Report 23835, "Requirements for Temporary Tieback Earth Anchors". 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 installation and testing of the anchors should be observed by a representative of this firm. Minor caving during drilling of the anchors should be anticipated.

Tieback anchors shall be detensioned upon engagement of the basement walls as required by the City of West Hollywood.

Anchor Installation

Tied-back anchors may be installed between 20 and 40 degrees below the horizontal. Caving of the anchor shafts, particularly within sand deposits, should be anticipated and the following provisions should be implemented in order to minimize such caving. The anchor shafts should be filled with concrete by pumping from the tip out, and the concrete should extend from the tip of the anchor to the active wedge. In order to minimize the chances of caving, it is recommended that the portion of the anchor shaft within the active wedge be backfilled with sand before testing the anchor. This portion of the shaft should be filled tightly and flush with the face of the excavation. The sand backfill should be placed by pumping; the sand may contain a small amount of cement to facilitate pumping.

Deflection

It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized that some deflection will occur. It is estimated that the deflection could be on the order of one inch at the top of the shored embankment. If greater deflection occurs during construction, additional bracing may be necessary to minimize settlement of adjacent buildings and utilities in adjacent street and alleys. If desired to reduce the deflection, a greater active pressure could be used in the shoring design. Where internal bracing is used, the rakers should be tightly wedged to minimize deflection. The proper installation of the raker braces and the wedging will be critical to the performance of the shoring.

Monitoring

Because of the depth of the excavation, some mean of monitoring the performance of the shoring system is suggested. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all soldier piles and the lateral movement along the entire lengths of selected soldier piles. Also, some means of periodically checking the load on selected anchors will be necessary, where applicable. Survey and monitoring reports shall be provided to this firm for review in a timely manner.

Some movement of the shored embankments should be anticipated as a result of the relatively deep excavation. It is recommended that photographs and surveys of the existing buildings on the adjacent properties be made during construction to record any movements for use in the event of a dispute.

Shoring Observations

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

SLABS ON GRADE

Outdoor Flatwork

Outdoor concrete flatwork should be a minimum of 4 inches in thickness, and should be reinforced with a minimum of #3 steel bars on 24-inch centers each way. Outdoor concrete flatwork should be cast over undisturbed natural geologic materials or properly compacted fill. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 90 percent of the maximum dry density.

Concrete Crack Control

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

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

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

PAVEMENTS

Prior to placing paving, the existing grade should be scarified to a minimum depth of 12 inches, moistened as required to obtain optimum moisture content, and recompacted to 90 percent of the maximum density as determined by the most recent revision of ASTM D 1557. The client should be aware that removal of all existing fill in the area of new paving is not required; however, pavement constructed over uncompacted fill or disturbed native soils will most likely have a shorter design life and increased maintenance costs. The following pavement sections are recommended based upon estimated traffic indices.

RECOMMENDED ASPHALT PAVING SECTIONS		
Vehicular Service – Traffic Index (TI)	Asphalt Pavement Thickness (Inches)	Thickness of Base Course (Inches)
Passenger Vehicles (TI = 4)	3	4
Moderate Trucks (TI = 6)	4	6
Heavy Truck (TI = 8)	6	8

A subgrade modulus of 100 pounds per cubic inch may be assumed for design of concrete paving. Concrete paving for support of passenger cars and moderate truck traffic shall be a minimum of 6 inches in thickness underlain by a minimum of 6 inches of compacted aggregate base material. Concrete paving for heavy truck traffic shall be a minimum of 7½ inches in thickness, and shall be underlain by 6 inches of aggregate base. For standard crack control, maximum expansion joint spacing of 15 feet should not be exceeded. Lesser spacing would provide greater crack control. Joints at curves and angle points are recommended.

Base materials which underlie the asphalt or concrete pavement should be compacted to a minimum of 95 percent of the laboratory determined maximum dry density (ASTM D1557). Base materials may consist of Crushed Aggregate Base which conforms with Section 200-2.2 of the most recent edition of “Standard Specifications for Public Works Construction” (Green Book). Crushed Miscellaneous Base is an acceptable substitute for an aggregate base which is addressed in Section 200-2.4.

SITE DRAINAGE

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

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

STORMWATER DISPOSAL

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

Due to the historically highest groundwater level and the depth of the proposed subterranean levels and foundation elements, it is the opinion of this firm that infiltration of stormwater is not feasible for the project site.

DESIGN REVIEW

Engineering of the proposed project should not begin until approval of the geotechnical report by the Building Official is obtained in writing. Significant changes in the geotechnical recommendations may result during the building department review process.

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

CONSTRUCTION MONITORING

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

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

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

EXCAVATION CHARACTERISTICS

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

CLOSURE AND LIMITATIONS

The purpose of this report is to aid in the design and completion of the described project. Implementation of the advice presented in this report is intended to reduce certain risks associated with construction projects. The professional opinions and geotechnical advice contained in this report are sought because of special skill in engineering and geology and were prepared in accordance with generally accepted geotechnical engineering practice. Geotechnologies, Inc. has a duty to exercise the ordinary skill and competence of members of the engineering profession. Those who hire Geotechnologies, Inc. are not justified in expecting infallibility, but can expect reasonable professional care and competence.

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

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

GEOTECHNICAL TESTING

Classification and Sampling

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

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

Moisture and Density Relationships

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

Direct Shear Testing

Shear tests are performed by the most recent revision of ASTM D 3080 with a strain controlled, direct shear machine manufactured by Soil Test, Inc. or a Direct Shear Apparatus manufactured by GeoMatic, Inc. The rate of deformation is approximately 0.005 inches per minute. Each sample is sheared under varying confining pressures in order to determine the Mohr-Coulomb shear strength parameters of the cohesion intercept and the angle of internal friction. Samples are generally tested in an artificially saturated condition. Depending upon the sample location and future site conditions, samples may be tested at field moisture content. The results are plotted on the "Shear Test Diagram," B-Plates.

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

Consolidation Testing

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

Expansion Index Testing

The expansion tests performed on the remolded samples are in accordance with the Expansion Index testing procedures, as described in the most recent revision of ASTM D4829. The soil sample is compacted into a metal ring at a saturation degree of 50 percent. The ring sample is then placed in a consolidometer, under a vertical confining pressure of 1 lbf/square inch and inundated with distilled water. The deformation of the specimen is recorded for a period of 24 hour or until the rate of deformation becomes less than 0.0002 inches/hour, whichever occurs first. The expansion index, EI, is determined by dividing the difference between final and initial height of the ring sample by the initial height, and multiplied by 1,000. Results are presented in Plate D of this report.

Laboratory Compaction Characteristics

The maximum dry unit weight and optimum moisture content of a soil are determined by use of the most recent revision of ASTM D 1557. A soil at a selected moisture content is placed in five layers into a mold of given dimensions, with each layer compacted by 25 blows of a 10-pound hammer dropped from a distance of 18 inches subjecting the soil to a total compactive effort of about 56,000 pounds per cubic foot. The resulting dry unit weight is determined. The procedure is repeated for a sufficient number of moisture contents to establish a relationship between the dry unit weight and the water content of the soil. The data when plotted represent a curvilinear relationship known as the compaction curve. The values of optimum moisture content and modified maximum dry unit weight are determined from the compaction curve. Results are presented in Plate D of this report.

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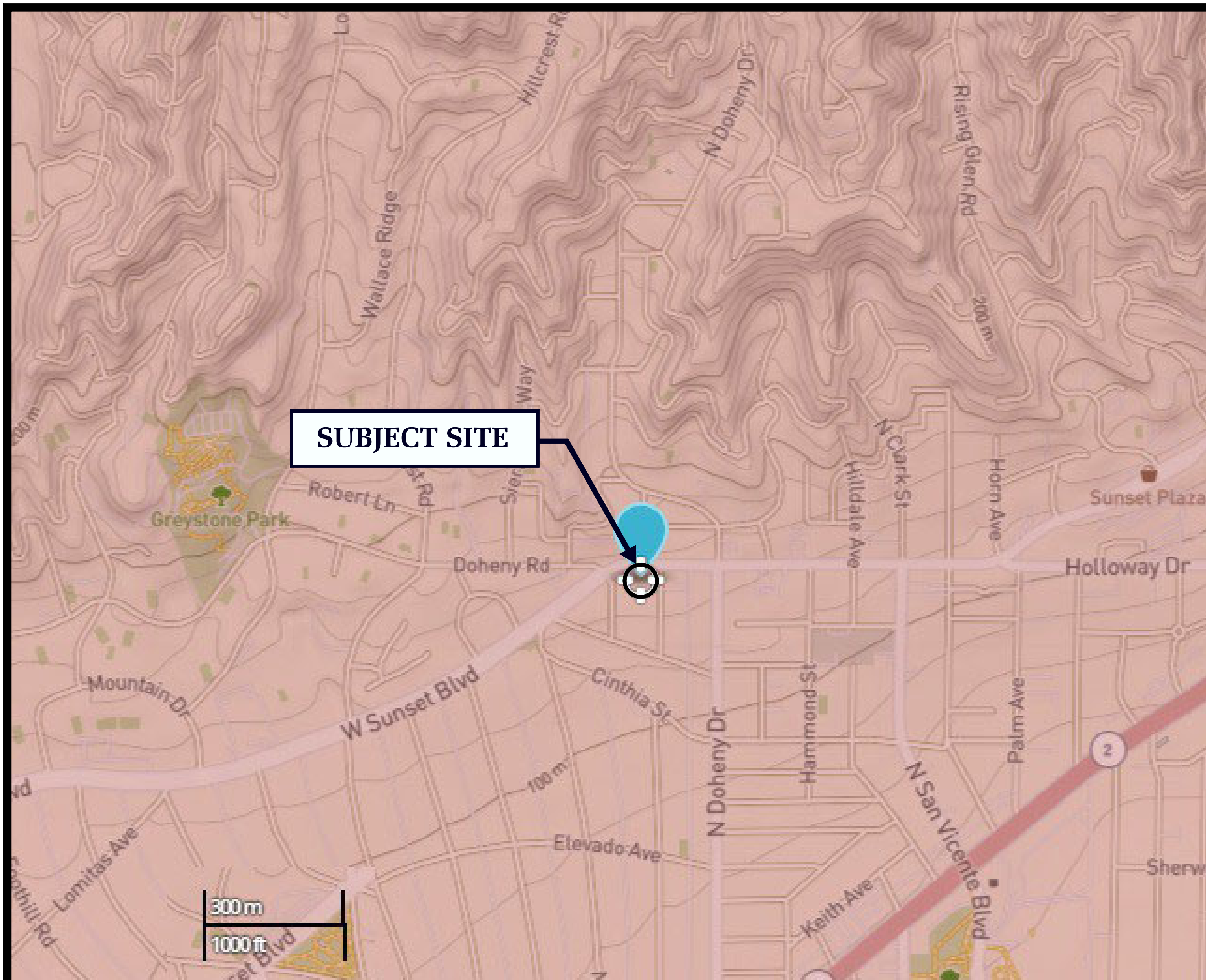

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VICINITY MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

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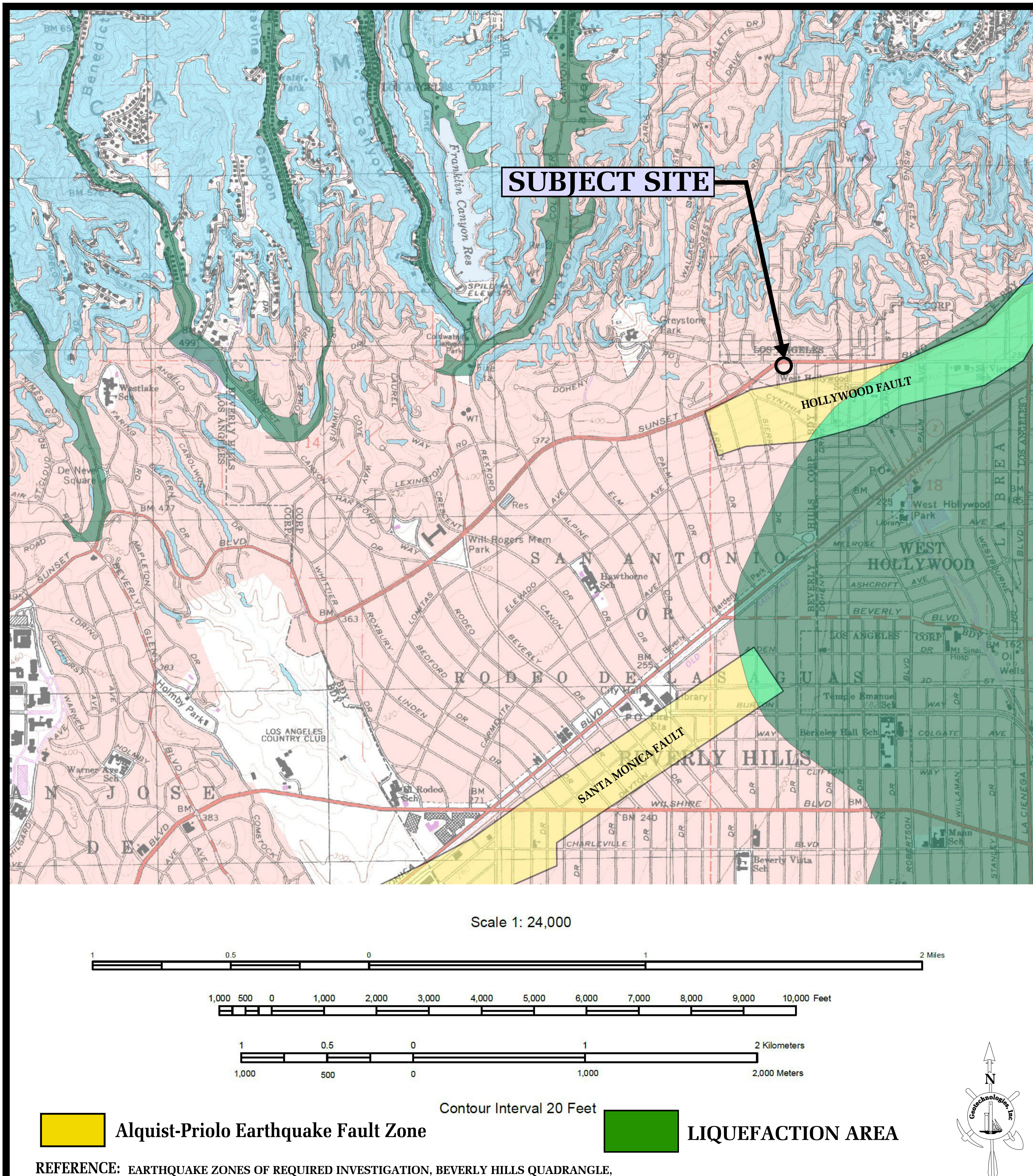
FILE No. 22055



REFERENCE: EARTHQUAKE ZONES OF REQUIRED INVESTIGATION, BEVERLY HILLS QUADRANGLE, CALIFORNIA GEOLOGICAL SURVEY, (CGS,2018)

EARTHQUAKE ZONES OF REQUIRED INVESTIGATION MAP

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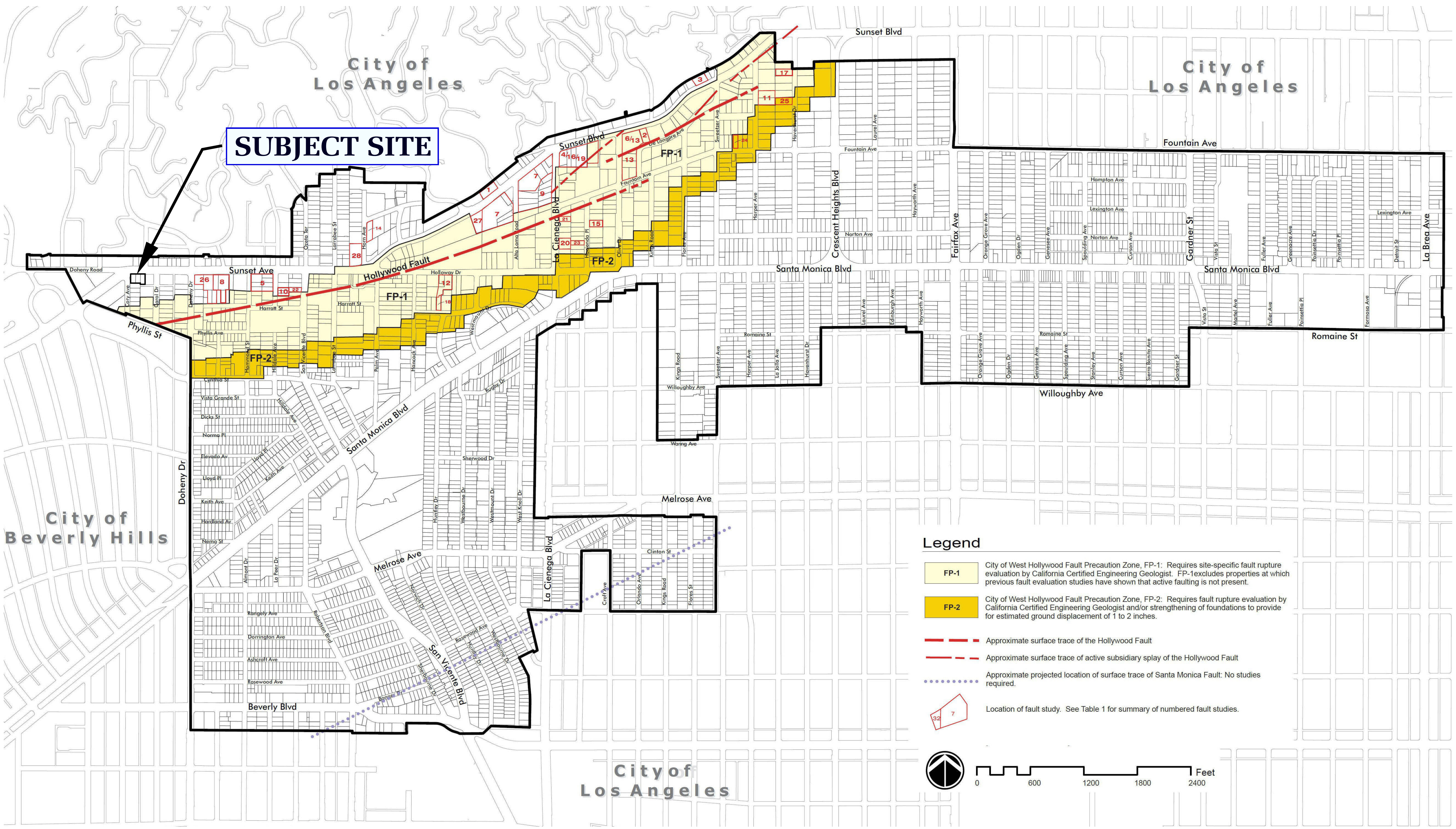
Consulting Geotechnical Engineers

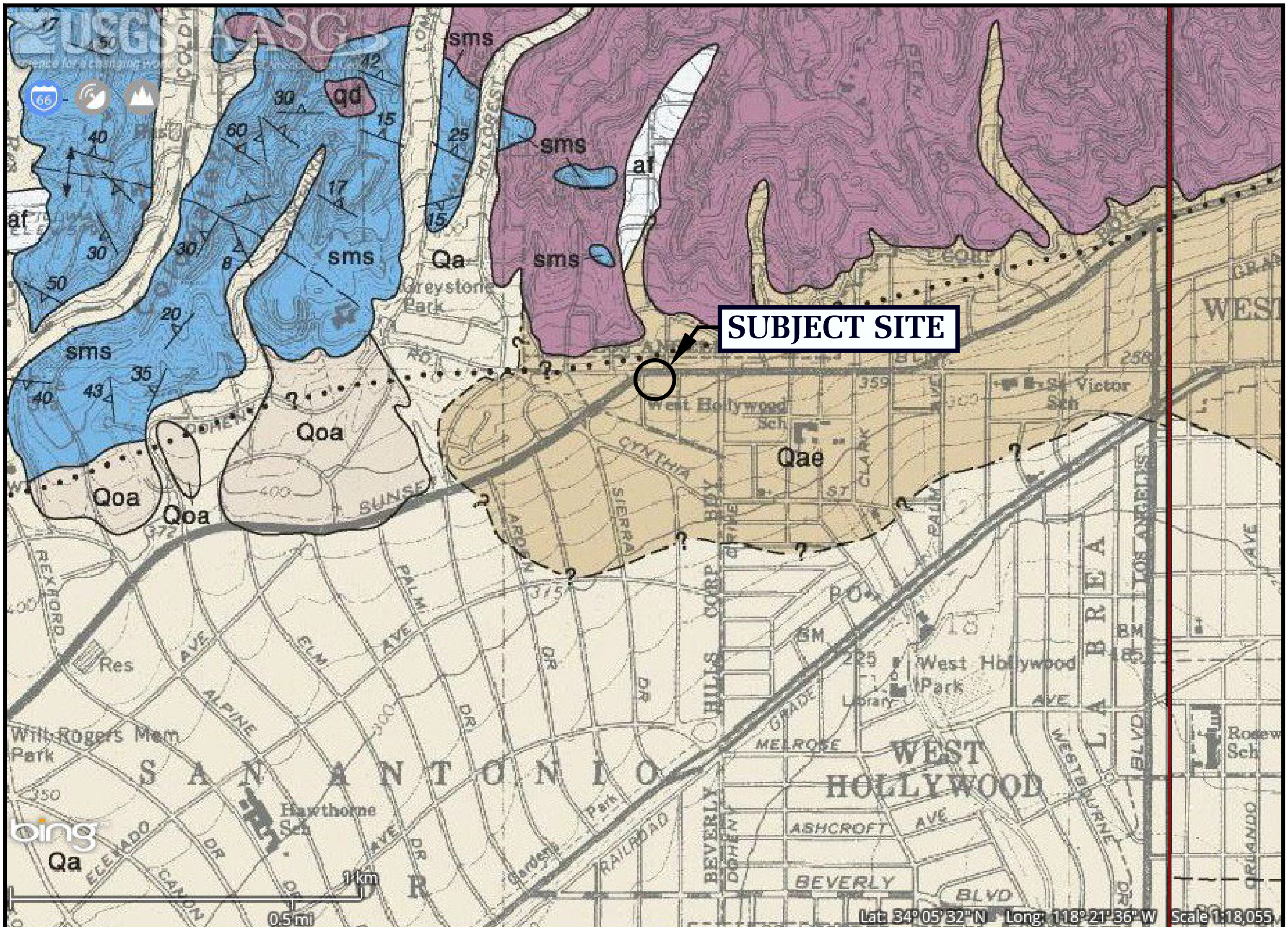
THE JOHN BUCK COMPANY

FILE No. 22055



REFERENCE: City Fault Location and Precaution Zone Map by KFM Geoscience, dated March 2010

CITY FAULT LOCATION AND PRECAUTION ZONE MAP	
Geotechnologies, Inc. Consulting Geotechnical Engineers	THE JOHN BUCK COMPANY
	FILE No. 22055



LEGEND

af: Surficial Sediments - Artificial cut and fill
Qa: Surficial Sediments - alluvium: gravel, sand and clay
Qae: Older Surficial Sediments - alluvial fan sediments of granitic sand at West Hollywood
Qd: Crystalline Basement Rocks - Quartz Diorite: gray to light gray granitic rock
sms: Santa Monica Slate - dark bluish gray slate-phyllite, weathers brown
—••••? Fault - dashed where indefinite or inferred, dotted where concealed, queried where existence is doubtful

REFERENCE: DIBBLEE, T.W., (1991) GEOLOGIC MAP OF THE BEVERLY HILLS & VAN NUYS (SOUTH HALF) QUADRANGLES (#DF-31)

LOCAL GEOLOGIC MAP

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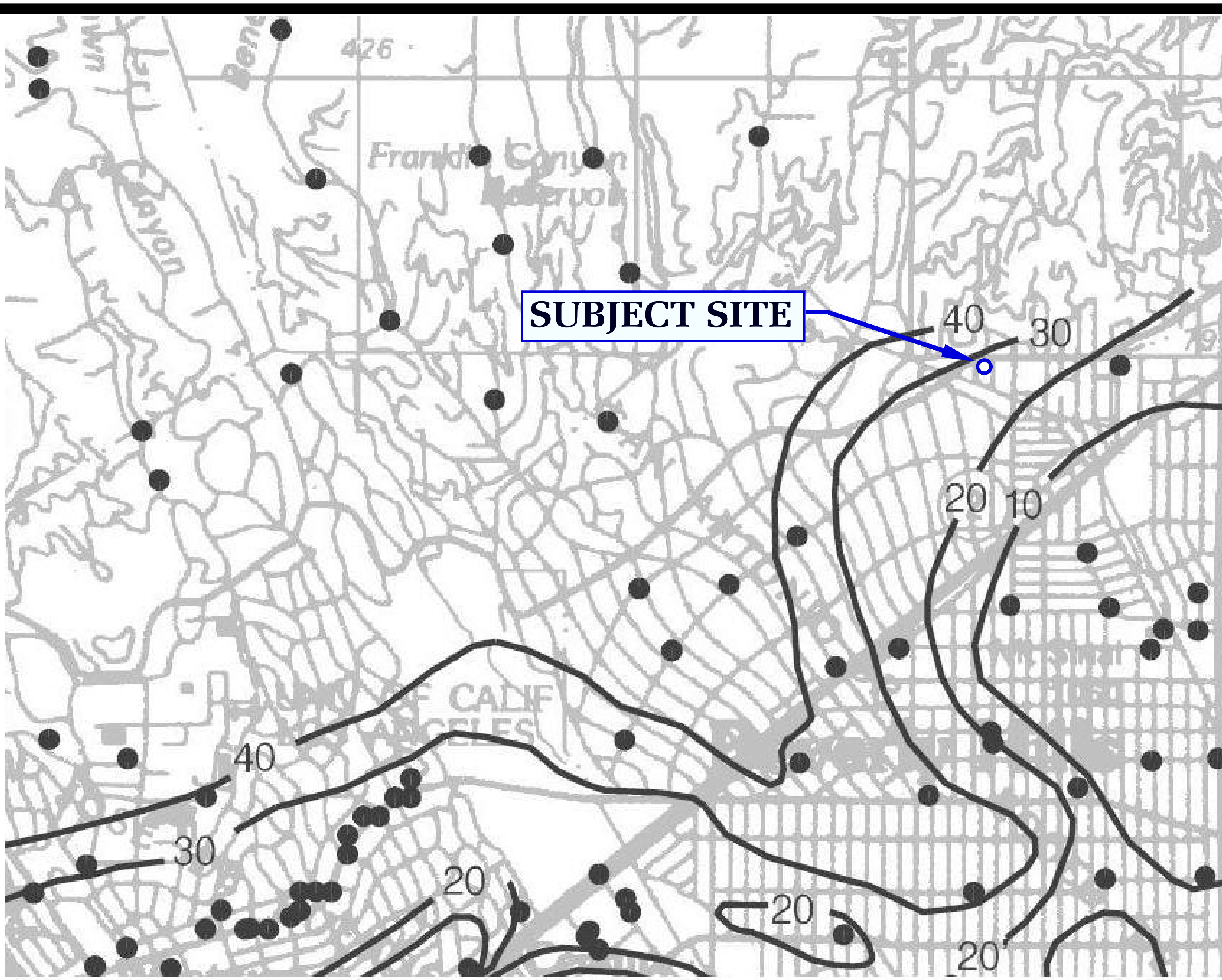
Consulting Geotechnical Engineers

THE JOHN BUCK COMPANY

FILE No. 22055



ONE MILE

SCALE

20 Depth to groundwater in feet

REFERENCE: PLATE 1.2, GROUNDWATER MAP, SEISMIC HAZARD EVALUATION REPORT, OFR 98-14
7.5 MINUTE QUADRANGLES, BEVERLY HILLS, CA QUADRANGLE

N

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HISTORICALLY HIGHEST GROUNDWATER LEVELS

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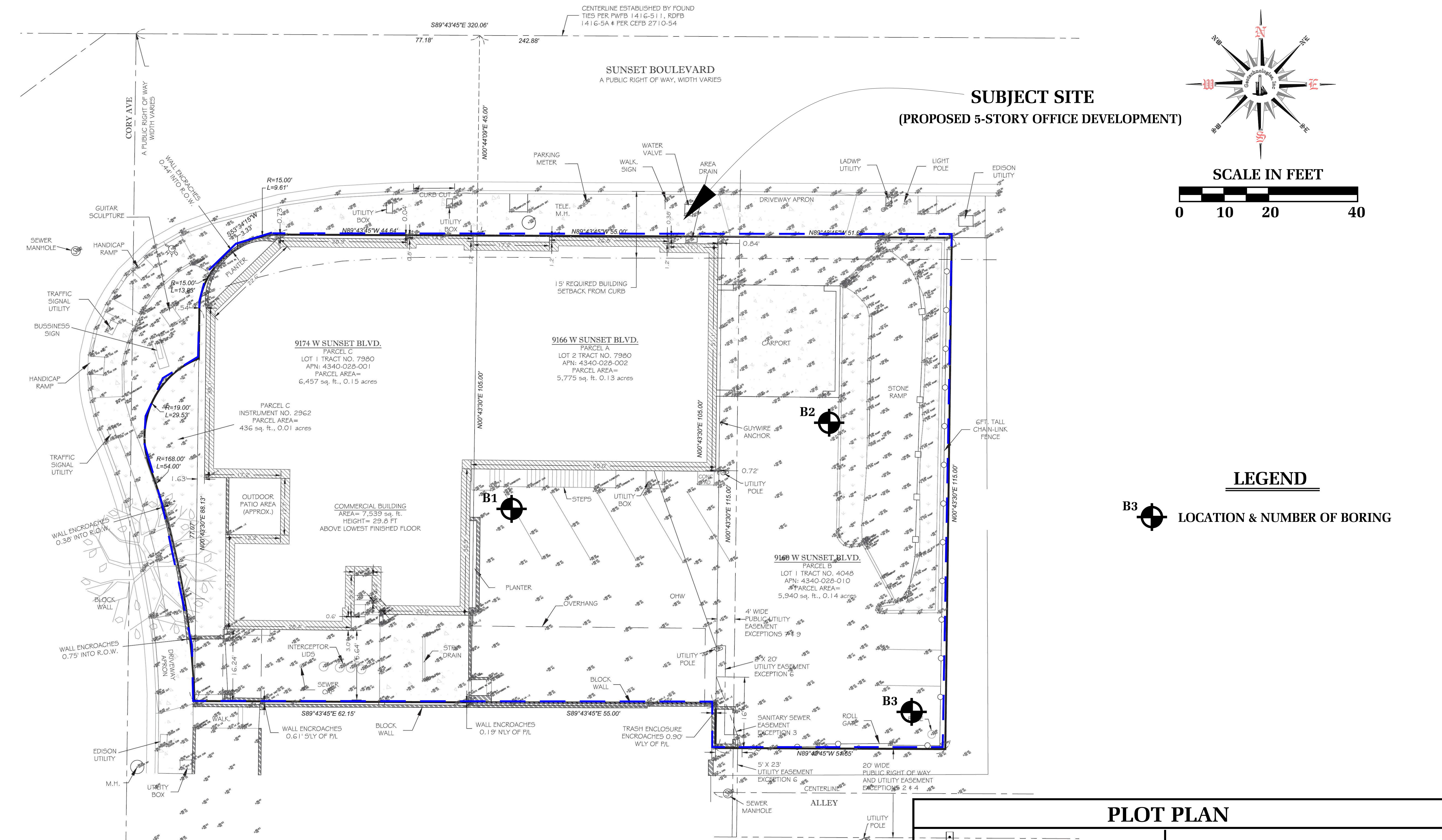
THE JOHN BUCK COMPANY

FILE No. 22055



PLOT PLAN

THE JOHN BUCK COMPANY

Geotechnologies, Inc.
Consulting Geotechnical Engineers

FILE No. 22055	DRAWN BY: TC
DATE: November 2020	

REFERENCE: ALTA/NSPS LAND TITLE SURVEY PROVIDED BY CHRISTENSEN & PLOUF LAND SURVEYING DATED JULY 10, 2019

BORING LOG NUMBER 1

John Buck Company — **Date: 11/02/20** — **Elevation: 99.8'***

File No. 22055 — **Method: 8-inch diameter Hollow Stem Auger**

dy

*Reference: Land Title Survey by Christensen & Plouff Land Surveying (Dated 7/10/19)

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphaltic Paving
				0 --		4-inch Asphalt over 6-inch Base
				-		
				1 --		FILL: Silty Sand, light brown to brown, slightly moist, medium dense, fine grained to coarse grained, some gravel, debris fragments
				-		
				2 --		
2.5	112	3.9	106.9	-		
				3 --		
				-		
				4 --		
				-		
5	6	3.6	vSPT	5 --		- - - - - - - - - - - - -
				-		some Clay
				6 --		
				-		
				7 --		
7.5	11	5.2	113.6	-		
				8 --	SM	NATIVE SOILS (OLDER ALLUVIAL FAN): Silty Sand, brown, moist, medium dense, fine grained, some gravel, trace amounts of clay
				-		
				9 --		
				-		
10	8	4.9	SPT	10 --		- - - - - - - - - - - - -
				-		fine to coarse grained
				11 --		
				-		
				12 --		
12.5	14	3.0	122.6	-		
				13 --	SW-SM	Sand with silt, brown, moist, medium dense, fine to coarse grained, few gravel
				-		
				14 --		
				-		
15	8	5.1	SPT	15 --		
				-	SM	Silty Sand, brown, moist, medium dense, fine to coarse grained, few gravel
				16 --		
				-		
				17 --		
17.5	16	6.8	120.6	-		- - - - - - - - - - - - -
				18 --		some Clay
				-		
				19 --		
				-		
20	10	5.3	SPT	20 --		
				-		
				21 --		
				-		
				22 --		
22.5	17	9.8	124.9	-		
				23 --	SC/CL	Sandy Clay to Clayey Sand, dark brown, moist, medium dense, very stiff, fine to medium grained
				-		
				24 --		
				-		
25	14	10.7	SPT	25 --		
				-		

BORING LOG NUMBER 1

John Buck Company

File No. 22055

dy

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
27.5	32	12.2	121.8	-		
				28 --		few gravels
				-		
				29 --		
				-		
30	23	13.9	SPT	30 --		
				-		
				31 --		
				-		
				32 --		
32.5	63	11.5	128.9	-		
				33 --		
				-		
				34 --		
				-		
35	47	13.2	SPT	35 --		
				-		
				36 --		
				-		
				37 --		
37.5	74	13.8	123.4	-		
				38 --		dark brown
				-		
				39 --		
				-		
40	41	12.8	SPT	40 --		
				-	CL	Sandy Clay, brown, moist, very stiff
				41 --		
				-		
				42 --		
42.5	25	12.2	123.1	-		
	50/5"			43 --		
				-		
				44 --		
				-		
45	46	13.3	SPT	45 --		
				-		
				46 --		
				-		
				47 --		
47.5	76	15.9	125.4	-		
				48 --		
				-		
				49 --		
				-		
50	37	15.2	SPT	50 --		
				-		

BORING LOG NUMBER 1

John Buck Company

File No. 22055

dy

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
52.5	87	14.3	125.8	-		
				53 --		dark reddish brown, very moist
				-		
				54 --		
				-		
55	37	14.5	SPT	55 --		
				-		wet
				56 --		
				-		
				57 --		
57.5	30 50/5"	12.9	125.1	-		
				58 --		some mottling
				-		
				59 --		
				-		
60	46	16.8	SPT	60 --		
				-		
				61 --		
				-		
				62 --		
62.5	50/5"	14.8	120.7	-		
				63 --	SC	Clayey Sand, mottled brown, wet, very dense, fine to coarse grained
				-		
				64 --		
				-		
65	35	15.3	SPT	65 --		
				-		mottled brown to gray, few mineral precipitates
				66 --		
				-		
				67 --		
67.5	33 50/4"	16.8	117.7	-		
				68 --		bluish gray
				-		
				69 --		
				-		
70	43	13.8	SPT	70 --		
				-		
				71 --		
				-		
				72 --		
72.5	50/4"	15.2	121.3	-		
				73 --		
				-		
				74 --		
				-		
75	42	13.4	SPT	75 --		
				-		

BORING LOG NUMBER 1

John Buck Company

File No. 22055

dy

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				76 --		
				-		
				77 --		
77.5		No Recovery		-		
				78 --		
				-		
				79 --		
				-		
80	41	14.2	SPT	80 --		
				-		Total Depth: 80 feet
				81 --		Water at 53 feet
				-		Fill to 7.5 feet
				82 --		
				-		NOTE: The stratification lines represent the approximate
				83 --		boundary between earth types; the transition may be gradual.
				-		
				84 --		Used 8-inch diameter Hollow-Stem Auger
				-		140-lb. Automatic Hammer, 30-inch drop
				85 --		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 2

John Buck Company — **Date: 11/02/20** — **Elevation: 102.2'***

File No. 22055 — **Method: 8-inch diameter Hollow Stem Auger**

dy

*Reference: Land Title Survey by Christensen & Plouff Land Surveying (Dated 7/10/19)

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphaltic Paving
				0 --		4-in Asphalt over 5-in base
				1 --		FILL: Silty Sand, brown, moist, medium dense, fine to coarse grained, few gravel, some Clay, debris fragments
				2 --		
2.5	9	8.0	112.9	-		
				3 --	SM	NATIVE SOILS (OLDER ALLUVIAL FAN): Silty Sand, brown, moist, medium dense, fine to coarse grained, minor clay
				4 --		
5	13	5.5	114.5	5 --		
				6 --		
				7 --		
7.5	13	3.5	109.8	-		
				8 --	SW-SM	Sand with Silt, brown, moist, medium dense, fine to coarse grained, some gravel
				9 --		
10	15	6.6	116.3	10 --	SM	Silty Sand, brown, moist, medium dense, fine to medium grained
				11 --		
				12 --		
				13 --		
				14 --		
15	14	7.3	115.5	15 --		
				16 --		
				17 --		
				18 --		
				19 --		
20	18	13.9	123.1	20 --	CL	Sandy Clay, brown, moist, stiff
				21 --		
				22 --		
				23 --		
				24 --		
25	50	13.4	122.9	25 --		very stiff

BORING LOG NUMBER 2

John Buck Company

File No. 22055

dy

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
				28		
				29		
30	74	10.0	126.1	30		
						few gravels
				31		
				32		
				33		
				34		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
35	64	13.1	122.3	35		
						Used 8-inch diameter Hollow-Stem Auger
				36		140-lb. Automatic Hammer, 30-inch drop
						Modified California Sampler used unless otherwise noted
				37		
				38		
				39		
40	69	10.5	122.7	40		
					CL/SC	Silty Clay to Clayey Sand, brown, moist, very stiff, dense, fine to coarse grained
				41		
				42		
				43		
				44		
45	63	16.4	118.3	45		
					SC	Clayey Sand, dark reddish brown, moist, dense, fine to coarse grained
				46		
				47		
				48		
				49		
						wet
50	69	13.9	122.3	50		
						Total Depth: 50 feet Water at 50 feet Fill to 3 feet

BORING LOG NUMBER 3

John Buck Company **Date: 11/04/20** **Elevation: 93.9'***

File No. 22055 **Method: 8-inch diameter Hollow Stem Auger**

dy

*Reference: Land Title Survey by Christensen & Plouff Land Surveying (Dated 7/10/19)

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphaltic Paving
				0 --		5-inch Asphalt over 3-inch Base
				1 --		FILL: Silty Sand, brown, moist, medium dense, fine to medium grained, some gravel, few cobbles, debris fragments, trace amount of clay
				2 --		
				3 --		
				4 --		
5	20	5.3	117.4	5 --		
				-	SP-SC	NATIVE SOILS (OLDER ALLUVIAL FAN): Sand with Clay, brown, moist, medium dense fine to coarse grained
				6 --		
				7 --		
				8 --		
				9 --		
10	15	5.1	123.2	10 --		
				11 --		
				12 --		
				13 --		
				14 --		
15	16	7.5	117.7	15 --		
				16 --		
				17 --		
				18 --		
				19 --		
20	24	5.6	117.8	20 --		
				21 --		
				22 --		
				23 --		
				24 --		
25	55	16.0	118.9	25 --		
				-	SC/CL	Sandy Clay to Clayey Sand, brown, moist, very stiff, very dense

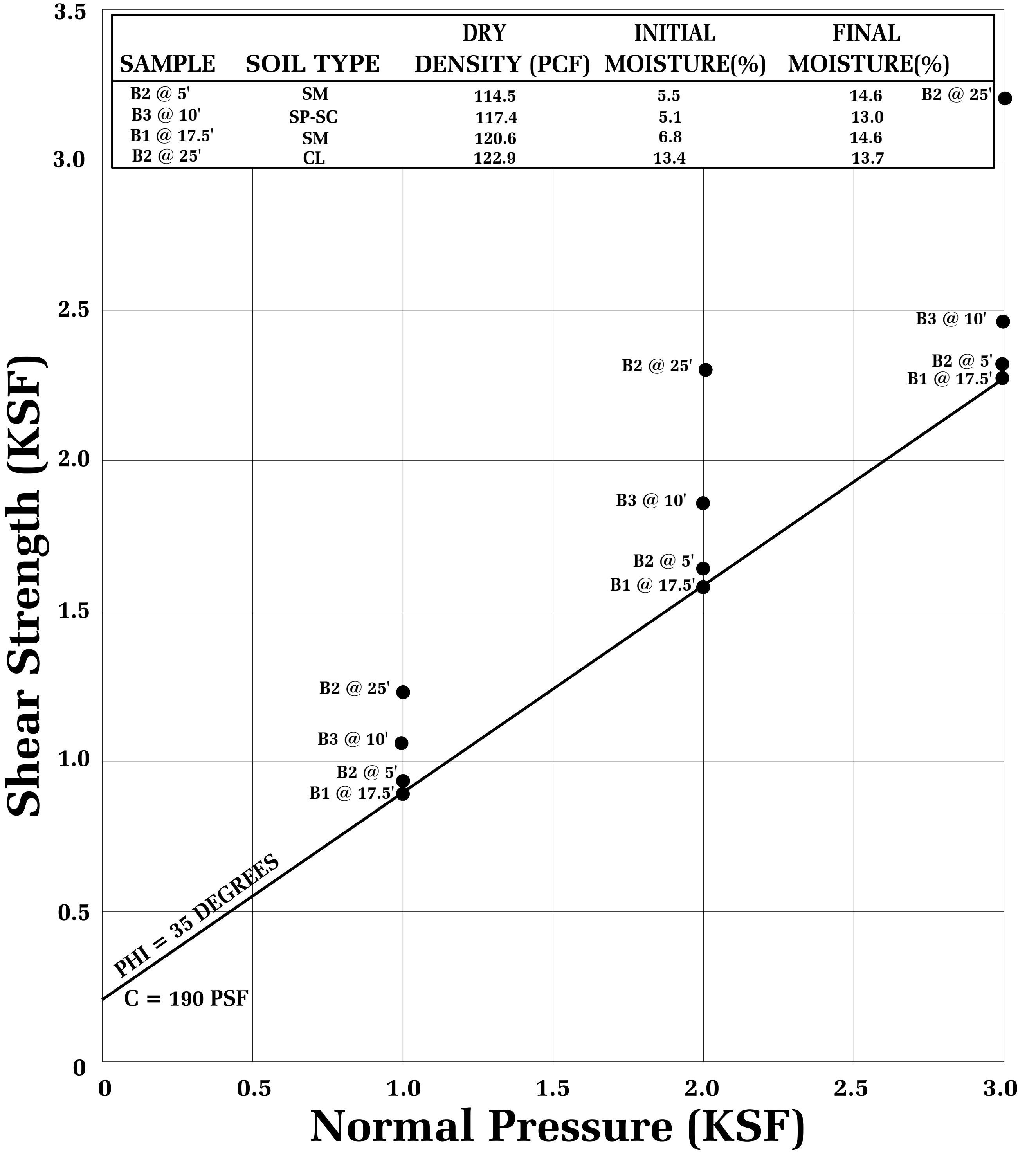
BORING LOG NUMBER 3

John Buck Company

File No. 22055

dy

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
30	24	12.6	126.8	30 --		
	50/6"			-		mottled brown
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
35	45	11.6	125.3	35 --		
	50/3"			-		few gravels
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
40	37	16.5	119.5	40 --		
	50/5"			-		dark reddish brown
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
45	50/6"	14.5	113.0	45 --		
				-		NOTE: The stratification lines represent the approximate
				46 --		boundary between earth types; the transition may be gradual.
				-		
				47 --		Used 8-inch diameter Hollow-Stem Auger
				-		140-lb. Automatic Hammer, 30-inch drop
				48 --		Modified California Sampler used unless otherwise noted
				-		
				49 --		
				-		
50	66	13.8	128.0	50 --		
				-		Total Depth: 50 feet
						No Ground Water
						Fill to 5 feet



SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B2 @ 5'	SM	114.5	5.5	14.6
B3 @ 10'	SP-SC	117.4	5.1	13.0
B1 @ 17.5'	SM	120.6	6.8	14.6
B2 @ 25'	CL	122.9	13.4	13.7

● Direct Shear, Saturated

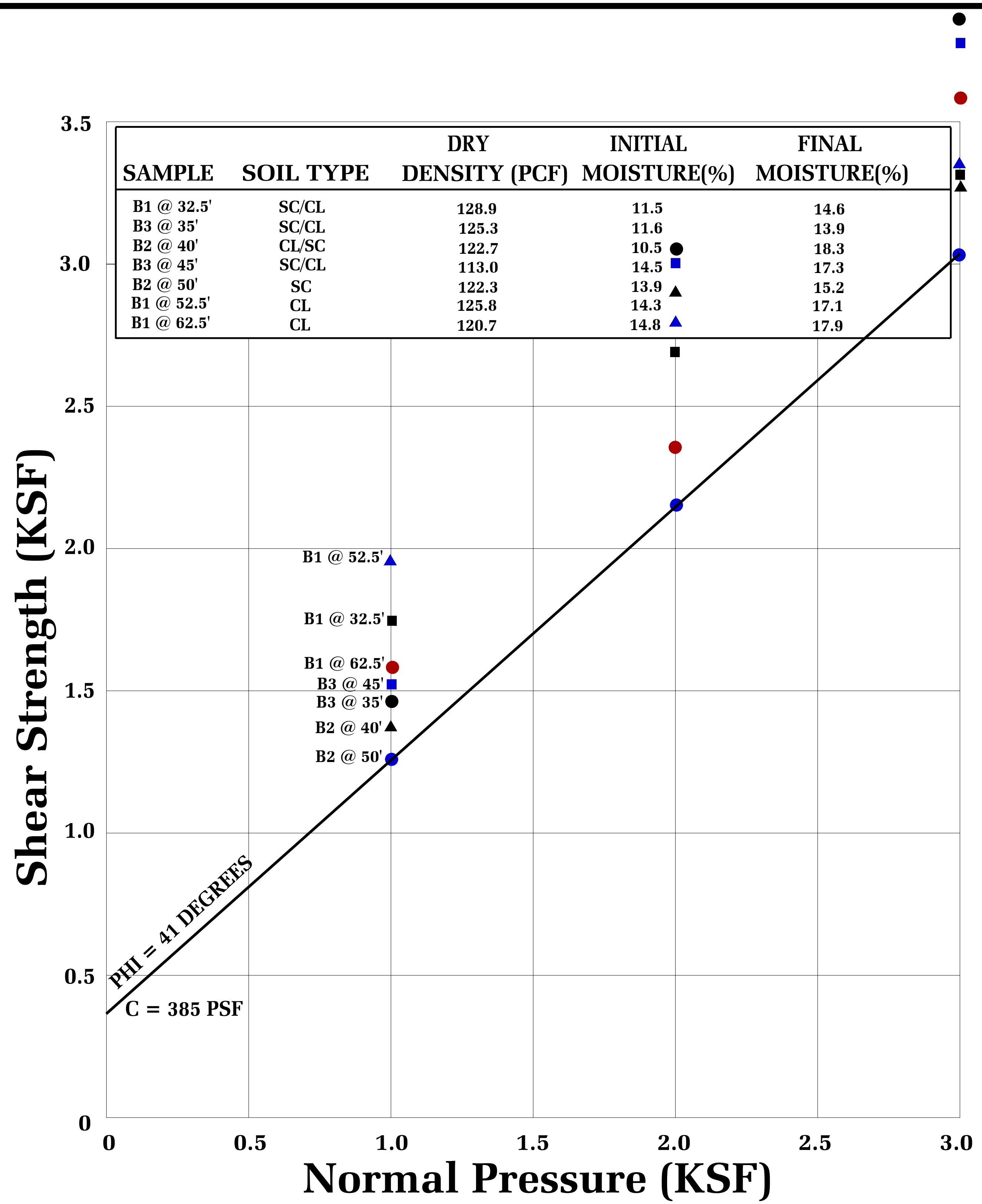
SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

THE JOHN BUCK COMPANY

FILE NO. 22055 | PLATE: B-1

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B1 @ 32.5'	SC/CL	128.9	11.5	14.6
B3 @ 35'	SC/CL	125.3	11.6	13.9
B2 @ 40'	CL/SC	122.7	10.5	18.3
B3 @ 45'	SC/CL	113.0	14.5	17.3
B2 @ 50'	SC	122.3	13.9	15.2
B1 @ 52.5'	CL	125.8	14.3	17.1
B1 @ 62.5'	CL	120.7	14.8	17.9



● Direct Shear, Saturated

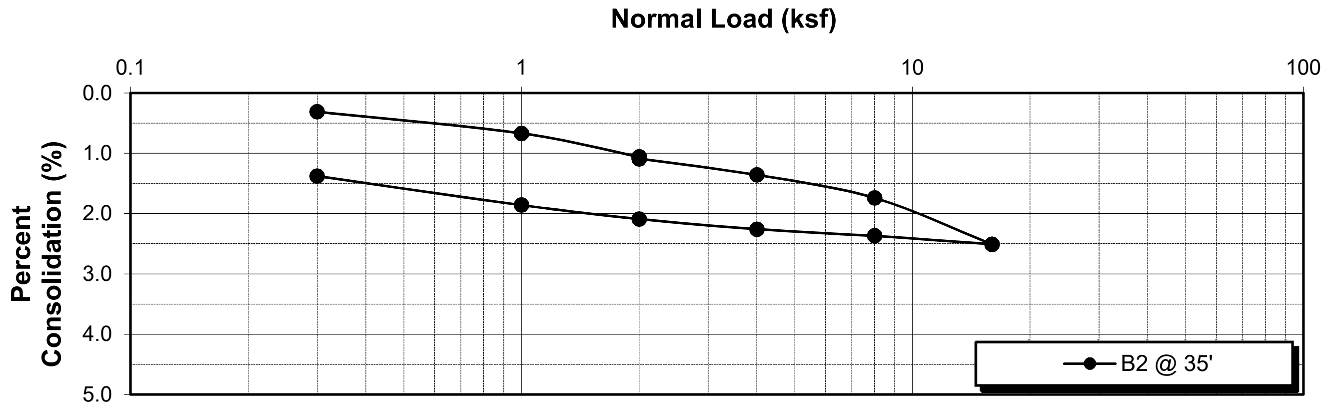
SHEAR TEST DIAGRAM

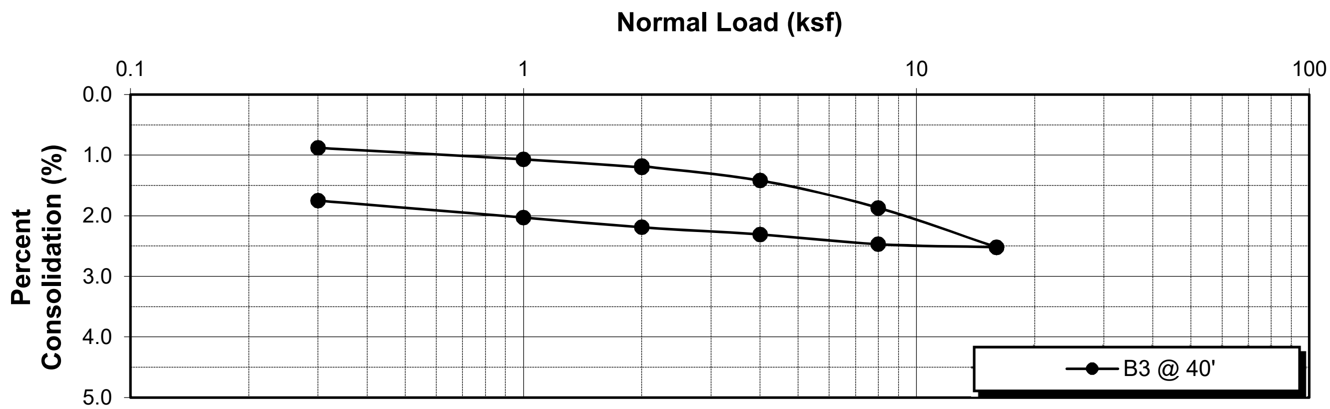
Geotechnologies, Inc.
Consulting Geotechnical Engineers

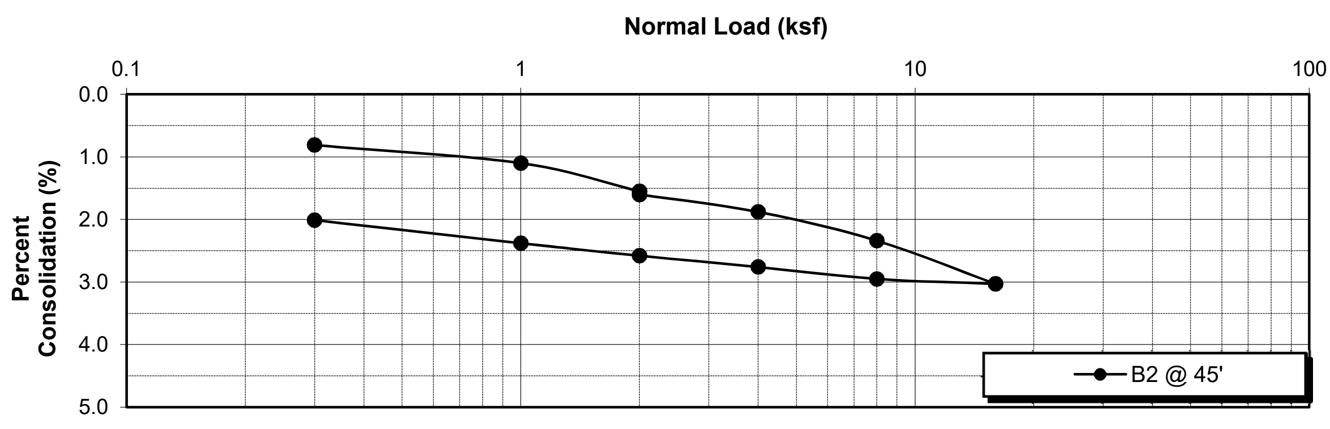
THE JOHN BUCK COMPANY

FILE NO. 22055

PLATE: B-2







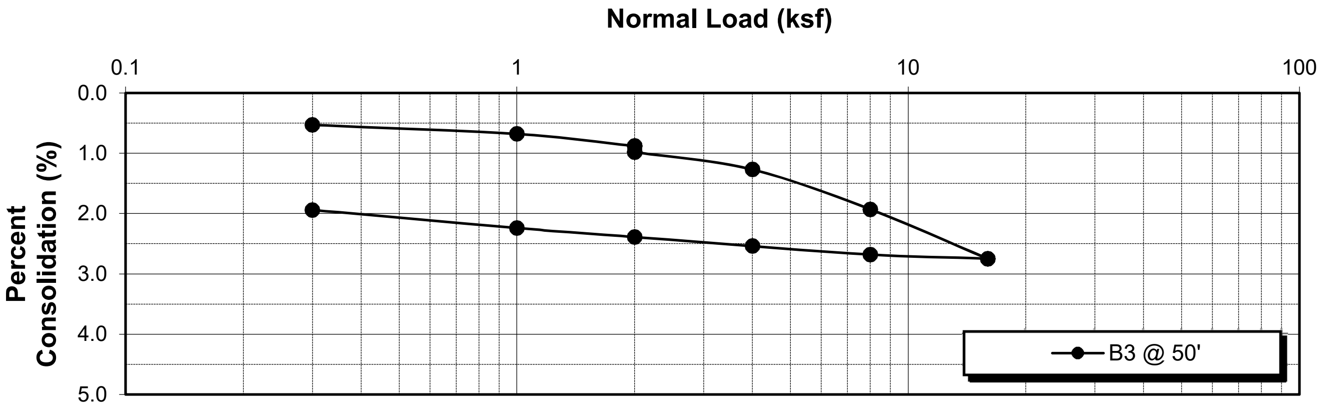
Water added at 2 KSF

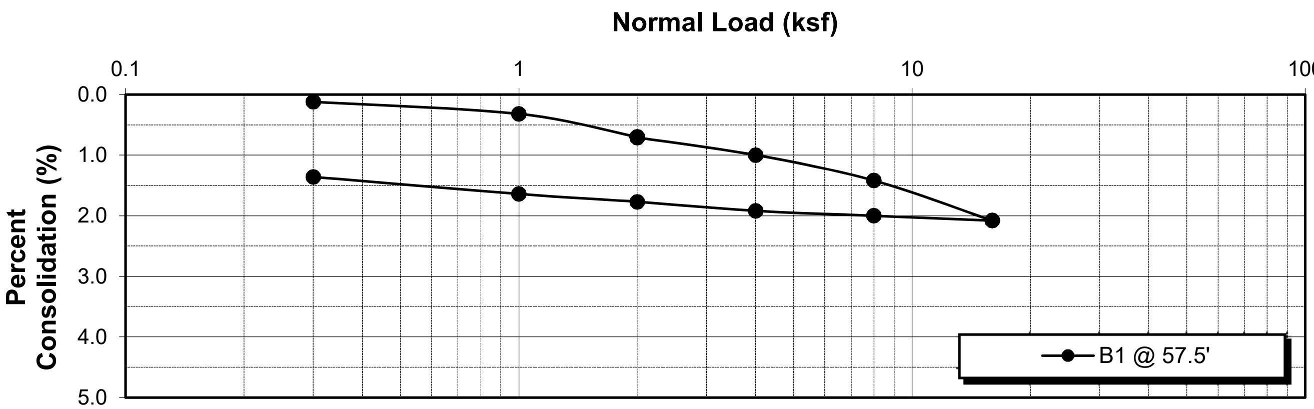
CONSOLIDATION

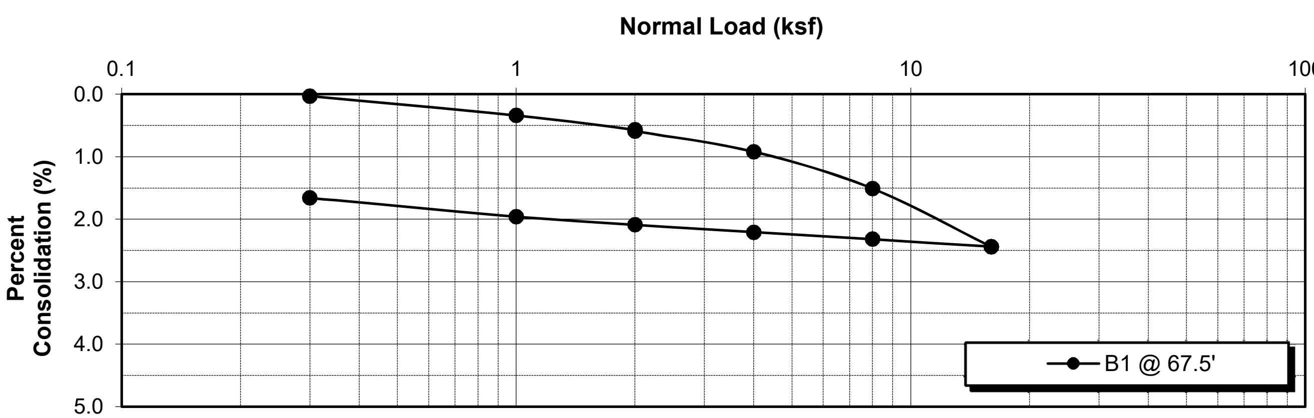
Geotechnologies, Inc.
Consulting Geotechnical Engineers

PROJECT: The John Buck Company

FILE NO.: 22055	PLATE: C-1







Water added at 2 KSF

CONSOLIDATION

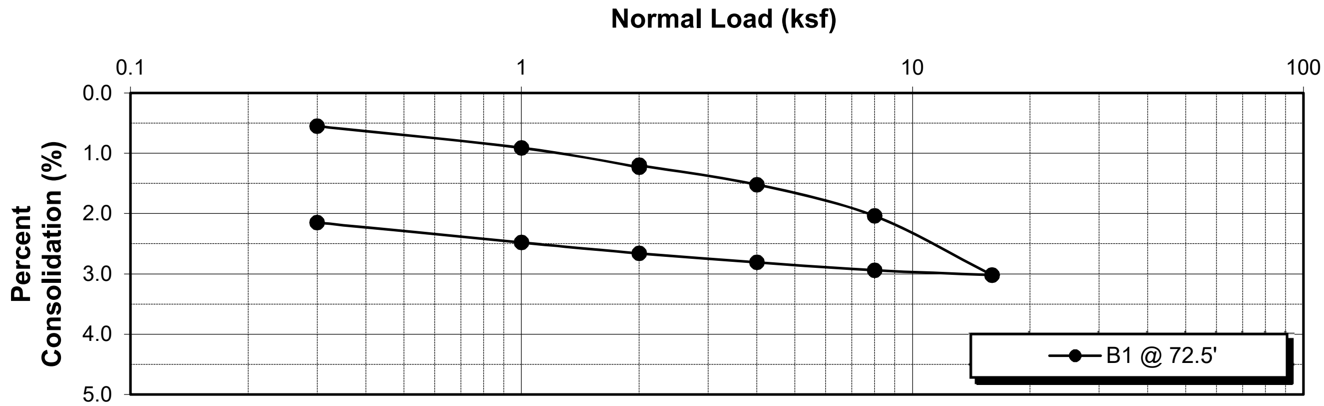
Geotechnologies, Inc.

Consulting Geotechnical Engineers

PROJECT: The John Buck Company

FILE NO.: 22055

PLATE: C-2



Water added at 2 KSF

CONSOLIDATION

Geotechnologies, Inc.
Consulting Geotechnical Engineers

PROJECT: The John Buck Company	
FILE NO.: 22055	PLATE: C-3

ASTM D-1557

SAMPLE	B1 @ 1-5'	B3 @ 1-5'
SOIL TYPE:	SM	SM
MAXIMUM DENSITY pcf.	134.4	136.3
OPTIMUM MOISTURE %	8.0	7.4

ASTM D 4829

SAMPLE	B1 @ 1-5'	B3 @ 1-5'
SOIL TYPE:	SM	SM
EXPANSION INDEX UBC STANDARD 18-2	7	10
EXPANSION CHARACTER	VERY LOW	VERY LOW

SULFATE CONTENT

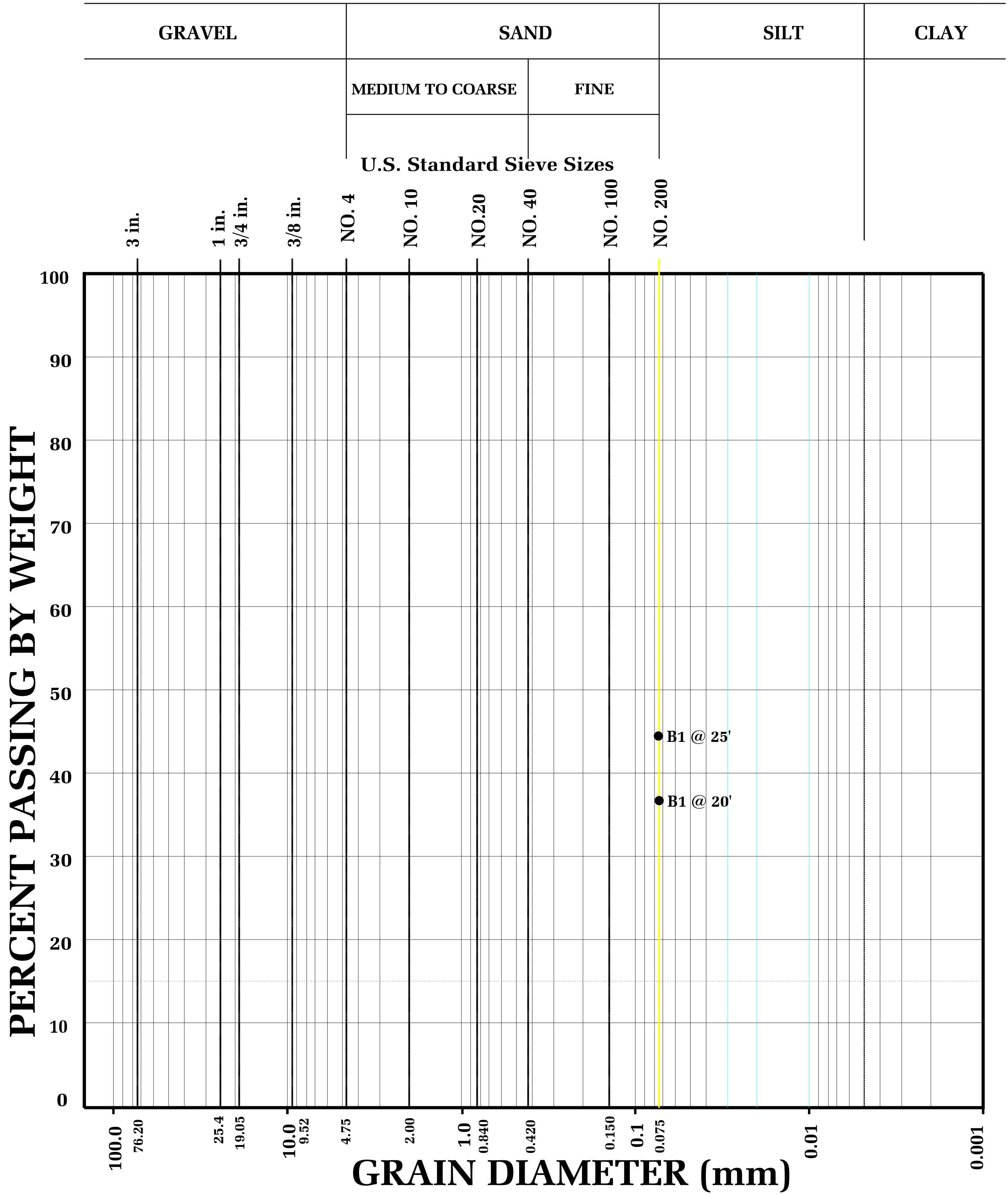
SAMPLE	B1 @ 1-5'	B3 @ 1-5'
SULFATE CONTENT: (percentage by weight)	< 0.10%	< 0.10%

COMPACTION/EXPANSION/SULFATE DATA SHEET

Geotechnologies, Inc.
Consulting Geotechnical Engineers

THE JOHN BUCK COMPANY

FILE NO. 22055 | PLATE: D



GRAIN SIZE DISTRIBUTION

Geotechnologies, Inc.
Consulting Geotechnical Engineers

THE JOHN BUCK COMPANY

FILE NO. 22055

PLATE: E

Geotechnologies, Inc.

Project: The John Buck Company
File No.: 22055
Description: Liquefaction Analysis - Maximum Considered Earthquake (2% Probability of Exceedance in 50 years, Return Period of 2,475 years)
Boring Number: 1

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	6.9
Peak Ground Horizontal Acceleration, PGA (g):	1.00
Calculated Mag.Wtg.Factor:	1.171

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	50.0
Historically Highest Groundwater Level* (ft):	29.0
Unit Weight of Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

BOREHOLE AND SAMPLER INFORMATION:

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

LIQUEFACTION BOUNDARY:

Plastic Index Cut Off (PI):	12
Minimum Liquefaction FS:	1.3

Depth to Base Layer (feet)	Total Unit Weight (pcf)	Current Water Level (feet)	Historical Water Level (feet)	Field SPT Blowcount N	Depth of SPT Blowcount (feet)	Fines Content #200 Sieve (%)	Plastic Index (PI)	Vetical Stress σ_{vc}, (psf)	Effective Vert. Stress σ_{vc}', (psf)	Fines Corrected $(N_1)_{60\text{-}cs}$	Stress Reduction Coeff, r_d	Cyclic Shear Ratio CSR	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Liquefaction Settlment ΔS_i (inches)
1	115.3	Unsaturated	Unsaturated	6	5	0.0	0	115.3	115.3	12.2	1.00	0.654	0.173	Non-Liq.	0.00
2	115.3	Unsaturated	Unsaturated	6	5	0.0	0	230.6	230.6	12.2	1.00	0.652	0.173	Non-Liq.	0.00
3	115.3	Unsaturated	Unsaturated	6	5	0.0	0	345.9	345.9	12.2	1.00	0.650	0.173	Non-Liq.	0.00
4	115.3	Unsaturated	Unsaturated	6	5	0.0	0	461.2	461.2	12.2	0.99	0.648	0.173	Non-Liq.	0.00
5	115.3	Unsaturated	Unsaturated	6	5	0.0	0	576.5	576.5	13.1	0.99	0.646	0.181	Non-Liq.	0.00
6	115.3	Unsaturated	Unsaturated	6	5	0.0	0	691.8	691.8	13.1	0.99	0.643	0.181	Non-Liq.	0.00
7	115.3	Unsaturated	Unsaturated	6	5	0.0	0	807.1	807.1	13.1	0.98	0.641	0.181	Non-Liq.	0.00
8	122.9	Unsaturated	Unsaturated	8	10	0.0	0	930.0	930.0	16.3	0.98	0.638	0.215	Non-Liq.	0.00
9	122.9	Unsaturated	Unsaturated	8	10	0.0	0	1052.9	1052.9	16.1	0.98	0.636	0.210	Non-Liq.	0.00
10	122.9	Unsaturated	Unsaturated	8	10	0.0	0	1175.8	1175.8	15.1	0.97	0.633	0.196	Non-Liq.	0.00
11	122.9	Unsaturated	Unsaturated	8	10	0.0	0	1298.7	1298.7	14.3	0.97	0.630	0.185	Non-Liq.	0.00
12	122.9	Unsaturated	Unsaturated	8	10	0.0	0	1421.6	1421.6	13.5	0.96	0.628	0.176	Non-Liq.	0.00
13	127.5	Unsaturated	Unsaturated	8	15	0.0	0	1549.1	1549.1	12.9	0.96	0.625	0.168	Non-Liq.	0.00
14	127.5	Unsaturated	Unsaturated	8	15	0.0	0	1676.6	1676.6	12.3	0.95	0.622	0.161	Non-Liq.	0.00
15	127.5	Unsaturated	Unsaturated	8	15	0.0	0	1804.1	1804.1	13.3	0.95	0.619	0.169	Non-Liq.	0.00
16	127.5	Unsaturated	Unsaturated	8	15	0.0	0	1931.6	1931.6	12.7	0.95	0.616	0.163	Non-Liq.	0.00
17	127.5	Unsaturated	Unsaturated	8	15	0.0	0	2059.1	2059.1	12.3	0.94	0.613	0.158	Non-Liq.	0.00
18	127.5	Unsaturated	Unsaturated	8	15	0.0	0	2186.6	2186.6	11.9	0.94	0.610	0.153	Non-Liq.	0.00
19	127.5	Unsaturated	Unsaturated	8	15	0.0	0	2314.1	2314.1	11.5	0.93	0.606	0.149	Non-Liq.	0.00
20	133.0	Unsaturated	Unsaturated	10	20	36.4	0	2447.1	2447.1	19.8	0.93	0.603	0.234	Non-Liq.	0.00
21	133.0	Unsaturated	Unsaturated	10	20	36.4	0	2580.1	2580.1	19.4	0.92	0.600	0.227	Non-Liq.	0.00
22	133.0	Unsaturated	Unsaturated	10	20	36.4	0	2713.1	2713.1	19.1	0.92	0.596	0.221	Non-Liq.	0.00
23	137.0	Unsaturated	Unsaturated	14	25	43.5	0	2850.1	2850.1	25.2	0.91	0.593	0.329	Non-Liq.	0.00
24	137.0	Unsaturated	Unsaturated	14	25	43.5	0	2987.1	2987.1	24.8	0.90	0.589	0.315	Non-Liq.	0.00
25	137.0	Unsaturated	Unsaturated	14	25	43.5	0	3124.1	3124.1	24.3	0.90	0.586	0.302	Non-Liq.	0.00
26	137.0	Unsaturated	Unsaturated	14	25	43.5	0	3261.1	3261.1	23.9	0.89	0.582	0.292	Non-Liq.	0.00
27	137.0	Unsaturated	Unsaturated	14	25	43.5	0	3398.1	3398.1	23.6	0.89	0.579	0.282	Non-Liq.	0.00
28	140.2	Unsaturated	Unsaturated	23	30	0.0	0	3538.3	3538.3	36.1	0.88	0.575	1.413	Non-Liq.	0.00
29	140.2	Unsaturated	Unsaturated	23	30	0.0	0	3678.5	3678.5	35.6	0.88	0.572	1.244	Non-Liq.	0.00
30	140.2	Unsaturated	Saturated	23	30	0.0	0	3818.7	3756.3	35.3	0.87	0.577	1.165	2.0	0.00
31	140.2	Unsaturated	Saturated	23	30	0.0	0	3958.9	3834.1	35.0	0.87	0.583	1.095	1.9	0.00
32	140.2	Unsaturated	Saturated	23	30	0.0	0	4099.1	3911.9	34.7	0.86	0.587	1.032	1.8	0.00
33	140.2	Unsaturated	Saturated	47	35	0.0	0	4239.3	3989.7	74.3	0.85	0.592	1.902	3.2	0.00
34	140.2	Unsaturated	Saturated	47	35	0.0	0	4379.5	4067.5	73.9	0.85	0.596	1.889	3.2	0.00
35	142.1	Unsaturated	Saturated	47	35	0.0	0	4521.6	4147.2	73.5	0.84	0.599	1.875	3.1	0.00
36	142.1	Unsaturated	Saturated	47	35	0.0	0	4663.7	4226.9	73.2	0.84	0.602	1.862	3.1	0.00
37	142.1	Unsaturated	Saturated	47	35	0.0	0	4805.8	4306.6	72.8	0.83	0.605	1.849	3.1	0.00
38	142.1	Unsaturated	Saturated	47	35	0.0	0	4947.9	4386.3	72.4	0.83	0.607	1.837	3.0	0.00
39	142.1	Unsaturated	Saturated	47	35	0.0	0	5090.0	4466.0	72.1	0.82	0.609	1.824	3.0	0.00
40	139.3	Unsaturated	Saturated	41	40	0.0	0	5229.3	4542.9	62.6	0.81	0.611	1.812	3.0	0.00
41	139.3	Unsaturated	Saturated	41	40	0.0	0	5368.6	4619.8	62.3	0.81	0.612	1.801	2.9	0.00
42	139.3	Unsaturated	Saturated	41	40	0.0	0	5507.9	4696.7	62.1	0.80	0.613	1.789	2.9	0.00
43	139.3	Unsaturated	Saturated	41	40	0.0	0	5647.2	4773.6	61.8	0.80	0.614	1.778	2.9	0.00
44	139.3	Unsaturated	Saturated	41	40	0.0	0	5786.5	4850.5	61.5	0.79	0.615	1.767	2.9	0.00
45	141.7	Unsaturated	Saturated	46	45	0.0	0	5928.2	4929.8	68.8	0.79	0.615	1.756	2.9	0.00
46	141.7	Unsaturated	Saturated	46	45	0.0	0	6069.9	5009.1	68.5	0.78	0.615	1.745	2.8	0.00
47	141.7	Unsaturated	Saturated	46	45	0.0	0	6211.6	5088.4	68.2	0.77	0.615	1.734	2.8	0.00
48	141.7	Unsaturated	Saturated	46	45	0.0	0	6353.3	5167.7	67.9	0.77	0.615	1.723	2.8	0.00
49	141.7	Unsaturated	Saturated	46	45	0.0	0	6495.0	5247.0	67.6	0.76	0.615	1.713	2.8	0.00
50	144.6	Unsaturated	Saturated	37	50	0.0	0	6639.6	5329.2	54.2	0.76	0.614	1.702	2.8	0.00
51	144.6	Saturated	Saturated	37	50	0.0	0	6784.2	5411.4	54.0	0.75	0.614	1.692	2.8	0.00
52	144.6	Saturated	Saturated	37	50	0.0	0	6928.8	5493.6	53.7	0.75	0.613	1.681	2.7	0.00
53	144.6	Saturated	Saturated	37	50	0.0	0	7073.4	5575.8	53.5	0.74	0.612	1.671	2.7	0.00
54	144.6	Saturated	Saturated	37	50	0.0	0	7218.0	5658.0	53.3	0.73	0.611	1.661	2.7	0.00
55	142.6	Saturated	Saturated	37	55	0.0	0	7360.6	5738.2	53.1	0.73	0.610	1.651	2.7	0.00
56	142.6	Saturated	Saturated	37	55	0.0	0	7503.2	5818.4	52.9	0.72	0.608	1.641	2.7	0.00
57	142.6	Saturated	Saturated	37	55	0.0	0	7645.8	5898.6	52.8	0.72	0.607	1.632	2.7	0.00
58	142.6	Saturated	Saturated	37	55	0.0	0	7788.4	5978.8	52.6	0.71	0.605	1.623	2.7	0.00
59	142.6	Saturated	Saturated	37	55	0.0	0	7931.0	6059.0	52.4	0.71	0.604	1.613	2.7	0.00
60	140.0	Saturated	Saturated	46	60	0.0	0	8071.0	6136.6	64.9	0.70	0.602	1.605	2.7	0.00
61	140.0	Saturated	Saturated	46	60	0.0	0	8211.0	6214.2	64.7	0.70	0.601	1.596	2.7	0.00
62	140.0	Saturated	Saturated	46	60	0.0	0	8351.0	6291.8	64.5	0.69	0.599	1.587	2.6	0.00
63	140.0	Saturated	Saturated	46	60	0.0	0	8491.0	6369.4	64.3	0.69	0.597	1.579	2.6	0.00
64	140.0	Saturated	Saturated	46	60	0.0	0	8631.0	6447.0	64.1	0.68	0.596	1.570	2.6	0.00
65	138.0	Saturated	Saturated	35	65	0.0	0	8769.0	6522.6	48.6	0.68	0.594	1.562	2.6	0.00
66	138.0	Saturated	Saturated	35	65	0.0	0	8907.0	6598.2	48.5	0.67	0.592	1.554	2.6	0.00
67	138.0	Saturated	Saturated	35	65	0.0	0	9045.0	6673.8	48.3	0.67	0.590	1.547	2.6	0.00
68	138.0	Saturated	Saturated	35	65	0.0	0	9183.0	6749.4	48.2	0.66	0.588	1.539	2.6	0.00
69	138.0	Saturated	Saturated	35	65	0.0	0	9321.0	6825.0	48.0	0.66	0.586	1.531	2.6	0.00
70	138.6	Saturated	Saturated	43	70	0.0	0	9459.6	6901.2	58.8	0.65	0.585	1.523	2.6	0.00
71	138.6	Saturated	Saturated	43	70	0.0	0	9598.2	6977.4	58.7	0.65	0.583	1.516	2.6	0.00
72	138.6	Saturated	Saturated	43	70	0.0	0	9736.8	7053.6	58.5	0.65	0.581	1.508	2.6	0.00
73	138.6	Saturated	Saturated	43	70	0.0	0	9875.4	7129.8	58.3	0.64	0.579	1.501	2.6	0.00
74	138.6	Saturated	Saturated	43	70	0.0	0	10014.0	7206.0	58.2	0.64	0.577	1.494	2.6	0.00
75	139.7	Saturated	Saturated	42	75	0.0	0	10153.7	7283.3	56.7	0.63	0.575	1.486	2.6	0.00
76	139.7	Saturated	Saturated	42	75	0.0	0	10293.4	7360.6	56.5	0.63	0.573	1.479	2.6	0.00
77	139.7	Saturated	Saturated	42	75	0.0	0	10433.1	7437.9	56.3	0.63	0.571	1.472	2.6	0.00
78	139.7	Saturated	Saturated	41	80	0.0	0	10572.8	7515.2	54.8	0.62	0.569	1.464	2.6	0.00
79	139.7	Saturated	Saturated	41	80	0.0	0	10712.5	7592.5	54.7	0.62	0.567	1.457	2.6	0.00
80	139.7	Saturated	Saturated	41	80	0.0	0	10852.2	7669.8	54.6	0.61	0.566	1.450	2.6	0.00

Total Liquefaction Settlement, S = 0.00 inches

December 4, 2020 via email: ebabayan@geoteq.com

GEOTECHNOLOGIES, INC.
439 Western Ave.
Glendale, CA 91201

Attention: Mr. Edmond Babayan

Re: Soil Corrosivity Study
The John Buck Company
West Hollywood, CA
HDR #20-0771SCS, CID #22055

Introduction

Laboratory tests have been completed on two soil samples provided for the referenced project. The purpose of these tests was to determine if the soils might have deleterious effects on underground utility piping, hydraulic elevator cylinders, and concrete structures. HDR Engineering, Inc. (HDR) assumes that the samples provided are representative of the most corrosive soils at the site.

The proposed structure has five stories and three subterranean levels. The site is located at 9176 Sunset Boulevard in West Hollywood, California, and the water table is reportedly 50 feet deep.

The scope of this study is limited to a determination of soil corrosivity and general corrosion control recommendations for materials likely to be used for construction. HDR's recommendations do not constitute, and are not meant as a substitute for, design documents for the purpose of construction. If the architects and/or engineers desire more specific information, designs, specifications, or review of design, HDR will be happy to work with them as a separate phase of this project.

431 W. Baseline Road, Claremont, CA 91711-1608
(909) 626-0967

Laboratory Soil Corrosivity Tests

The electrical resistivity of each sample was measured in a soil box per ASTM G187 in its as-received condition and again after saturation with distilled water. Resistivities are at about their lowest value when the soil is saturated. The pH of the saturated samples were measured per ASTM G51. A 5:1 water:soil extract from each sample was chemically analyzed for the major soluble salts commonly found in soil per ASTM D4327, ASTM D6919, and Standard Method 2320-B[1]. Laboratory test results are shown in the attached Table 1.

Soil Corrosivity

A major factor in determining soil corrosivity is electrical resistivity. The electrical resistivity of a soil is a measure of its resistance to the flow of electrical current. Corrosion of buried metal is an electrochemical process in which the amount of metal loss due to corrosion is directly proportional to the flow of electrical current (DC) from the metal into the soil. Corrosion currents, following Ohm's Law, are inversely proportional to soil resistivity. Lower electrical resistivities result from higher moisture and soluble salt contents and indicate corrosive soil.

A correlation between electrical resistivity and corrosivity toward ferrous metals is:[2]

Soil Resistivity in ohm-centimeters	Corrosivity Category
Greater than 10,000	Mildly Corrosive
2,001 to 10,000	Moderately Corrosive
1,001 to 2,000	Corrosive
0 to 1,000	Severely Corrosive

Other soil characteristics that may influence corrosivity towards metals are pH, soluble salt content, soil types, aeration, anaerobic conditions, and site drainage.

Electrical resistivities were in the mildly corrosive and moderately corrosive categories with as-received moisture. When saturated, the resistivities were in the moderately corrosive category. Some as-received resistivities were at or near their saturated values.

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[1] American Public Health Association (APHA). 2012. Standard Methods of Water and Wastewater. 22nd ed. American Public Health Association, American Water Works Association, Water Environment Federation publication. APHA, Washington D.C.
[2] Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, pp. 166–167.

Soil pH values varied from 7.8 to 7.9. This range is neutral to mildly alkaline.[3] These values do not particularly increase soil corrosivity.

The soluble salt content of the samples was low. Chloride and sulfate were found at low concentrations.

Nitrate was detected in low concentrations. Ammonium was not detected.

Tests were not made for sulfide and oxidation-reduction (redox) potential because these samples did not exhibit characteristics typically associated with anaerobic conditions.

This soil is classified as moderately corrosive to ferrous metals.

Corrosion Control Recommendations

The life of buried materials depends on thickness, strength, loads, construction details, soil moisture, etc., in addition to soil corrosivity, and is, therefore, difficult to predict. Of more practical value are corrosion control methods that will increase the life of materials that would be subject to significant corrosion.

The following recommendations are based on the soil conditions discussed in the Soil Corrosivity section above. Unless otherwise indicated, these recommendations apply to the entire site or alignment.

Steel Pipe

1. Underground steel pipe with rubber gasketed, mechanical, grooved end, or other nonconductive type joints should be bonded for electrical continuity. Electrical continuity is necessary for corrosion monitoring and the possible future application of cathodic protection.
2. Install corrosion monitoring test stations to facilitate corrosion monitoring and the possible future application of cathodic protection:
    a. At each end of the pipeline.
    b. At each end of all casings.
    c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.

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[3] *Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, p. 8.*

3. To prevent dissimilar metal corrosion cells and to facilitate the possible future application of cathodic protection, electrically isolate each buried steel pipeline per NACE SP0286 from:

    a. Dissimilar metals.

    b. Dissimilarly coated piping (cement-mortar vs. dielectric).

    c. Above ground steel pipe.

    d. All existing piping.

    Insulated joints should be placed above grade or in vaults where possible. Wrap all buried insulators with wax tape per AWWA C217.

4. Choose one of the following corrosion control options:

    OPTION 1

    a. Apply a suitable dielectric coating intended for underground use such as:

        i. Polyurethane per AWWA C222 *or*

        ii. Extruded polyethylene per AWWA C215 *or*

        iii. A tape coating system per AWWA C214 *or*

        iv. Hot applied coal tar enamel per AWWA C203 *or*

        v. Fusion bonded epoxy per AWWA C213.

    b. Although it is customary to cathodically protect bonded dielectrically coated structures, cathodic protection is not recommended at this time due to moderately corrosive soils. Joint bonds, test stations, and insulated joints should still be installed and will facilitate the application of cathodic protection in the future if needed to control leaks.

    OPTION 2

    As an alternative to dielectric coating and possible future cathodic protection, apply a ¾-inch cement mortar coating per AWWA C205 or encase in concrete three inches thick, using any type of ASTM C150 cement. Joint bonds, test stations, and insulated joints are still recommended for this alternative.

NOTE: Some steel piping systems, such as for oil, gas, and high-pressure piping systems, have special corrosion and cathodic protection requirements that must be evaluated for each specific application.

Hydraulic Elevators

1. Choose one of the following corrosion control options for the hydraulic steel cylinders.

   OPTION 1

   a. Coat hydraulic elevator cylinders with a suitable dielectric coating intended for underground use such as:

      i. Polyurethane per AWWA C222 or

      ii. Extruded polyethylene per AWWA C215 or

      iii. A tape coating system per AWWA C214 or

      iv. Hot applied coal tar enamel per AWWA C203 or

      v. Fusion bonded epoxy per AWWA C213.

   b. Electrically insulate each cylinder from building metals by installing dielectric material between the piston platen and car, insulating the bolts, and installing an insulated joint in the oil line.

   c. Apply cathodic protection to hydraulic cylinders as per NACE SP0169.

   OPTION 2

   As an alternative to electrical insulation and cathodic protection, place each cylinder in a plastic casing with a plastic watertight seal at the bottom.

2. The elevator oil line should be placed above ground if possible but, if underground, should be protected by one of the following corrosion control options:

   OPTION 1

   a. Provide a bonded dielectric coating.

   b. Electrically isolate the pipeline.

   c. Apply cathodic protection to steel piping as per NACE SP0169.

   OPTION 2

   Place the oil line in a PVC casing pipe with solvent-welded joints and sealed at both ends to prevent contact with soil and moisture.

Ductile Iron Pipe

1. To prevent dissimilar metal corrosion cells and to facilitate the possible future application of cathodic protection, electrically insulate underground iron pipe from dissimilar metals and from above ground iron pipe with insulating joints per NACE SP0286.
2. Bond all nonconductive type joints for electrical continuity. Electrical continuity is necessary for corrosion monitoring and possible future application of cathodic protection.
3. Install corrosion monitoring test stations to facilitate corrosion monitoring and the possible future application of cathodic protection:
    a. At each end of the pipeline.
    b. At each end of any casings.
    c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.
4. Choose one of the following corrosion control options:

    OPTION 1

    a. Apply a suitable coating intended for underground use such as:
        i. Polyethylene encasement per AWWA C105; *or*
        ii. Epoxy coating; *or*
        iii. Polyurethane; *or*
        iv. Wax tape.

        NOTE: The thin factory-applied asphaltic coating applied to ductile iron pipe for transportation and aesthetic purposes does not constitute a corrosion control coating.

    b. Although it is customary to cathodically protect coated structures, cathodic protection is not recommended at this time due to moderately corrosive soils. Joint bonds, test stations, and insulated joints should still be installed and will facilitate the application of cathodic protection in the future if needed to control leaks.

OPTION 2

As an alternative to the coating systems described in Option 1 and possible future cathodic protection, concrete encase all buried portions of metallic piping so that there is a minimum of three inches of concrete cover provided over and around surfaces of pipe, fittings, and valves using any type of ASTM C150 cement

NOTE: Some iron piping systems, such as for fire water piping, have special corrosion and cathodic protection requirements that must be evaluated for each specific application.

Cast Iron Soil Pipe

1. Protect cast iron soil pipe with either a double wrap 4-mil or single wrap 8-mil polyethylene encasement per AWWA C105.
2. It is not necessary to bond the pipe joints or apply cathodic protection.
3. Provide six inches of clean sand backfill all around the pipe.

Clean Sand Backfill

1. Clean sand backfill must have the following parameters:
   a. Minimum saturated resistivity of no less than 3,000 ohm-cm; *and*
   b. pH between 6.0 and 8.0.
2. All backfill testing should be performed by a corrosion engineering laboratory.

Copper Tubing

1. Electrically insulate underground copper pipe from dissimilar metals and from above ground copper pipe with insulating devices per NACE SP0286.
2. Electrically insulate cold water piping from hot water piping systems.
3. Place cold water copper tubing in an 8-mil polyethylene sleeve or encase in double 4-mil thick polyethylene sleeves and bed and backfill with clean sand at least two inches thick surrounding the tubing. Copper tubing for cold water can also be treated the same as for hot water.

4. Hot water tubing may be subject to a higher corrosion rate. Protect hot copper tubing by one of the following measures:
    a. Preventing soil contact. Soil contact may be prevented by placing the tubing above ground or encasing the tubing with PVC pipe with solvent-welded joints. *or*
    b. Applying cathodic protection per NACE SP0169. The amount of cathodic protection current needed can be minimized by coating the tubing.

Plastic and Vitrified Clay Pipe

1. No special corrosion control measures are required for plastic and vitrified clay piping placed underground.
2. Protect all metallic fittings and valves with wax tape per AWWA C217, or with epoxy and appropriately sized cathodic protection per NACE SP0169.

All Pipe

1. On all pipes, appurtenances, and fittings not protected by cathodic protection, coat bare metal such as valves, bolts, flange joints, joint harnesses, and flexible couplings with wax tape per AWWA C217 after assembly.
2. Where metallic pipelines penetrate concrete structures such as building floors, vault walls, and thrust blocks use plastic sleeves, rubber seals, or other dielectric material to prevent pipe contact with the concrete and reinforcing steel.

Concrete Structures and Pipe

1. From a corrosion standpoint, any type of ASTM C150 cement may be used for concrete structures and pipe because the sulfate concentration is negligible, from 0 to 0.10 percent.[4,5,6]
2. Standard concrete cover over reinforcing steel may be used for concrete structures and pipe in contact with these soils due to the low chloride concentrations[7] found onsite. Limit the water-soluble chloride ion content in the concrete mix design to less than 0.3 percent by weight of cement.

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[4] *2015 International Building Code (IBC) which refers to American Concrete Institute (ACI) 318-14 Table 19.3.2.1*
[5] *2015 International Residential Code (IRC) which refers to American Concrete Institute (ACI) 318-14 Table 19.3.2.1*
[6] *2016 California Building Code (CBC) which refers to American Concrete Institute (ACI) 318-14 Table 19.3.2.1*
[7] *Design Manual 303: Concrete Cylinder Pipe. Ameron. p.65*

Closure

The analysis and recommendations presented in this report are based upon data obtained from the laboratory samples. This report does not reflect variations that may occur across the site or due to the modifying effects of construction. If variations appear, HDR should be notified immediately so that further evaluation and supplemental recommendations can be provided.

HDR's services have been performed with the usual thoroughness and competence of the engineering profession. No other warranty or representation, either expressed or implied, is included or intended.

Please call if you have any questions.

Respectfully Submitted,
HDR Engineering, Inc.

James T. Keegan
Corrosion and Lab Services Section Manager

Marc E. N. Wegner, PE
Sr. Corrosion Project Manager

Enc: Table 1 – Laboratory Tests on Soil Samples

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Table 1 - Laboratory Tests on Soil Samples

Geotechnologies, Inc.
The John Buck Company
Your #22055, HDR Lab #20-0771SCS
19-Nov-20

Sample ID

			B1 @ 2.5'	B2 @ 30'
Resistivity		**Units**		
as-received		ohm-cm	44,000	6,000
saturated		ohm-cm	3,640	2,760
pH			7.9	7.8
Electrical Conductivity		mS/cm	0.12	0.06
Chemical Analyses				
Cations				
calcium	Ca^{2+}	mg/kg	37	36
magnesium	Mg^{2+}	mg/kg	6.3	10
sodium	Na^{1+}	mg/kg	91	29
potassium	K^{1+}	mg/kg	12	9.7
Anions				
carbonate	CO_3^{2-}	mg/kg	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	186	201
fluoride	F^{1-}	mg/kg	1.6	1.8
chloride	Cl^{1-}	mg/kg	14	4.5
sulfate	SO_4^{2-}	mg/kg	152	58
phosphate	PO_4^{3-}	mg/kg	ND	ND
Other Tests				
ammonium	NH_4^{1+}	mg/kg	ND	ND
nitrate	NO_3^{1-}	mg/kg	19	7.9
sulfide	S^{2-}	qual	na	na
Redox		mV	na	na

Resistivity per ASTM G187, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.
Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

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