APPENDIX A

Geotechnical Report

Geotechnologies, Inc.
Consulting Geotechnical Engineers

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

July 1, 2015
File Number 20971

Rodrigues Holdings, LLC
303 North Glenoaks Boulevard
Suite M180
Burbank, California 91502

Attention: Gary Rodrigues

Subject: Geotechnical Engineering Investigation
Proposed Mixed-Use Development
6001 - 6059 Van Nuys Boulevard, Van Nuys, California

Dear Mr. Rodrigues:

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

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

Should you have any questions please contact this office.

Respectfully submitted,
GEOTECHNOLOGIES, INC.

REGISTERED PROFESSIONAL ENGINEER
GREGORIO VARELA
No. 81201
Exp. 9/30/15
CIVIL
STATE OF CALIFORNIA

GREGORIO VARELA
R.C.E. 81201

GV:km

Distribution: (5) Addressee

Email to: [mwareham@mackurban.com]

TABLE OF CONTENTS

SECTION **PAGE**

TABLE OF CONTENTS

SECTION **PAGE**

TABLE OF CONTENTS

GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
6001 - 6059 VAN NUYS BOULEVARD
VAN NUYS, CALIFORNIA

INTRODUCTION

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

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

PROPOSED DEVELOPMENT

Information concerning the proposed development was obtained by review of the Architectural Plans (Progress Set) prepared by the Métier Group, Inc., dated June 22, 2015, as well as communication with the office of Mack Urban. The site is proposed to be developed with a mixed-use structure. The proposed structure will be five stories in height, built over a subterranean parking level. It is anticipated that the finished grade of the subterranean level will extend between 6 and 15 feet below the existing grade. The enclosed Plot Plan and cross section show the anticipated location, alignment, and depth of the proposed structure.

Column loads are estimated to be between 300 and 800 kips. Wall loads are estimated to be between 8 and 20 kips per lineal foot. These loads reflect the dead plus live load. It is anticipated that grading may consist of excavations to an approximated maximum depth of 20 feet for construction of the proposed subterranean level and foundation elements.

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

SITE CONDITIONS

The site located at 6001 through 6059 Van Nuys Boulevard, in the Van Nuys area of the City of Los Angeles, California. The site is rectangular in shape, and approximately 4 acres in area. The site is bounded by the Orange Line Busway to the north, Van Nuys Boulevard to the east, Oxnard Street to the south, and Vesper Avenue to the west. The site is currently bisected along the east-west direction by Aetna Street, which will be vacated and incorporated into the project site. The site is shown relative to nearby topographic features in the enclosed Vicinity Map.

The site is currently developed with an automobile dealership complex. The complex is composed of several one and two-story structures, as well as several asphalt-paved parking lots. It is anticipated that all existing structures will be demolished prior to construction.

The existing site's grade descends gently to the south. Based on the elevations contained in the Site Plan prepared by the Métier Group, Inc., dated June 22, 2015, there is an approximate elevation difference of 5 feet across the site, ranging from elevation 698 feet along the northern property line, to elevation 693 along the southern property line. The enclosed Cross Sections A-A' and B-B' illustrate the approximate existing grade elevation difference.

Vegetation on the site consists of mature trees, grass lawns, and bushes, contained in planter areas. Drainage across the site appears to be by sheetflow to the city streets to the south.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was explored on May 11 and 12, 2015 by excavating eight borings. The borings were drilled to depths ranging between 30 and 60 feet below the existing grade with the aid of a truck-mounted drilling machine using 8-inch diameter hollowstem augers. The exploration locations are shown on the enclosed Plot Plan and the geologic materials encountered are logged on Plates A-1 through A-8.

The location of exploratory excavations was determined from hardscape features shown on the attached Plot Plan. The location of the exploratory excavations should be considered accurate only to the degree implied by the method used.

Geologic Materials

Fill materials were encountered in all exploratory borings to depths ranging between 3 and 7½ feet below the existing grade. The fill consists of a mixture of clay, silt and sand, which ranges from yellowish brown to dark brown in color, and is moist, stiff, or medium dense to very dense, and fine grained.

The fill materials are in turn underlain by native alluvial soils, consisting of interlayered mixtures of sand, silt and clay. The native soils range from yellowish brown to dark brown in color, and are moist, medium dense to very dense, or stiff, and fine to coarse grained, with occasional gravel. More detailed descriptions of the earth materials encountered may be obtained from individual logs of the subsurface excavations.

Groundwater

Groundwater was not encountered during exploration, conducted to a maximum depth of 60 feet below the existing site grade. The historically highest groundwater level was established by review of the Van Nuys 7½ Minute Quadrangle Seismic Hazard Evaluation Report, Plate 1.2, Historically Highest Ground Water Contours (CDMG, 2005). Review of this plate indicates that the historically highest groundwater level at the site was on the order of 15 feet below grade. A copy of this plate is included in the Appendix as the Historically Highest Groundwater Levels Map.

Based on the elevations contained in the Site Plan prepared by the Métier Group, Inc., dated June 22, 2015, the existing site grade varies between elevations 693 and 698 feet. Based on an average site elevation of 695.5 feet, it is the opinion of this firm that the historically highest groundwater level at the site may be considered to correspond to elevation 680.5 feet.

Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, and other factors not evident at the time of the measurements reported herein. Fluctuations also may occur across the site. High groundwater levels can result in changed conditions.

Caving

Caving could not be directly observed during exploration due to the type of excavation equipment utilized. However, based on the experience of this firm, large diameter excavations that encounter granular, cohesionless soils 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.

SEISMIC HAZARDS AND DESIGN CONSIDERATIONS

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

Surface Rupture

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

CGS policy is to delineate a boundary from 200 to 500 feet wide on each side of the known fault trace based on the location precision, the complexity, or the regional significance of the fault. If a site lies within an Earthquake Fault Zone, a geologic fault rupture investigation must be performed that demonstrates that the proposed building site is not threatened by surface displacement from the fault before development permits may be issued.

Ground surface rupture is defined as surface displacement which occurs along the surface trace of the causative fault during an earthquake. Based on research of available literature, no known active or potentially active faults underlie the subject site.

Based on these considerations, the potential for surface ground rupture at the subject site is considered low.

2013 California Building Code Seismic Parameters

Based on information derived from the subsurface investigation, the subject site is classified as Site Class D, which corresponds to a "Stiff Soil" Profile, according to Table 20.3-1 of ASCE 7-10. This information and the site coordinates were input into the USGS U.S. Seismic Design Maps tool (Version 3.1.0) to calculate the ground motions for the site.

2013 CALIFORNIA BUILDING CODE SEISMIC PARAMETERS	
Site Class	D
Mapped Spectral Acceleration at Short Periods (S_S)	2.231g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	2.231g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.487g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.776g
Site Coefficient (F_v)	1.5
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	1.165g
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.776g

Liquefaction

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

The Seismic Hazard Map for the Van Nuys Quadrangle by the State of California (CDMG, 1998), classifies the site as part of a "Liquefiable" area. This determination is based on historic groundwater depth records, soil type, and distance to a fault capable of producing a substantial earthquake. A copy of this map has been included in the Appendix.

Two site-specific liquefaction analyses were performed following the Recommended Procedures for Implementation of the California Geologic Survey Special Publication 117A, Guidelines for Analyzing and Mitigating Seismic Hazards in California (CGS, 2008). The enclosed liquefaction analyses were performed using the spreadsheet template LIQ2_30.WQ1 developed by Thomas F. Blake (Blake, 1996). This program utilizes the 1996 NCEER method of analysis. This semi-empirical method is based on a correlation between measured values of Standard Penetration Test (SPT) resistance and field performance data.

Groundwater was not encountered during exploration to a depth of 60 feet below the ground surface. According to the Seismic Hazard Zone Report of the Van Nuys 7½-Minute Quadrangle (CDMG, 1998), the historically highest groundwater level for the site was on the order of 15 feet below ground surface. The historically highest groundwater level was conservatively utilized for the enclosed liquefaction analyses.

The peak ground acceleration (PGA_M) and modal magnitude were obtained from the USGS websites, using the Probabilistic Seismic Hazard Deaggregation program (USGS, 2008) and the U.S. Seismic Design Maps tool (USGS, 2013). A Site Class "D" and a published shear wave velocity of 230 meters per second were utilized for Vs30 (Tinsley and Fumal, 1985) in the USGS seismic programs. A modal magnitude (M_W) of 6.6 is obtained using the USGS Probabilistic Seismic Hazard Deaggregation program (USGS, 2008). Peak ground acceleration of 0.79g was obtained using the U.S. Seismic Design Map Tools. These parameters are used in the enclosed liquefaction analyses.

The enclosed two "Empirical Estimation of Liquefaction Potential" calculation sheets are based on Borings 3 and Boring 7. Standard Penetration Test (SPT) data were collected at 5-foot intervals. Samples of the collected materials were conveyed to the laboratory for testing and analysis. Fines content, as defined by percentage passing the #200 sieve, were utilized for the fines correction factor in computing the corrected blow count of selected soil layers. Fine contents of selected samples are presented in Plate E of this report. In addition, Atterberg Limit tests were performed for selected samples and the results are presented in Plates F of this report.

The procedure presented in the SP117A guidelines was followed in analyzing the liquefaction potential of the subject site. The SP 117A guidelines were developed based on a paper titled, "Assessment of the Liquefaction Susceptibility of Fine-Grained Soils", by Bray and Sancio (2006). According to the SP117A, soils having a Plastic Index greater than 18 exhibit clay-like behavior, and the liquefaction potential of these soils are considered to be low. Therefore, where the results of Atterberg Limits testing showed a Plastic Index greater than 18, the soils would be considered non-liquefiable, and the analysis of these soil layers was turned off in the liquefaction susceptibility column.

The site-specific liquefaction analyses included in the Appendix, indicate that the site soils would not be prone to liquefaction during the ground motion expected during the design-based seismic event.

Dynamic Dry Settlement

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

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

Tsunamis, Seiches and Flooding

Tsunamis are large ocean waves generated by sudden water displacement caused by a submarine earthquake, landslide, or volcanic eruption. Review of the County of Los Angeles Flood and Inundation Hazards Map, Leighton (1990), indicates the site does not lie within the mapped tsunami inundation boundaries. The site is far and/or high enough from the ocean or lakes such that it would not be prone to hazards of a tsunami or seiche.

Review of the County of Los Angeles Flood and Inundation Hazards Map (Leighton, 1990), indicates the site lies within inundation boundaries of reservoirs within the San Fernando Valley region. A determination of whether a higher site elevation would remove the site from the potential inundation zones is beyond the scope of this investigation.

Landsliding

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

CONCLUSIONS AND RECOMMENDATIONS

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

Fill materials were encountered during exploration to depths ranging between 3 and 7½ feet below the existing site grade. The existing fill materials are unsuitable for support of new foundations and concrete slabs-on-grade. It is anticipated that the existing fill will be removed during excavation for the proposed subterranean level, which is expected to extend to depths ranging between 6 and 15 feet below the existing site grade. The proposed structure may be supported by conventional foundations bearing in the native alluvial soils expected at the subgrade of the proposed subterranean levels.

In the event that fill materials are encountered within portions of the exposed subterranean subgrade, the proposed foundations shall be deepened through the existing fill to bear in undisturbed native soils, and the fill shall be completely removed and recompacted for support of the new concrete slab-on-grade.

Any existing or abandoned utilities located within the footprint of the proposed structure should be removed or relocated as appropriate. This includes any utilities located below the portion of Aetna Street which will be vacated and incorporated into the site. In the event that relocating some of these utilities is not possible, this firm shall be contacted so the appropriate recommendations are provided.

Based on the elevations contained in the Site Plan prepared by the Métier Group, Inc., dated June 22, 2015, the existing site grade varies between elevations 693 and 698 feet. Based on an average site elevation of 695.5 feet, it is the opinion of this firm that the historically highest groundwater level at the site may be considered to correspond to elevation 680.5 feet. As illustrated in the enclosed Cross Section A-A', the finished floor elevation of proposed subterranean level is not expected to extend below elevation 682 feet. It is the opinion of this firm that the groundwater level would not be expected to rise to the finished grade of the proposed subterranean level during the life of the structure. Permanent dewatering is not anticipated for the proposed structure.

The proposed subterranean level will extend adjacent to the property lines. Therefore, where a temporary sloped embankment is not possible, the excavation for the proposed subterranean level will require temporary shoring in order to provide a stable vertical excavation. Shoring recommendations are provided in the "Excavations" section of this report.

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 excavations on the site as indicated and should in no way be construed to reflect any variations which may occur between these excavations or which may result from changes in subsurface conditions. Any changes in the design, 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.

EXPANSIVE SOILS

The onsite geologic materials are in the low to moderate expansion range. The Expansion Index was found to be 40 and 56 for representative bulk samples. Recommended reinforcing is provided in the "Foundation Design" and "Slab-On-Grade" sections of this report.

WATER-SOLUBLE SULFATES

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

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

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

CITY OF LOS ANGELES METHANE ZONE

Based on review of the NavigateLA Website, developed by the City of Los Angeles, Bureau of Engineering, Department of Public Works, the subject site is not located within the limits of a City of Los Angeles Methane Zone or Methane Buffer Zone.

GRADING GUIDELINES

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

Site Preparation

- 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 should be mechanically compacted in layers not more than 8 inches thick. Based on the moderate expansion index of some of the site soils, it is recommended that fill materials are moisture conditioned to approximately 3 percent over optimum moisture content before recompaction.

The City of Los Angeles Department of Building and Safety requires a minimum comparative compaction of 95 percent of the laboratory maximum density where the soils to be utilized in the fill have less than 15 percent finer than 0.005 millimeters. Fill materials having more than 15 percent finer than 0.005 millimeters may be compacted to a minimum of 90 percent of the maximum density. The maximum density shall be determined by the laboratory operated by Geotechnologies, Inc. using the test method described in the most recent revision of ASTM D 1557.

Field observation and testing shall be performed by a representative of the geotechnical engineer during grading to assist the contractor in obtaining the required degree of compaction and the proper moisture content. Where compaction is less than required, additional compactive effort shall be made with adjustment of the moisture content, as necessary, until a minimum of 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) compaction is obtained.

Acceptable Materials

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

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

Utility Trench Backfill

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

Wet Soils

At the time of exploration the soils which will be exposed at the bottom of the excavation were locally above optimum moisture content.

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

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

Shrinkage

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

Weather Related Grading Considerations

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

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

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

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

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

Conventional

The proposed structure may be supported by conventional foundations bearing in the native alluvial soils expected at the subgrade of the proposed subterranean levels. In the event that fill materials are exposed at areas of the subterranean subgrade, the proposed foundations shall be deepened through the fill to bear in undisturbed alluvial soils. Where foundations are deepened, the deepened portion should consist of hard rock concrete having the same strength as the planned structural footing.

Continuous foundations may be designed for a bearing capacity of 2,250 pounds per square foot, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material.

Column foundations may be designed for a bearing capacity of 2,750 pounds per square foot, and should be a minimum of 24 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material.

The bearing capacity increase for each additional foot of width is 50 pounds per square foot. The bearing capacity increase for each additional foot of depth is 250 pounds per square foot. The maximum recommended bearing capacity is 5,000 pounds per square foot.

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

Foundation Reinforcement

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

Lateral Design

Resistance to lateral loading may be provided by friction acting at the base of foundations and by passive earth pressure. An allowable coefficient of friction of 0.27 may be used with the dead load forces.

Passive geologic pressure for the sides of foundations poured against undisturbed or recompacted soil may be computed as an equivalent fluid having a density of 300 pounds per cubic foot with a maximum earth pressure of 1,500 pounds per square foot.

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

Foundation Settlement

Settlement of the foundation system is expected to occur on initial application of loading. The maximum settlement is expected to be 1 inch and occur below the heaviest loaded columns. Differential settlement is not expected to exceed ¼-inch.

Foundation Observations

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

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

RETAINING WALL DESIGN

Based on the anticipated depth of the subterranean level relative to the existing site grade, it is anticipated that retaining walls up to 15 feet in height may be required for the project. As a precautionary measure, recommendations for the design of underground retaining walls up to a height of 17 feet have been provided herein. Retaining walls may be designed as indicated below, depending on whether the walls will be restrained or cantilevered. Retaining wall foundations may be designed in accordance with the provisions of the "Foundation Design" section of this report.

Additional pressure should be added for a surcharge condition due to vehicular traffic or adjacent structures. Based on review of the enclosed Plot Plan, it is not anticipated that the proposed retaining walls will be surcharged by existing structures. However, vehicular traffic is expected in the vicinity of the proposed structure. For traffic surcharge, the upper 10 feet of any 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 traffic surcharge. If the traffic is more than 10 feet from the retaining walls, the traffic surcharge may be neglected.

Cantilever Retaining Walls

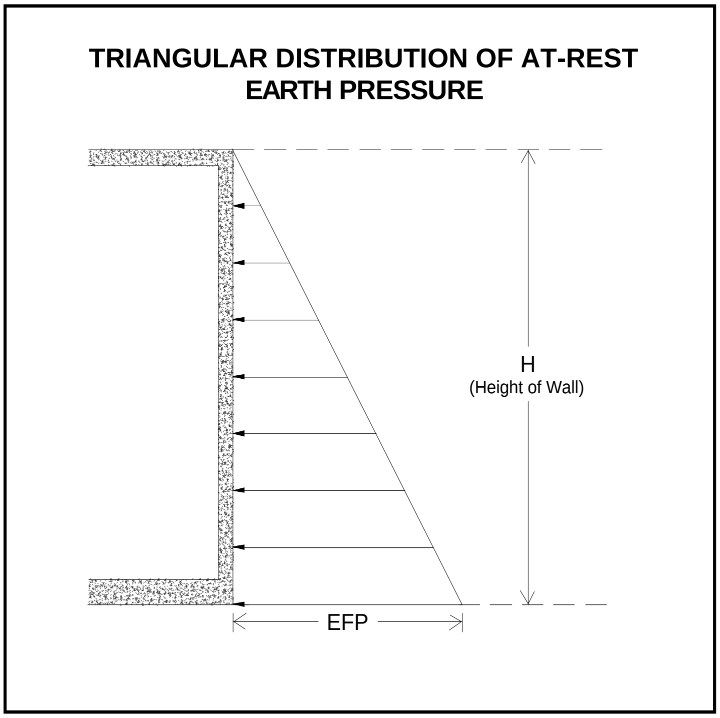
Retaining walls supporting a level backslope may be designed utilizing a triangular distribution of pressure. Cantilever retaining walls may be designed utilizing the following table:

HEIGHT OF WALL (feet)	EQUIVALENT FLUID PRESSURE (pounds per cubic foot)
Up to 9	30
9 to 12	39
12 to 17	48

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

Restrained Drained Retaining Walls

Restrained retaining walls may be designed to resist a triangular pressure distribution of at-rest earth pressure as indicated in the diagram below. For the purpose of designing restrained retaining walls up to 17 feet in height, the at-rest pressure would be 79 pounds per cubic foot.



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

Dynamic (Seismic) Earth Pressure

Retaining walls exceeding 6 feet in height shall be designed to resist the additional earth pressure caused by seismic ground shaking. A triangular pressure distribution should be utilized for the additional seismic loads, with an equivalent fluid pressure of 22 pounds per cubic foot. When using the load combination equations from the building code, the seismic earth pressure should be combined with the lateral active earth pressure for analyses of restrained basement walls under seismic loading condition. The dynamic earth pressure may be omitted where the retaining wall is 6 feet in height or less.

Surcharge from Adjacent Structures

Based on review of the enclosed Plot Plan, it is not anticipated that the proposed retaining walls, or temporary shoring walls, will be surcharged by existing structures. In the event that new retaining walls or temporary shoring walls will be surcharged by neighboring structures, the following surcharge equation provided in the LADBS Information Bulletin Document No. P/BC 2008-83, may be utilized to determine the surcharge loads on basement walls and shoring system for existing structures located within the 1:1 (h:v) surcharge influence zone of the excavation and basement.

Resultant lateral force: $R = (0.3*P*h^2)/(x^2+h^2)$

Location of lateral resultant: $d = x*[(x^2/h^2+1)*\tan^{-1}(h/x)-(x/h)]$

where:

R	=	resultant lateral force measured in pounds per foot of wall width.
P	=	resultant surcharge loads of continuous or isolated footings measured in pounds per foot of length parallel to the wall.
x	=	distance of resultant load from back face of wall measured in feet.
h	=	depth below point of application of surcharge loading to top of wall footing measured in feet.
d	=	depth of lateral resultant below point of application of surcharge loading measure in feet.
$\tan^{-1}(h/x)$	=	the angle in radians whose tangent is equal to h/x.

The structural engineer and shoring engineer may use this equation to determine the surcharge loads based on the loading of the adjacent structures located within the surcharge influence zone.

Retaining Wall Drainage

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

As an alternative to the standard perforated subdrain pipe and gravel drainage system, the use of gravel pockets and weepholes is an acceptable drainage method. Weepholes shall be a minimum of 2 inches in diameter, placed at 8 feet on center along the base of the wall. Gravel pockets shall be a minimum of 1 cubic foot in dimension, and may consist of three-quarter inch to one inch crushed rocks, wrapped in filter fabric. A collector pipe shall be installed to direct collected waters to a sump

Certain types of subdrain pipe are not acceptable to the various municipal agencies, it is recommended that prior to purchasing subdrainage pipe, the type and brand is cleared with the proper municipal agencies. Subdrainage pipes should outlet to an acceptable location. Some municipalities do not allow the use of flat-drainage products, such as Miradrain. The use of such a product should be researched with the building official. The City of Los Angeles only allows the use of flat drainage products when in conjunction with a conventional perforated subdrain pipe and gravel, or gravel pockets and weepholes.

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

Sump Pump Design

The purpose of the recommended retaining wall backdrainage system is to relieve hydrostatic pressure. Groundwater was not encountered during exploration, conducted to a depth of 60 feet below the existing grade. As discussed previously in this report, the groundwater level is not expected to rise to the finished grade of the proposed subterranean level during the life of the structure. Therefore the only water which could affect the proposed retaining walls would be irrigation water and precipitation. Additionally, the proposed site grading is such that all drainage is directed to the street and the structure has been designed with adequate non-erosive drainage devices.

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

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 (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) relative compaction, obtainable by the most recent revision of ASTM D 1557 method of compaction. Flooding should not be permitted. Compaction within 5 feet, measured horizontally, behind a retaining structure should be achieved by use of light weight, hand operated compaction equipment.

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

TEMPORARY EXCAVATIONS

Excavations up to a maximum depth of 20 feet below the existing grade may be anticipated for construction of the proposed subterranean level and foundation elements. The excavations are expected to expose fill and dense native soils, which are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic or structures. Vertical excavations exceeding 5 feet, or excavations which will be surcharged by adjacent traffic or structures should be shored.

Where sufficient space is available, temporary unsurcharged embankments could be cut at a uniform 1:1 slope gradient to a maximum depth of 20 feet. A uniform sloped excavation is sloped from bottom to top and does not have a vertical component.

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

Excavation Observations

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

SHORING DESIGN

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

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

Soldier Piles

Drilled cast-in-place soldier piles should be placed no closer than 2 diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the earth materials. For design purposes, an allowable passive value for the earth materials below the bottom plane of excavation may be assumed to be 500 pounds per square foot per foot. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed earth materials.

Groundwater was not encountered during exploration to a depth of 60 feet below grade. Proposed shoring pile excavations are not anticipated to encounter water. Caving may be experienced within the granular soil layers. If caving is experienced during drilling, casing should be used. If casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet.

The frictional resistance between the soldier piles and retained earth material may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.27 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 500 pounds per square foot. The minimum depth of embedment for shoring piles is 5 feet below the bottom of the footing excavation or 5 feet below the bottom of excavated plane whichever is deeper.

<u>Lagging</u>

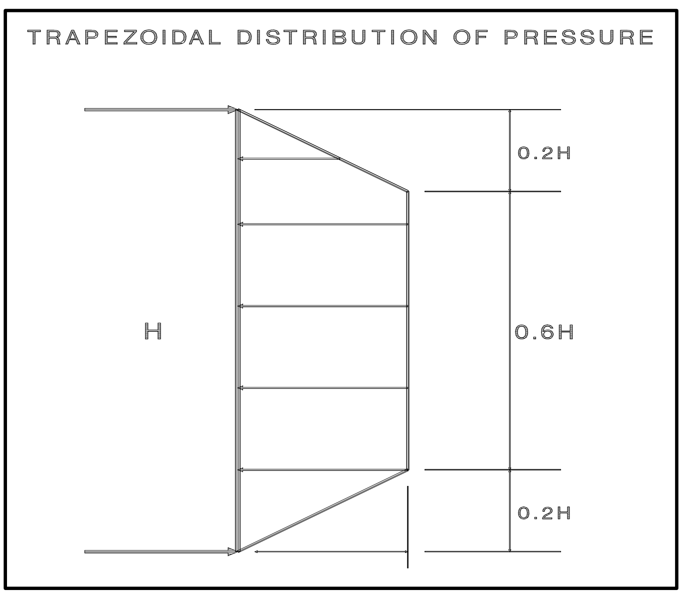
Soldier piles and anchors should be designed for the full anticipated pressures. Due to arching in the geologic materials, the pressure on the lagging will be less. It is recommended that the lagging should be designed for the full design pressure but be limited to a maximum of 400 pounds per square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

Lateral Pressures

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

HEIGHT OF SHORING "H" (feet)	EQUIVALENT FLUID PRESSURE (pounds per cubic foot)
Up to 12	28
12 to 15	35
15 to 20	42

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



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

HEIGHT OF SHORING "H" (feet)	DESIGN SHORING FOR (Where H is the height of the wall)
Up to 12	18H
12 to 15	22H
15 to 20	27H

Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination. Additional active pressure should be applied where the shoring will be surcharged by adjacent traffic or structures. It is anticipated that the proposed shoring walls will be surcharged by neighboring structures to the north, east and south. Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination.

Tied-Back Anchors

Tied-back anchors may be used to resist lateral loads. Friction anchors are recommended. For design purposes, it may be assumed that the active wedge adjacent to the shoring is defined by a plane drawn 35 degrees with the vertical through the bottom plane of the excavation. Friction anchors should extend a minimum of 20 feet beyond the potentially active wedge. Anchors should be placed at least 6 feet on center to be considered isolated.

Drilled friction anchors constructed without utilizing pressure-grouting techniques may be designed for a skin friction of 350 pounds per square foot. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. Where belled anchors are utilized, the capacity of belled anchors may be designed by applying the skin friction over the surface area of the bonded anchor shaft. The diameter of the bell may be utilized as the diameter of the bonded anchor shaft when determining the surface area. This implies that in order for the belled anchor to fail, the entire parallel soil column must also fail.

Depending on the techniques utilized, and the experience of the contractor performing the installation, it is anticipated that a skin friction of 2,000 pounds per square foot could be utilized for post-grouted anchors, provided the design does not rely on end-bearing plates to provide the necessary capacity. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads.

Anchor Installation

Tied-back anchors may be installed between 20 and 40 degrees below the horizontal. Where caving of the anchor shafts is experienced, 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.

Tieback Anchor Testing

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

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

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

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

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

Internal Bracing

Rakers may be utilized to brace the soldier piles in lieu of tieback anchors. The raker bracing could be supported laterally by temporary concrete footings (deadmen) or by the permanent interior footings. An allowable bearing pressure of 3,000 pounds per square foot may be used for the design a raker foundations. This bearing pressure is based on a raker foundation a minimum of 18 inches in width and length as well as 18 inches in depth into native alluvial soils. The base of the raker foundations should be horizontal. Care should be employed in the positioning of raker foundations so that they do not interfere with the foundations for the proposed structure.

Deflection

The City of Los Angeles Department of Building and Safety requires limiting shoring deflection to ½ inch at the top of the shored embankment where a structure is within a 1:1 plane projected up from the base of the excavation. A maximum deflection of 1-inch is allowed provided there are no structures within a 1:1 plane drawn upward from the base of the excavation. If the observed deflection is greater than the criteria established by the City of Los Angeles Department of Building and Safety, additional bracing for shoring walls 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.

Monitoring

Because of the depth of the excavation, some means of monitoring the performance of the shoring system is suggested. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all soldier piles and the lateral movement along the entire lengths of selected soldier piles. Also, some means of periodically checking the load on selected anchors will be necessary, where applicable.

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

Shoring Observations

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

SLABS ON GRADE

Concrete Slabs-on Grade

Concrete floor slabs should be a minimum of 5 inches in thickness. Slabs-on-grade should be cast over undisturbed native alluvial soils or properly controlled fill materials. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum dry density.

Outdoor concrete flatwork should be a minimum of 4 inches in thickness. Outdoor concrete flatwork should be cast over undisturbed native alluvial soils or properly controlled fill materials. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum dry density.

Design of Slabs That Receive Moisture-Sensitive Floor Coverings

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

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

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

Where a vapor retarder is used, a low-slump concrete should be used to minimize possible curling of the slabs. The barrier can be covered with a layer of trimable, compactible, granular fill, where it is thought to be beneficial. Where a granular fill layer is used, this layer should be a minimum of 2 inches in thickness. See ACI 302.2R-32, Chapter 7 for information on the placement of vapor retarders and the use of a fill layer.

Concrete Crack Control

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

For standard control of concrete cracking, a maximum crack control joint spacing of 12 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended. The crack control joints should be installed as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. Construction joints should be designed by a structural engineer.

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

Slab Reinforcing

Concrete slabs-on-grade should be reinforced with a minimum of #4 steel bars on 16-inch centers each way. Outdoor flatwork should be reinforced with a minimum of #3 steel bars on 18-inch centers each way.

PAVEMENTS

Prior to placing paving, the existing grade should be scarified to a depth of 12 inches, moistened as required to obtain optimum moisture content, and recompacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) relative compaction, as determined by the most recent revision of ASTM D 1557. The client should be aware that removal of all existing fill in the area of new paving is not required, however, pavement constructed in this manner will most likely have a shorter design life and increased maintenance costs. The following pavement sections are recommended:

Service	**Asphalt Pavement Thickness Inches**	**Base Course Inches**
Passenger Car Traffic	3	6
Moderate Truck Traffic	4	8

Concrete paving may also be used on the project. For passenger cars and moderate truck traffic, concrete paving should be 6 inches of concrete over 4 inches of compacted base. For standard crack control maximum expansion joint spacing of 12 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended. Concrete paving should be reinforced with a minimum of #3 steel bars on 18-inch centers each way.

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

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

SITE DRAINAGE

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

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

STORMWATER DISPOSAL

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

At this time, stormwater infiltration at the subject site has not been proposed, and percolation testing has not been performed by this firm. It is recommended that this office is notified should stormwater infiltration be considered for the proposed project so that adequate testing and recommendations are provided.

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

DESIGN REVIEW

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

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

If corrosion sensitive improvements are planned, it is recommended that a comprehensive corrosion study should be commissioned. The study will develop recommendations to avoid premature corrosion of buried pipes and concrete structures in direct contact with the soils.

GEOTECHNICAL TESTING

Classification and Sampling

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

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

Moisture and Density Relationships

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

Direct Shear Testing

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

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

Consolidation Testing

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

Expansion Index Testing

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

Grain Size Distribution

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

The most recent revision of ASTM D 422 is used to determine particle sizes smaller than the Number 200 sieve. The grain size distributions are plotted on the E-Plate presented in the Appendix of this report.

Atterberg Limits

ASTM D 4318 is used to determine the liquid limits, plastic limits, and plasticity index of a soil. These test methods are used to characterize the fine grained fractions of the soil. Results from Atterberg Limits tests are presented in the F-Plates of this report.

REFERENCES

Blake, T.F., 1994-1996, LIQ2_30.WQ1 - A QUATTRO-PRO Spreadsheet Computer Program for the determination of liquefaction potential using the NCEER (1996) semi-empirical method.

Bray, J. D., Sancio, R. B., 2006, Assessment of the Liquefaction Susceptibility of Fine-Grained Soils, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 132, No. 9, pp. 1165-1177.

California Department of Conservation, Division of Mines and Geology, 1997 (Revised 2005), Seismic Hazard Zone Report of the Nan Nuys 7½-Minute Quadrangle, Los Angeles County, California., C.D.M.G. Seismic Hazard Zone Report 08, map scale 1:24,000.

California Department of Conservation, Division of Mines and Geology, 1998, Seismic Hazard Zones Map, Hollywood 7½-minute Quadrangle, CDMG Seismic Hazard Zone Mapping Act of 1990.

California Geological Survey, 2008, Guidelines for Evaluation and Mitigation of Seismic Hazards in California, Special Publication 117A.

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

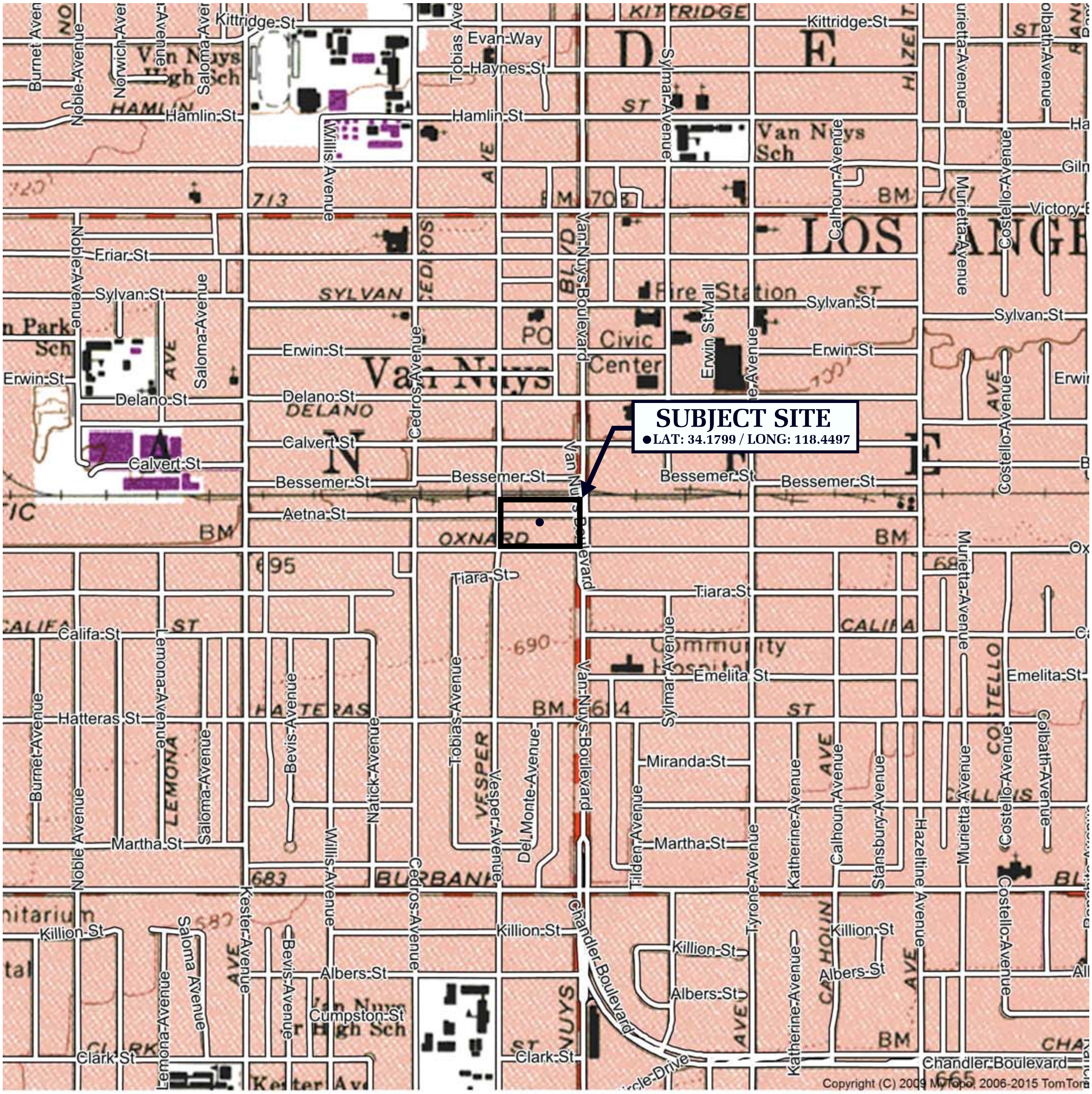
Department of Water Resources, 1961, Planned Utilization of the Ground Water Basins of the Coastal Plain of Los Angeles County, Bulletin 104, Appendix A, 181 pages.

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

Tinsley, J.C., and Fumal, T.E., 1985, Mapping Quaternary Sedimentary Deposits for Areal Variations in Shaking Response, in Evaluation Earthquake Hazards in the Los Angeles Region-An Earth Science Perspective, U.S. Geological Survey Professional Paper 1360, Ziony, J.I. ed., pp 101-125.

United States Geological Survey, 2008, U.S.G.S. Interactive Deaggregation Program. http://eqint.cr.usgs.gov/deaggint/2008/index.php.

United States Geological Survey, 2013, U.S.G.S. U.S. Seismic Design Maps tool (Version 3.1.0). http://geohazards.usgs.gov/designmaps/us/application.php



SCALE 1:12000

0.0 0.1 0.2 0.3 0.4 0.5 0.6
MILES

0 1000 2000 3000
FEET

N

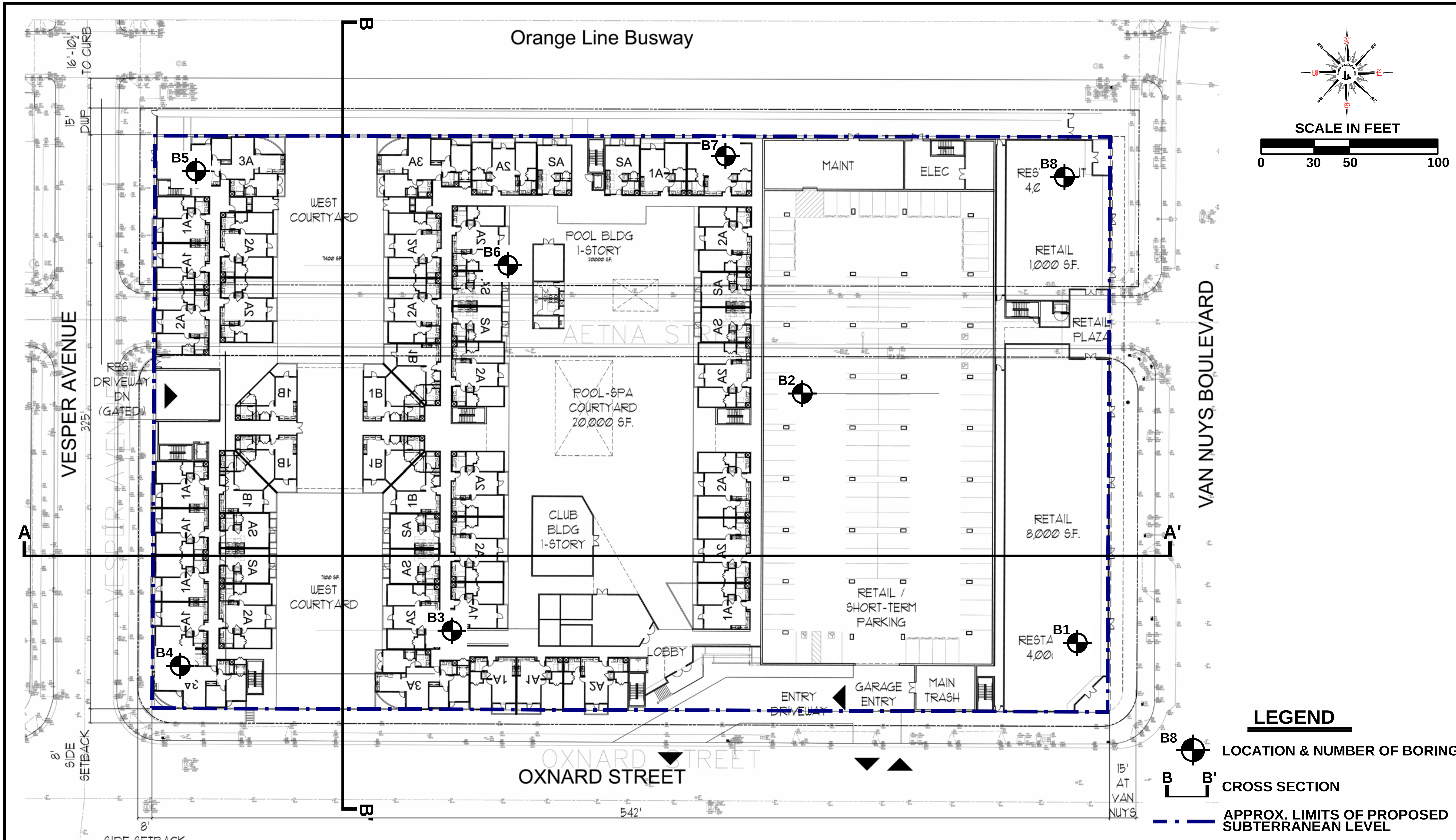
REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES,
VAN NUYS, CA QUADRANGLE

VICINITY MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

RODRIGUES HOLDINGS

FILE NO. 20971



LEGEND

B8 — LOCATION & NUMBER OF BORING

B B' — CROSS SECTION

APPROX. LIMITS OF PROPOSED SUBTERRANEAN LEVEL

REFERENCE: SITE PLAN BY METIER DATED 6/22/15

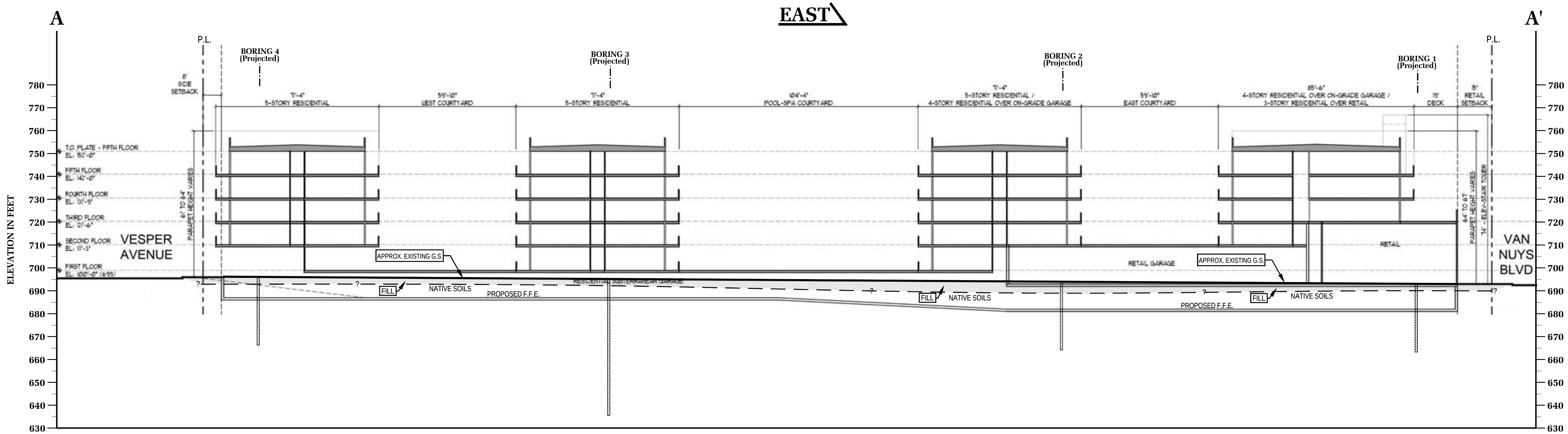
Geotechnologies, Inc.
Consulting Geotechnical Engineers

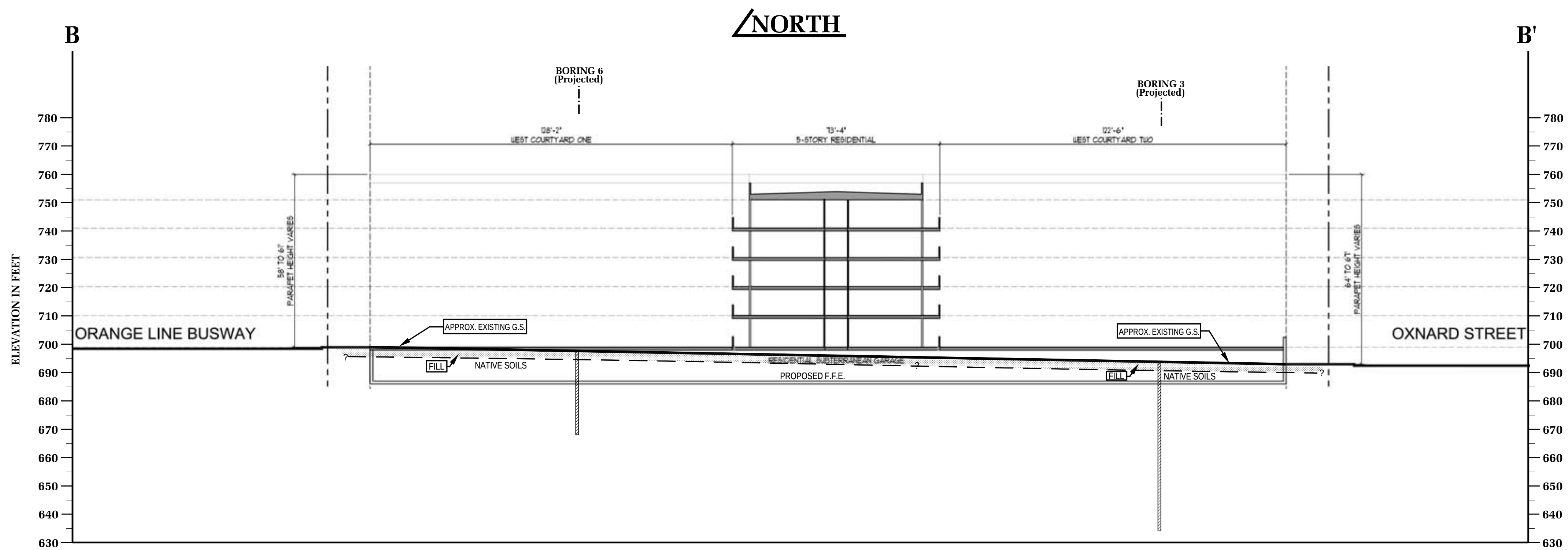
PLOT PLAN

RODRIGUES HOLDINGS

File No.: 20971

Date: June '15





SCALE IN FEET

0 30 50 100

REFERENCE: SITE PLAN BY METIER DATED 6/22/15

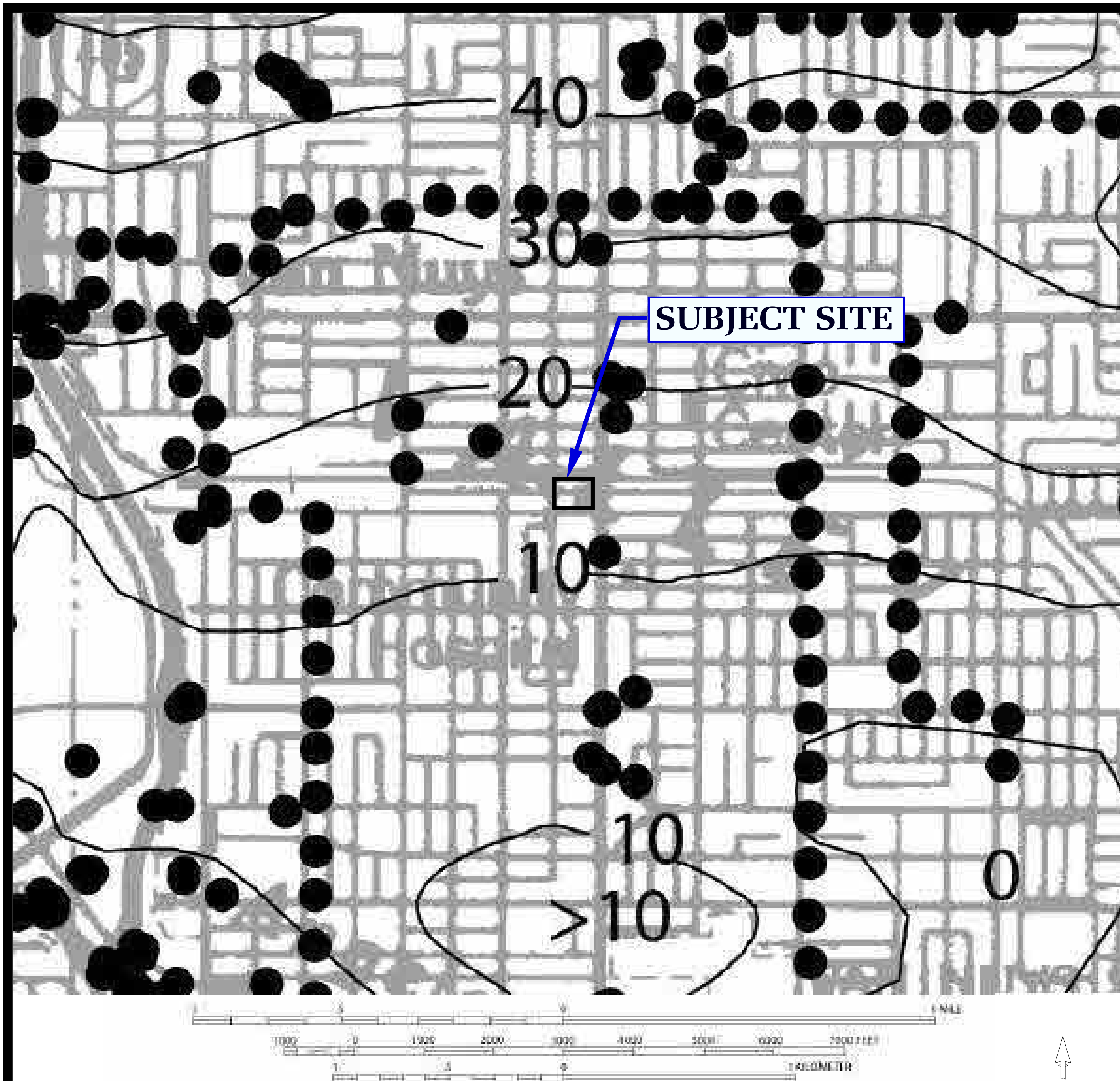
Geotechnologies, Inc.
Consulting Geotechnical Engineers

CROSS SECTIONS A-A' AND B-B'

RODRIGUES HOLDINGS

File No.: 20971

Date: June '15



30 Depth to groundwater in feet

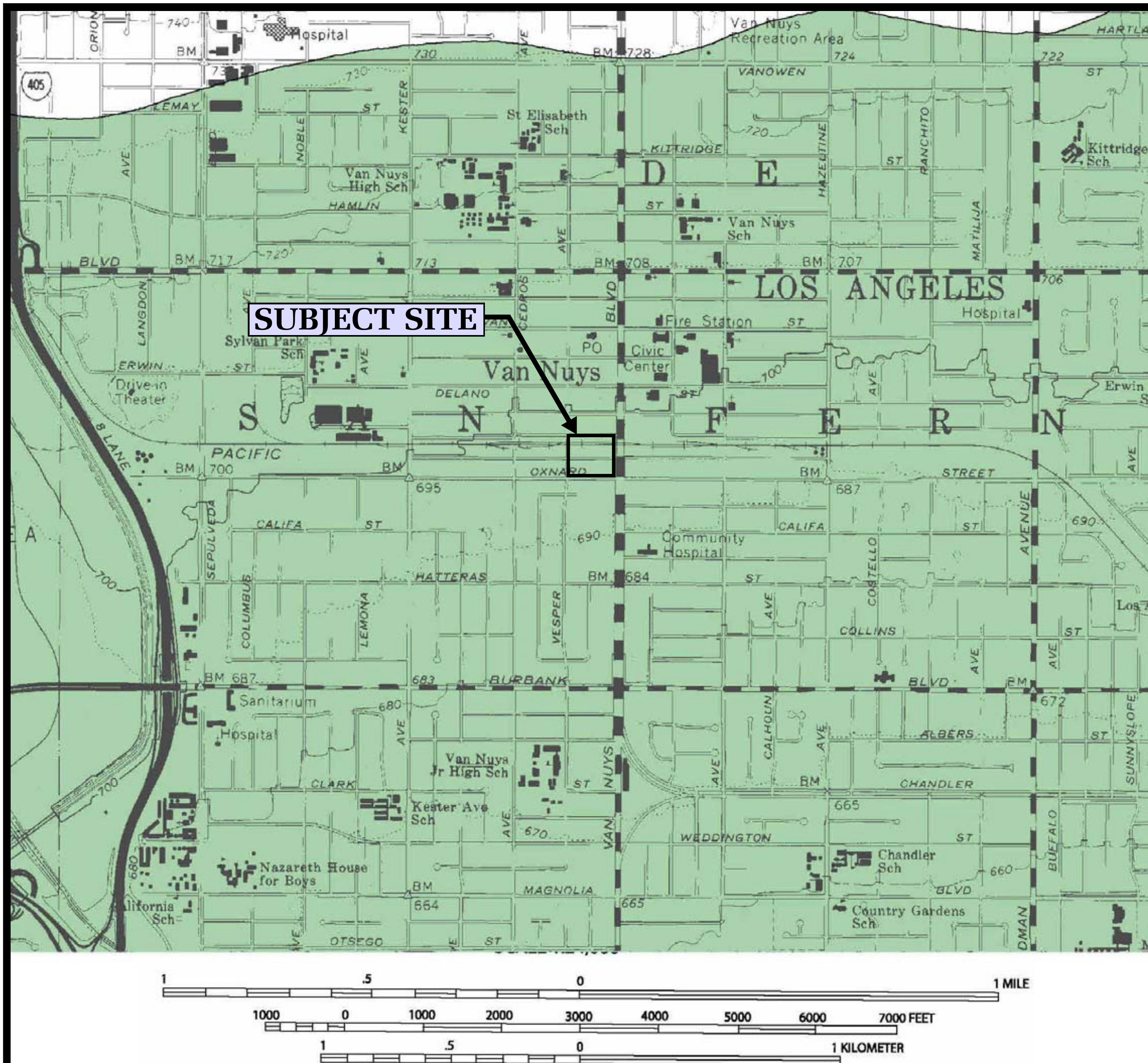
REFERENCE: CDMG, SEISMIC HAZARD ZONE REPORT, 08
VAN NUYS 7.5 - MINUTE QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA (1997, REVISED 2005)

HISTORICALLY HIGHEST GROUNDWATER LEVELS

Geotechnologies, Inc.
Consulting Geotechnical Engineers

RODRIGUES HOLDINGS

FILE NO. 20971



LIQUEFACTION AREA

REFERENCE: SEISMIC HAZARD ZONES, VAN NUYS QUADRANGLE OFFICIAL MAP (CDMG, 1998)

SEISMIC HAZARD ZONE MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

RODRIGUES HOLDINGS

FILE NO. 20971

BORING LOG NUMBER 1

Rodrigues Holdings, LLC **Date: 05/11/15**

File No. 20971 **Method: 8-inches Diameter Hollow Stem Auger**

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		3-inches Asphalt over 5-inches Base
				-		
				1 --		FILL: Sandy Silt, dark brown, moist, stiff
				-		
				2 --		
2.5	10	9.3	107.4	-		
				3 --		
				-	ML	NATIVE SOILS: Sandy Silt, dark brown, moist, stiff
				4 --		
				-		
5	12	13.2	95.3	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	16	15.6	93.9	-		
				8 --		
				-		
				9 --		
				-		
10	15	16.2	100.6	10 --		
				-	ML/CL	Sandy Silt to Sandy Clay, medium brown, moist, stiff
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	22	8.7	130.0	15 --		
				-	SM	Silty Sand, dark brown, moist, medium dense, fine grained
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	59	8.5	132.8	20 --		
				-		dark to grayish brown, dense
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	40	16.0	120.3	25 --		
				-		dark brown, medium dense, minor gravel

BORING LOG NUMBER 1

Rodrigues Holdings, LLC

File No. 20971

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-	SP	Sand, dark brown, moist, dense, fine to medium grained
30	64	2.0	119.2	30 --		
				-		Total Depth 30 feet
				31 --		No Water
				-		Fill to 3 feet
				32 --		
				-		
				33 --		
				-		
				34 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				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 --		
				-		
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
				45 --		
				-		
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
				50 --		
				-		

BORING LOG NUMBER 2

Rodrigues Holdings, LLC **Date: 05/11/15**

File No. 20971 **Method: 8-inches Diameter Hollow Stem Auger**

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		4-inches Asphalt over 5-inches Base
				1 --		FILL: Sandy Silt to Sandy Clay, dark brown, moist, stiff
				2 --		
2.5	29	18.7	104.5	-		
				3 --		
				4 --		
5	12	19.0	110.0	5 --		
				-	ML/CL	NATIVE SOILS: Sandy Silt to Sandy Clay, dark to yellowish brown, moist, stiff
				6 --		
				7 --		
7.5	16	n/a	n/a	-		
				8 --		
				9 --		
10	23	19.9	107.4	10 --		
				-	CL	Sandy Clay, medium to dark brown, moist, stiff
				11 --		
				12 --		
				13 --		
				14 --		
15	23	17.4	115.7	15 --		
				16 --		
				17 --		
				18 --		
				19 --		
20	27	10.7	127.7	20 --		
				-	SM	Silty Sand, dark to yellowish brown, moist, medium dense, fine grained
				21 --		
				22 --		
				23 --		
				24 --		
25	64	6.8	134.1	25 --		
				-		dark and grayish brown

BORING LOG NUMBER 2

Rodrigues Holdings, LLC

File No. 20971

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-	SP	Sand, orange brown, moist, very dense, fine to medium grained
30	50/6"	4.0	105.3	30 --		
				-		Total Depth 30 feet
				31 --		No Water
				-		Fill to 5 feet
				32 --		
				-		
				33 --		
				-		
				34 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				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 --		
				-		
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
				45 --		
				-		
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
				50 --		
				-		

BORING LOG NUMBER 3

Rodrigues Holdings, LLC **Date: 05/11/15**

File No. 20971 **Method: 8-inch Diameter Hollow Stem Auger**

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		4-inches Asphalt over 1-inch Base
				-		FILL: Sandy Silt, dark to yellowish brown, moist, stiff
				1 --		
				-		
				2 --		
2.5	14	10.1	101.9	-		
				3 --		
				-	ML	NATIVE SOILS: Sandy Silt, dark to yellowish brown, moist, stiff
				4 --		
				-		
5	6	11.7	SPT	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	18	13.2	103.0	-		
				8 --		
				-		
				9 --		
				-		
10	9	14.0	SPT	10 --		
				-	CL	Sandy to Silty Clay, yellowish brown, moist, stiff
				11 --		
				-		
				12 --		
12.5	23	10.9	119.9	-		
				13 --		
				-		
				14 --		
				-		
15	10	11.9	SPT	15 --		
				-		
				16 --		
				-		
				17 --		
17.5	39	7.2	120.8	-		
				18 --	SM/ML	Sandy Silt to Silty Sand, dark brown, moist, medium dense, stiff
				-		
				19 --		
				-		
20	33	7.3	SPT	20 --		
				-		
				21 --		
				-		
				22 --		
22.5	43	11.6	117.7	-		
				23 --	SM	Silty Sand, dark to yellowish brown, moist, medium dense, fine grained
				-		
				24 --		
				-		
25	22	11.1	SPT	25 --		
				-		

BORING LOG NUMBER 3

Rodrigues Holdings, LLC

File No. 20971

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
27.5	70	4.3	128.1	-		
				28 --		dark to grayish brown, moist, dense, fine to medium grained, minor gravel
				-		
				29 --		
				-		
30	30	4.3	SPT	30 --		
				-		medium dense
				31 --		
				-		
				32 --		
32.5	50/6"	2.7	114.9	-		
				33 --	SP	Sand, dark to grayish brown, moist, very dense, fine grained
				-		
				34 --		
				-		
35	42	1.7	SPT	35 --		
				-	SM/SP	Silty Sand to Sand, dark to yellowish brown, moist, medium dense to dense, fine grained
				36 --		
				-		
				37 --		
37.5	49	15.3	116.3	-		
				38 --	SM	Silty Sand, dark and grayish brown, moist, medium dense, fine grained
				-		
				39 --		
				-		
40	34	4.1	SPT	40 --		
				-	SM/SP	Silty Sand to Sand, dark to grayish brown, moist, medium dense, fine to medium grained, minor gravel
				41 --		
				-		
				42 --		
42.5	50/6"	3.8	114.4	-		
				43 --	SP	Sand, dark to yellowish brown, moist, very dense, fine to medium grained
				-		
				44 --		
				-		
45	41	4.3	SPT	45 --		
				-		
				46 --		
				-		
				47 --		
47.5	50/5"	2.7	119.2	-		
				48 --	SP/SW	Sand to Gravelly Sand, dark to yellowish brown, moist, very dense, fine to coarse grained
				-		
				49 --		
				-		
50	45	3.1	SPT	50 --		
				-	SP	Sand, dark to yellowish brown, moist, medium dense to dense, fine to medium grained, minor gravel

BORING LOG NUMBER 3

Rodrigues Holdings, LLC

File No. 20971

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				- 51 -- - 52 --		
52.5	87	4.0	127.4	- 53 -- - 54 -- -	SP/SM	Sand to Silty Sand, dark to grayish brown, moist, dense, fine to medium grained
55	65	5.9	SPT	55 -- - 56 -- - 57 --	SP	Sand, dark to grayish brown, moist, dense, fine to medium grained, minor gravel
57.5	100/7"	5.6	116.9	- 58 -- - 59 -- -		
60	86	2.8	SPT	60 -- - 61 -- - 62 -- - 63 -- - 64 -- - 65 -- - 66 -- - 67 -- - 68 -- - 69 -- - 70 -- - 71 -- - 72 -- - 73 -- - 74 -- - 75 -- -		Total Depth 60 feet No Water Fill to 3 feet NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual. Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted SPT=Standard Penetration Test

BORING LOG NUMBER 4

Rodrigues Holdings, LLC **Date: 05/11/15**

File No. 20971 **Method: 8-inches Diameter Hollow Stem Auger**

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		3.5-inches Asphalt over 2.5-inches Base
				-		
				1 --		FILL: Sandy Silt, dark brown, moist, stiff
				-		
				2 --		
2.5	9	14.8	98.0	-		
				3 --		
				-	ML	NATIVE SOILS: Sandy Silt, dark to yellowish brown, moist, stiff
				4 --		
				-		
5	15	12.4	98.4	5 --		
				-		
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	19	15.0	98.4	10 --		
				-	CL/ML	Sandy Clay to Sandy Silt, yellowish brown, moist, stiff
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	27	15.7	107.0	15 --		
				-	CL/SC	Sandy Clay to Clayey Sand, dark brown, moist, stiff or medium dense, fine grained
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	19	8.1	100.2	20 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark to yellowish brown, moist, medium dense, fine grained, stiff
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	30	24.9	114.3	25 --		
				-	CL	Sandy Clay, dark to grayish brown, moist, stiff

BORING LOG NUMBER 4

Rodrigues Holdings, LLC

File No. 20971

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		Silty Sand to Sandy Silt, dark to grayish brown, moist, medium
				-	SM/ML	dense, stiff, fine grained
30	39	12.3	121.9	30 --		
				-		Total Depth 30 feet
				31 --		No Water
				-		Fill to 5 feet
				32 --		
				-		
				33 --		
				-		
				34 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				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 --		
				-		
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
				45 --		
				-		
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
				50 --		
				-		

BORING LOG NUMBER 5

Rodrigues Holdings, LLC **Date: 05/12/15**

File No. 20971 **Method: 8-inches Diameter Hollow Stem Auger**

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		3-inches Asphalt over 3-inches Base
				-		
				1 --		FILL: Silty Sand, dark brown, moist, medium dense, fine grained, minor gravel
				-		
				2 --		
2.5	15	15.2	110.9	-		
				3 --		
				-	CL/ML	NATIVE SOILS: Silty Clay to Clayey Silt, dark brown, moist, stiff
				4 --		
				-		
5	14	17.5	106.7	5 --		
				-		
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	17	20.6	95.1	10 --		
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	17	17.1	107.5	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	18	13.3	103.0	20 --		
				-	ML	Sandy Silt, dark to yellowish brown, moist, medium dense, fine grained, stiff
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	38	16.0	117.7	25 --		
				-	CL/ML	Silty Clay to Clayey Silt, dark to grayish brown, moist, medium dense, fine grained, stiff

BORING LOG NUMBER 5

Rodrigues Holdings, LLC

File No. 20971

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
				28		
				29		
30	25	23.9	111.2	30		
				31		Total Depth 30 feet No Water Fill to 3 feet
				32		
				33		
				34		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				35		
				36		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				37		
				38		
				39		
				40		
				41		
				42		
				43		
				44		
				45		
				46		
				47		
				48		
				49		
				50		

BORING LOG NUMBER 6

Rodrigues Holdings, LLC **Date: 05/12/15**

File No. 20971 **Method: 8-inches Diameter Hollow Stem Auger**

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		4-inches Asphalt over 2-inches Base
				-		
				1 --		FILL: Sandy Silt, dark brown, moist, stiff
				-		
				2 --		
2.5	9	18.6	98.3	-		
				3 --		
				-	CL/ML	NATIVE SOILS: Silty Clay to Clayey Silt, dark to yellowish brown, moist, stiff
				4 --		
				-		
5	13	19.0	96.6	5 --		
				-		
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	22	19.8	101.3	10 --		
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	14	17.3	110.6	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	34	14.4	117.0	20 --		
				-		reddish brown
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	39	20.1	111.0	25 --		
				-		

BORING LOG NUMBER 6

Rodrigues Holdings, LLC

File No. 20971

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26 --		
				27 --		
				28 --		
				29 --		
30	37	13.6	122.0	30 --		
				31 --		Total Depth 30 feet No Water Fill to 3 feet
				32 --		
				33 --		
				34 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				35 --		
				36 --		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				37 --		
				38 --		
				39 --		
				40 --		
				41 --		
				42 --		
				43 --		
				44 --		
				45 --		
				46 --		
				47 --		
				48 --		
				49 --		
				50 --		

BORING LOG NUMBER 7

Rodrigues Holdings, LLC **Date: 05/12/15**

File No. 20971 **Method: 8-inch Diameter Hollow Stem Auger**

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		4-inches Asphalt over 8-inches Base
				-		
				1 --		
				-		FILL: Silty Sand to Sandy Silt, dark brown, moist, medium
				2 --		dense, fine grained, stiff
2.5	36	5.8	125.0	-		
				3 --		
				-	CL/ML	NATIVE SOILS: Silty Clay to Clayey Silt, dark brown, moist,
				4 --		stiff
				-		
5	9	18.5	SPT	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	18	16.1	111.5	-		
				8 --		
				-		
				9 --		
				-		
10	9	17.2	SPT	10 --		
				-		
				11 --		
				-		
				12 --		
12.5	22	17.7	106.8	-		
				13 --		
				-		
				14 --		
				-		
15	10	19.5	SPT	15 --		
				-		
				16 --		
				-		
				17 --		
17.5	20	18.8	106.4	-		
				18 --		
				-		
				19 --		
				-		
20	18	21.8	SPT	20 --		
				-	ML	Sandy Silt, yellowish brown, moist, stiff
				21 --		
				-		
				22 --		
22.5	44	12.2	125.3	-		
				23 --		
				-		
				24 --		
				-		
25	22	14.3	SPT	25 --		
				-		

BORING LOG NUMBER 7

Rodrigues Holdings, LLC

File No. 20971

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
27.5	42	21.6	111.4			- - - - - - - - - - - - - -
				28		reddish brown
				29		
30	21	17.6	SPT	30		
				31		
				32		
32.5	55	9.8	126.1			
				33	ML/SM	Sandy Silt to Silty Sand, dark to grayish brown, moist, medium dense, fine grained
				34		
35	24	9.7	SPT	35		
				36		
				37		
37.5	48	11.0	128.6			
				38		
				39		
40	25	13.0	SPT	40		
				41		
				42		
42.5	55	4.5	125.0			
				43	SM	Silty Sand, dark to grayish brown, moist, dense, fine grained
				44		
45	27	12.1	SPT	45		
				46		
				47		
47.5	50/5"	5.1	113.3			
				48	SP/SM	Sand to Silty Sand, dark to yellowish brown, moist, very dense, fine grained
				49		
50	35	4.2	SPT	50		

BORING LOG NUMBER 7

Rodrigues Holdings, LLC

File No. 20971

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				51		
				52		
52.5	50/5"	4.2	113.4			
				53		minor gravel
				54		
55	33	3.3	SPT	55		
				56		
				57		
57.5	50/5"	2.7	111.6			
				58		
				59		
60	72	2.2	SPT	60		
						Total Depth 60 feet
				61		No Water
						Fill to 3 feet
				62		
				63		
						NOTE: The stratification lines represent the approximate
				64		boundary between earth types; the transition may be gradual.
				65		Used 8-inch diameter Hollow-Stem Auger
						140-lb. Automatic Hammer, 30-inch drop
				66		Modified California Sampler used unless otherwise noted
				67		SPT=Standard Penetration Test
				68		
				69		
				70		
				71		
				72		
				73		
				74		
				75		

BORING LOG NUMBER 8

Rodrigues Holdings, LLC **Date: 05/12/15**

File No. 20971 **Method: 8-inches Diameter Hollow Stem Auger**

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		2.5-inches Asphalt over 2.5-inches Base
				-		
				1 --		FILL: Sandy Silt to Silty Sand, dark brown, moist, stiff, medium dense, fine grained
				-		
				2 --		
2.5	72	8.6	128.8	-		
				3 --		Silty Sand, dark brown to gray, moist, very dense, fine grained
				-		
				4 --		
				-		
5	60	9.2	125.3	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	10	15.6	96.5	-		
				8 --	CL/ML	NATIVE SOILS: Silty Clay to Clayey Silt, dark brown, moist, stiff
				-		
				9 --		
				-		
10	14	14.4	105.1	10 --		
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	22	16.3	102.6	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	20	1.0	123.4	20 --		
				-		
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	36	6.6	120.6	25 --		
				-	ML/SM	Sandy Silt to Silty Sand, dark and grayish brown mottling, moist, dense, fine grained

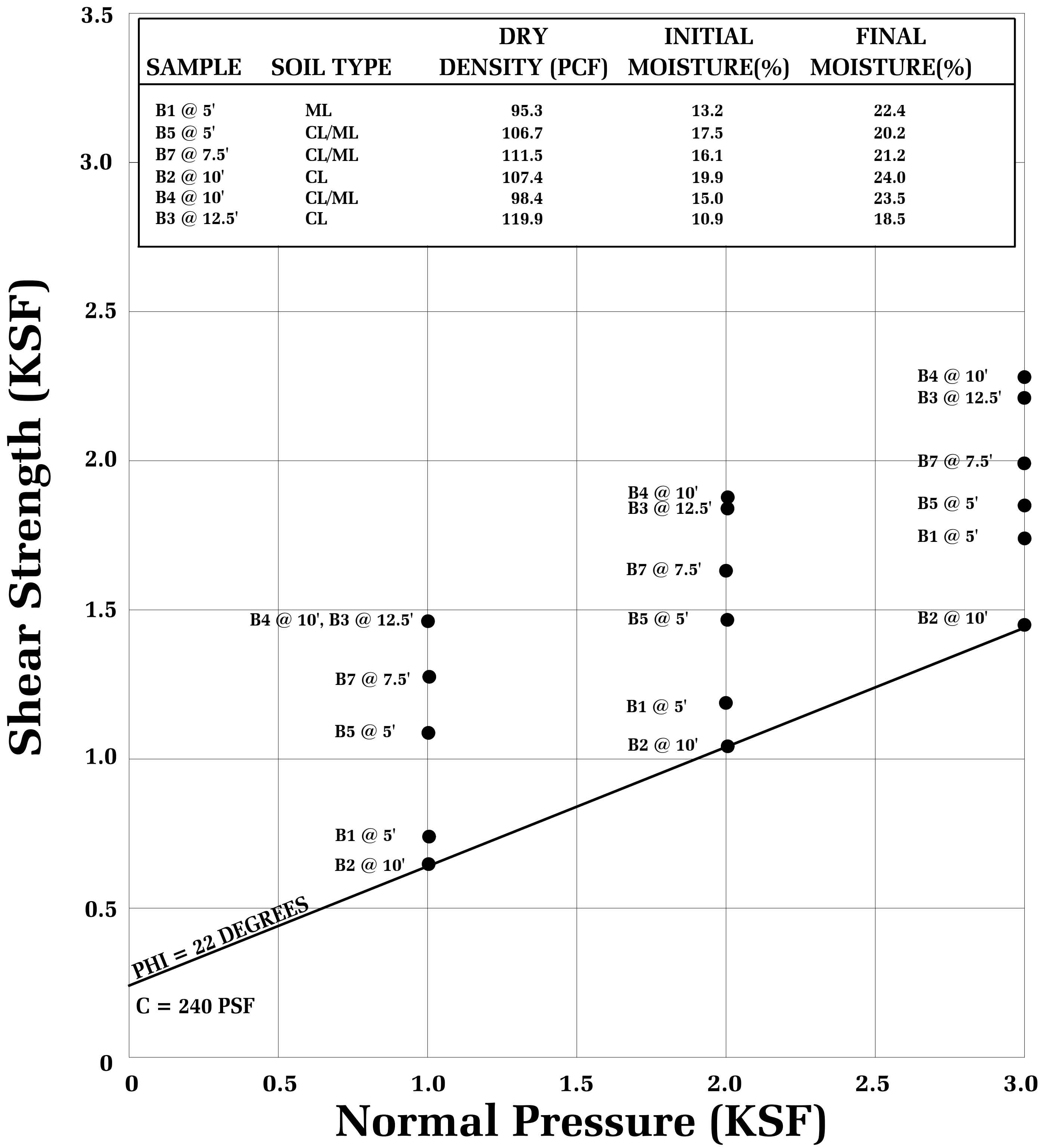
BORING LOG NUMBER 8

Rodrigues Holdings, LLC

File No. 20971

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
30	59	9.0	131.7	30 --		
				-		Total Depth 30 feet
				31 --		No Water
				-		Fill to 7.5 feet
				32 --		
				-		
				33 --		
				-		
				34 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				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 --		
				-		
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
				45 --		
				-		
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
				50 --		
				-		



SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B1 @ 5'	ML	95.3	13.2	22.4
B5 @ 5'	CL/ML	106.7	17.5	20.2
B7 @ 7.5'	CL/ML	111.5	16.1	21.2
B2 @ 10'	CL	107.4	19.9	24.0
B4 @ 10'	CL/ML	98.4	15.0	23.5
B3 @ 12.5'	CL	119.9	10.9	18.5

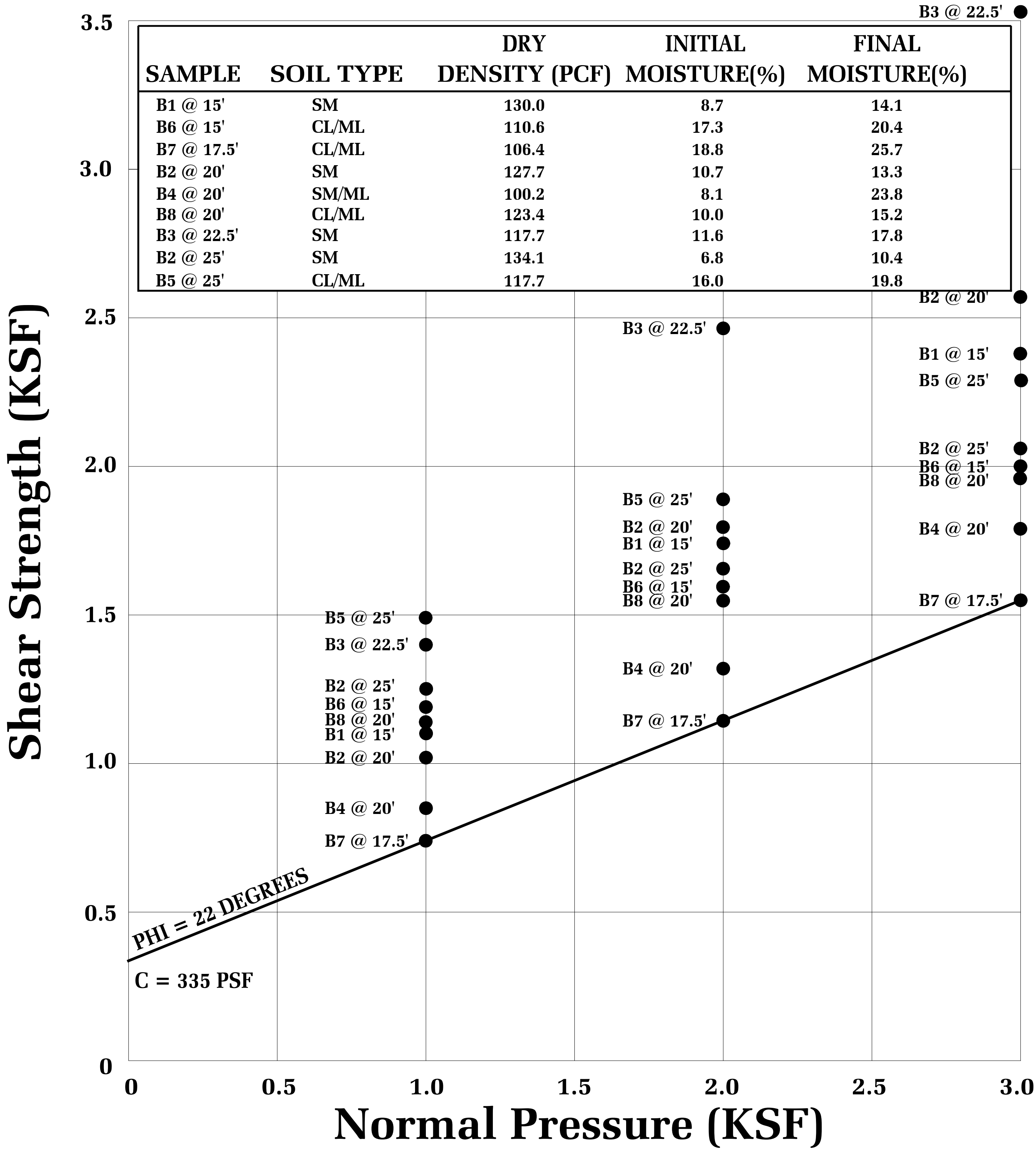
● Direct Shear, Saturated

SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

RODRIGUES HOLDINGS, LLC

FILE NO. 20971 | PLATE: B-1



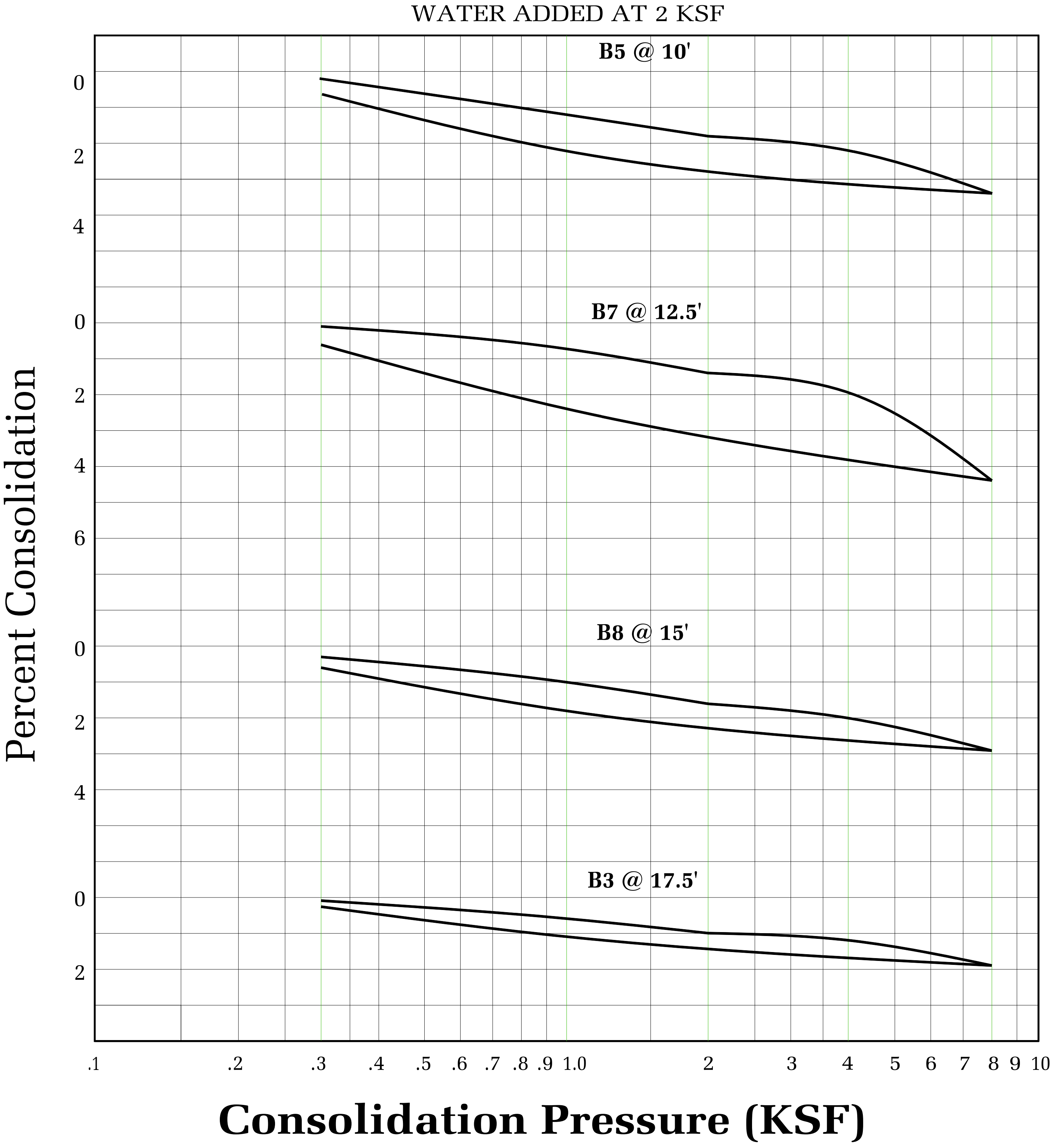
SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B1 @ 15'	SM	130.0	8.7	14.1
B6 @ 15'	CL/ML	110.6	17.3	20.4
B7 @ 17.5'	CL/ML	106.4	18.8	25.7
B2 @ 20'	SM	127.7	10.7	13.3
B4 @ 20'	SM/ML	100.2	8.1	23.8
B8 @ 20'	CL/ML	123.4	10.0	15.2
B3 @ 22.5'	SM	117.7	11.6	17.8
B2 @ 25'	SM	134.1	6.8	10.4
B5 @ 25'	CL/ML	117.7	16.0	19.8

SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

RODRIGUES HOLDINGS,LLC

FILE NO. 20971 | PLATE: B-2

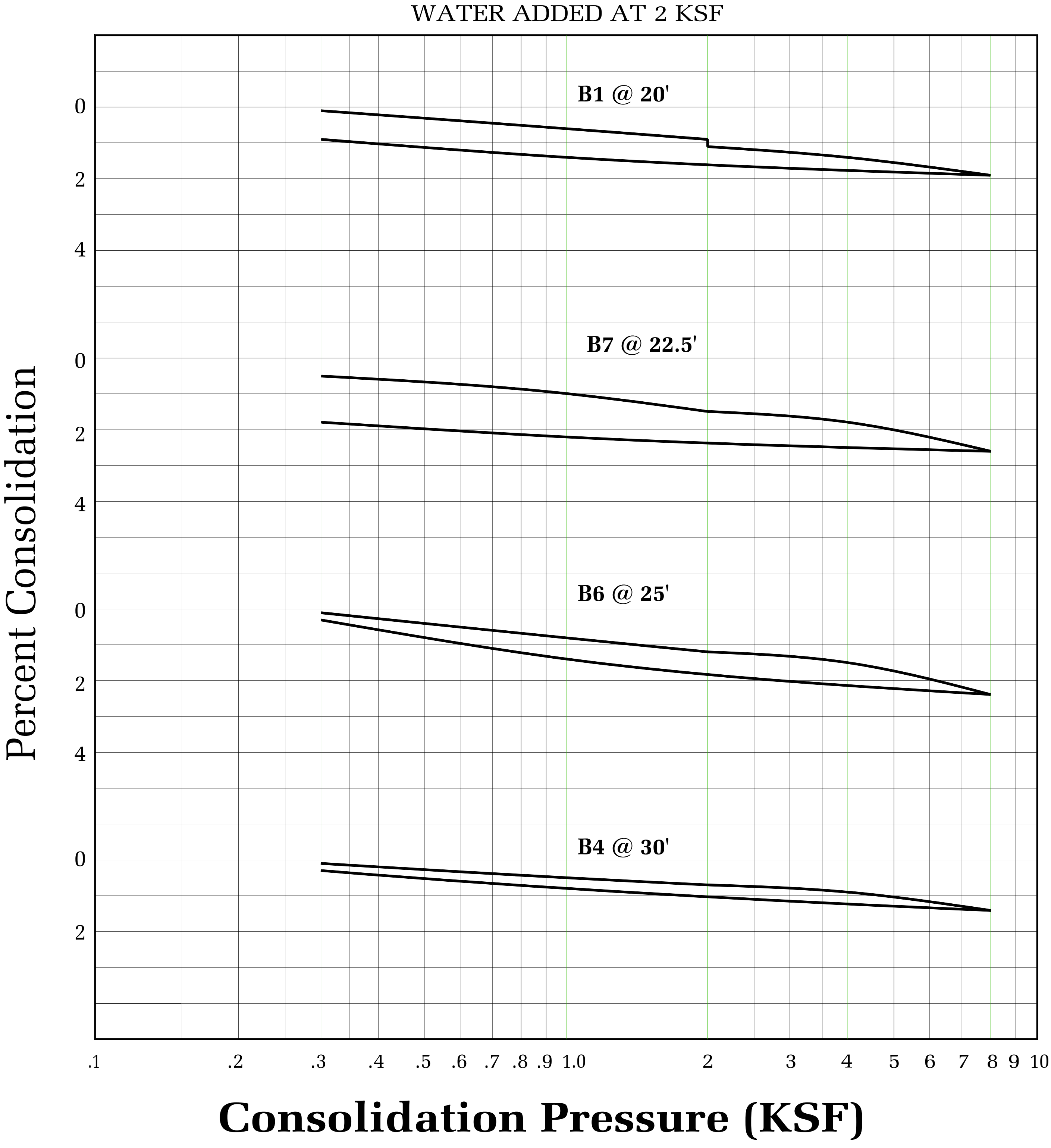


CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

RODRIGUES HOLDINGS, LLC

FILE NO. 20971 | **PLATE: C-1**



CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

RODRIGUES HOLDINGS, LLC

FILE NO. 20971	PLATE: C-2

ASTM D-1557

SAMPLE	B1 @ 1-5'	B5 @ 1-5'
SOIL TYPE:	ML	ML/CL
MAXIMUM DENSITY pcf.	131.6	134.2
OPTIMUM MOISTURE %	10.2	9.4

ASTM D 4829

SAMPLE	B1 @ 1-5'	B5 @ 1-5'
SOIL TYPE:	ML	ML/CL
EXPANSION INDEX UBC STANDARD 18-2	56	40
EXPANSION CHARACTER	MODERATE	LOW

SULFATE CONTENT

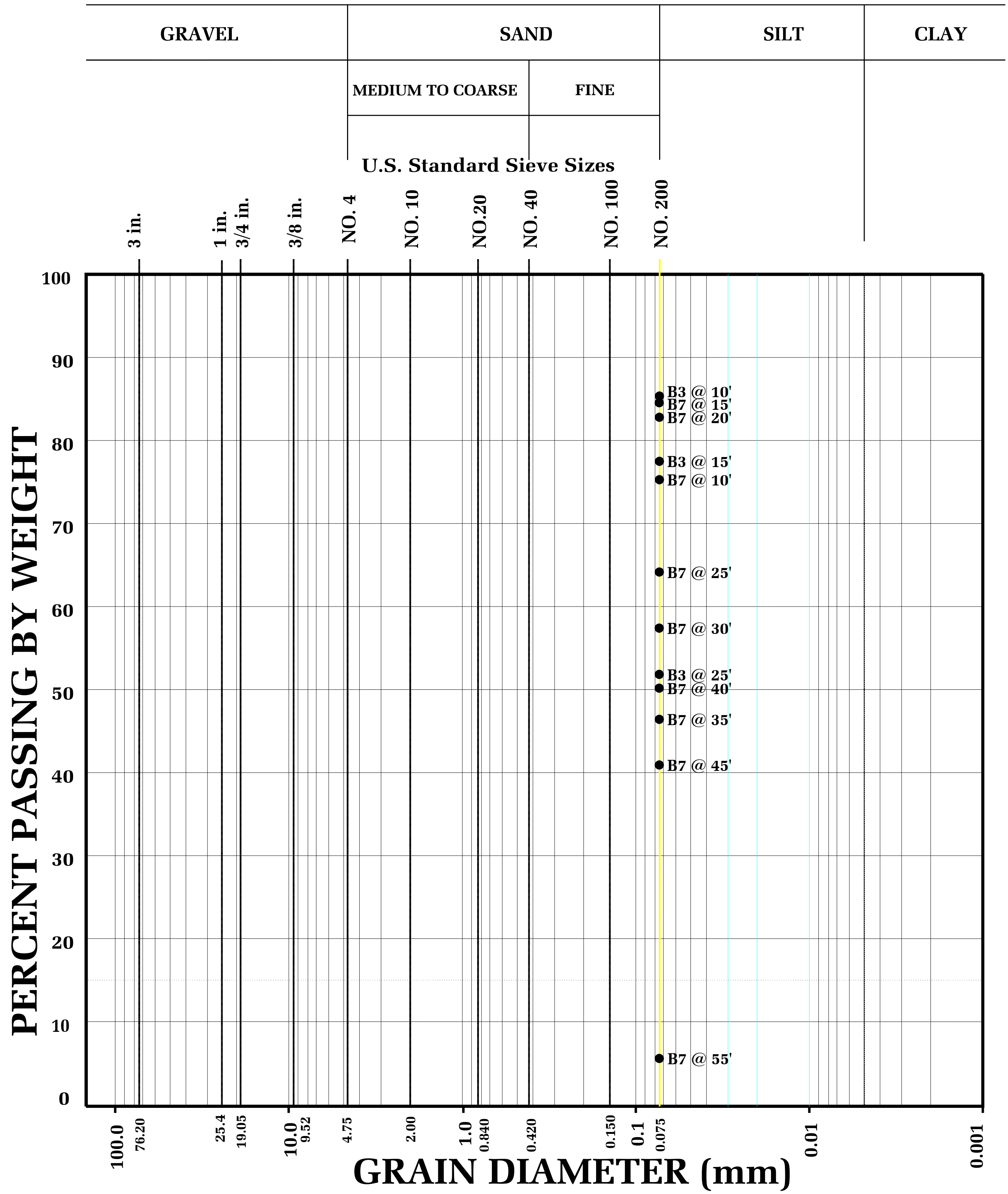
SAMPLE	B1 @ 1-5'	B5 @ 1-5'
SULFATE CONTENT: (percentage by weight)	< 0.10%	< 0.10%

COMPACTION/EXPANSION/SULFATE DATA SHEET

Geotechnologies, Inc.
Consulting Geotechnical Engineers

RODRIGUES HOLDINGS, LLC

FILE NO. 20971 | PLATE: D



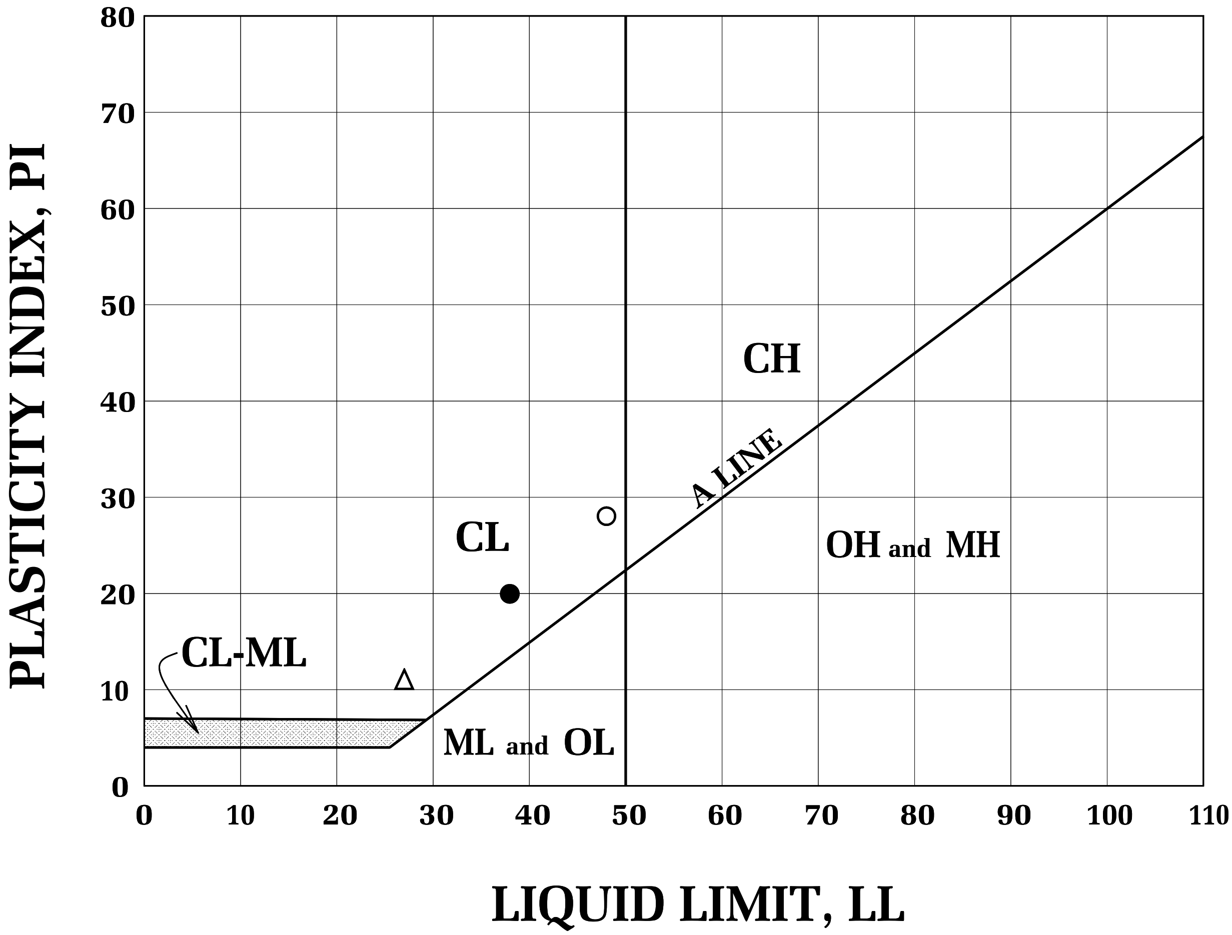
GRAIN SIZE DISTRIBUTION

Geotechnologies, Inc.
Consulting Geotechnical Engineers

RODRIGUES HOLDINGS, LLC

FILE NO. 20971	PLATE: E

ASTM D4318



BORING NUMBER	DEPTH (FEET)	TEST SYMBOL	LL	PL	PI	DESCRIPTION
B3	10	○	48	20	28	CL
B3	15	●	38	18	20	CL
B3	25	△	27	16	11	CL

ATTERBERG LIMITS DETERMINATION

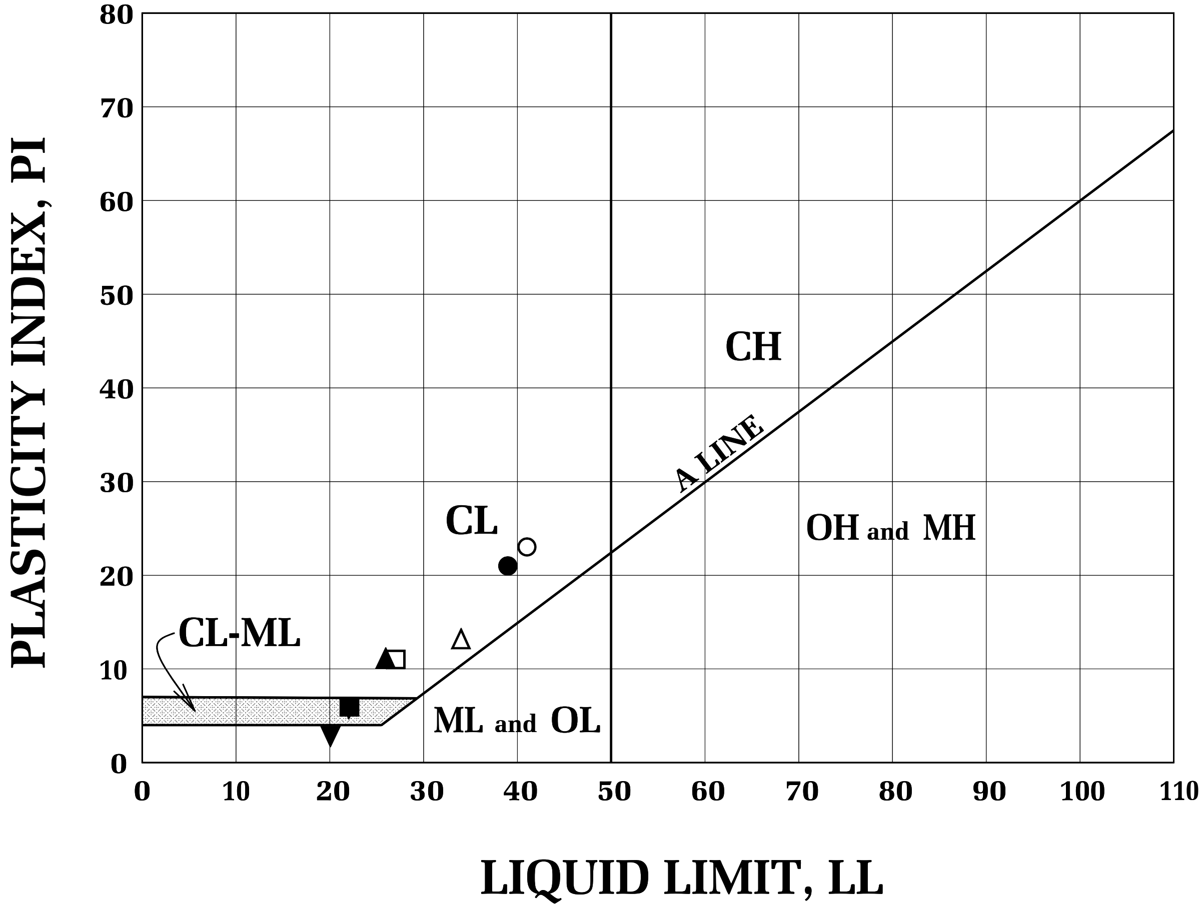
Geotechnologies, Inc.
Consulting Geotechnical Engineers

RODRIGUES HOLDINGS, LLC

FILE NO. 20971

PLATE: F-1

ASTM D4318



BORING NUMBER	DEPTH (FEET)	TEST SYMBOL	LL	PL	PI	DESCRIPTION
B7	10	○	41	18	23	CL
B7	15	●	39	18	21	CL
B7	20	△	34	21	13	CL
B7	25	▲	26	15	11	CL
B7	30	□	27	16	11	CL
B7	35	■	22	16	6	CL-ML
B7	40	▽	22	16	6	CL-ML
B7	45	▼	20	17	3	ML

ATTERBERG LIMITS DETERMINATION

Geotechnologies, Inc.
Consulting Geotechnical Engineers

RODRIGUES HOLDINGS, LLC

FILE NO. 20971 PLATE: F-2

Geotechnologies, Inc.

Project: Rodrigues Holdings, LLC
File No.: 20971
Description: Liquefaction Analysis
Boring Number: 3

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL

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

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.6
Peak Horiz. Acceleration (g):	0.79
Calculated Mag.Wtg.Factor:	0.724

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	61.0
Historic Highest Groundwater Level* (ft):	15.0
Unit Wt. Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

ENERGY & ROD CORRECTIONS:

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

LIQUEFACTION CALCULATIONS:

Depth to Base (ft)	Total Unit Wt. (pcf)	Current Water Level (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected $(N_1)_{60}$	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	130.0	0	NA	1.0	0	0.0		2.000	0.0	~	0.998	0.369	~
2.0	130.0	0	NA	1.0	0	0.0		########	#VALUE!	~	0.993	0.367	~
3.0	112.2	0	6.0	5.0	0	0.0		1.979	13.9	~	0.989	0.365	~
4.0	112.2	0	6.0	5.0	0	0.0		1.979	13.9	~	0.984	0.364	~
5.0	112.2	0	6.0	5.0	0	0.0		1.979	13.9	~	0.979	0.362	~
6.0	112.2	0	6.0	5.0	0	0.0		1.979	13.9	~	0.975	0.360	~
7.0	112.2	0	6.0	5.0	0	0.0		1.979	13.9	~	0.970	0.359	~
8.0	116.6	0	6.0	5.0	0	0.0		1.979	13.9	~	0.966	0.357	~
9.0	116.6	0	6.0	5.0	0	0.0		1.979	13.9	~	0.961	0.355	~
10.0	132.9	0	9.0	10.0	0	85.4		1.374	21.5	~	0.957	0.354	~
11.0	132.9	0	9.0	10.0	0	85.4		1.374	21.5	~	0.952	0.352	~
12.0	132.9	0	9.0	10.0	0	85.4		1.374	21.5	~	0.947	0.350	~
13.0	132.9	0	9.0	10.0	0	85.4		1.374	21.5	~	0.943	0.349	~
14.0	132.9	0	9.0	10.0	0	85.4		1.374	21.5	~	0.938	0.347	~
15.0	132.9	0	10.0	15.0	0	77.5		1.089	20.7	~	0.934	0.345	~
16.0	132.9	0	10.0	15.0	0	77.5		1.089	20.7	~	0.929	0.343	~
17.0	132.9	0	10.0	15.0	0	77.5		1.089	20.7	~	0.925	0.342	~
18.0	129.5	0	33.0	20.0	1	0.0	97	0.931	42.9	Infin.	0.920	0.340	Non-Liq.
19.0	129.5	0	33.0	20.0	1	0.0	97	0.931	42.9	Infin.	0.915	0.338	Non-Liq.
20.0	129.5	0	33.0	20.0	1	0.0	97	0.931	42.9	Infin.	0.911	0.337	Non-Liq.
21.0	129.5	0	33.0	20.0	1	0.0	97	0.931	42.9	Infin.	0.906	0.335	Non-Liq.
22.0	129.5	0	33.0	20.0	1	0.0	97	0.931	42.9	Infin.	0.902	0.333	Non-Liq.
23.0	131.3	0	22.0	25.0	1	51.8	74	0.827	34.1	Infin.	0.897	0.332	Non-Liq.
24.0	131.3	0	22.0	25.0	1	51.8	74	0.827	34.1	Infin.	0.893	0.330	Non-Liq.
25.0	131.3	0	22.0	25.0	1	51.8	74	0.827	34.1	Infin.	0.888	0.328	Non-Liq.
26.0	131.3	0	22.0	25.0	1	51.8	74	0.827	34.1	Infin.	0.883	0.327	Non-Liq.
27.0	131.3	0	22.0	25.0	1	51.8	74	0.827	34.1	Infin.	0.879	0.325	Non-Liq.
28.0	133.7	0	22.0	25.0	1	51.8	74	0.827	34.1	Infin.	0.874	0.323	Non-Liq.
29.0	133.7	0	22.0	25.0	1	51.8	74	0.827	34.1	Infin.	0.870	0.321	Non-Liq.
30.0	133.7	0	30.0	30.0	1	0.0	81	0.751	35.1	Infin.	0.865	0.320	Non-Liq.
31.0	133.7	0	30.0	30.0	1	0.0	81	0.751	35.1	Infin.	0.861	0.318	Non-Liq.
32.0	133.7	0	30.0	30.0	1	0.0	81	0.751	35.1	Infin.	0.856	0.316	Non-Liq.
33.0	118.0	0	30.0	30.0	1	0.0	81	0.751	35.1	Infin.	0.851	0.315	Non-Liq.
34.0	118.0	0	30.0	30.0	1	0.0	81	0.751	35.1	Infin.	0.847	0.313	Non-Liq.
35.0	118.0	0	42.0	35.0	1	0.0	91	0.695	45.5	Infin.	0.842	0.311	Non-Liq.
36.0	118.0	0	42.0	35.0	1	0.0	91	0.695	45.5	Infin.	0.838	0.310	Non-Liq.
37.0	118.0	0	42.0	35.0	1	0.0	91	0.695	45.5	Infin.	0.833	0.308	Non-Liq.
38.0	134.1	0	42.0	35.0	1	0.0	91	0.695	45.5	Infin.	0.829	0.306	Non-Liq.
39.0	134.1	0	42.0	35.0	1	0.0	91	0.695	45.5	Infin.	0.824	0.305	Non-Liq.
40.0	134.1	0	34.0	40.0	1	0.0	78	0.650	34.5	Infin.	0.819	0.303	Non-Liq.
41.0	134.1	0	34.0	40.0	1	0.0	78	0.650	34.5	Infin.	0.815	0.301	Non-Liq.
42.0	134.1	0	34.0	40.0	1	0.0	78	0.650	34.5	Infin.	0.810	0.300	Non-Liq.
43.0	118.8	0	41.0	45.0	1	0.0	82	0.612	39.2	Infin.	0.806	0.298	Non-Liq.
44.0	118.8	0	41.0	45.0	1	0.0	82	0.612	39.2	Infin.	0.801	0.296	Non-Liq.
45.0	118.8	0	41.0	45.0	1	0.0	82	0.612	39.2	Infin.	0.797	0.294	Non-Liq.
46.0	118.8	0	41.0	45.0	1	0.0	82	0.612	39.2	Infin.	0.792	0.293	Non-Liq.
47.0	118.8	0	41.0	45.0	1	0.0	82	0.612	39.2	Infin.	0.787	0.291	Non-Liq.
48.0	122.5	0	41.0	45.0	1	0.0	82	0.612	39.2	Infin.	0.783	0.289	Non-Liq.
49.0	122.5	0	41.0	45.0	1	0.0	82	0.612	39.2	Infin.	0.778	0.288	Non-Liq.
50.0	122.5	0	45.0	50.0	1	0.0	83	0.600	42.1	Infin.	0.774	0.286	Non-Liq.
51.0	122.5	0	45.0	50.0	1	0.0	83	0.600	42.1	Infin.	0.769	0.284	Non-Liq.
52.0	122.5	0	45.0	50.0	1	0.0	83	0.600	42.1	Infin.	0.765	0.283	Non-Liq.
53.0	132.5	0	45.0	50.0	1	0.0	83	0.600	42.1	Infin.	0.760	0.281	Non-Liq.
54.0	132.5	0	45.0	50.0	1	0.0	83	0.600	42.1	Infin.	0.755	0.279	Non-Liq.
55.0	123.4	0	65.0	55.0	1	0.0	96	0.600	60.8	Infin.	0.751	0.278	Non-Liq.
56.0	123.4	0	65.0	55.0	1	0.0	96	0.600	60.8	Infin.	0.746	0.276	Non-Liq.
57.0	123.4	0	65.0	55.0	1	0.0	96	0.600	60.8	Infin.	0.742	0.274	Non-Liq.
58.0	123.4	0	65.0	55.0	1	0.0	96	0.600	60.8	Infin.	0.737	0.272	Non-Liq.
59.0	123.4	0	65.0	55.0	1	0.0	96	0.600	60.8	Infin.	0.733	0.271	Non-Liq.
60.0	123.4	0	86.0	60.0	1	0.0	107	0.600	80.5	Infin.	0.728	0.269	Non-Liq.

Geotechnologies, Inc.

Project: Rodrigues Holdings, LLC
File No.: 20971
Description: Liquefaction Analysis
Boring Number: 7

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL

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

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.6
Peak Horiz. Acceleration (g):	0.79
Calculated Mag.Wtg.Factor:	0.724

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	61.0
Historic Highest Groundwater Level* (ft):	15.0
Unit Wt. Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

ENERGY & ROD CORRECTIONS:

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

LIQUEFACTION CALCULATIONS:

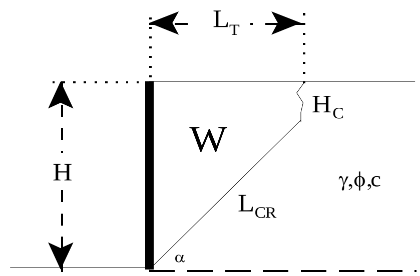
Depth to Base (ft)	Total Unit Wt. (pcf)	Current Water Level (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected $(N_1)_{60}$	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	130.0	0	NA	1.0	0	0.0		2.000	0.0	~	0.998	0.369	~
2.0	130.0	0	NA	1.0	0	0.0		########	#VALUE!	~	0.993	0.367	~
3.0	132.3	0	9.0	5.0	0	0.0		1.893	19.9	~	0.989	0.365	~
4.0	132.3	0	9.0	5.0	0	0.0		1.893	19.9	~	0.984	0.364	~
5.0	132.3	0	9.0	5.0	0	0.0		1.893	19.9	~	0.979	0.362	~
6.0	132.3	0	9.0	5.0	0	0.0		1.893	19.9	~	0.975	0.360	~
7.0	132.3	0	9.0	5.0	0	0.0		1.893	19.9	~	0.970	0.359	~
8.0	129.4	0	9.0	5.0	0	0.0		1.893	19.9	~	0.966	0.357	~
9.0	129.4	0	9.0	5.0	0	0.0		1.893	19.9	~	0.961	0.355	~
10.0	129.4	0	9.0	10.0	0	75.3		1.304	20.7	~	0.957	0.354	~
11.0	129.4	0	9.0	10.0	0	75.3		1.304	20.7	~	0.952	0.352	~
12.0	129.4	0	9.0	10.0	0	75.3		1.304	20.7	~	0.947	0.350	~
13.0	125.7	0	9.0	10.0	0	75.3		1.304	20.7	~	0.943	0.349	~
14.0	125.7	0	9.0	10.0	0	75.3		1.304	20.7	~	0.938	0.347	~
15.0	125.7	0	10.0	15.0	0	84.6		1.060	20.3	~	0.934	0.345	~
16.0	125.7	0	10.0	15.0	0	84.6		1.060	20.3	~	0.929	0.343	~
17.0	125.7	0	10.0	15.0	0	84.6		1.060	20.3	~	0.925	0.342	~
18.0	126.4	0	10.0	15.0	0	84.6		1.060	20.3	~	0.920	0.340	~
19.0	126.4	0	10.0	15.0	0	84.6		1.060	20.3	~	0.915	0.338	~
20.0	126.4	0	18.0	20.0	1	82.8	71	0.918	30.1	Infin.	0.911	0.337	Non-Liq.
21.0	140.6	0	18.0	20.0	1	82.8	71	0.918	30.1	Infin.	0.906	0.335	Non-Liq.
22.0	140.6	0	18.0	20.0	1	82.8	71	0.918	30.1	Infin.	0.902	0.333	Non-Liq.
23.0	140.6	0	18.0	20.0	1	82.8	71	0.918	30.1	Infin.	0.897	0.332	Non-Liq.
24.0	140.6	0	18.0	20.0	1	82.8	71	0.918	30.1	Infin.	0.893	0.330	Non-Liq.
25.0	140.6	0	22.0	25.0	1	64.2	73	0.812	33.6	Infin.	0.888	0.328	Non-Liq.
26.0	140.6	0	22.0	25.0	1	64.2	73	0.812	33.6	Infin.	0.883	0.327	Non-Liq.
27.0	140.6	0	22.0	25.0	1	64.2	73	0.812	33.6	Infin.	0.879	0.325	Non-Liq.
28.0	135.5	0	22.0	25.0	1	64.2	73	0.812	33.6	Infin.	0.874	0.323	Non-Liq.
29.0	135.5	0	22.0	25.0	1	64.2	73	0.812	33.6	Infin.	0.870	0.321	Non-Liq.
30.0	135.5	0	21.0	30.0	1	57.4	67	0.737	31.1	Infin.	0.865	0.320	Non-Liq.
31.0	135.5	0	21.0	30.0	1	57.4	67	0.737	31.1	Infin.	0.861	0.318	Non-Liq.
32.0	135.5	0	21.0	30.0	1	57.4	67	0.737	31.1	Infin.	0.856	0.316	Non-Liq.
33.0	138.5	0	24.0	35.0	1	46.4	68	0.679	32.4	Infin.	0.851	0.315	Non-Liq.
34.0	138.5	0	24.0	35.0	1	46.4	68	0.679	32.4	Infin.	0.847	0.313	Non-Liq.
35.0	138.5	0	24.0	35.0	1	46.4	68	0.679	32.4	Infin.	0.842	0.311	Non-Liq.
36.0	138.5	0	24.0	35.0	1	46.4	68	0.679	32.4	Infin.	0.838	0.310	Non-Liq.
37.0	138.5	0	24.0	35.0	1	46.4	68	0.679	32.4	Infin.	0.833	0.308	Non-Liq.
38.0	142.7	0	24.0	35.0	1	46.4	68	0.679	32.4	Infin.	0.829	0.306	Non-Liq.
39.0	142.7	0	24.0	35.0	1	46.4	68	0.679	32.4	Infin.	0.824	0.305	Non-Liq.
40.0	142.7	0	25.0	40.0	1	50.2	66	0.633	31.7	Infin.	0.819	0.303	Non-Liq.
41.0	142.7	0	25.0	40.0	1	50.2	66	0.633	31.7	Infin.	0.815	0.301	Non-Liq.
42.0	142.7	0	25.0	40.0	1	50.2	66	0.633	31.7	Infin.	0.810	0.300	Non-Liq.
43.0	130.7	0	27.0	45.0	1	40.9	65	0.600	32.3	Infin.	0.806	0.298	Non-Liq.
44.0	130.7	0	27.0	45.0	1	40.9	65	0.600	32.3	Infin.	0.801	0.296	Non-Liq.
45.0	130.7	0	27.0	45.0	1	40.9	65	0.600	32.3	Infin.	0.797	0.294	Non-Liq.
46.0	130.7	0	27.0	45.0	1	40.9	65	0.600	32.3	Infin.	0.792	0.293	Non-Liq.
47.0	130.7	0	27.0	45.0	1	40.9	65	0.600	32.3	Infin.	0.787	0.291	Non-Liq.
48.0	119.0	0	35.0	50.0	1	0.0	72	0.600	32.8	Infin.	0.783	0.289	Non-Liq.
49.0	119.0	0	35.0	50.0	1	0.0	72	0.600	32.8	Infin.	0.778	0.288	Non-Liq.
50.0	119.0	0	35.0	50.0	1	0.0	72	0.600	32.8	Infin.	0.774	0.286	Non-Liq.
51.0	119.0	0	35.0	50.0	1	0.0	72	0.600	32.8	Infin.	0.769	0.284	Non-Liq.
52.0	119.0	0	35.0	50.0	1	0.0	72	0.600	32.8	Infin.	0.765	0.283	Non-Liq.
53.0	118.2	0	35.0	50.0	1	0.0	72	0.600	32.8	Infin.	0.760	0.281	Non-Liq.
54.0	118.2	0	35.0	50.0	1	0.0	72	0.600	32.8	Infin.	0.755	0.279	Non-Liq.
55.0	118.2	0	33.0	55.0	1	5.6	67	0.600	31.0	Infin.	0.751	0.278	Non-Liq.
56.0	118.2	0	33.0	55.0	1	5.6	67	0.600	31.0	Infin.	0.746	0.276	Non-Liq.
57.0	118.2	0	33.0	55.0	1	5.6	67	0.600	31.0	Infin.	0.742	0.274	Non-Liq.
58.0	114.6	0	33.0	55.0	1	5.6	67	0.600	31.0	Infin.	0.737	0.272	Non-Liq.
59.0	114.6	0	33.0	55.0	1	5.6	67	0.600	31.0	Infin.	0.733	0.271	Non-Liq.
60.0	114.6	0	72.0	60.0	1	0.0	96	0.600	67.4	Infin.	0.728	0.269	Non-Liq.

Geotechnologies, Inc.
Project: Rodrigues Holdings
File No.: 20971
Description: Retaining Wall up to 9 feet High

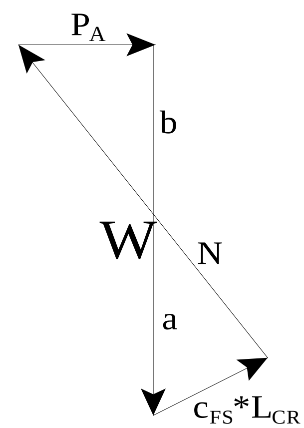
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	9.00	feet
Unit Weight of Retained Soils	(γ)	125.0	pcf
Friction Angle of Retained Soils	(ϕ)	22.0	degrees
Cohesion of Retained Soils	(c)	240.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters:	(ϕ_{FS})	15.1	degrees
	(c_{FS})	160.0	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	3.8	40	4941.6	8.0	2949.4	1992.1	925.8
41	3.7	39	4814.9	8.0	2830.1	1984.9	964.9
42	3.7	37	4686.1	8.0	2716.2	1969.9	1000.5
43	3.6	36	4556.1	7.9	2607.9	1948.2	1032.6
44	3.6	35	4425.6	7.8	2505.0	1920.7	1061.4
45	3.5	34	4295.2	7.8	2407.2	1888.1	1086.8
46	3.5	33	4165.4	7.7	2314.4	1851.0	1108.9
47	3.4	32	4036.4	7.6	2226.2	1810.1	1127.8
48	3.4	31	3908.4	7.5	2142.5	1765.9	1143.5
49	3.4	30	3781.7	7.5	2063.0	1718.7	1156.0
50	3.4	29	3656.4	7.4	1987.3	1669.1	1165.4
51	3.3	28	3532.5	7.3	1915.3	1617.2	1171.7
52	3.3	27	3410.0	7.2	1846.6	1563.4	1174.9
53	3.3	26	3289.0	7.1	1781.0	1508.1	1175.1
54	3.3	25	3169.5	7.0	1718.3	1451.3	1172.1
55	3.4	24	3051.5	6.9	1658.2	1393.3	1166.0
56	3.4	23	2934.8	6.8	1600.5	1334.3	1156.8
57	3.4	23	2819.4	6.7	1544.9	1274.5	1144.5
58	3.4	22	2705.4	6.6	1491.4	1214.0	1129.1
59	3.5	21	2592.5	6.5	1439.6	1152.9	1110.4
60	3.5	20	2480.7	6.4	1389.3	1091.4	1088.6
61	3.5	19	2370.0	6.2	1340.4	1029.6	1063.4
62	3.6	18	2260.1	6.1	1292.5	967.6	1034.9
63	3.7	17	2151.1	6.0	1245.6	905.5	1003.0
64	3.7	16	2042.8	5.9	1199.4	843.4	967.7
65	3.8	15	1935.0	5.7	1153.5	781.4	928.8



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\,max}$ 1175.1 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 29.0 pcf

Design Wall for an Equivalent Fluid Pressure: 30 pcf

Geotechnologies, Inc.

Project: Rodrigues Holdings

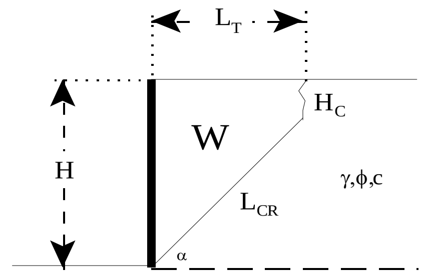
File No.: 20971

Description: Retaining Wall 9 to 12 feet High

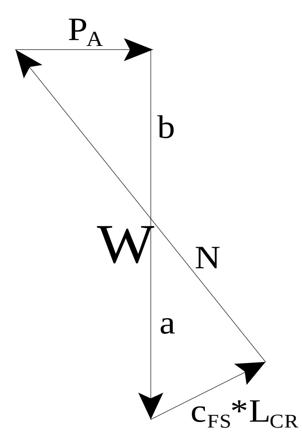
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	12.00	feet
Unit Weight of Retained Soils	(γ)	125.0	pcf
Friction Angle of Retained Soils	(ϕ)	22.0	degrees
Cohesion of Retained Soils	(c)	240.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters:	(ϕ_{FS})	15.1	degrees
	(c_{FS})	160.0	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	3.8	77	9634.1	12.7	4660.4	4973.7	2311.4
41	3.7	75	9344.5	12.6	4446.0	4898.5	2381.3
42	3.7	72	9059.2	12.4	4245.9	4813.3	2444.6
43	3.6	70	8778.6	12.3	4059.1	4719.5	2501.5
44	3.6	68	8503.0	12.2	3884.4	4618.6	2552.2
45	3.5	66	8232.7	12.0	3721.1	4511.7	2596.9
46	3.5	64	7967.8	11.9	3568.1	4399.7	2635.8
47	3.4	62	7708.1	11.7	3424.6	4283.5	2668.8
48	3.4	60	7453.8	11.6	3290.0	4163.8	2696.3
49	3.4	58	7204.5	11.4	3163.3	4041.2	2718.1
50	3.4	56	6960.3	11.3	3044.1	3916.2	2734.5
51	3.3	54	6721.0	11.1	2931.7	3789.3	2745.5
52	3.3	52	6486.3	11.0	2825.6	3660.7	2751.1
53	3.3	50	6256.2	10.8	2725.2	3531.0	2751.3
54	3.3	48	6030.3	10.7	2630.1	3400.2	2746.1
55	3.4	46	5808.5	10.6	2539.8	3268.8	2735.5
56	3.4	45	5590.7	10.4	2453.9	3136.8	2719.6
57	3.4	43	5376.5	10.3	2372.0	3004.4	2698.1
58	3.4	41	5165.8	10.1	2293.9	2871.9	2671.1
59	3.5	40	4958.4	10.0	2219.0	2739.4	2638.5
60	3.5	38	4754.0	9.8	2147.2	2606.9	2600.1
61	3.5	36	4552.5	9.7	2078.0	2474.6	2555.8
62	3.6	35	4353.7	9.5	2011.2	2342.6	2505.5
63	3.7	33	4157.3	9.4	1946.4	2210.9	2449.0
64	3.7	32	3963.2	9.2	1883.4	2079.8	2386.2
65	3.8	30	3771.1	9.0	1821.8	1949.2	2316.8



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\,max}$ 2751.3 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 38.2 pcf

Design Wall for an Equivalent Fluid Pressure: 39 pcf

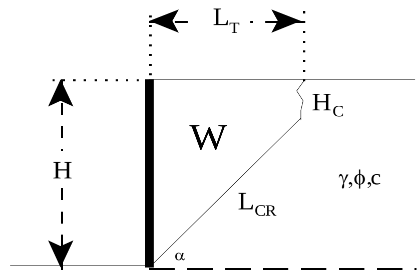
Geotechnologies, Inc.

Project: Rodrigues Holdings
File No.: 20971
Description: Retaining Wall 12 to 17 feet High

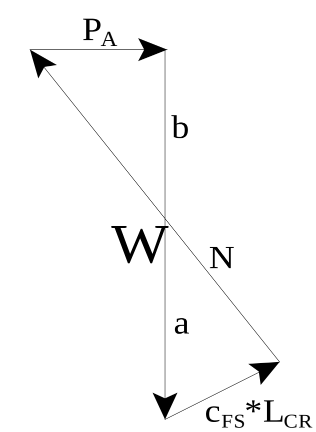
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	17.00 feet
Unit Weight of Retained Soils	(γ)	125.0 pcf
Friction Angle of Retained Soils	(ϕ)	22.0 degrees
Cohesion of Retained Soils	(c)	240.0 psf
Factor of Safety	(FS)	1.50
Factored Parameters:	(ϕ_{FS})	15.1 degrees
	(c_{FS})	160.0 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	3.8	163	20434.3	20.5	7512.0	12922.4	6005.3
41	3.7	158	19769.7	20.2	7139.1	12630.6	6139.9
42	3.7	153	19124.1	19.9	6795.3	12328.8	6261.5
43	3.6	148	18496.9	19.6	6477.6	12019.3	6370.6
44	3.6	143	17887.5	19.4	6183.6	11704.0	6467.6
45	3.5	138	17295.2	19.1	5910.9	11384.3	6552.9
46	3.5	134	16719.3	18.8	5657.7	11061.7	6626.9
47	3.4	129	16159.1	18.6	5422.0	10737.1	6689.8
48	3.4	125	15613.7	18.3	5202.3	10411.3	6741.9
49	3.4	121	15082.4	18.1	4997.3	10085.2	6783.4
50	3.4	117	14564.7	17.8	4805.5	9759.2	6814.5
51	3.3	112	14059.7	17.6	4625.8	9433.8	6835.2
52	3.3	109	13566.7	17.3	4457.3	9109.4	6845.8
53	3.3	105	13085.2	17.1	4298.9	8786.4	6846.2
54	3.3	101	12614.6	16.9	4149.8	8464.8	6836.4
55	3.4	97	12154.2	16.7	4009.1	8145.0	6816.4
56	3.4	94	11703.4	16.4	3876.3	7827.1	6786.1
57	3.4	90	11261.7	16.2	3750.6	7511.2	6745.3
58	3.4	87	10828.7	16.0	3631.3	7197.3	6694.0
59	3.5	83	10403.7	15.8	3518.1	6885.6	6632.0
60	3.5	80	9986.3	15.6	3410.2	6576.0	6558.8
61	3.5	77	9576.0	15.4	3307.3	6268.6	6474.4
62	3.6	73	9172.3	15.2	3208.9	5963.5	6378.3
63	3.7	70	8774.9	15.0	3114.4	5660.5	6270.1
64	3.7	67	8383.3	14.8	3023.5	5359.8	6149.5
65	3.8	64	7997.0	14.5	2935.7	5061.3	6015.8



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$
$b = W-a$
$P_A = b*\tan(\alpha-\phi_{FS})$
$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 6846.2 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 47.4 pcf

Design Wall for an Equivalent Fluid Pressure: 48 pcf

Geotechnologies, Inc.
Project: Rodrigues Holdings
File No.: 20971

Soil Weight	γ	125	pcf
Internal Friction Angle	ϕ	22	degrees
Cohesion	c	0	psf
Height of Retaining Wall	H	17	feet

Restrained Retaining Wall Design based on At Rest Earth Pressure

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

$K_o = 1 - \sin\phi$	0.625	
$\sigma'_v = \gamma H$	2125.0	psf

σ'_h =	1329.0	psf	
EFP =	78.2	pcf	
P_o =	11296.2	lbs/ft	(based on a triangular distribution of pressure)

Design wall for an EFP of 79 pcf

Geotechnologies, Inc.
Project: Rodrigues Holdings
File No.: 20971

Seismically Induced Lateral Soil Pressure on Retaining Wall

Input:

Height of Retaining Wall:	(H)	17.0	feet
Retained Soil Unit Weight:	(γ)	125.0	pcf
Horizontal Ground Acceleration:	(k_h)	0.26	g

Seismic Increment (ΔP_{AE}):

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

$\Delta P_{AE} =$ 3522.2 lbs/ft

Force applied at 0.6H above the base of the wall
Transfer load to 2/3 of the height of the wall

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

T = 3170.0 lbs/ft

$EFP = 2*T/H^2$

EFP = 22 pcf

triangular distribution of pressure

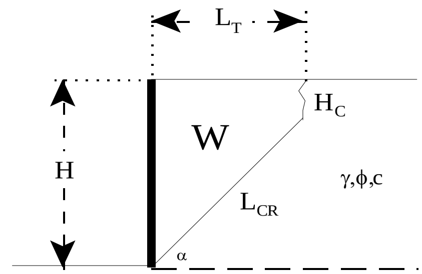
Geotechnologies, Inc.

Project: Rodrigues Holdings
File No.: 20971
Description: Temporary Shoring Wall up to 12 feet High

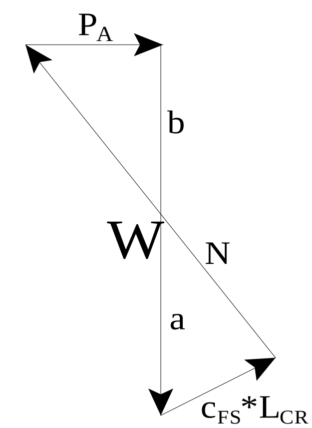
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	12.00	feet
Unit Weight of Retained Soils	(γ)	125.0	pcf
Friction Angle of Retained Soils	(ϕ)	22.0	degrees
Cohesion of Retained Soils	(c)	240.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(ϕ_{FS})	17.9	degrees
	(c_{FS})	192.0	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	5.1	70	8808.3	10.8	5235.1	3573.2	1450.0
41	4.9	69	8599.9	10.8	5014.6	3585.3	1528.4
42	4.8	67	8383.7	10.7	4804.0	3579.7	1600.4
43	4.7	65	8162.5	10.7	4603.7	3558.8	1666.2
44	4.6	64	7938.2	10.6	4413.5	3524.7	1725.8
45	4.5	62	7712.3	10.6	4233.2	3479.0	1779.4
46	4.5	60	7485.9	10.5	4062.5	3423.4	1827.0
47	4.4	58	7260.1	10.4	3900.9	3359.2	1868.8
48	4.4	56	7035.4	10.3	3748.0	3287.4	1904.7
49	4.3	54	6812.3	10.2	3603.1	3209.2	1935.0
50	4.3	53	6591.1	10.1	3465.7	3125.4	1959.6
51	4.3	51	6372.1	10.0	3335.5	3036.6	1978.7
52	4.2	49	6155.5	9.9	3211.8	2943.7	1992.2
53	4.2	48	5941.3	9.7	3094.2	2847.2	2000.2
54	4.2	46	5729.7	9.6	2982.2	2747.5	2002.7
55	4.2	44	5520.5	9.5	2875.3	2645.2	1999.7
56	4.2	43	5313.8	9.4	2773.2	2540.6	1991.2
57	4.3	41	5109.4	9.2	2675.5	2434.0	1977.2
58	4.3	39	4907.4	9.1	2581.6	2325.8	1957.7
59	4.3	38	4707.6	9.0	2491.3	2216.3	1932.6
60	4.4	36	4509.9	8.8	2404.2	2105.6	1901.8
61	4.4	35	4314.1	8.7	2319.9	1994.2	1865.3
62	4.5	33	4120.0	8.5	2238.0	1882.0	1823.1
63	4.5	31	3927.7	8.4	2158.2	1769.5	1774.9
64	4.6	30	3736.7	8.2	2080.0	1656.7	1720.8
65	4.7	28	3547.0	8.0	2003.2	1543.8	1660.6



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\,max}$ 2002.7 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 27.8 pcf

Design Shoring for an Equivalent Fluid Pressure: 28 pcf

Geotechnologies, Inc.

Project: Rodrigues Holdings

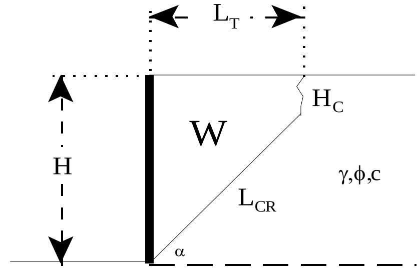
File No.: 20971

Description: Temporary Shoring Wall 12 to 15 feet High

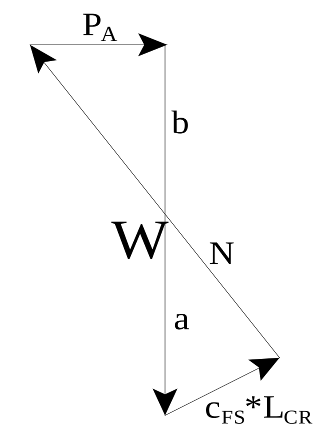
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	15.00	feet
Unit Weight of Retained Soils	(γ)	125.0	pcf
Friction Angle of Retained Soils	(ϕ)	22.0	degrees
Cohesion of Retained Soils	(c)	240.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(ϕ_{FS})	17.9	degrees
	(c_{FS})	192.0	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	5.1	119	14841.5	15.4	7502.6	7338.9	2978.2
41	4.9	115	14423.6	15.3	7145.0	7278.6	3102.8
42	4.8	112	14006.2	15.2	6810.9	7195.3	3216.8
43	4.7	109	13591.4	15.1	6499.0	7092.4	3320.5
44	4.6	105	13180.5	14.9	6207.7	6972.9	3414.2
45	4.5	102	12774.8	14.8	5935.4	6839.3	3498.1
46	4.5	99	12374.7	14.6	5680.8	6693.9	3572.4
47	4.4	96	11981.0	14.5	5442.5	6538.5	3637.5
48	4.4	93	11593.7	14.3	5219.1	6374.6	3693.5
49	4.3	90	11213.0	14.2	5009.5	6203.5	3740.5
50	4.3	87	10839.0	14.0	4812.6	6026.5	3778.6
51	4.3	84	10471.7	13.8	4627.3	5844.3	3808.1
52	4.2	81	10110.8	13.7	4452.8	5658.0	3829.0
53	4.2	78	9756.2	13.5	4288.0	5468.2	3841.4
54	4.2	75	9407.8	13.3	4132.3	5275.5	3845.3
55	4.2	73	9065.3	13.2	3984.8	5080.5	3840.7
56	4.2	70	8728.5	13.0	3844.9	4883.5	3827.5
57	4.3	67	8397.1	12.8	3711.9	4685.1	3805.9
58	4.3	65	8070.8	12.6	3585.2	4485.6	3775.6
59	4.3	62	7749.5	12.5	3464.2	4285.2	3736.7
60	4.4	59	7432.7	12.3	3348.4	4084.3	3688.9
61	4.4	57	7120.3	12.1	3237.2	3883.0	3632.1
62	4.5	54	6811.8	11.9	3130.2	3681.6	3566.3
63	4.5	52	6507.1	11.7	3026.8	3480.4	3491.1
64	4.6	50	6205.8	11.5	2926.5	3279.3	3406.3
65	4.7	47	5907.6	11.3	2828.9	3078.8	3311.8



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\,max}$ 3845.3 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 34.2 pcf

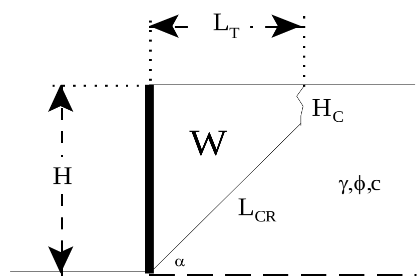
Design Shoring for an Equivalent Fluid Pressure: 35 pcf

Geotechnologies, Inc.
Project: Rodrigues Holdings
File No.: 20971
Description: Temporary Shoring Wall 15 to 20 feet High

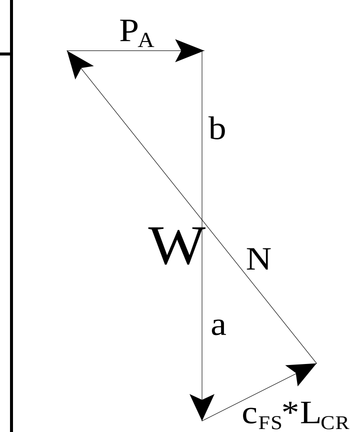
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	20.00	feet
Unit Weight of Retained Soils	(γ)	125.0	pcf
Friction Angle of Retained Soils	(ϕ)	22.0	degrees
Cohesion of Retained Soils	(c)	240.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(ϕ_{FS})	17.9	degrees
	(c_{FS})	192.0	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	5.1	223	27876.3	23.2	11281.9	16594.5	6734.3
41	4.9	216	27005.8	23.0	10695.6	16310.2	6952.9
42	4.8	209	26153.5	22.7	10155.7	15997.8	7152.2
43	4.7	203	25320.4	22.4	9657.9	15662.5	7332.9
44	4.6	196	24506.6	22.1	9198.0	15308.7	7495.7
45	4.5	190	23712.3	21.9	8772.4	14939.9	7641.2
46	4.5	183	22937.0	21.6	8377.9	14559.1	7769.9
47	4.4	177	22180.3	21.3	8011.6	14168.7	7882.4
48	4.4	172	21441.8	21.0	7670.9	13770.9	7978.9
49	4.3	166	20720.8	20.8	7353.5	13367.3	8059.9
50	4.3	160	20016.7	20.5	7057.3	12959.4	8125.6
51	4.3	155	19328.7	20.3	6780.4	12548.3	8176.4
52	4.2	149	18656.1	20.0	6521.0	12135.0	8212.4
53	4.2	144	17998.2	19.8	6277.8	11720.4	8233.6
54	4.2	139	17354.4	19.5	6049.2	11305.2	8240.3
55	4.2	134	16723.8	19.3	5834.0	10889.8	8232.3
56	4.2	129	16105.9	19.0	5631.1	10474.8	8209.8
57	4.3	124	15500.0	18.8	5439.4	10060.6	8172.5
58	4.3	119	14905.3	18.5	5257.9	9647.4	8120.5
59	4.3	115	14321.4	18.3	5085.7	9235.6	8053.4
60	4.4	110	13747.5	18.1	4922.1	8825.4	7971.0
61	4.4	105	13183.0	17.8	4766.1	8416.9	7873.1
62	4.5	101	12627.4	17.6	4617.1	8010.3	7759.2
63	4.5	97	12080.1	17.3	4474.4	7605.6	7629.1
64	4.6	92	11540.4	17.1	4337.3	7203.1	7482.1
65	4.7	88	11007.9	16.9	4205.0	6802.9	7317.7



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 8240.3 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 41.2 pcf

Design Shoring for an Equivalent Fluid Pressure: 42 pcf