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Celebrating 50 YEARS OF SERVICE 1971-2021

September 24, 2021
File No. 22167

Bardas Investment Group
1015 North Fairfax Avenue
West Hollywood, California 90046

Attention: Collin Monsour

Subject: Geotechnical Engineering Investigation
Proposed Adaptive Re-Use Development
1200 through 1210 North Cahuenga Boulevard, 6337 through 6351 West Lexington Avenue, and 6332 through 6356 West La Mirada Avenue, Los Angeles, California

Ladies and Gentlemen:

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.

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TABLE OF CONTENTS

SECTION **PAGE**

TABLE OF CONTENTS

SECTION **PAGE**

TABLE OF CONTENTS

SECTION **PAGE**

GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED ADAPTIVE RE-USE DEVELOPMENT
1200 THROUGH 1210 NORTH CAHUENGA BOULEVARD,
6337 THROUGH 6351 WEST LEXINGTON AVENUE,
AND 6332 THROUGH 6356 WEST LA MIRADA AVENUE,
LOS ANGELES, CALIFORNIA

INTRODUCTION

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

This investigation included four 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 indicated on the enclosed Plot Plans. The results of the exploration and the laboratory testing are presented in the Appendix of this report.

Previous Site Investigations

This firm has obtained geotechnical engineering reports by previous consultants for the site. A report was prepared by Hakimian Geotechnical Consultants, Inc, Project No. H 01-1102, dated December 17, 2001, which included exploratory excavations in the northern and eastern portions of the project site. This investigation was submitted for a development consisting of a two-story school building with subterranean parking and a playground area underlain by subterranean parking. The report included four exploratory borings and laboratory testing. This previous geotechnical report was reviewed and approved by the City of Los Angeles Department of

Building and Safety in the letter dated December 17, 2001 (Log# 37757). The findings presented in the previous investigation by Hakimian Geotechnical Consultants were considered during the preparation of this report. The Plot Plan and corresponding boring logs from this document are included in the Appendix.

A subsequent site investigation was prepared by Irvine Geotechnical, Inc, Project No. IC 16007-C, dated February 22, 2016, which consisted of exploratory excavations in the southwest corner of the project site. This investigation included recommendations for a development consisting of an interior remodeling and seismic retrofit of an existing school building. The report included five exploratory test pit excavations and laboratory testing. This previous geotechnical report was reviewed and approved by the City of Los Angeles Department of Building and Safety in the letter dated April 4, 2016 (Log# 92540). The findings presented in the previous investigation by Irvine Geotechnical were considered during the preparation of this report. The Plot Plan and corresponding boring logs from this document are included in the Appendix.

PROPOSED DEVELOPMENT

Information concerning the proposed development was furnished by the client. The anticipated development will consists of an adaptive re-use of an existing school campus which will include the demolition of existing school buildings, construction of newly proposed office structures and renovation of an existing school building for commercial purposes. Development details are presented as follows:

The northern portion of the site is currently occupied by recreational playground areas and a single subterranean parking level underlying the existing playfield. It is proposed that a four-story office building designated as “Building A” will be constructed in this region of the site along La Mirada Avenue. The proposed structure is anticipated to include a subterranean parking level significantly deep enough to accommodate double-stack parking systems. The finish floor elevation of the existing subterranean level is estimated at 307 feet above sea level and will be replaced with a deeper subterranean level with a finish floor elevation of 299.5 feet. Details of the proposed “Building A” is provided on the enclosed Plot Plan – Proposed Development and Cross-Sections A-A’ and B-B’.

The existing school building designated as “Building B” in the southeast section of the site consists of a two-story structure above ground surface and includes a single subterranean parking level. It is anticipated that this structure will undergo interior renovations for re-use as a commercial office building. Modification or expansion of the existing foundation system is not anticipated.

The southwest corner of the site is currently occupied by an at-grade, two-story school building. This structure will be demolished and replaced with an at-grade office building designated as “Building C” and will consist of three sections ranging from two to four stories in height. Architectural details of the proposed “Building C” are indicated on the attached Plot Plan – Proposed Development.

Column loads are estimated to range between 200 and 500 kips. Wall loads are estimated to range from 5 kips to 10 kips per lineal foot. It is anticipated that grading will consist of excavations to an approximate depth of 20 feet below the existing grade for construction of the proposed subterranean garage level for “Building A’ including foundation elements and elevator pit enclosures. In addition, removal and recompaction of existing site soils will be required to create a certified building pad for support of “Building C”.

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 is located at 1200 Cahuenga Boulevard in the City of Los Angeles, California. The site is roughly rectangular in shape and approximately 1.2 acres in area. The site is bounded by La Mirada Avenue to the north, a parking lot with a subterranean level and a three-story apartment building with a partial subterranean level toward the east, Lexington Avenue to the south and Cahuenga Boulevard to the west. The location of the site relative to nearby cultural features is indicated on the attached Vicinity Map. The site boundaries are indicated on the attached Plot Plans.

The northern portion of the site is currently developed with recreational areas underlain by a single subterranean parking level. The southern section of the site consists of at-grade, two-story school buildings including a single level of subterranean parking underlying the structure located in the southeast portion of the site designated as “Building B”. Vegetation includes planter islands with mature trees. Details of the existing development is indicated on the Plot Plan – Existing Development and Cross-Sections A-A’ and B-B’ enclosed herein.

Elevations on the site range from 315 above mean sea level (AMSL) at the north perimeter to 310 feet AMSL at the south perimeter. The site gradient is approximately 30 to 1 sloping downward toward the south. Drainage across the project site is by sheetflow toward the south and to city streets.

The neighboring developments consist of commercial and residential structures ranging from two to three stories in height.

Subterranean Parking Level of Adjacent Development – East Perimeter

A single subterranean parking level servicing the adjacent property lies to the east and extends approximately 10 feet below existing ground surface as indicated on the enclosed Cross-Section A-A’. The precise depth, position and orientation of the subterranean retaining wall, foundations or structural elements underlying the adjacent parking lot should be determined prior to construction.

New foundations for the proposed development should not be allowed to surcharge the existing retaining wall, foundations or structural elements associated the subterranean parking level which lies to the east. New conventional foundations positioned in close proximity to the east perimeter of the site should extend below the surcharge zone boundary line as indicated on Cross-Section A-A'.

LOCAL GEOLOGY

The site is located south of the Hollywood Hills which are composed of mixture of granitic, metamorphic, and sedimentary rocks (Dibblee, 1991). The Hills are an east-west trending ridge that is dissected by canyons and smaller gullies. The canyons flow to the south depositing their sediments into an area of several coalescing alluvial fans. The alluvial fan sediments consist primarily of sand, silt with some clay and few gravels, dipping gently to the south. The geology of the site vicinity is indicated on the attached Local Geologic Map - Dibblee.

Hollywood Fault

The Hollywood Fault is part of a 200 km-long, east-west trending line of oblique, reverse and left lateral faults (Dolan, et al., 1997). The Hollywood Fault trends along the base of the Hollywood Hills to the north and connects with the Raymond Fault to the east and the Santa Monica Fault to the west.

The Hollywood Fault is reverse, north-dipping fault located along the southern edge of the eastern Santa Monica Mountains (Dolan, J.F., Stevens, D., and Rockwell, T.K., 2000). The fault juxtaposes Miocene sedimentary rocks over Pleistocene and Holocene alluvium (Dolan et al., 1997) identified several geologic features in the Hollywood area that were believed to be fault scarps; the nearest is located at the toe of a slope found at Carlos Avenue, approximately 4,000 feet to the northeast of the site.

Based on recent work by several geotechnical engineering consultants and information compiled by the California Geological Survey, the Hollywood Fault has been found to be sufficiently-active and well-defined based on the criteria established by the California Geological Survey (Hernandez and Treiman, 2014a and Hernandez, 2014b). The fault location as indicated on the CGS map is based on substantial subsurface work performed on nearby properties as well as very detailed comparisons of current and historical survey data.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was explored on July 24, 2021 by excavating two borings and two test pits. The boring excavations ranged in depth from 20 feet below the subterranean parking level to 70 feet below existing ground surface – both borings are located near the northern perimeter of the site. Boring B1 was drilled with the aid of a truck-mounted drilling machine using 8-inch diameter hollowstem augers. Boring B2 was excavated with the aid of a 4-inch diameter hand auger. The two test pits were excavated near the southwest corner of the site, to depths of 20 feet below ground surface. The test pits were completed with the aid of manual labor. The boring locations are indicated on the Plot Plans and the geologic materials encountered are logged on Plates A-1 through A-4.

Soil samples were taken in Boring B1 at alternating depths with a California-modified, split-spoon sampler, and with a Standard Penetration Test (SPT). The California-modified, split-spoon sampler was lined with 2.5-inch diameter brass rings. The sampler was advanced with a 140-pound weight dropped from a height of 30 inches using an automatic trip hammer.

The locations of the borings were determined from hardscape features indicated on the attached Plot Plan drawings. Borings from previous site investigations by Hakimian Geotechnical Consultants and Irvine Geotechnical, Inc are also indicated on the attached Plot Plans for convenience. The locations of the exploratory excavations should be considered accurate only to the degree implied by the method used.

Geologic Materials

The site is underlain by fill soil and older alluvium. The boring locations are shown on the attached Plot Plans. The subsurface distribution of the geologic materials is indicated on the attached Cross-Sections A-A' and B-B'.

The fill soil consists of silty to sandy clay which is dark brown in color, moist, stiff and fine grained. The fill soil ranges in thickness from one to three feet. Older alluvial soil underlies the fill.

The older alluvium consists of silty to sandy clay, clayey sand, and silty sand to sand with occasional gravel. The older alluvium is dark grayish to reddish brown in color, is moist to wet, medium dense to dense, stiff and fine to medium grained.

More detailed descriptions of the geologic materials encountered may be obtained from the individual logs of the subsurface excavations. Local geologic conditions are indicated on the Local Geologic Map provided in the Appendix of this report.

Groundwater

Groundwater was encountered in one of the borings drilled as part of this investigation and in two borings from a previous site investigation from another firm (Hakimian Geotechnical) as indicated in the following table:

BORING NUMBER	GROUND SURFACE ELEVATION (Feet)	DEPTH TO WATER (Feet)	WATER ELEVATION (AMSL in Feet)
B1 (Geotechnologies)	315	27	288
B1 (Hakimian)	313 (Est.)	25	288
B2 (Hakimian)	311 (Est.)	25	286

The California Geological Survey Seismic Hazard Evaluation for the Los Angeles Quadrangle (1998 revised 2006) indicates the historic high groundwater level at a depth of 40 feet below ground surface. A copy of this plate is included in the Appendix as Historically Highest Groundwater Levels Map.

It is the assessment of this firm (based on water measurement observations) that the groundwater encountered during exploration represents the static groundwater level even though historically highest groundwater is estimated to be significantly deeper. Groundwater levels reported by Hakimian Geotechnical are conservatively assumed represent static groundwater as there was no mention of a perched or seepage groundwater condition underlying the site.

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. Groundwater is not anticipated to be encountered during excavation to the subgrade elevation for the proposed single-level basement of “Building A”.

Caving

Caving could not be directly observed during exploration due to the continuously cased design of the hollowstem augers. Based on the experience of this firm, large diameter excavations that encounter granular, cohesionless soils, and excavations below the groundwater table, will most likely experience caving.

SEISMIC EVALUATION

REGIONAL GEOLOGIC SETTING

The subject site is located in the northern portion of the Peninsular Ranges Geomorphic Province. The Peninsular Ranges are characterized by northwest-trending blocks of mountain ridges and sediment-floored valleys. The dominant geologic structural features are northwest trending fault zones that either die out to the northwest or terminate at east-trending reverse faults that form the southern margin of the Transverse Ranges.

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

REGIONAL FAULTING

Based on criteria established by the California Division of Mines and Geology (CDMG) now called California Geologic Survey (CGS), Faults may be categorized as Holocene-active, Pre-Holocene faults, and Age-undetermined faults. Holocene-active faults are those which show evidence of surface displacement within the last 11,700 years. Pre-Holocene faults are those that have not moved in the past 11,700 years. Age-undetermined faults are faults where the recency of fault movement has not been determined.

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. As revised in 2018, The Act defines "Holocene-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 11,700 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.

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. In addition, the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. The fault zone nearest to the project site is approximately 3,600 feet to the north and is identified as the Hollywood fault zone as indicated on the attached Earthquake Zones of Required Investigation map. Based on these considerations, the potential for surface ground rupture at the subject site is considered low.

Liquefaction

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

According to the Earthquake Zones of Required Investigation Map (CGS, 2014) the site is not located within a potentially liquefiable area. This determination is based on groundwater depth records, soil type and distance to a fault capable of producing a substantial earthquake. A copy of the Earthquake Zones of Required Investigation map is included in the Appendix of this report.

A site-specific liquefaction analysis was performed in accordance with 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 City of Los Angeles Information Bulletin P/BC 2020-151, and the EERI Monograph (MNO-12) by Idriss and Boulanger (2008). This semi-empirical method is based on a correlation between measured values of Standard Penetration Test (SPT) resistance and field performance data.

Groundwater was encountered in Boring 1 during exploration at a depth of 27 feet below ground surface. According to the Seismic Hazard Zone Report of the Hollywood 7½-Minute Quadrangle (CDMG, 1998, Revised 2006), the historically high groundwater level for the subject site is estimated at 40 feet below ground surface. A groundwater level of 27 feet below ground surface was conservatively used in the enclosed liquefaction analyses.

The peak ground acceleration (PGA_M) and modal magnitude were obtained from the USGS website using the Probabilistic Seismic Hazard Deaggregation program (USGS, 2021) and the Structural Engineers Association of California in collaboration with the Office of Statewide Health Planning and Development (SEAOCC/OSHPD, 2021), ground motion utility tool. A Site Class "D" ("Stiff Soil" Profile) was utilized in the USGS seismic and SEAOCC/OSHPD ground motion utility tools. A modal magnitude (MW) of 6.9 was obtained using the USGS Probabilistic Seismic Hazard Deaggregation program (USGS, 2021). A peak ground acceleration PGA_M of 0.99g, corresponding to a seismic event with a mean return interval of 2,475 years (2% exceedance in 50 years) was obtained using the SEAOCC/OSHPD seismic hazard utility tool. The peak ground acceleration for seismic event corresponding to 2/3 PGA_M was estimated by multiplying 0.99g by 2/3 for a result of 0.66g. These parameters were used in the enclosed liquefaction analyses.

Samples of the collected soils were conveyed to the laboratory for testing and analysis. The percent passing a Number 200 sieve and Atterberg limit test results of representative samples of the soils encountered in the exploratory boring are presented on the enclosed E-Plate and F-Plate for Boring 1. Based on CGS Special Publication 117A (CDMG, 2008), the vast majority of liquefaction hazards are associated with sandy soils and silty soils of low plasticity.

The procedure presented in the SP117A guidelines was followed in analyzing the liquefaction potential of the subject site. The SP117A 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 and LADBS Information Bulletin P/BC 2020-151, soils having a Plastic Index greater than 18 exhibit clay-like behavior, the liquefaction potential of these soils are considered to be low. Where the results of Atterberg Limits testing showed a Plastic Index greater than 18, or where the Plastic Index is between 7 and 18 with a saturated moisture content less than 80 percent of the liquid limit, the soils would be considered non-liquefiable and the analysis of these soil layers was deactivated in the liquefaction susceptibility column.

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

Dynamic Dry Settlement

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

Some seismically-induced settlement of the proposed structure 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.

Seiches are oscillations generated in enclosed bodies of water which can be caused by ground shaking associated with an earthquake. The County of Los Angeles Flood and Inundation Hazards Map, Leighton (1990) was reviewed. This map identifies areas that would be impacted in the event of a catastrophic failure of an upgradient dam. The map indicates the site lies within a mapped inundation boundary caused by a seiche or a breach of the Hollywood Reservoir. A determination of whether a higher site elevation would preclude flooding from this source 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 in slope 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 office building development is considered feasible from a geotechnical engineering standpoint provided the advice and recommendations presented herein are followed and implemented during construction.

Geology and Geologic Hazards

The site is underlain by fill soil and older alluvial soil. The fill soil was observed to range in thickness from one to three feet and consists of silty to sandy clay which is dark brown in color, moist, stiff and fine grained. Older alluvial soil underlies the fill and consists of silty to sandy clay, clayey sand, and silty sand to sand with occasional gravel. The older alluvium is dark grayish to reddish brown in color, is moist to wet, medium dense to dense, stiff and fine to medium grained.

Groundwater was encountered in one of the borings drilled as part of this investigation and in two borings from a previous site investigation by Hakimian Geotechnical Consultants. Groundwater observed during site exploration and in previous borings by Hakimian Geotechnical ranged from 25 feet to 27 feet below ground surface. The historically highest groundwater level is estimated at 40 feet below ground surface.

The site is not located within an earthquake fault zone. The liquefaction potential of the site was considered to be remote based on a site specific liquefaction analysis.

Foundation Design – "Building A"

The existing fill soil is not suitable for support of the proposed foundations, floor slabs or additional fill. The existing subterranean parking level which currently exists along the northern perimeter of the site will be demolished to accommodate a deeper parking level designed to accommodate double-stack parking systems for the proposed “Building A”. Excavation of the deeper subterranean level for “Building A” is anticipated to remove unsuitable fill soil within the building footprint. The proposed “Building A” structure may be supported by conventional foundations bearing in older alluvial soil exposed at the base of the proposed excavation.

An adjacent development to the east of the subject site consists of an existing parking lot underlain by a single subterranean parking level. New foundations for the proposed development shall not be allowed to surcharge the existing retaining wall, foundations or structural elements associated the adjacent parking structure which lies to the east. Conventional foundations positioned in close proximity to the east perimeter of the site should be deepened to extend below the surcharge zone boundary line as indicated on Cross-Section A-A’.

The excavation for the proposed subterranean level will require shoring to provide a stable working area due to the proposed depth of excavation, and the proximity of adjacent structures. Solder pile excavations (if required) will likely encounter groundwater and may require mitigation measures for caving and concrete construction as recommended in this report.

Foundation Design – “Building C”

All existing fill materials and any soils disturbed as a result of demolition of the existing school structure shall be completely removed within the building area and recompacted for foundation and slab support. The proposed “Building C” may be supported on conventional foundations bearing in an engineered building pad consisting of certified recompacted fill. The proposed engineered building pad shall extend a minimum of five feet below the existing site grade, or a minimum of three feet below the bottom of the proposed foundations, whichever is greater. In addition, the proposed recompacted building pad shall be over-excavated a minimum of three feet horizontally beyond the edge of foundations or for a distance equal to the depth of fill below the foundations, whichever is greater. If the required overexcavation cannot be achieved for exterior foundations immediately adjacent to the property line or adjacent structures, foundations should extend through the compacted fill to bear in the underlying competent alluvial soils. Any imported fill materials shall be verified and tested by this office prior to usage on site.

New foundations for the proposed development should not be allowed to surcharge the existing retaining wall, foundations or structural elements associated the adjacent structure ("Building B") which lies to the east. Conventional foundations positioned in close proximity to the existing "Building B" shall be deepened so that existing foundations or retaining walls are not surcharged.

General

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

The validity of the conclusions and design recommendations presented herein is dependent upon review of the geotechnical aspects of the proposed construction by this firm. The subsurface conditions described herein have been projected from 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.

SEISMIC DESIGN CONSIDERATION

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 7-16. This information and the site coordinates were input into the SEAOC/OSHPD seismic utility program in order to calculate ground motion parameters for the site.

CALIFORNIA BUILDING CODE SEISMIC PARAMETERS	
California Building Code	2019
ASCE Design Standard	7-16
Risk Category	II
Site Class	D
Mapped Spectral Acceleration at Short Periods (S_S)	2.096g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	2.096g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.398g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.747g
Site Coefficient (F_v)	1.7*
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	1.270g*
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.847g*

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

FILL SOIL

Fill depths ranging from one foot to three feet were encountered during site exploration. The existing fill soils are not suitable for the support of foundations, floor slabs or additional fill but may be reused as compacted fill.

EXPANSIVE SOILS

The onsite geologic materials are in the very low to moderate expansion range. The Expansion Index was found range from 15 to 68 for bulk samples taken from a depth of 1 to 5 feet below ground surface. Building slab reinforcement recommendations are provided in the "Slabs-on Grade" section 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 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, the sulfate exposure is considered to be negligible for geologic materials with less than 0.1% and Type I cement may be utilized for concrete foundations in contact with the site soils.

METHANE ZONES

Based on review of the 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

Site Preparation

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

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

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

- Subsequent to the indicated removals, the exposed grade shall be scarified to a depth of six inches, moistened to within three percent of 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.

Subgrade Preparation and Soil Mixing

Once the onsite soils have been removed it is recommended that they should be well blended to reduce the overall expansion index of the newly placed controlled fill. Where the site grading will result in a net export, the sandier or more granular materials should be segregated from the stockpiled soils and the more clayey or expansive materials should be exported. Where the importation of soil will be needed, it is recommended that the imported soil consist of granular materials, with low expansion properties. Samples of the segregated, imported and/or blended soils should be tested by this office to ascertain the expansion index prior to placement and compaction.

Recommended Overexcavation

The area designated for construction of "Building C" shall be excavated to a minimum depth of five feet below the existing site grade, or three feet below the bottom of the proposed foundations, whichever is greater, to create an engineered fill pad for support of the proposed structure. In addition, the proposed recompacted fill pad shall be overexcavated a minimum of three feet horizontally beyond the edge of foundations or for a distance equal to the depth of fill below the foundations, whichever is greater. It is very important that the position of the proposed structure is accurately located so that the limits of the graded area are accurate and the grading operation proceeds efficiently.

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

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 soils are considered satisfactory for reuse in the controlled fills as long as any debris and/or organic matter is removed. Any imported soils shall be observed and tested by the representative of the geotechnical engineer prior to use in fill areas. Imported soils should contain sufficient fines so as to result in a stable subgrade when compacted. Any required import soils should consist of geologic materials with an expansion index of less than 40. The water-soluble sulfate content of the import soils should be less than 0.1% percentage by weight.

Imported soils 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 soils 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 some of the soils which will be exposed during grading and at the bottom of the excavations were locally above optimum moisture content. It is anticipated that the some of the excavated material to be placed as compacted fill, and some of the materials exposed at the bottom of excavated planes may require drying and aeration prior to recompaction.

Pumping (also known as 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 and Bulking

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 alluvial soils on the site to an average comparative compaction of 96 percent.

Weather Related Grading Considerations

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

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

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

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

Abandoned Seepage Pits

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

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

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

LEED Considerations

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

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

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

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

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

Building A

The proposed "Building A" may be supported by conventional foundations bearing in competent older alluvial soil. It is anticipated that the excavation for the proposed deeper subterranean parking level will remove any existing fill soil and expose competent older alluvium at the subgrade.

Conventional Foundation Bearing Capacity – Building A

Continuous foundations bearing in alluvial soils may be designed for a bearing capacity of 3,000 pounds per square foot and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material.

Column foundations bearing in alluvial soils may be designed for a bearing capacity of 3,500 pounds per square foot and should be a minimum of 24 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material.

The bearing capacity increase for each additional foot of width is 200 pounds per square foot. The bearing capacity increase for each additional foot of depth is 600 pounds per square foot. The maximum recommended bearing capacity is 6,000 pounds per square foot.

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

A minimum factor of safety of 3 was utilized in determining the allowable bearing capacities. The bearing values indicated above are for the total of dead and frequently applied live loads and may be increased by one third for short duration loading, which includes the effects of wind or seismic forces. Since the recommended bearing value is a net value, the weight of concrete in the foundations may be taken as 50 pounds per cubic foot and the weight of the soil backfill may be neglected when determining the downward load on the foundations.

Conventional Foundation Lateral Design – Building A

Resistance to lateral loading may be provided by friction acting at the base of foundations and by passive earth pressure. An allowable coefficient of friction of 0.35 may be used with the dead load forces.

Passive geologic pressure for the sides of foundations poured against undisturbed alluvial soil may be computed as an equivalent fluid having a density of 220 pounds per cubic foot with a maximum earth pressure of 2,200 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.

Building C

The proposed "Building C" may be supported on conventional foundations bearing in an engineered building pad consisting of certified recompacted fill. The proposed engineered building pad shall extend a minimum of five feet below the existing site grade, or a minimum of three feet below the bottom of the proposed foundations, whichever is greater. In addition, the proposed recompacted building pad shall be over-excavated a minimum of three feet horizontally beyond the edge of foundations or for a distance equal to the depth of fill below the foundations, whichever is greater. If the required overexcavation cannot be achieved for exterior foundations immediately adjacent to the property line or adjacent structures, foundations should extend through the compacted fill to bear in the underlying competent alluvial soils.

Conventional Foundation Bearing Capacity – Building C

Continuous foundations bearing in recompacted fill or alluvial soils may be designed for a bearing capacity of 2,800 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 bearing in recompacted fill or alluvial soils may be designed for a bearing capacity of 3,300 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 130 pounds per square foot. The bearing capacity increase for each additional foot of depth is 500 pounds per square foot. The maximum recommended bearing capacity is 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.

A minimum factor of safety of 3 was utilized in determining the allowable bearing capacities. The bearing values indicated above are for the total of dead and frequently applied live loads and may be increased by one third for short duration loading, which includes the effects of wind or seismic forces. Since the recommended bearing value is a net value, the weight of concrete in the foundations may be taken as 50 pounds per cubic foot and the weight of the soil backfill may be neglected when determining the downward load on the foundations.

Conventional Foundation Lateral Design – Building C

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 0.33 may be used with the dead load forces.

Passive geologic pressure for the sides of foundations poured against certified, recompacted soil or alluvium may be computed as an equivalent fluid having a density of 200 pounds per cubic foot with a maximum earth pressure of 2,000 pounds per square foot.

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

<u>Miscellaneous Conventional Foundations</u>

Conventional foundations for structures such as privacy walls or trash enclosures which will not be rigidly connected to the proposed building may be deepened through any existing fill to bear in undisturbed native soils. Continuous footings may be designed for a bearing capacity of 1,500 pounds per square foot and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material. No bearing capacity increases are recommended.

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

Conventional Foundations Adjacent to Buildings or Property Lines

Foundations for the proposed “Building A” and “Building C” should not be allowed to surcharge any existing subterranean retaining wall or deep foundations of an adjacent development. The surcharge zone of a foundation is defined as a line drawn down and away at a declined slope gradient of 1 to 1 (horizontal to vertical) from the outer bottom surface of a foundation as indicated on Cross-Section A-A’. Where necessary, the proposed foundations in near proximity to subterranean retaining walls or deep foundations shall be deepened such that the existing retaining walls or foundations do not fall within the surcharge zone of the proposed foundation. Where new foundations are proposed immediately adjacent to an existing deep foundation, the new foundations should be deepened to match the depth of the adjacent foundation. Where foundation excavations will leave an adjacent foundation unsupported, the foundation excavation should be slot cut or shored.

Deepened Footings

Conventional footings may be required to extend into native alluvial soil when in close proximity to property lines or adjacent structures and compacted fill overexcavation cannot be achieved. In addition, deepened foundations may be required to prevent surcharge of an existing foundation or retaining wall.

The deepened portion of the footings may be filled with concrete of the same mix as that specified for the footing. The initial pour would not require reinforcing as it is simply passing the load through to the recommended bearing material. Once the initial pour has hardened, the footing may be reinforced and poured on top of the first pour. Some method of creating a positive bond between the two pours should be employed. 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.

Conventional 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.

Conventional Foundation Settlement

The majority of the foundation settlement is expected to occur on initial application of loading. Based on static settlement calculations, it is anticipated that a maximum settlement on the order of 1-inch will occur beneath the heaviest loaded column foundations for "Building A" and "Building C". Differential settlement is not expected to exceed 0.5-inch within a span of 30 feet.

Conventional Foundation Observations

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

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

RETAINING WALL DESIGN

Cantilever Retaining Walls

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

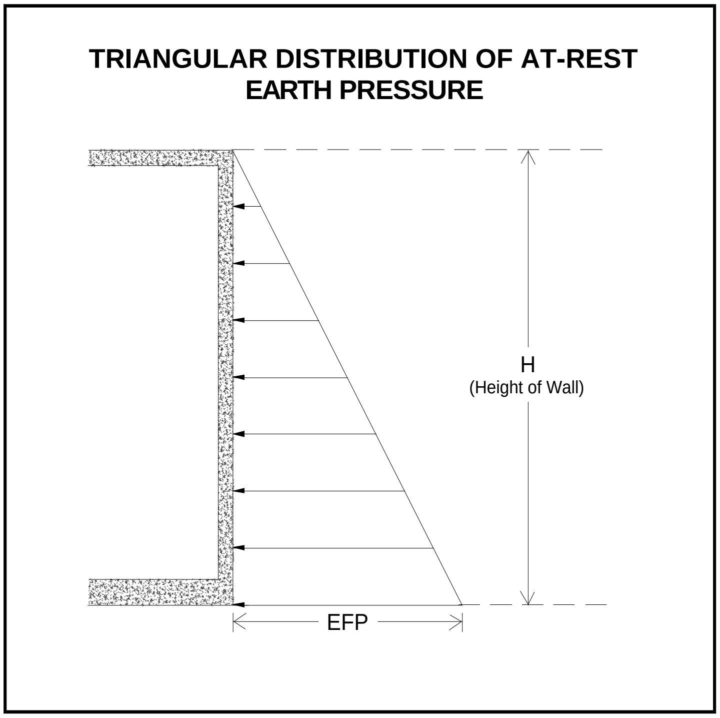
HEIGHT OF RETAINING WALL "H" (feet)	EQUIVALENT FLUID PRESSURE (pounds per cubic foot)
Up to 10	30
10 to 15	34

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

In addition to the recommended earth pressure, the upper ten feet of the retaining wall adjacent to streets, driveways or parking areas should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the walls due to normal street traffic. If the traffic is kept back at least ten feet from the retaining walls, the traffic surcharge may be neglected.

Restrained Drained Retaining Walls

Restrained retaining walls may be designed to resist a triangular pressure distribution of at-rest earth pressure as indicated in the diagram below. The at-rest pressure for design purposes would be 67 pounds per cubic foot. Additional earth pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures.



In addition to the recommended earth pressure, the upper ten feet of the retaining wall adjacent to streets, driveways or parking areas should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the walls due to normal street traffic. If the traffic is kept back at least ten feet from the retaining walls, the traffic surcharge may be neglected.

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

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 27.8 pounds per cubic foot. When using the load combination equations from the building code, the seismic earth pressure should be combined with the lateral active earth pressure for analyses of restrained basement walls under seismic loading condition.

Surcharge from Adjacent Structures

As indicated herein, additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures for retaining walls and shoring design.

The following surcharge equation provided in the LADBS Information Bulletin Document No. P/BC 2020-083, 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 bottom of wall footing measured in feet.
d	=	depth of lateral resultant below point of application of surcharge loading measure in feet.
$\tan^{-1}(h/x)$	=	the angle in radians whose tangent is equal to h/x.

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

Waterproofing

Moisture affecting retaining walls is one of the most common post construction complaints. Poorly applied or omitted waterproofing can lead to efflorescence or standing water inside the building. Efflorescence is a process in which a powdery substance is produced on the surface of the concrete by the evaporation of water. The white powder usually consists of soluble salts such as gypsum, calcite, or common salt. Efflorescence is common to retaining walls and does not affect their strength or integrity.

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

Retaining Wall Drainage

All retaining walls shall be provided with a subdrain 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 4 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 back-drainage system is to relieve hydrostatic pressure. Groundwater was encountered during site exploration and during previous site investigations at depths ranging from 25 feet to 27 feet below ground surface. The historically highest groundwater level is estimated at 40 feet below ground surface.

For retaining wall drainage systems extending less than 15 feet below existing ground surface, the water anticipated from the wall drainage system will be from rainfall, watering and leaky pipes, etc. A pump capacity of 5 gallons per minute is considered sufficient.

Retaining Wall Backfill

Any required backfill should be mechanically compacted in layers not more than 8 inches thick, to at least 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum density obtainable by the most recent revision of ASTM

D1557 method of compaction. Flooding is not permitted. Compaction within five feet, measured horizontally, behind a retaining structure should be achieved by use of light weight, hand operated compaction equipment.

A compacted fill blanket or other seal shall be provided at the surface. Retaining walls may be backfilled with gravel adjacent to the wall to within two feet of the ground surface. The onsite earth materials are acceptable for use as retaining wall backfill as long as they are compacted to a minimum of 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum density as determined by ASTM D1557.

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

TEMPORARY EXCAVATIONS

Excavations on the order of 20 feet in vertical height may be required for the subterranean parking level, anticipated elevator pit enclosures, and foundation elements of “Building A’. The excavations are expected to expose fill and dense native soils, which are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic or structures. Excavations which will be surcharged by adjacent traffic or structures should be slot cut or shored.

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

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

Excavations Adjacent to Existing Foundations, Buildings or Property Lines

Where excavations will leave an adjacent property or adjacent foundation unsupported, the proposed excavation should be slot cut or shored. The slot cutting method employs the earth as a buttress and allows the earth excavation to proceed in phases. Alternate "A" slots of 8 feet may be worked. The remaining earth buttresses ("B" and "C" slots) should each be 8 feet in width for a combined intervening length of 16 feet. The grading should be completed or the foundation should be poured in the "A" slots before the "B" slots are excavated. After completing the grading or foundation in the "B" slots, finally the "C" slots may be excavated.

Calculations indicating that slots 8 feet in width will be stable for the maximum recommended height of 8 feet have been included in the appendix of this report. These calculations include a conservative surcharge load to be produced by adjacent foundations or vehicular traffic.

Trench Shoring

Temporary vertical excavations exceeding a height of 5 feet, or excavations that will be surcharged by adjacent foundations during construction, may require stabilization with a temporary trench shoring system. Temporary trench shoring may consist of plywood, timber struts and angle braces, or a hydraulic trench shoring system. Temporary shoring and bracing systems up to 12 feet in height should be designed for a triangular pressure distribution with a minimum equivalent fluid pressure of 25 pounds per cubic foot. Additional active pressure should be added for a surcharge condition due to adjacent structures, foundations or vehicular traffic. It is recommended that a qualified shoring contractor be retained to determine the acceptable materials and procedures to be utilized for shoring.

The design team and contractor must be aware that the use of temporary shoring may impede the continuous construction of foundations. Foundations may require to be poured in several phases to accommodate for the removal of the trench shoring, while maintaining a stable excavation.

Temporary Bracing and False-Work

Temporary support of existing building elements while retaining walls and foundations are constructed may be necessary. Provisions for this phase of construction are expected to include temporary bracing or false-work, temporary foundations and trench shoring. Temporary foundations may bear in natural alluvial soils and may be designed in accordance with the "Conventional Foundation Design" section of this report.

Excavation Observations

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

SHORING DESIGN

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

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

Soldier Piles – Drilled and Poured

Drilled cast-in-place soldier piles should be placed no closer than 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 geologic materials. For design purposes, an allowable passive value for the geologic materials below the bottom plane of excavation may be assumed to be 550 pounds per square foot per foot for isolated piles. Piles are considered isolated if spaced at least 3 diameters on center. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed geologic materials.

Groundwater was observed at depths ranging from 25 feet to 27 feet below ground surface based on this investigation and a previous site investigation by Hakimian Geotechnical. Piles placed below the water level require the use of a tremie to place the concrete into the bottom of the hole. A tremie shall consist of a water-tight tube having a diameter of not less than 4 inches and connected to a concrete pump. The tube shall be equipped with a valve that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie shall be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end shall be closed at the start of the work to prevent water entering the tube and shall be entirely sealed at all times, except when the concrete is being placed. The tremie tube shall be kept full of concrete. The flow shall be continuous until the work is completed and the resulting concrete seal shall be monolithic and homogeneous. The tip of the tremie tube shall always be kept about five feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

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

Drilling mud or drilling polymer may be required if caving is encountered in granular (or saturated) geologic materials. If mud or polymer is used, the concrete shall be tremied into the hole as described in the paragraphs above. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet.

The frictional resistance between the soldier piles and retained geologic material may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.35 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 450 pounds per square foot. The minimum depth of embedment for shoring piles is 5 feet below the bottom of the footing excavation or 7 feet below the bottom of excavated plane whichever is deeper. These values assumed that the shoring piles will not be vibrated into place.

Lagging

Soldier piles and anchors should be designed for the full anticipated pressures. Due to arching in the geologic materials, the pressure on the lagging will be less. It is recommended that the lagging should be designed for the full design pressure but 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.

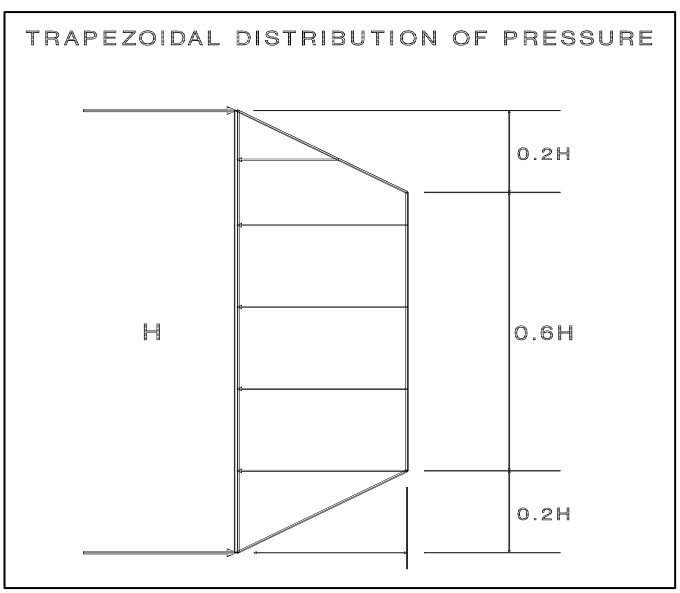
Lateral Pressures

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 15	25
15 to 20	31

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

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 15	18H
16 to 20	20H

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

<u>Tied-Back Anchors</u>

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 may be designed for a skin friction of 450 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 2,500 pounds per square foot could be utilized for post-grouted anchors. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads.

Anchor Installation

Tied-back anchors may be installed between 20 and 45 degrees below the horizontal. Caving of the anchor shafts, particularly within saturated sand deposits, should be anticipated and the following provisions should be implemented in order to minimize such caving. The anchor shafts should be filled with concrete by pumping from the tip out, and the concrete should extend from the tip of the anchor to the active wedge. In order to minimize the chances of caving, it is recommended that the portion of the anchor shaft within the active wedge be backfilled with sand before testing the anchor. This portion of the shaft should be filled tightly and flush with the face of the excavation. The sand backfill should be placed by pumping; the sand may contain a small amount of cement to facilitate pumping.

Tieback Anchor Testing

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

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

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

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

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

Raker Brace Foundations

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 4 feet in width and length as well as 4 feet in depth. The base of the raker foundations should be horizontal. Care should be employed in the positioning of raker foundations so that they do not interfere with the foundations for the proposed structure.

Deflection

It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized that some deflection will occur. It is estimated that the deflection could be on the order of ½ inch at the top of the shored embankment. If greater deflection occurs during construction, additional bracing may be necessary to minimize settlement of adjacent buildings and utilities in adjacent street and alleys. If desired to reduce the deflection, a greater active pressure could be used in the shoring design.

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.

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 for slabs not subjected to vehicular loading. Slabs-on-grade should be cast over certified compacted fill. 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 for concrete not subjected to vehicular loading. Outdoor concrete flatwork should be cast over undisturbed 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, where necessary, it is recommended that a qualified consultant should 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 on various components of the structure.

Where any dampness would be objectionable or where the slab will be cast below the historic high groundwater level, it is recommended that floor slabs should be waterproofed. A qualified waterproofing consultant should be engaged in order to recommend a product and/or method which would provide protection from unwanted moisture.

Based on ACI 302.2R-30, Chapter 7, for projects which do not have vapor sensitive coverings or humidity-controlled areas, a vapor retarder is not necessary. Where a vapor retarder is considered necessary, the design of the slab and the installation of the vapor retarder should comply with the most recent revisions of ASTM E1643 and ASTM E1745. The vapor retarder should comply with ASTM E1745 Class A requirements. The necessity of a vapor retarder is not a geotechnical issue and should be confirmed by qualified members of the design team.

Based on ACI 302.2R-30, Chapter 7, for projects with vapor sensitive coverings, a vapor barrier should be provided. The concrete slab should be poured directly on the vapor barrier. Where humidity-controlled areas are proposed and the base materials and slabs will not be within a water-tight system, the barrier should be covered with a 4-inch layer of dry granular material. ACI notes that the decision whether to locate the material in direct contact with the slab or beneath a layer of granular fill should be made on a case by case basis. The necessity of a vapor retarder as well as the use of dry granular material, as discussed above, is not a geotechnical issue and should be confirmed by qualified members of the design team.

ACI 302.2R-30, Chapter 7 discusses benefits derived from concrete poured on a granular layer as well as directly on the vapor retarder. Changes to the concrete used, such as slump, mix or admixtures are also discussed. This is also not a geotechnical issue and should be confirmed by qualified members of the design team. It is the recommendation of this firm that the design team become familiar with ACI 302.2R-30, Chapter 7.

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 95 percent of the maximum density as determined by the most recent revision of ASTM D1557. 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:

PAVING DESIGN SECTIONS				
Service Level	Asphalt Pavement		Concrete Pavement	
	Asphalt Pavement Thickness (Inches)	Asphalt Pavement Base Course (Inches)	Concrete Pavement Thickness (Inches)	Concrete Pavement Base Course (Inches)
Passenger Cars	3	5	6	4
Moderate Truck	4	8	6	4
Heavy Trucks	5	11	7.5	4

Aggregate base should be compacted to a minimum of 95 percent of the most recent revision of ASTM D1557 laboratory maximum dry density. Base materials should consist of Crushed Aggregate Base which conforms with Section 200-2.2 of the most recent edition of “Standard Specifications for Public Works Construction”, (Green Book). Crushed Misc. Base is addressed in Section or 200-2.4.

Concrete paving may be used on the project. A subgrade modulus of 75 pounds per cubic inch may be assumed for design of concrete paving. 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. Concrete paving should be reinforced with a minimum of #3 steel bars on 18-inch centers each way for paving not subjected to hydrostatic pressures. Concrete paving required to resist hydrostatic forces may require a revised design. Construction joints should be designed by a structural engineer.

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.

Percolation testing of the on-site soils was not conducted by this firm. However, based on the fines content of the majority of the site soils, it is the opinion of this firm that these soils will have poor infiltration capabilities. Allowing stormwater infiltration would result in a perched water condition.

In addition, groundwater was encountered below the subject site at depths between 25 and 27 feet below the ground surface during exploration. Current regulations require that the bottom of infiltration systems maintain a minimum vertical separation of 10 feet above the groundwater level. Based on the required vertical separation, and the shallowest depth to groundwater observed during exploration, any potential stormwater infiltration to be conducted at the site would have to occur within the upper 15 feet of soils. Stormwater infiltration is not recommendable within these upper soils, as it would saturate the strata which will provide primary support to the proposed foundations.

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

It is recommended that the design team (including the structural engineer, waterproofing consultant, plumbing engineer, and landscape architect) be consulted in regards to the design and construction of filtration systems.

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. Excavation and drilling in these areas may require full size equipment and coring capability. The contractor should be familiar with the site and the geologic materials in the vicinity.

CLOSURE AND LIMITATIONS

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

The recommendations of this report pertain only to the site investigated and are based upon the assumption that the geologic conditions do not deviate from those disclosed in the investigation. If any variations are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geotechnologies, Inc. should be notified so that supplemental recommendations can be prepared.

This report is issued with the understanding that it is the responsibility of the owner, or the owner's representatives, to ensure that the information and recommendations contained herein are brought to the attention of the project architect and engineer and are incorporated into the plans. The owner is also responsible to see that the contractor and subcontractors carry out the geotechnical recommendations during construction.

The findings of this report are valid as of the date of this report. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside control of this firm. Therefore, this report is subject to review and should not be relied upon after a period of three years.

Geotechnical observations and testing during construction are considered to be a continuation of the geotechnical investigation. It is, therefore, most prudent to employ the consultant performing the initial investigative work to provide observation and testing services during construction. This practice enables the project to flow smoothly from the planning stages through to completion.

Should another geotechnical firm be selected to provide the testing and observation services during construction, that firm should prepare a letter indicating their assumption of the responsibilities of geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for review. The letter should acknowledge the concurrence of the new geotechnical engineer with the recommendations presented in this report.

EXCLUSIONS

Geotechnologies, Inc. does not practice in the fields of methane gas, radon gas, environmental engineering, waterproofing, dewatering organic substances or the presence of corrosive soils or wetlands which could affect the proposed development including mold and toxic mold. Nothing in this report is intended to address these issues and/or their potential effect on the proposed development. A competent professional consultant should be retained in order to address environmental issues, waterproofing, organic substances and wetlands which might affect the proposed development.

GEOTECHNICAL TESTING

Classification and Sampling

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

Samples of the geologic materials encountered in the exploratory excavations were collected and transported to the laboratory. Undisturbed samples of soil are obtained at frequent intervals. Unless noted on the excavation logs as an SPT sample, samples acquired while utilizing a hollow-stem auger drill rig are obtained by driving a thin-walled, California Modified Sampler with successive 30-inch drops of a 140-pound hammer. The soil is retained in brass rings of 2.50 inches outside diameter and 1.00 inch in height. The central portion of the samples are stored in close fitting, waterproof containers for transportation to the laboratory. Samples noted on the excavation logs as SPT samples are obtained in accordance with the most recent revision of ASTM D1586. 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 D4959 or ASTM D4643. 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 D3080 with a strain controlled, direct shear machine manufactured by Soil Test, Inc. or a Direct Shear Apparatus manufactured by GeoMatic, Inc. 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 D3080 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 D2435. 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 D1557. A soil at a selected moisture content is placed in five layers into as mold of given dimensions, with each layer compacted by 25 blows of a 10-pound hammer dropped from a distance of 18 inches subjecting the soil to a total compactive effort of about 56,000 pounds per cubic foot. The resulting dry unit weight is determined. The procedure is repeated for a sufficient number of moisture contents to establish a relationship between the dry unit weight and the water content of the soil. The data when plotted represent a curvilinear relationship known as the compaction curve. The values of optimum moisture content and modified maximum dry unit weight are determined from the compaction curve. 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 D422 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

Depending on their moisture content, cohesive soils can be solid, plastic, or liquid. The water contents corresponding to the transitions from solid to plastic or plastic to liquid are known as the Atterberg Limits. The transitions are called the plastic limit and liquid limit. The difference between the liquid and plastic limits is known as the plasticity index. ASTM D 4318 is utilized to determine the Atterberg Limits. The results are shown on the enclosed Plate F.

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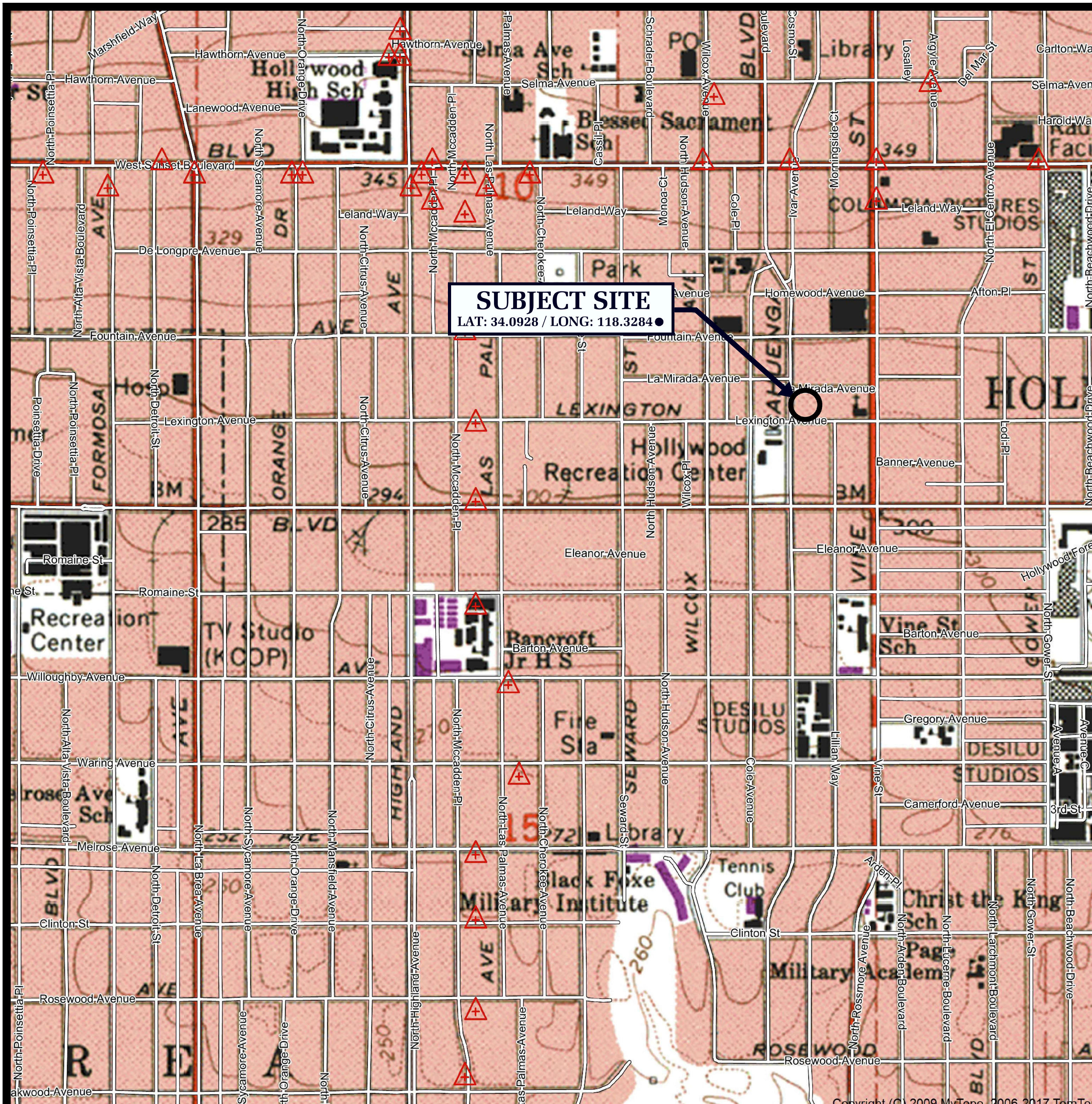

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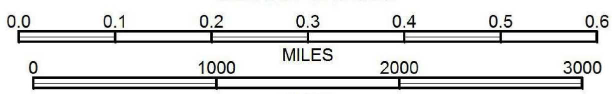

REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, HOLLYWOOD, CA QUADRANGLE

VICINITY MAP

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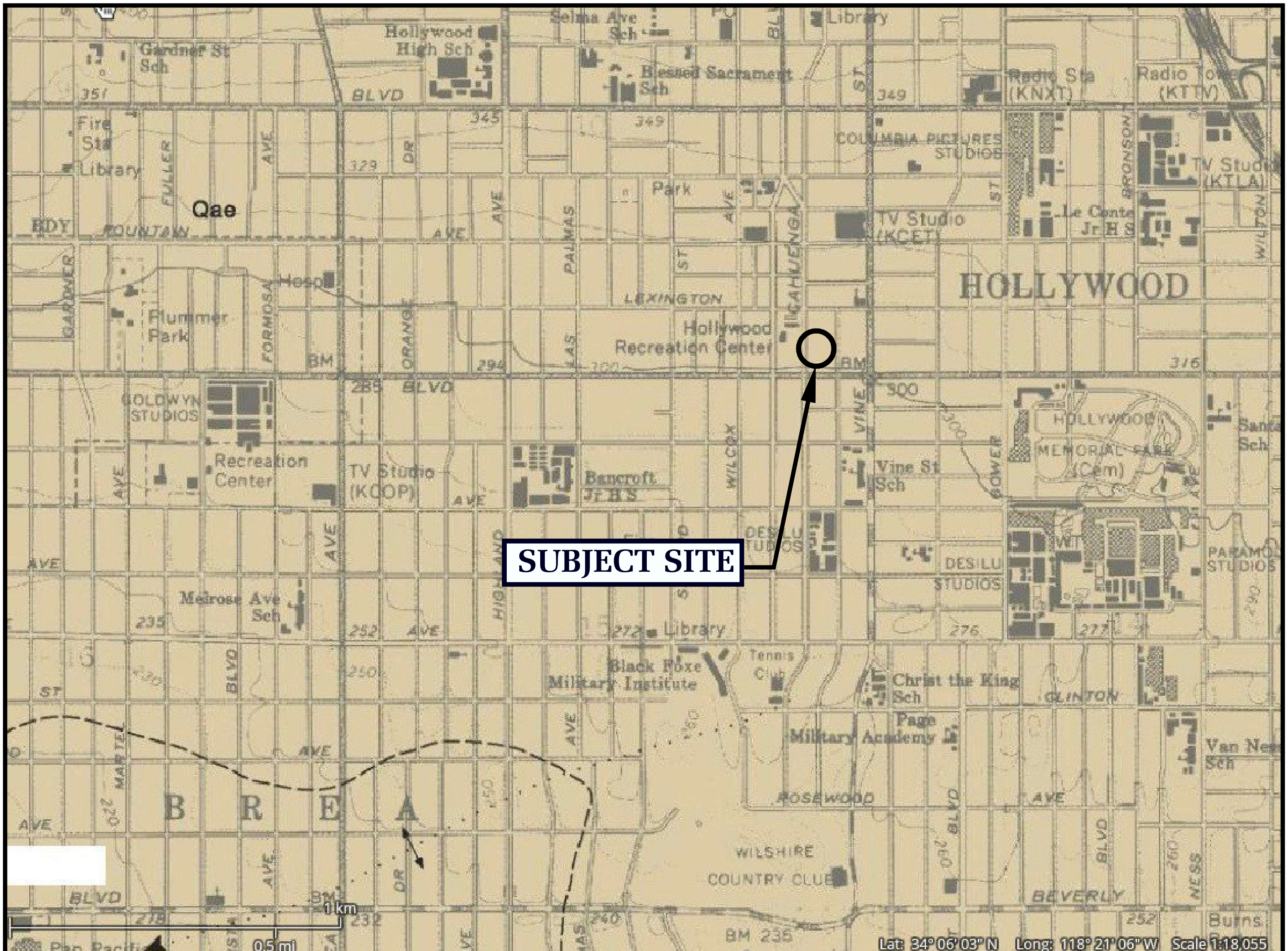
Consulting Geotechnical Engineers

BARDAS INVESTMENT GROUP
1200 CAHUENGA BLVD., LOS ANGELES

FILE NO. 22167



LEGEND

Qae: Older Surficial Sediments - alluvium: gravel, sand and clay, but slightly elevated and dissected

Folds - arrow on axial trace of fold indicates direction of plunge

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

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

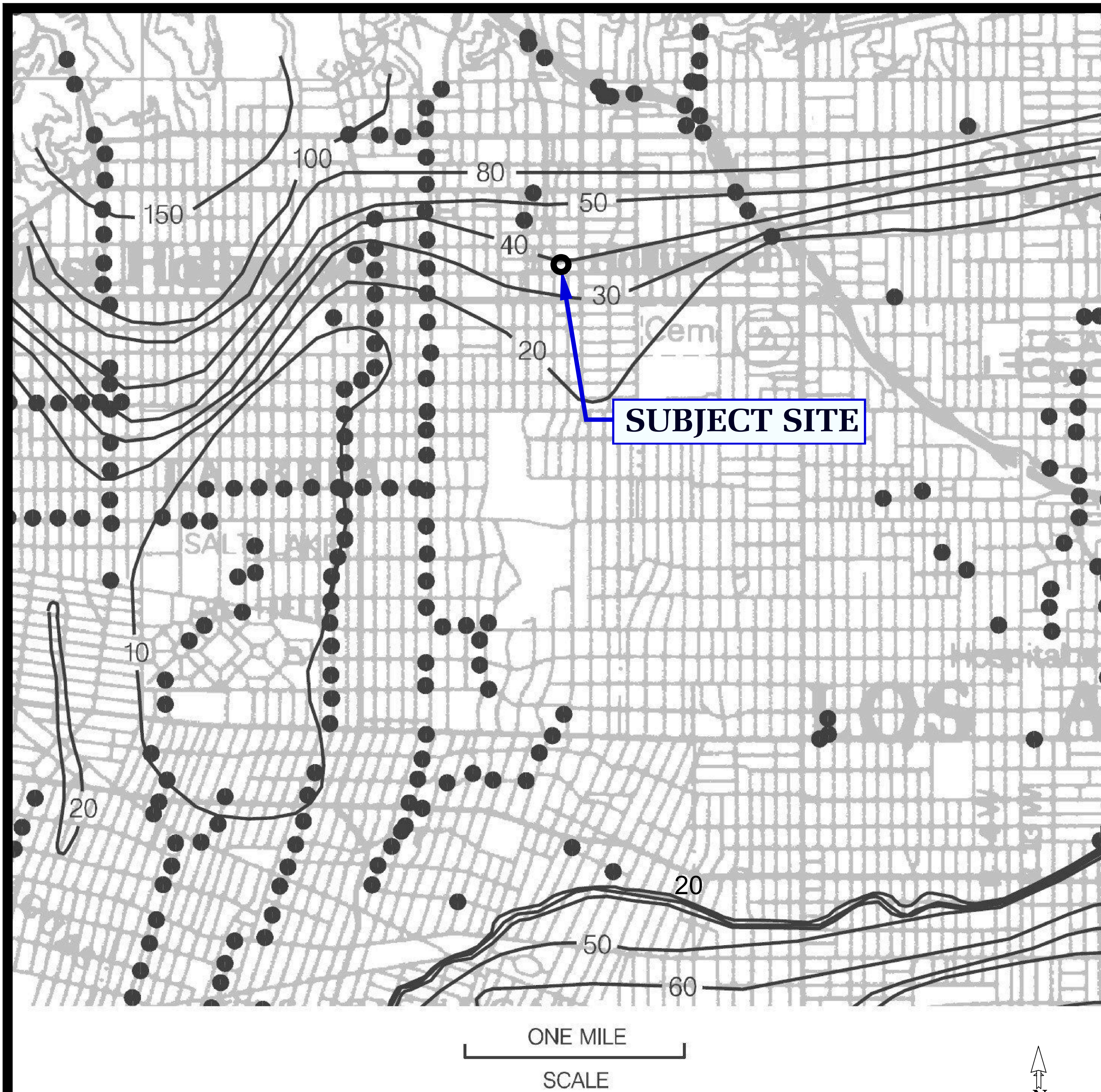
N

LOCAL GEOLOGIC MAP - DIBBLEE

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BARDAS INVESTMENT GROUP
1200 CAHUENGA BLVD. LOS ANGELES

FILE NO. 22167



20 Depth to groundwater in feet

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

HISTORICALLY HIGHEST GROUNDWATER LEVELS

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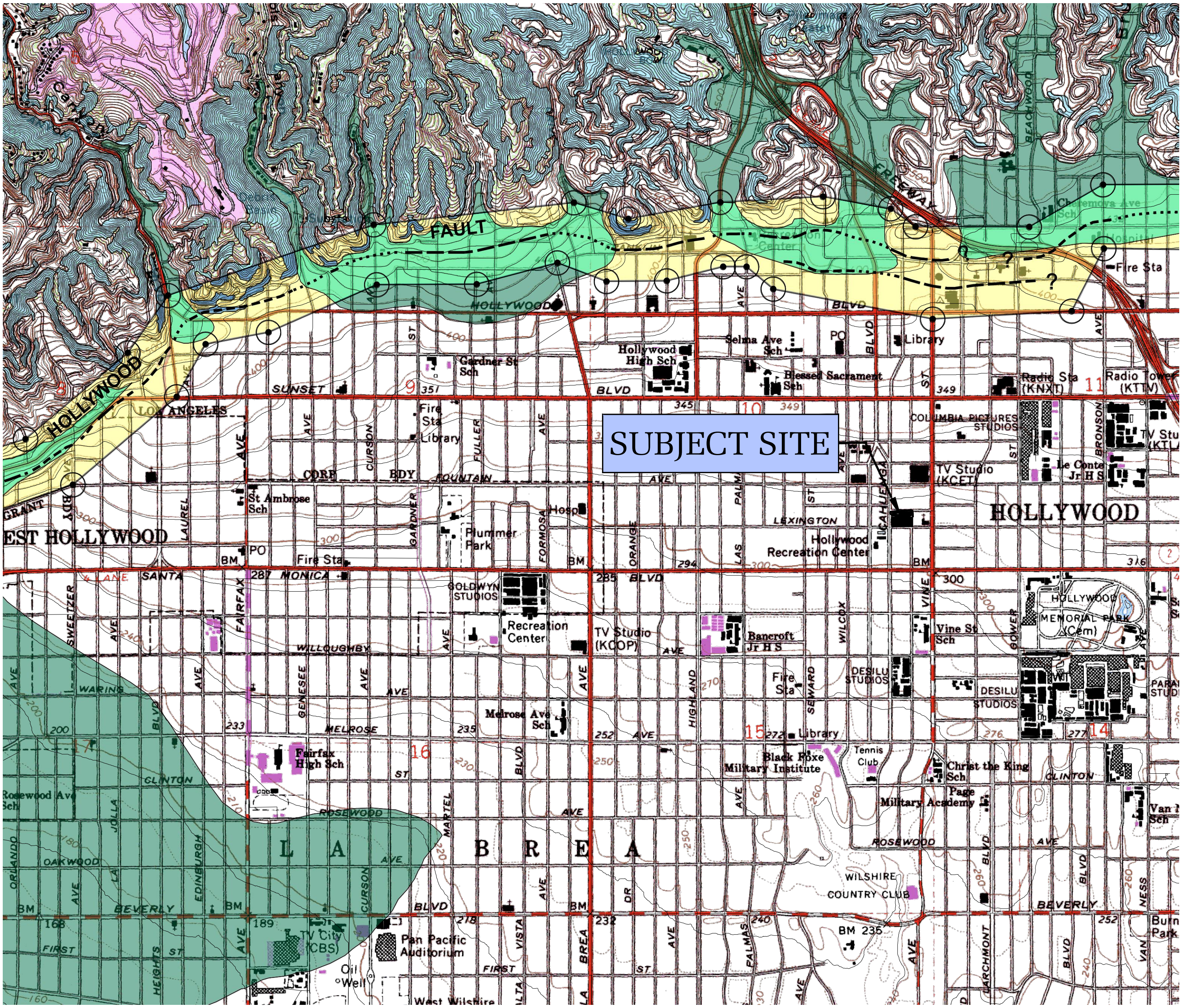
1200 CAHUENGA BLVD., LOS ANGELES

FILE NO. 22167



1 0.5 0 1 2 MILES

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

1 0.5 0 1 2 KILOMETERS

1000 500 0 1000 2000 METERS

Contour Interval 20 Feet

LIQUEFACTION ZONES

EARTHQUAKE FAULT ZONES

REFERENCE: EARTHQUAKE ZONES OF REQUIRED INVESTIGATION, HOLLYWOOD QUADRANGLE (CGS, 2014)

SPECIAL STUDIES ZONE MAP

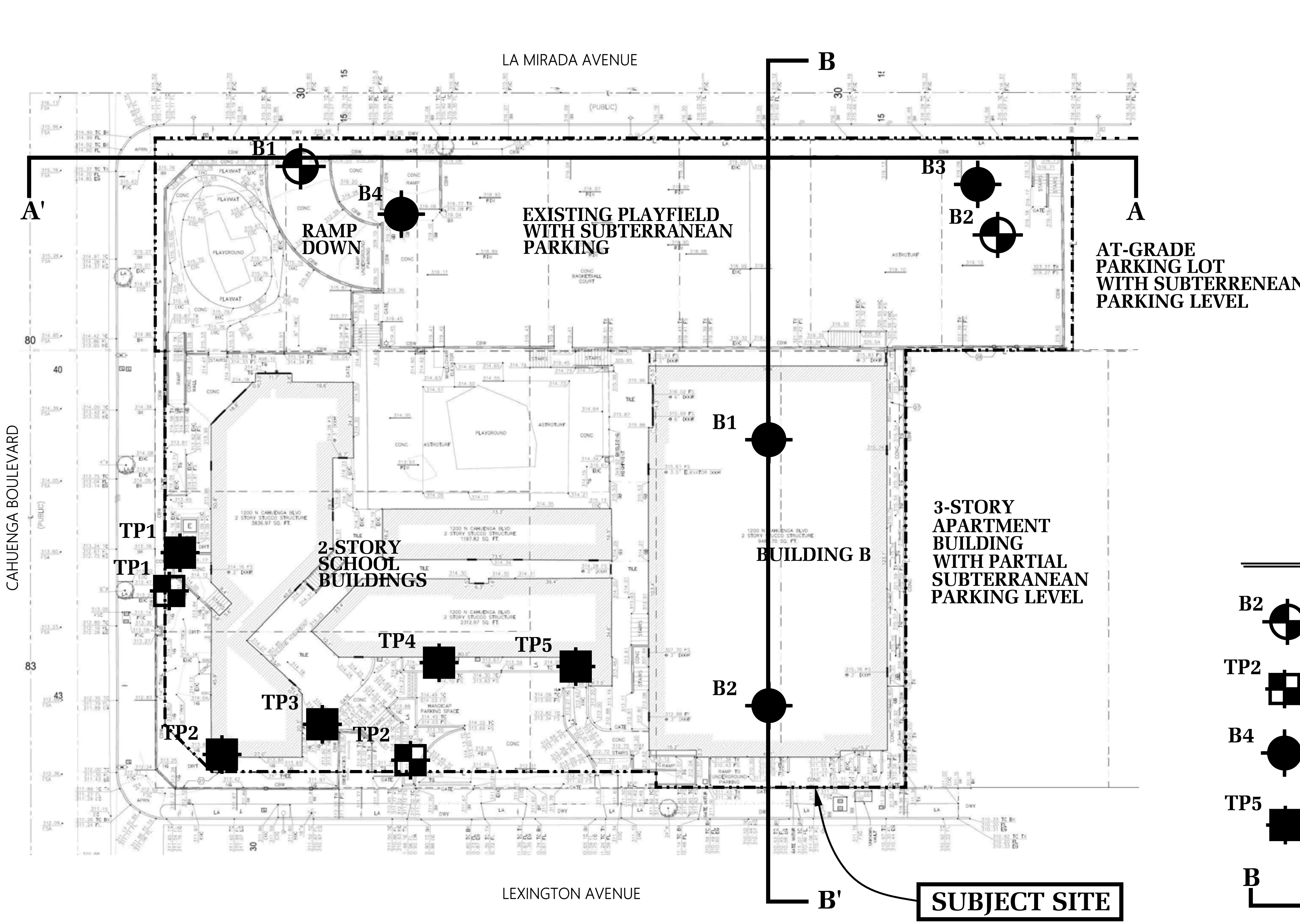
Geotechnologies, Inc.

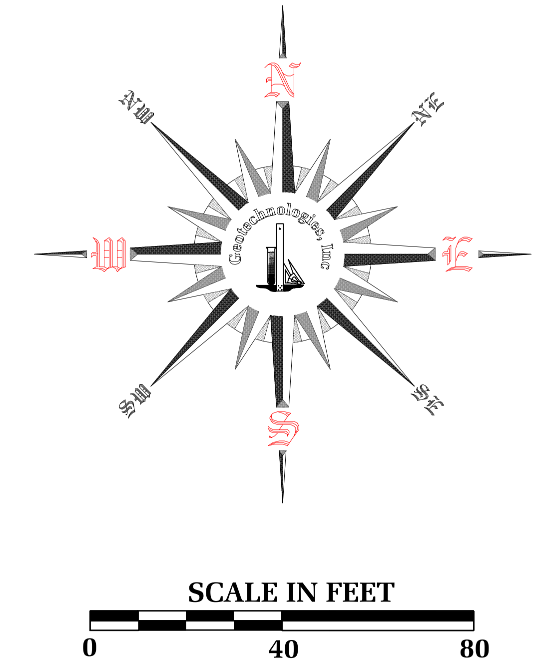
Consulting Geotechnical Engineers

BARDAS INVESTMENT GROUP

1200 CAHUENGA BLVD. LOS ANGELES

FILE NO: 22167





LEGEND

B2	LOCATION & NUMBER OF BORING
TP2	LOCATION & NUMBER OF TEST PIT
B4	APPROXIMATE LOCATION & NUMBER OF BORINGS PERFORMED BY HAKIMIAN GEOTECHNICAL CONSULTANTS, INC.
TP5	APPROXIMATE LOCATION & NUMBER OF TEST PITS PERFORMED BY IRVINE GEOTECHNICAL, INC.
B B'	CROSS SECTION

REFERENCE: DESIGN SURVEY PLAN PROVIDED BY KPFF
DATED: August 8, 2021

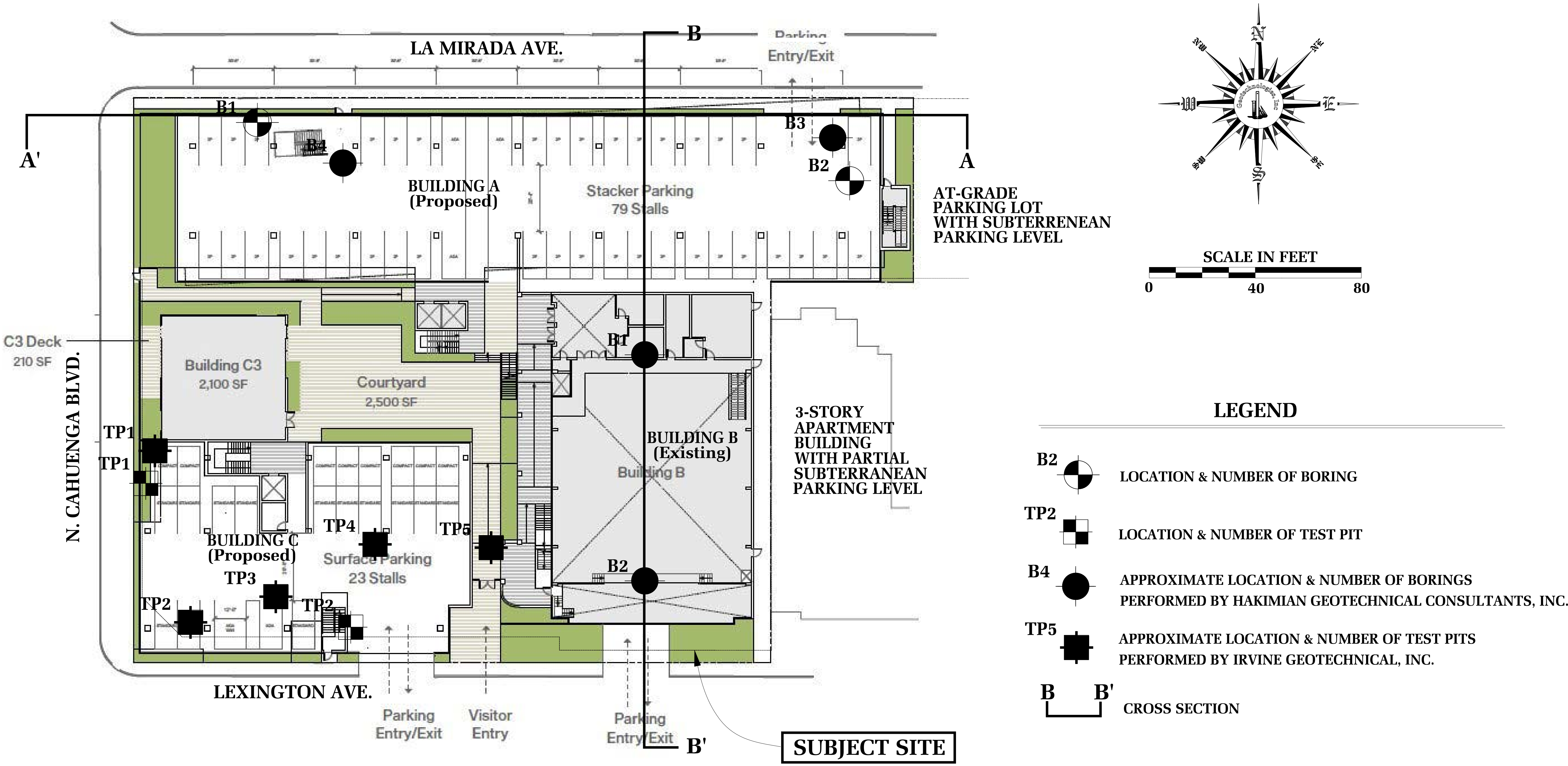
PLOT PLAN - EXISTING DEVELOPMENT

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BARDAS INVESTMENT GROUP
1200 CAHUENGA BLVD., LOS ANGELES

DRAWN BY: GN	FILE No. 22167

DATE: September 2021



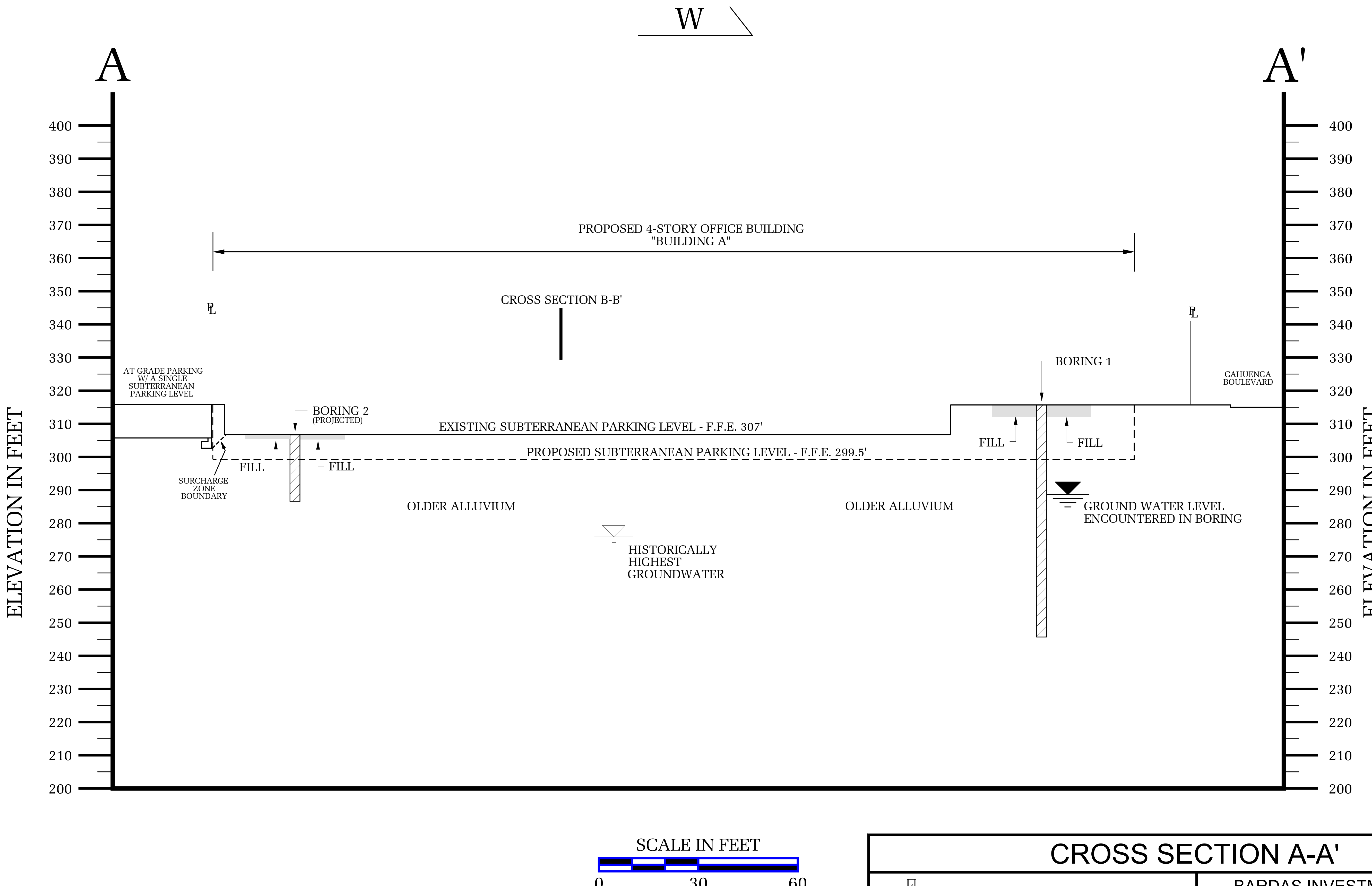
REFERENCE: LEVEL 01 PLAN PROVIDED BY HOUSE & ROBERTSON ARCHITECTS
DATED: May 27, 2021

PLOT PLAN - PROPOSED DEVELOPMENT

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BARDAS INVESTMENT GROUP
1200 CAHUENGA BLVD., LOS ANGELES

Drawn by: YD	File No. 22167
Date: August 2021	

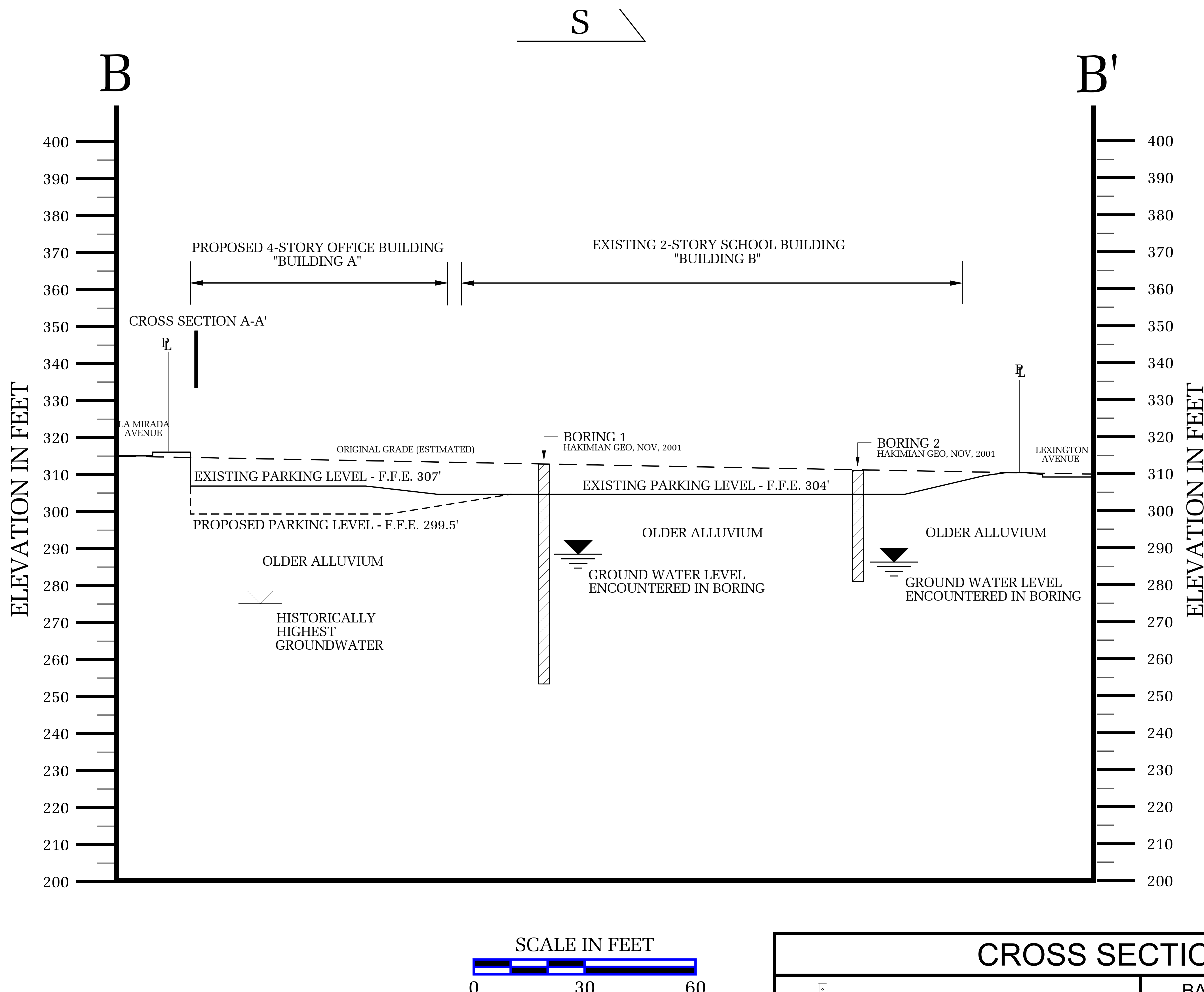


CROSS SECTION A-A'

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BARDAS INVESTMENT GROUP
1200 CAHUENGA BLVD., LOS ANGELES

Drawn by: YD	File No. 22167
Date: September 2021	



CROSS SECTION B-B'

Geotechnologies, Inc.

Consulting Geotechnical Engineers

BARDAS INVESTMENT GROUP

1200 CAHUENGA BLVD., LOS ANGELES

Drawn by: YD	File No. 22167
Date: September 2021	

BORING LOG NUMBER 1

Bardas Investment Group | **Date: 07/24/21** | **Elevation: 315'***

File No. 22167 | **Method: 8-Inch Diameter Hollow Stem Auger**

typist initials | * Design Survey by KPFF, dated August 8, 2021

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Concrete For Driveway
				0 --		6.8 Inches Concrete, 5 Inch Base
				-		
				1 --		
				-		FILL: Sandy Clay, dark brown, moist, stiff, fine grained
				2 --		
2.5	52	15.4	119.3	-		
				3 --		
				-	CL	OLDER ALLUVIUM: Sandy to Silty Clay, dark grayish and reddish brown, moist, stiff, fine grained
				4 --		
				-		
5	22	16.8	SPT	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	38	18.0	114.8	-		
				8 --		reddish brown
				-		
				9 --		
				-		
10	9	14.1	SPT	10 --		
				-		
				11 --		
				-		
				12 --		
12.5	24	19.1	101.5	-		
				13 --		
				-		
				14 --		
				-		
15	8	15.2	SPT	15 --		
				-	CL/SC	Sandy Clay to Clayey Sand, dark reddish brown, moist, dense, stiff, fine grained
				16 --		
				-		
				17 --		
17.5	27	25.3	100.3	-		
				18 --	CL	Silty Clay, dark reddish brown, moist, stiff, fine grained
				-		
				19 --		
				-		
20	13	11.2	SPT	20 --		
				-	SC/CL	Clayey Sand to Sandy Clay, dark reddish brown, moist, medium dense, stiff, fine grained
				21 --		
				-		
				22 --		
22.5	48	15.7	114.1	-		
				23 --	CL	Sandy to Silty Clay, dark and reddish brown, moist, stiff, fine grained
				-		
				24 --		
				-		
25	21	14.8	SPT	25 --		
				-		

BORING LOG NUMBER 1

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
27.5	78	13.7	120.8			
				28		sandy
				29		
30	27	16.9	SPT	30		silty, dark grayish brown
				31		
				32		
32.5	45	15.4	116.5			
				33		reddish brown
				34		
35	29	18.0	SPT	35	SC	Clayey Sand, dark and reddish brown, moist, dense, fine grained
				36		
				37		
37.5	36	20.5	105.9			
				38		clayey
				39		
40	9	22.4	SPT	40	SC/CL	Clayey Sand to Sandy Clay, dark and reddish brown, moist medium dense, stiff, fine grained
				41		
				42		
42.5	34	No Recovery				
				43		
				44		
45	22	16.6	SPT	45	SC	Clayey Sand, dark and reddish brown, moist, dense, fine grained
				46		
				47		
47.5	50	16.5	115.6			
				48		
				49		
50	25	15.1	SPT	50		sandy

BORING LOG NUMBER 1

Bardas Investment Group

File No. 22167

typist initials

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
52.5	56	11.8	127.3	-		
				53 --		clayey
				-		
				54 --		
				-		
55	47	12.6	SPT	55 --		
				-	SC/CL	Clayey Sand to Sandy Clay, dark reddish brown, moist,
				56 --		medium dense, stiff, fine grained
				-		
				57 --		
57.5	68	12.7	123.5	-		
				58 --	SC	Clayey Sand, dark reddish brown, moist, dense, fine grained
				-		minor pebbles
				59 --		
				-		
60	29	17.6	SPT	60 --		
				-	SM/SP	Silty Sand to Sand, reddish brown, wet, dense, fine grained
				61 --		
				-		
				62 --		
62.5	44	14.0	122.2	-		
				63 --	CL/SC	Sandy Clay to Clayey Sand, dark reddish brown, moist,
				-		medium dense, stiff, fine grained
				64 --		
				-		
65	26	18.5	SPT	65 --		
				-		
				66 --		
				-		
				67 --		
67.5	29	14.3	113.3	-		
	50/4"			68 --		
				-		
				69 --		
				-		Silty Sand to Sand, dark reddish brown, wet, dense, fine to
70	62	11.3	SPT	70 --	SM/SP	medium grained, minor pebbles
				-		Total Depth: 70 Feet
				71 --		Groundwater At 27 Feet
				-		Fill To 3 Feet
				72 --		
				-		NOTE: The stratification lines represent the approximate
				73 --		boundary between earth types; the transition may be gradual.
				-		
				74 --		Used 8-inch diameter Hollow-Stem Auger
				-		140-lb. Automatic Hammer, 30-inch drop
				75 --		Modified California Sampler used unless otherwise noted
				-		
						SPT=Standard Penetration Test

BORING LOG NUMBER 2

Bardas Investment Group **Date: 07/24/21** **Elevation: 307'***

File No. 22167 **Method: Hand Auger**

ln

* Architectural Elevation View by West of West, dated May 27, 2021

Sample Depth ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Concrete For Parking
			0 --		6.5 Inches Concrete, 11.5 Inch Base
			-		
			1 --		
			-		
2	11.5	106.4	2 --	CL/SC	OLDER ALLUVIUM: Silty Sand, dark brown, moist, medium
			-		dense, fined grained
			3 --		
			-		
4	17.7	95.5	4 --		
			-		
			5 --		
			-		
			6 --		
			-		
7	7.4	111.8	7 --		
			-		
			8 --	SM/SC	Silty to Clayey Sand, dark reddish brown, moist, dense, fine
			-		grained
			9 --		
			-		
10	33.4	88.1	10 --		
			-	CL	Silty Clay, dark reddish brown, moist, stiff, fine grained
			11 --		
			-		
			12 --		
			-		
			13 --		
			-		
			14 --		
			-		
15	14.8	113.4	15 --		
			-	SC/CL	Sandy Clay to Clayey Sand, dark and grayish brown, moist,
			16 --		medium dense, stiff, fine grained
			-		
			17 --		
			-		
			18 --		
			-		
			19 --		
			-		more moist
20	12.4	120.9	20 --		
			-		Total Depth: 20 Feet
			21 --		No Water
			-		No Fill
			22 --		
			-		NOTE: The stratification lines represent the approximate
			23 --		boundary between earth types; the transition may be gradual.
			-		
			24 --		Used 4-inch diameter Hand-Augering Equipment; Hand Sampler
			-		
			25 --		
			-		

LOG OF TEST PIT NUMBER 1

Bardas Investment Group | **Date: 07/24/21** | **Elevation: 314'***

File No.: 22167 | **Method: Hand Auger And Test Pit**

In

* Design Survey by KPFF, dated August 8, 2021

Sample Depth ft.	Moisture Content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
					Surface Conditions: Bare Ground
			0 --		FILL: Sandy Clay, dark brown, slightly moist, stiff, fine grained
			-		
1	5.1	99.0	1 --		
			-		moist, medium dense
			2 --		
			-	CL	OLDER ALLUVIUM: Silty to Sandy Clay, dark grayish brown, moist, stiff, fine grained
3	12.0	102.4	3 --		
			-		
			4 --		
			-		
5	13.3	113.7	5 --		
			-		
			6 --		
			-		
7	17.2	106.5	7 --		
			-		
			8 --		
			-		
			9 --		
			-		
10	14.7	110.5	10 --		
			-		
			11 --		
			-		
			12 --		
			-		
			13 --		
			-		
			14 --		
			-		
15	21.0	87.0	15 --		
			-		reddish brown
			16 --		
			-		
			17 --		
			-		
			18 --		
			-		
			19 --		
			-		grayish mottling
20	26.1	100.0	20 --		
			-		Total Depth: 20 Feet
			21 --		No Water
			-		Fill to 2 Feet
			22 --		
			-		NOTE: The stratification lines represent the approximate
			23 --		boundary between earth types; the transition may be gradual.
			-		
			24 --		Used 4-inch diameter Hand-Augering Equipment; Hand Sampler
			-		
			25 --		
			-		

LOG OF TEST PIT NUMBER 2

Bardas Investment Group **Date: 07/24/21** **Elevation: 313'***

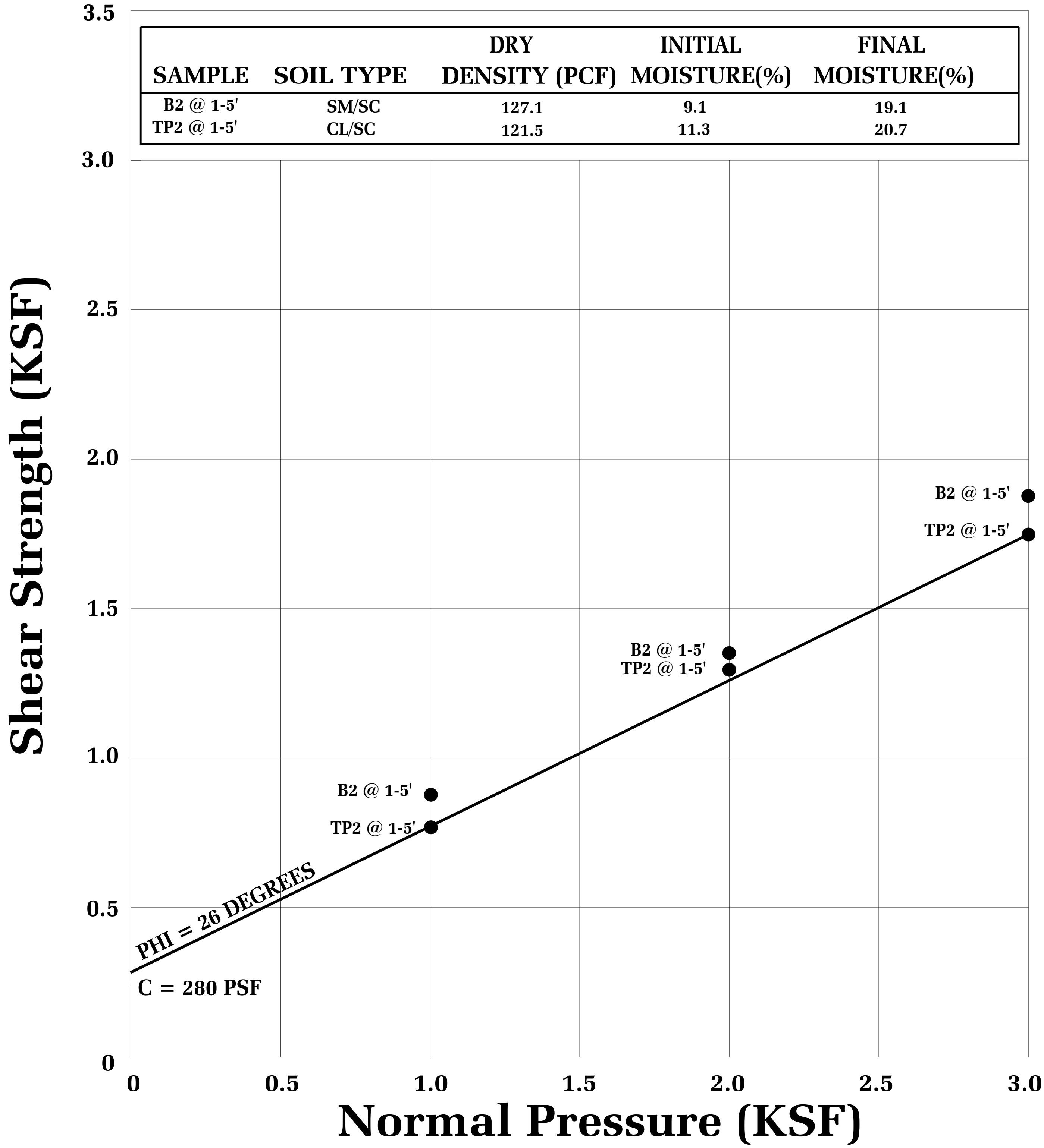
File No.: 22167 **Method: Hand Dig And Auger**

ln * Design Survey by KPFF, dated August 8, 2021

Sample Depth ft.	Moisture Content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
			0 --		FILL: Silty to Sandy Clay, dark brown, moist, medium dense, fine grained
			-		
			1 --		
			-		
2	11.3	99.0	2 --		
			-	CL	OLDER ALLUVIUM: Silty Clay, dark grayish brown, moist, stiff, fine grained
			3 --		
			-		
4	12.9	108.0	4 --		
			-		
			5 --		
			-		
			6 --		
			-		
7	13.5	114.2	7 --		
			-		
			8 --		
			-		
			9 --		
			-		
10	13.4	103.0	10 --		
			-		
			11 --		
			-		
			12 --		
			-		
			13 --		
			-		
			14 --		
			-		
15	15.6	93.4	15 --		
			-	CL/SC	Sandy Clay to Clayey Sand, dark reddish brown, moist, medium dense, fine grained
			16 --		
			-		
			17 --		
			-		
			18 --		
			-		
			19 --		
			-		grayish brown
20	10.6	98.1	20 --		
			-		Total Depth: 20 Feet
			21 --		No Water
			-		Fill To 2 Feet
			22 --		
			-		NOTE: The stratification lines represent the approximate
			23 --		boundary between earth types; the transition may be gradual.
			-		
			24 --		Used 4-inch diameter Hand-Augering Equipment; Hand Sampler
			-		
			25 --		
			-		

BULK SAMPLE REMOLDED TO 90 PERCENT OF THE MAXIMUM LABORATORY DENSITY

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B2 @ 1-5'	SM/SC	127.1	9.1	19.1
TP2 @ 1-5'	CL/SC	121.5	11.3	20.7



● Direct Shear, Saturated

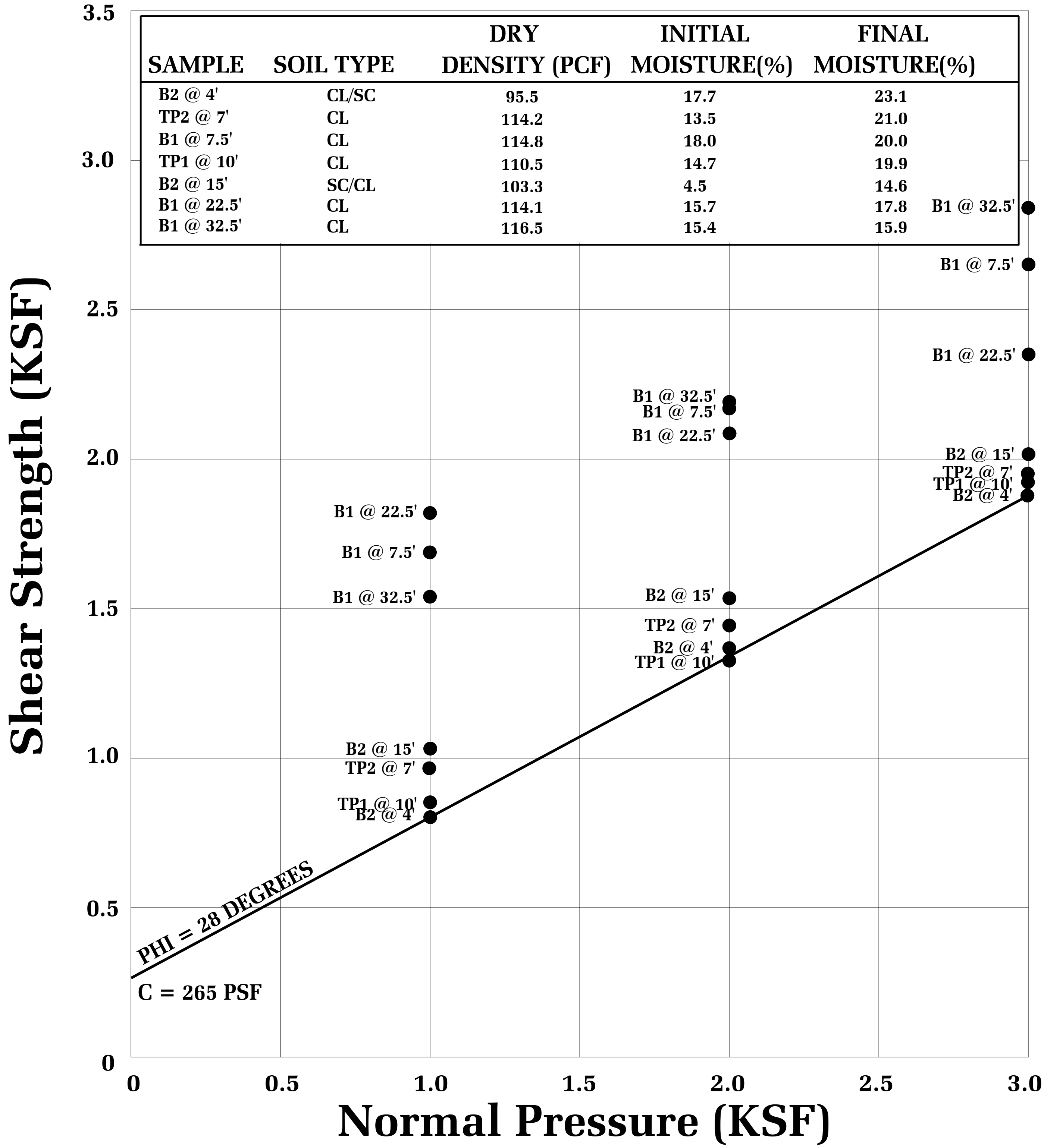
SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BARDAS INVESTMENT GROUP	
FILE NO. 22167	PLATE: B-1

OLDER ALLUVIUM

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B2 @ 4'	CL/SC	95.5	17.7	23.1
TP2 @ 7'	CL	114.2	13.5	21.0
B1 @ 7.5'	CL	114.8	18.0	20.0
TP1 @ 10'	CL	110.5	14.7	19.9
B2 @ 15'	SC/CL	103.3	4.5	14.6
B1 @ 22.5'	CL	114.1	15.7	17.8
B1 @ 32.5'	CL	116.5	15.4	15.9



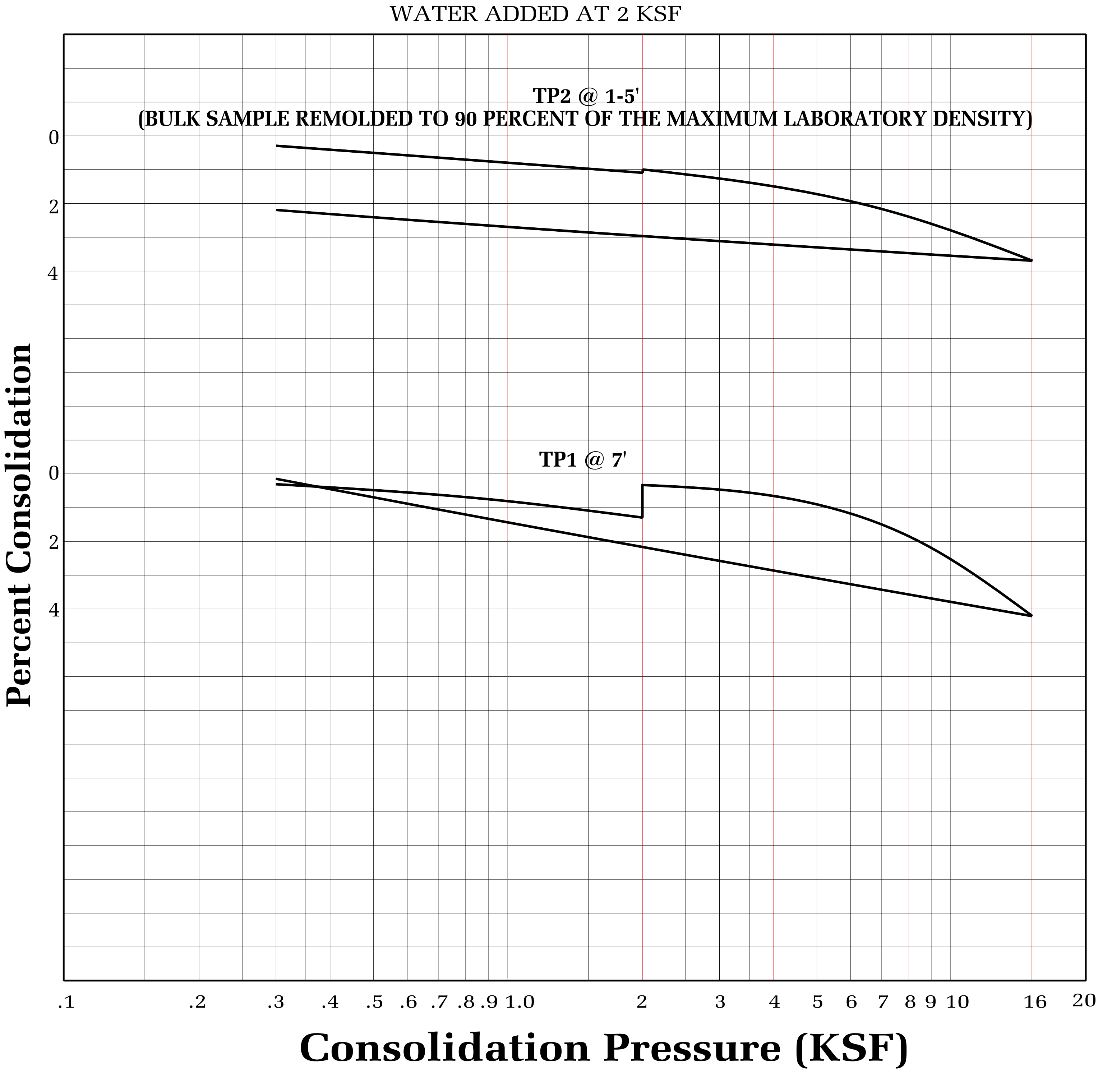
● Direct Shear, Saturated

SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BARDAS INVESTMENT GROUP

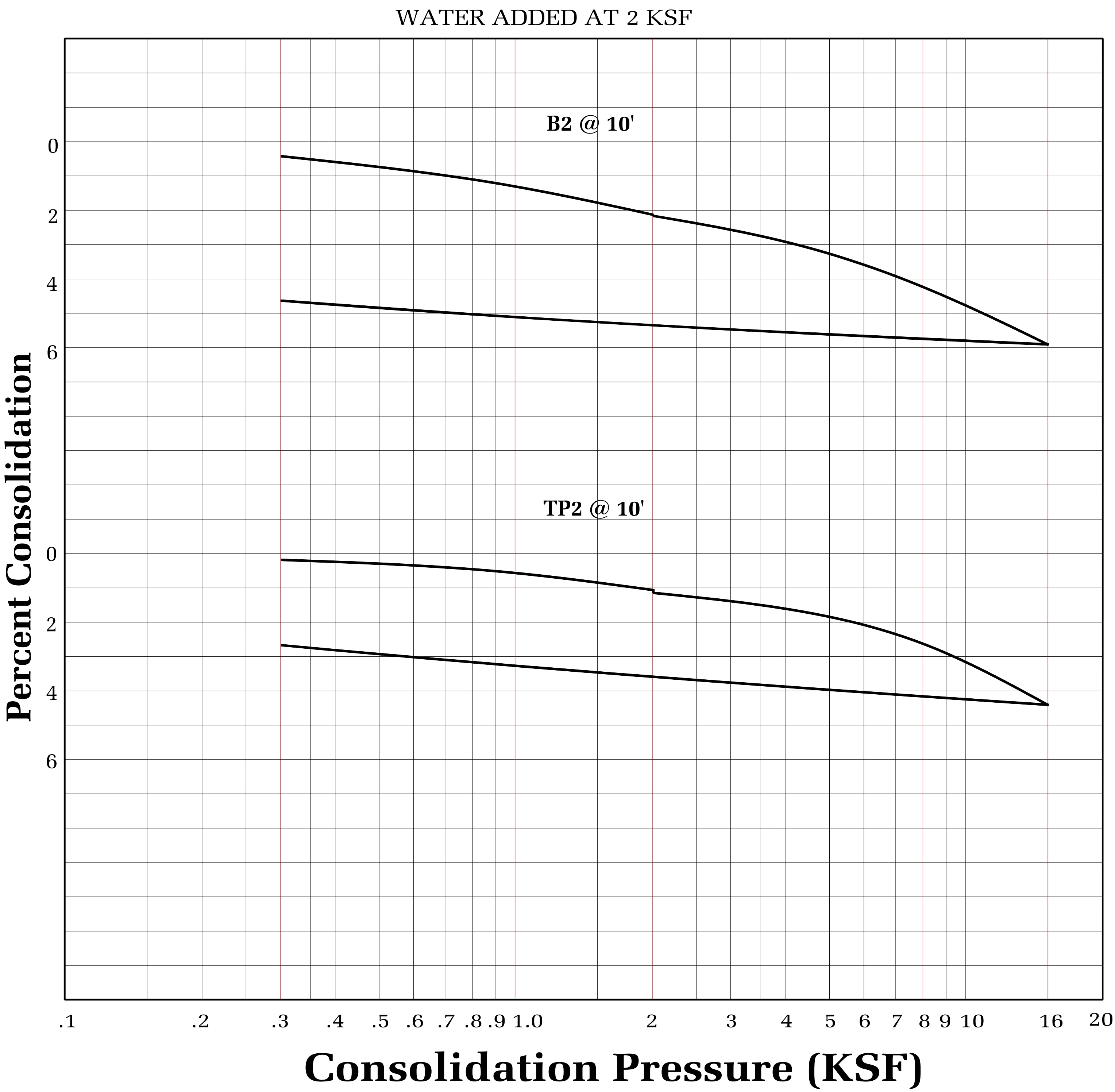
FILE NO. 22167 | PLATE: B-2



CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

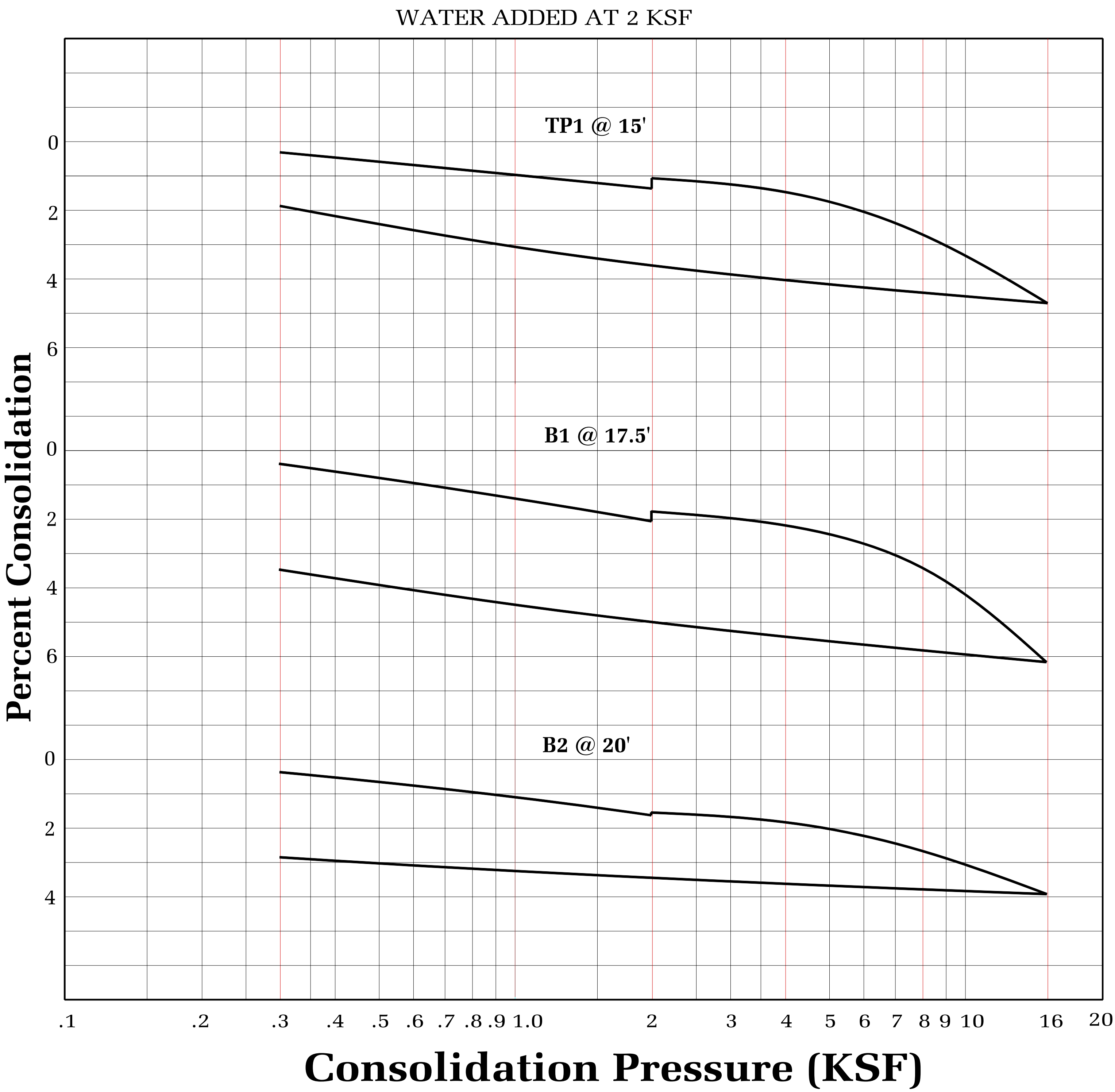
BARDAS INVESTMENT GROUP	
FILE NO. 22167	**PLATE: C-1**



CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

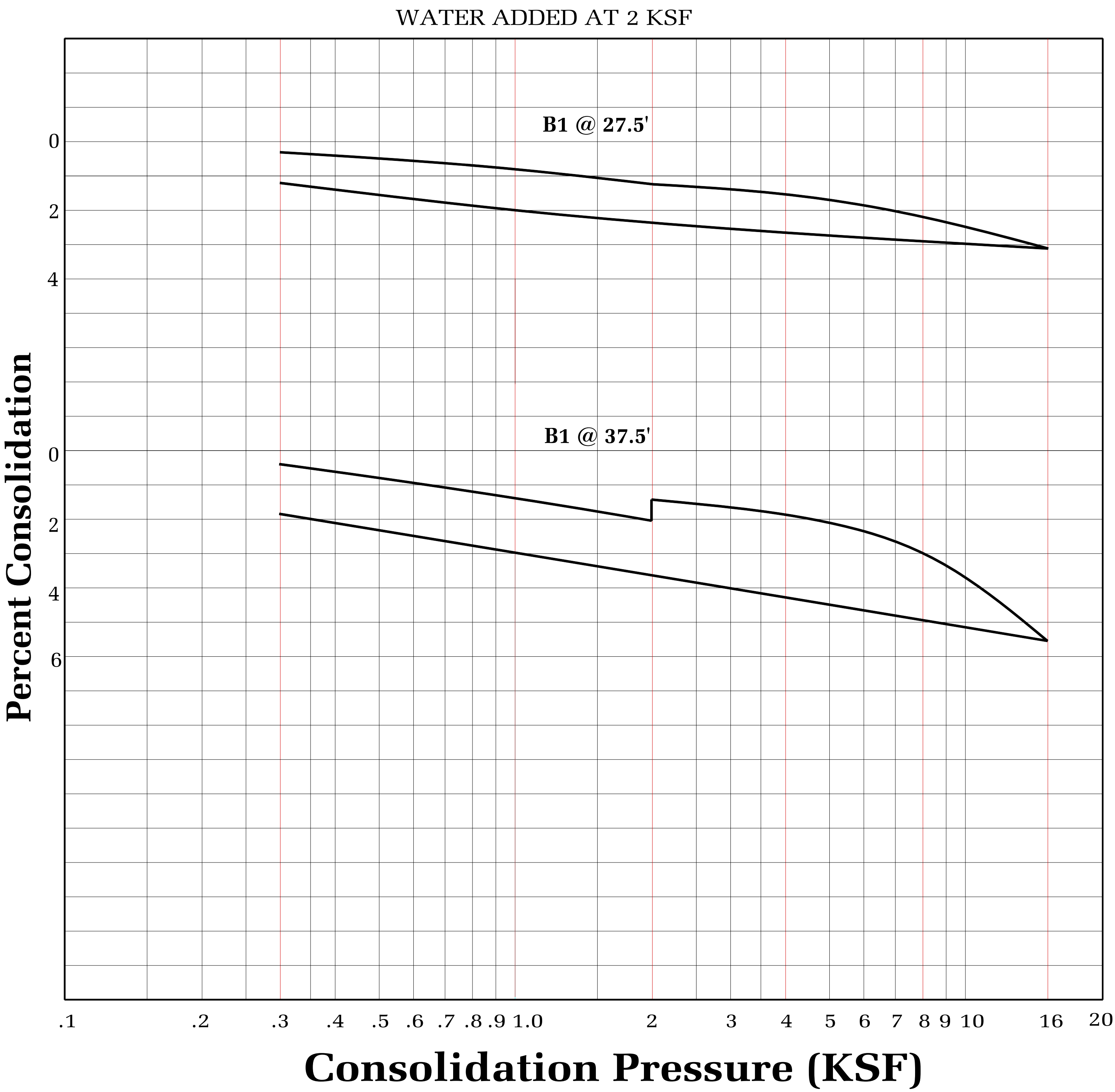
BARDAS INVESTMENT GROUP	
FILE NO. 22167	PLATE: C-2



CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BARDAS INVESTMENT GROUP	
FILE NO. 22167	PLATE: C-3



CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BARDAS INVESTMENT GROUP	
FILE NO. 22167	**PLATE: C-4**

ASTM D-1557

SAMPLE	B2 @ 1-5'	TP2 @ 1-5'
SOIL TYPE:	SM/SC	CL/SC
MAXIMUM DENSITY pcf.	127.1	121.5
OPTIMUM MOISTURE %	9.1	11.3

ASTM D 4829-03

SAMPLE	B2 @ 1-5'	TP2 @ 1-5'
SOIL TYPE:	SM/SC	CL/SC
EXPANSION INDEX UBC STANDARD 18-2	15	68
EXPANSION CHARACTER	VERY LOW	MODERATE

SULFATE CONTENT

SAMPLE	B2 @ 1-5'	TP2 @ 1-5'	B1 @ 20'	B2 @ 7'
SULFATE CONTENT: (percentage by weight)	< 0.1 %	< 0.1 %	< 0.1 %	< 0.1 %

COMPACTION/EXPANSION/SULFATE DATA SHEET

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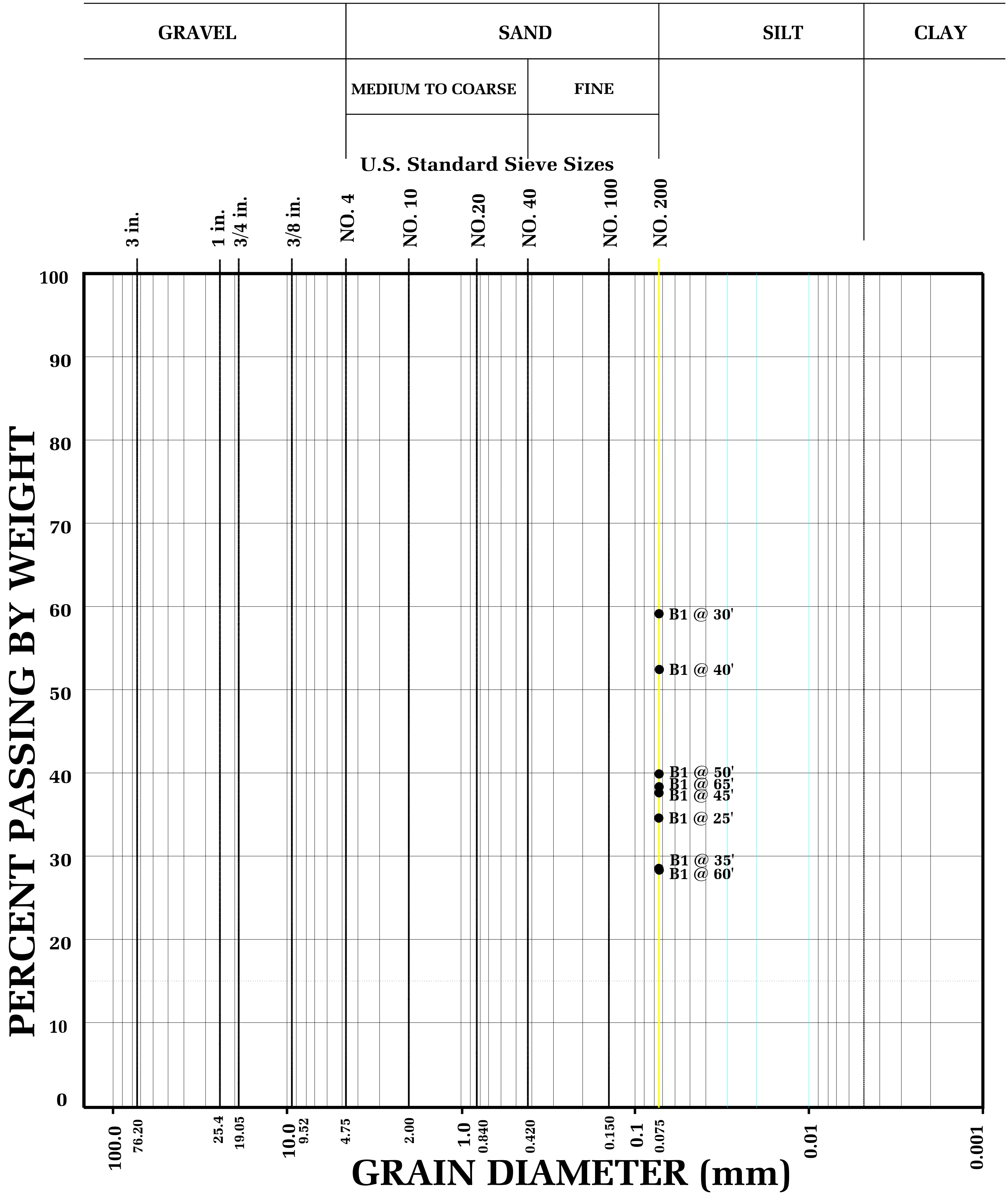
Consulting Geotechnical Engineers

BARDAS INVESTMENT GROUP

FILE NO. 22167 PLATE: D



GRAIN SIZE DISTRIBUTION

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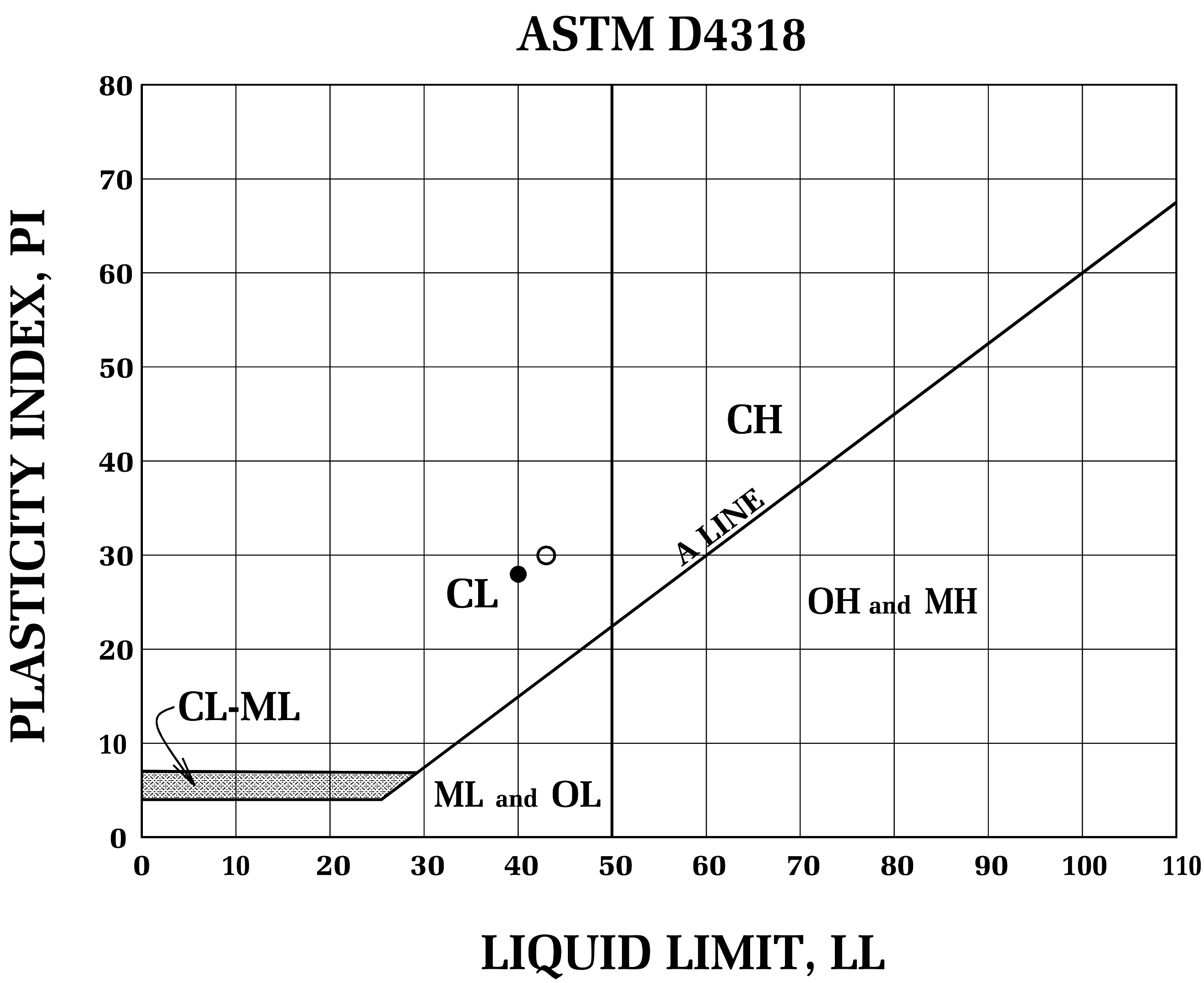
Consulting Geotechnical Engineers

BARDAS INVESTMENT GROUP

FILE NO. 22167

PLATE: E



BORING NUMBER	DEPTH (FEET)	TEST SYMBOL	LL	PL	PI	DESCRIPTION
B1	30	○	43	13	30	CL
B1	40	●	40	12	28	CL

ATTERBERG LIMITS DETERMINATION

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Consulting Geotechnical Engineers

BARDAS INVESTMENT GROUP

FILE NO. 22167

PLATE: F

Geotechnologies, Inc.

Project: Bardas Investment Group
File No.: 22167
Description Liquefaction Analysis (PGA_M)
Boring No: 1

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	6.9
Peak Ground Horizontal Acceleration, PGA (g):	0.99
Calculated Mag.Wtg.Factor:	1.171

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	27.0
Historically Highest Groundwater Level* (ft):	27.0
Unit Weight of Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

BOREHOLE AND SAMPLER INFORMATION:

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

LIQUEFACTION BOUNDARY:

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

Depth to Base Layer (feet)	Total Unit Weight (pcf)	Current Water Level (feet)	Historical Water Level (feet)	Field SPT Blowcount N	Depth of SPT Blowcount (feet)	Fines Content #200 Sieve (%)	Plastic Index (PI)	Vetical Stress σ_{vc}, (psf)	Effective Vert. Stress σ_{vc}', (psf)	Fines Corrected $(N_1)_{60\text{-}cs}$	Stress Reduction Coeff, r_d	Cyclic Shear Ratio CSR	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Liquefaction Settlment ΔS_i (inches)
1	137.6	Unsaturated	Unsaturated	22	5	0.0	0	137.6	137.6	52.4	1.00	0.645	2.000	Non-Liq.	0.00
2	137.6	Unsaturated	Unsaturated	22	5	0.0	0	275.2	275.2	52.4	1.00	0.643	2.000	Non-Liq.	0.00
3	137.6	Unsaturated	Unsaturated	22	5	0.0	0	412.8	412.8	47.4	1.00	0.641	2.000	Non-Liq.	0.00
4	137.6	Unsaturated	Unsaturated	22	5	0.0	0	550.4	550.4	44.4	0.99	0.639	2.000	Non-Liq.	0.00
5	137.6	Unsaturated	Unsaturated	22	5	0.0	0	688.0	688.0	44.6	0.99	0.637	2.000	Non-Liq.	0.00
6	137.6	Unsaturated	Unsaturated	22	5	0.0	0	825.6	825.6	42.8	0.99	0.634	2.000	Non-Liq.	0.00
7	135.5	Unsaturated	Unsaturated	22	5	0.0	0	961.1	961.1	41.3	0.98	0.632	2.000	Non-Liq.	0.00
8	135.5	Unsaturated	Unsaturated	22	5	0.0	0	1096.6	1096.6	40.0	0.98	0.629	2.000	Non-Liq.	0.00
9	135.5	Unsaturated	Unsaturated	22	5	0.0	0	1232.1	1232.1	40.9	0.98	0.627	2.000	Non-Liq.	0.00
10	135.5	Unsaturated	Unsaturated	9	10	0.0	0	1367.6	1367.6	15.4	0.97	0.624	0.196	Non-Liq.	0.00
11	135.5	Unsaturated	Unsaturated	9	10	0.0	0	1503.1	1503.1	14.7	0.97	0.622	0.187	Non-Liq.	0.00
12	135.5	Unsaturated	Unsaturated	9	10	0.0	0	1638.6	1638.6	14.0	0.96	0.619	0.178	Non-Liq.	0.00
13	120.9	Unsaturated	Unsaturated	9	10	0.0	0	1759.5	1759.5	13.5	0.96	0.616	0.172	Non-Liq.	0.00
14	120.9	Unsaturated	Unsaturated	9	10	0.0	0	1880.4	1880.4	13.0	0.95	0.613	0.166	Non-Liq.	0.00
15	120.9	Unsaturated	Unsaturated	8	15	0.0	0	2001.3	2001.3	12.5	0.95	0.610	0.160	Non-Liq.	0.00
16	120.9	Unsaturated	Unsaturated	8	15	0.0	0	2122.2	2122.2	12.1	0.95	0.607	0.156	Non-Liq.	0.00
17	120.9	Unsaturated	Unsaturated	8	15	0.0	0	2243.1	2243.1	11.7	0.94	0.604	0.152	Non-Liq.	0.00
18	125.7	Unsaturated	Unsaturated	8	15	0.0	0	2368.8	2368.8	11.3	0.94	0.601	0.148	Non-Liq.	0.00
19	125.7	Unsaturated	Unsaturated	8	15	0.0	0	2494.5	2494.5	11.0	0.93	0.598	0.144	Non-Liq.	0.00
20	125.7	Unsaturated	Unsaturated	13	20	0.0	0	2620.2	2620.2	18.7	0.93	0.595	0.217	Non-Liq.	0.00
21	125.7	Unsaturated	Unsaturated	13	20	0.0	0	2745.9	2745.9	18.2	0.92	0.591	0.210	Non-Liq.	0.00
22	125.7	Unsaturated	Unsaturated	13	20	0.0	0	2871.6	2871.6	17.7	0.92	0.588	0.204	Non-Liq.	0.00
23	132.1	Unsaturated	Unsaturated	21	25	34.6	0	3003.7	3003.7	37.0	0.91	0.585	1.858	Non-Liq.	0.00
24	132.1	Unsaturated	Unsaturated	21	25	34.6	0	3135.8	3135.8	36.4	0.90	0.581	1.574	Non-Liq.	0.00
25	132.1	Unsaturated	Unsaturated	21	25	34.6	0	3267.9	3267.9	35.8	0.90	0.578	1.355	Non-Liq.	0.00
26	132.1	Unsaturated	Unsaturated	21	25	34.6	0	3400.0	3400.0	35.2	0.89	0.574	1.184	Non-Liq.	0.00
27	132.1	Unsaturated	Unsaturated	21	25	34.6	0	3532.1	3532.1	34.6	0.89	0.571	1.048	Non-Liq.	0.00
28	137.4	Saturated	Saturated	21	25	34.6	0	3669.5	3607.1	36.4	0.88	0.577	1.497	2.6	0.00
29	137.4	Saturated	Saturated	21	25	34.6	0	3806.9	3682.1	36.1	0.88	0.583	1.386	2.4	0.00
30	137.4	Saturated	Saturated	27	30	59.1	30	3944.3	3757.1	48.5	0.87	0.588	1.944	Non-Liq.	0.00
31	137.4	Saturated	Saturated	27	30	59.1	30	4081.7	3832.1	48.2	0.87	0.593	1.930	Non-Liq.	0.00
32	137.4	Saturated	Saturated	27	30	59.1	30	4219.1	3907.1	48.0	0.86	0.597	1.917	Non-Liq.	0.00
33	134.5	Saturated	Saturated	27	30	59.1	30	4353.6	3979.2	47.7	0.85	0.601	1.904	Non-Liq.	0.00
34	134.5	Saturated	Saturated	27	30	59.1	30	4488.1	4051.3	47.4	0.85	0.604	1.892	Non-Liq.	0.00
35	134.5	Saturated	Saturated	29	35	28.5	0	4622.6	4123.4	50.6	0.84	0.607	1.879	3.1	0.00
36	134.5	Saturated	Saturated	29	35	28.5	0	4757.1	4195.5	50.4	0.84	0.610	1.867	3.1	0.00
37	134.5	Saturated	Saturated	29	35	28.5	0	4891.6	4267.6	50.1	0.83	0.612	1.856	3.0	0.00
38	127.6	Saturated	Saturated	29	35	28.5	0	5019.2	4332.8	49.9	0.83	0.615	1.845	3.0	0.00
39	127.6	Saturated	Saturated	29	35	28.5	0	5146.8	4398.0	49.7	0.82	0.616	1.835	3.0	0.00
40	127.6	Saturated	Saturated	9	40	53.1	28	5274.4	4463.2	15.1	0.81	0.618	0.168	Non-Liq.	0.00
41	127.6	Saturated	Saturated	9	40	53.1	28	5402.0	4528.4	15.0	0.81	0.620	0.167	Non-Liq.	0.00
42	127.6	Saturated	Saturated	9	40	53.1	28	5529.6	4593.6	14.9	0.80	0.621	0.166	Non-Liq.	0.00
43	127.6	Saturated	Saturated	9	40	53.1	28	5657.2	4658.8	14.8	0.80	0.622	0.165	Non-Liq.	0.00
44	127.6	Saturated	Saturated	9	40	53.1	28	5784.8	4724.0	14.7	0.79	0.622	0.164	Non-Liq.	0.00
45	134.7	Saturated	Saturated	22	45	37.6	0	5919.5	4796.3	34.0	0.79	0.623	0.847	1.4	0.00
46	134.7	Saturated	Saturated	22	45	37.6	0	6054.2	4868.6	33.8	0.78	0.623	0.810	1.3	0.00
47	134.7	Saturated	Saturated	22	45	37.6	0	6188.9	4940.9	33.5	0.77	0.623	0.776	1.2	0.00
48	134.7	Saturated	Saturated	22	45	37.6	0	6323.6	5013.2	33.3	0.77	0.623	0.745	1.2	0.00
49	134.7	Saturated	Saturated	22	45	37.6	0	6458.3	5085.5	33.1	0.76	0.622	0.716	1.2	0.00
50	134.7	Saturated	Saturated	25	50	39.9	0	6593.0	5157.8	38.7	0.76	0.622	1.725	2.8	0.00
51	134.7	Saturated	Saturated	25	50	39.9	0	6727.7	5230.1	38.5	0.75	0.621	1.715	2.8	0.00
52	134.7	Saturated	Saturated	25	50	39.9	0	6862.4	5302.4	38.2	0.75	0.620	1.706	2.8	0.00
53	142.2	Saturated	Saturated	25	50	39.9	0	7004.6	5382.2	38.0	0.74	0.619	1.695	2.7	0.00
54	142.2	Saturated	Saturated	25	50	39.9	0	7146.8	5462.0	37.7	0.73	0.618	1.685	2.7	0.00
55	142.2	Saturated	Saturated	47	55	0.0	0	7289.0	5541.8	68.1	0.73	0.616	1.675	2.7	0.00
56	142.2	Saturated	Saturated	47	55	0.0	0	7431.2	5621.6	67.9	0.72	0.615	1.665	2.7	0.00
57	142.2	Saturated	Saturated	47	55	0.0	0	7573.4	5701.4	67.6	0.72	0.613	1.655	2.7	0.00
58	139.2	Saturated	Saturated	47	55	0.0	0	7712.6	5778.2	67.4	0.71	0.612	1.646	2.7	0.00
59	139.2	Saturated	Saturated	47	55	0.0	0	7851.8	5855.0	67.1	0.71	0.610	1.637	2.7	0.00
60	139.2	Saturated	Saturated	29	60	28.3	0	7991.0	5931.8	44.4	0.70	0.608	1.628	2.7	0.00
61	139.2	Saturated	Saturated	29	60	28.3	0	8130.2	6008.6	44.1	0.70	0.607	1.619	2.7	0.00
62	139.2	Saturated	Saturated	29	60	28.3	0	8269.4	6085.4	43.9	0.69	0.605	1.610	2.7	0.00
63	139.3	Saturated	Saturated	26	65	38.3	0	8408.7	6162.3	37.6	0.69	0.603	1.602	2.7	0.00
64	139.3	Saturated	Saturated	26	65	38.3	0	8548.0	6239.2	37.3	0.68	0.601	1.518	2.5	0.00
65	139.3	Saturated	Saturated	26	65	38.3	0	8687.3	6316.1	37.1	0.68	0.599	1.429	2.4	0.00
66	139.3	Saturated	Saturated	26	65	38.3	0	8826.6	6393.0	36.9	0.67	0.597	1.351	2.3	0.00
67	139.3	Saturated	Saturated	26	65	38.3	0	8965.9	6469.9	36.7	0.67	0.595	1.284	2.2	0.00
68	129.5	Saturated	Saturated	26	65	38.3	0	9095.4	6537.0	36.5	0.66	0.593	1.230	2.1	0.00
69	129.5	Saturated	Saturated	26	65	38.3	0	9224.9	6604.1	36.3	0.66	0.591	1.179	2.0	0.00
70	129.5	Saturated	Saturated	62	70	0.0	0	9354.4	6671.2	85.6	0.65	0.590	1.547	2.6	0.00

Total Liquefaction Settlement, S = 0.00 inches

Geotechnologies, Inc.

Project: Bardas Investment Group
File No.: 22167
Description Liquefaction Analysis (2/3 PGA_M)
Boring No: 1

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	6.9
Peak Ground Horizontal Acceleration, PGA (g):	0.66
Calculated Mag.Wtg.Factor:	1.171

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	27.0
Historically Highest Groundwater Level* (ft):	27.0
Unit Weight of Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

BOREHOLE AND SAMPLER INFORMATION:

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

LIQUEFACTION BOUNDARY:

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

Depth to Base Layer (feet)	Total Unit Weight (pcf)	Current Water Level (feet)	Historical Water Level (feet)	Field SPT Blowcount N	Depth of SPT Blowcount (feet)	Fines Content #200 Sieve (%)	Plastic Index (PI)	Vetical Stress σ_{vc}, (psf)	Effective Vert. Stress σ_{vc}', (psf)	Fines Corrected $(N_1)_{60\text{-}cs}$	Stress Reduction Coeff, r_d	Cyclic Shear Ratio CSR	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Liquefaction Settlment ΔS_i (inches)
1	137.6	Unsaturated	Unsaturated	22	5	0.0	0	137.6	137.6	52.4	1.00	0.431	2.000	Non-Liq.	0.00
2	137.6	Unsaturated	Unsaturated	22	5	0.0	0	275.2	275.2	52.4	1.00	0.430	2.000	Non-Liq.	0.00
3	137.6	Unsaturated	Unsaturated	22	5	0.0	0	412.8	412.8	47.4	1.00	0.428	2.000	Non-Liq.	0.00
4	137.6	Unsaturated	Unsaturated	22	5	0.0	0	550.4	550.4	44.4	0.99	0.427	2.000	Non-Liq.	0.00
5	137.6	Unsaturated	Unsaturated	22	5	0.0	0	688.0	688.0	44.6	0.99	0.425	2.000	Non-Liq.	0.00
6	137.6	Unsaturated	Unsaturated	22	5	0.0	0	825.6	825.6	42.8	0.99	0.424	2.000	Non-Liq.	0.00
7	135.5	Unsaturated	Unsaturated	22	5	0.0	0	961.1	961.1	41.3	0.98	0.422	2.000	Non-Liq.	0.00
8	135.5	Unsaturated	Unsaturated	22	5	0.0	0	1096.6	1096.6	40.0	0.98	0.421	2.000	Non-Liq.	0.00
9	135.5	Unsaturated	Unsaturated	22	5	0.0	0	1232.1	1232.1	40.9	0.98	0.419	2.000	Non-Liq.	0.00
10	135.5	Unsaturated	Unsaturated	9	10	0.0	0	1367.6	1367.6	15.4	0.97	0.417	0.196	Non-Liq.	0.00
11	135.5	Unsaturated	Unsaturated	9	10	0.0	0	1503.1	1503.1	14.7	0.97	0.415	0.187	Non-Liq.	0.00
12	135.5	Unsaturated	Unsaturated	9	10	0.0	0	1638.6	1638.6	14.0	0.96	0.413	0.178	Non-Liq.	0.00
13	120.9	Unsaturated	Unsaturated	9	10	0.0	0	1759.5	1759.5	13.5	0.96	0.412	0.172	Non-Liq.	0.00
14	120.9	Unsaturated	Unsaturated	9	10	0.0	0	1880.4	1880.4	13.0	0.95	0.410	0.166	Non-Liq.	0.00
15	120.9	Unsaturated	Unsaturated	8	15	0.0	0	2001.3	2001.3	12.5	0.95	0.408	0.160	Non-Liq.	0.00
16	120.9	Unsaturated	Unsaturated	8	15	0.0	0	2122.2	2122.2	12.1	0.95	0.406	0.156	Non-Liq.	0.00
17	120.9	Unsaturated	Unsaturated	8	15	0.0	0	2243.1	2243.1	11.7	0.94	0.404	0.152	Non-Liq.	0.00
18	125.7	Unsaturated	Unsaturated	8	15	0.0	0	2368.8	2368.8	11.3	0.94	0.401	0.148	Non-Liq.	0.00
19	125.7	Unsaturated	Unsaturated	8	15	0.0	0	2494.5	2494.5	11.0	0.93	0.399	0.144	Non-Liq.	0.00
20	125.7	Unsaturated	Unsaturated	13	20	0.0	0	2620.2	2620.2	18.7	0.93	0.397	0.217	Non-Liq.	0.00
21	125.7	Unsaturated	Unsaturated	13	20	0.0	0	2745.9	2745.9	18.2	0.92	0.395	0.210	Non-Liq.	0.00
22	125.7	Unsaturated	Unsaturated	13	20	0.0	0	2871.6	2871.6	17.7	0.92	0.393	0.204	Non-Liq.	0.00
23	132.1	Unsaturated	Unsaturated	21	25	34.6	0	3003.7	3003.7	37.0	0.91	0.391	1.858	Non-Liq.	0.00
24	132.1	Unsaturated	Unsaturated	21	25	34.6	0	3135.8	3135.8	36.4	0.90	0.388	1.574	Non-Liq.	0.00
25	132.1	Unsaturated	Unsaturated	21	25	34.6	0	3267.9	3267.9	35.8	0.90	0.386	1.355	Non-Liq.	0.00
26	132.1	Unsaturated	Unsaturated	21	25	34.6	0	3400.0	3400.0	35.2	0.89	0.384	1.184	Non-Liq.	0.00
27	132.1	Unsaturated	Unsaturated	21	25	34.6	0	3532.1	3532.1	34.6	0.89	0.381	1.048	Non-Liq.	0.00
28	137.4	Saturated	Saturated	21	25	34.6	0	3669.5	3607.1	36.4	0.88	0.385	1.497	3.9	0.00
29	137.4	Saturated	Saturated	21	25	34.6	0	3806.9	3682.1	36.1	0.88	0.389	1.386	3.6	0.00
30	137.4	Saturated	Saturated	27	30	59.1	30	3944.3	3757.1	48.5	0.87	0.393	1.944	Non-Liq.	0.00
31	137.4	Saturated	Saturated	27	30	59.1	30	4081.7	3832.1	48.2	0.87	0.396	1.930	Non-Liq.	0.00
32	137.4	Saturated	Saturated	27	30	59.1	30	4219.1	3907.1	48.0	0.86	0.399	1.917	Non-Liq.	0.00
33	134.5	Saturated	Saturated	27	30	59.1	30	4353.6	3979.2	47.7	0.85	0.401	1.904	Non-Liq.	0.00
34	134.5	Saturated	Saturated	27	30	59.1	30	4488.1	4051.3	47.4	0.85	0.404	1.892	Non-Liq.	0.00
35	134.5	Saturated	Saturated	29	35	28.5	0	4622.6	4123.4	50.6	0.84	0.406	1.879	4.6	0.00
36	134.5	Saturated	Saturated	29	35	28.5	0	4757.1	4195.5	50.4	0.84	0.407	1.867	4.6	0.00
37	134.5	Saturated	Saturated	29	35	28.5	0	4891.6	4267.6	50.1	0.83	0.409	1.856	4.5	0.00
38	127.6	Saturated	Saturated	29	35	28.5	0	5019.2	4332.8	49.9	0.83	0.411	1.845	4.5	0.00
39	127.6	Saturated	Saturated	29	35	28.5	0	5146.8	4398.0	49.7	0.82	0.412	1.835	4.5	0.00
40	127.6	Saturated	Saturated	9	40	53.1	28	5274.4	4463.2	15.1	0.81	0.413	0.168	Non-Liq.	0.00
41	127.6	Saturated	Saturated	9	40	53.1	28	5402.0	4528.4	15.0	0.81	0.414	0.167	Non-Liq.	0.00
42	127.6	Saturated	Saturated	9	40	53.1	28	5529.6	4593.6	14.9	0.80	0.415	0.166	Non-Liq.	0.00
43	127.6	Saturated	Saturated	9	40	53.1	28	5657.2	4658.8	14.8	0.80	0.415	0.165	Non-Liq.	0.00
44	127.6	Saturated	Saturated	9	40	53.1	28	5784.8	4724.0	14.7	0.79	0.416	0.164	Non-Liq.	0.00
45	134.7	Saturated	Saturated	22	45	37.6	0	5919.5	4796.3	34.0	0.79	0.416	0.847	2.0	0.00
46	134.7	Saturated	Saturated	22	45	37.6	0	6054.2	4868.6	33.8	0.78	0.416	0.810	1.9	0.00
47	134.7	Saturated	Saturated	22	45	37.6	0	6188.9	4940.9	33.5	0.77	0.416	0.776	1.9	0.00
48	134.7	Saturated	Saturated	22	45	37.6	0	6323.6	5013.2	33.3	0.77	0.416	0.745	1.8	0.00
49	134.7	Saturated	Saturated	22	45	37.6	0	6458.3	5085.5	33.1	0.76	0.416	0.716	1.7	0.00
50	134.7	Saturated	Saturated	25	50	39.9	0	6593.0	5157.8	38.7	0.76	0.415	1.725	4.2	0.00
51	134.7	Saturated	Saturated	25	50	39.9	0	6727.7	5230.1	38.5	0.75	0.415	1.715	4.1	0.00
52	134.7	Saturated	Saturated	25	50	39.9	0	6862.4	5302.4	38.2	0.75	0.414	1.706	4.1	0.00
53	142.2	Saturated	Saturated	25	50	39.9	0	7004.6	5382.2	38.0	0.74	0.413	1.695	4.1	0.00
54	142.2	Saturated	Saturated	25	50	39.9	0	7146.8	5462.0	37.7	0.73	0.413	1.685	4.1	0.00
55	142.2	Saturated	Saturated	47	55	0.0	0	7289.0	5541.8	68.1	0.73	0.412	1.675	4.1	0.00
56	142.2	Saturated	Saturated	47	55	0.0	0	7431.2	5621.6	67.9	0.72	0.411	1.665	4.1	0.00
57	142.2	Saturated	Saturated	47	55	0.0	0	7573.4	5701.4	67.6	0.72	0.410	1.655	4.0	0.00
58	139.2	Saturated	Saturated	47	55	0.0	0	7712.6	5778.2	67.4	0.71	0.409	1.646	4.0	0.00
59	139.2	Saturated	Saturated	47	55	0.0	0	7851.8	5855.0	67.1	0.71	0.407	1.637	4.0	0.00
60	139.2	Saturated	Saturated	29	60	28.3	0	7991.0	5931.8	44.4	0.70	0.406	1.628	4.0	0.00
61	139.2	Saturated	Saturated	29	60	28.3	0	8130.2	6008.6	44.1	0.70	0.405	1.619	4.0	0.00
62	139.2	Saturated	Saturated	29	60	28.3	0	8269.4	6085.4	43.9	0.69	0.404	1.610	4.0	0.00
63	139.3	Saturated	Saturated	26	65	38.3	0	8408.7	6162.3	37.6	0.69	0.403	1.602	4.0	0.00
64	139.3	Saturated	Saturated	26	65	38.3	0	8548.0	6239.2	37.3	0.68	0.401	1.518	3.8	0.00
65	139.3	Saturated	Saturated	26	65	38.3	0	8687.3	6316.1	37.1	0.68	0.400	1.429	3.6	0.00
66	139.3	Saturated	Saturated	26	65	38.3	0	8826.6	6393.0	36.9	0.67	0.399	1.351	3.4	0.00
67	139.3	Saturated	Saturated	26	65	38.3	0	8965.9	6469.9	36.7	0.67	0.397	1.284	3.2	0.00
68	129.5	Saturated	Saturated	26	65	38.3	0	9095.4	6537.0	36.5	0.66	0.396	1.230	3.1	0.00
69	129.5	Saturated	Saturated	26	65	38.3	0	9224.9	6604.1	36.3	0.66	0.395	1.179	3.0	0.00
70	129.5	Saturated	Saturated	62	70	0.0	0	9354.4	6671.2	85.6	0.65	0.394	1.547	3.9	0.00

Total Liquefaction Settlement, S = 0.00 inches

Geotechnologies, Inc.

Project: Bardas Invesment Group

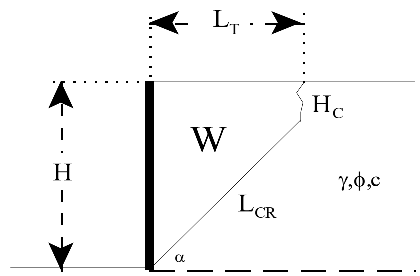
File No.: 22167

Description: Cantilever Retaining Walls (Up to 10)

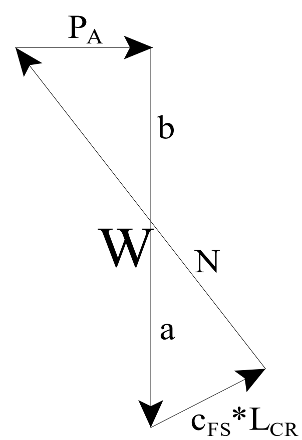
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	10.00	feet
Unit Weight of Retained Soils	(γ)	125.0	pcf
Friction Angle of Retained Soils	(ϕ)	28.0	degrees
Cohesion of Retained Soils	(c)	265.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters:	(ϕ_{FS})	19.5	degrees
	(c_{FS})	176.7	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	4.4	40	5051.6	7.9	3076.8	1974.8	941.2
46	4.3	39	4919.3	7.9	2958.7	1960.6	976.8
47	4.2	38	4784.1	7.9	2845.5	1938.6	1008.4
48	4.2	37	4646.8	7.8	2737.1	1909.7	1036.1
49	4.1	36	4508.2	7.8	2633.5	1874.8	1059.9
50	4.1	35	4369.0	7.7	2534.5	1834.6	1079.9
51	4.1	34	4229.6	7.7	2439.9	1789.8	1096.0
52	4.0	33	4090.4	7.6	2349.4	1740.9	1108.3
53	4.0	32	3951.5	7.5	2263.0	1688.5	1116.9
54	4.0	31	3813.2	7.4	2180.2	1633.0	1121.6
55	4.0	29	3675.7	7.3	2100.8	1574.8	1122.6
56	4.0	28	3538.9	7.2	2024.6	1514.3	1119.8
57	4.0	27	3403.1	7.1	1951.3	1451.7	1113.2
58	4.0	26	3268.1	7.0	1880.7	1387.4	1102.9
59	4.1	25	3134.0	6.9	1812.4	1321.6	1088.7
60	4.1	24	3000.7	6.8	1746.3	1254.5	1070.7
61	4.1	23	2868.3	6.7	1681.9	1186.3	1048.9
62	4.2	22	2736.6	6.6	1619.2	1117.4	1023.2
63	4.3	21	2605.5	6.4	1557.8	1047.7	993.6
64	4.3	20	2475.0	6.3	1497.4	977.6	960.1
65	4.4	19	2344.9	6.2	1437.6	907.2	922.6
66	4.5	18	2215.1	6.0	1378.3	836.7	881.2
67	4.6	17	2085.4	5.8	1319.0	766.3	835.8
68	4.7	16	1955.6	5.7	1259.4	696.2	786.4
69	4.9	15	1825.5	5.5	1199.0	626.5	733.1
70	5.0	14	1694.9	5.3	1137.3	557.6	676.0



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

$P_{A,\,max}$ 1122.6 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 22.5 pcf

Design Wall for an Equivalent Fluid Pressure: 30 pcf

Geotechnologies, Inc.

Project: Bardas Invesment Group

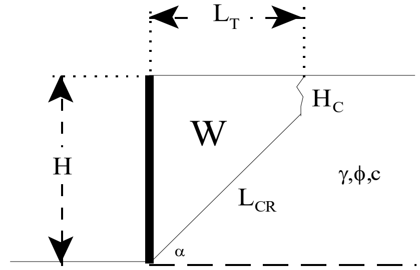
File No.: 22167

Description: Cantilever Retaining Walls (Up to 15)

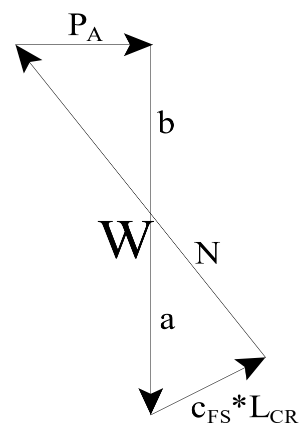
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	15.00	feet
Unit Weight of Retained Soils	(γ)	125.0	pcf
Friction Angle of Retained Soils	(ϕ)	28.0	degrees
Cohesion of Retained Soils	(c)	265.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters:	(ϕ_{FS})	19.5	degrees
	(c_{FS})	176.7	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	4.4	103	12864.1	15.0	5813.6	7050.6	3360.2
46	4.3	100	12463.8	14.9	5554.3	6909.5	3442.2
47	4.2	97	12069.3	14.7	5312.4	6756.9	3514.7
48	4.2	93	11681.2	14.6	5086.4	6594.8	3578.0
49	4.1	90	11299.5	14.4	4875.0	6424.5	3632.2
50	4.1	87	10924.5	14.2	4677.0	6247.5	3677.4
51	4.1	84	10556.1	14.1	4491.3	6064.8	3713.9
52	4.0	82	10194.2	13.9	4316.8	5877.3	3741.7
53	4.0	79	9838.6	13.8	4152.7	5686.0	3760.9
54	4.0	76	9489.3	13.6	3997.9	5491.4	3771.6
55	4.0	73	9146.0	13.4	3851.9	5294.2	3773.8
56	4.0	70	8808.5	13.3	3713.7	5094.8	3767.5
57	4.0	68	8476.6	13.1	3582.8	4893.8	3752.7
58	4.0	65	8149.9	12.9	3458.4	4691.5	3729.4
59	4.1	63	7828.2	12.8	3340.0	4488.2	3697.4
60	4.1	60	7511.3	12.6	3227.1	4284.2	3656.7
61	4.1	58	7198.8	12.4	3119.1	4079.8	3607.2
62	4.2	55	6890.5	12.2	3015.4	3875.1	3548.6
63	4.3	53	6586.2	12.0	2915.7	3670.5	3481.0
64	4.3	50	6285.4	11.9	2819.4	3466.0	3403.9
65	4.4	48	5987.9	11.7	2726.0	3261.9	3317.2
66	4.5	46	5693.4	11.5	2635.1	3058.3	3220.7
67	4.6	43	5401.6	11.3	2546.2	2855.4	3114.1
68	4.7	41	5112.0	11.1	2458.7	2653.3	2997.2
69	4.9	39	4824.5	10.8	2372.1	2452.4	2869.5
70	5.0	36	4538.4	10.6	2285.8	2252.6	2730.9



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 3773.8 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 33.5 pcf

Design Wall for an Equivalent Fluid Pressure: 34 pcf

Geotechnologies, Inc.

Project: Bardas Investment Group

File No.: 22167

Soil Weight	γ	125	pcf
Internal Friction Angle	ϕ	28	degrees
Cohesion	c	0	psf
Height of Retaining Wall	H	15	feet

Restrained Retaining Wall Design based on At Rest Earth Pressure

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

$K_o = 1 - \sin\phi$	0.531	
$\sigma'_v = \gamma H$	1875.0	psf

$\sigma'_h =$	994.7	psf	
EFP =	66.3	pcf	
$P_o =$	7460.6	lbs/ft	(based on a triangular distribution of pressure)

Design wall for an EFP of 67 pcf

Geotechnologies, Inc.
Project: Bardas Investment Group
File No.: 22167

Seismically Induced Lateral Soil Pressure on Retaining Wall

Input:

Height of Retaining Wall:	(H)	15.0	feet
Retained Soil Unit Weight:	(γ)	125.0	pcf
Horizontal Ground Acceleration:	(k_h)	0.33	g

Seismic Increment (ΔP_{AE}):

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

$\Delta P_{AE} =$ 3480.5 lbs/ft

Transfer load to 1/3 of the height of the wall

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

T = 3132.4 lbs/ft

$EFP = 2*T/H^2$

EFP = 27.8 pcf

triangular distribution of pressure applied to the proposed retaining wall.

Geotechnologies, Inc.

Project: Bardas Investment Group

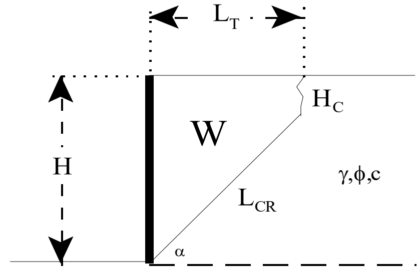
File No.: 22167

Description: Temporary Shoring Walls (Up to 15 feet)

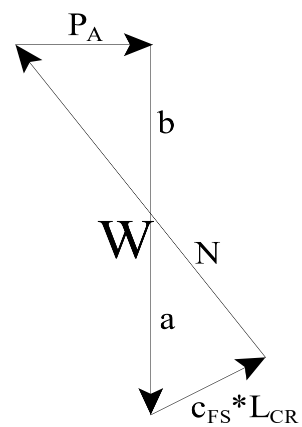
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	15.00 feet
Unit Weight of Retained Soils	(γ)	125.0 pcf
Friction Angle of Retained Soils	(ϕ)	28.0 degrees
Cohesion of Retained Soils	(c)	265.0 psf
Factor of Safety	(FS)	1.25
Factored Parameters:	(ϕ_{FS})	23.0 degrees
	(c_{FS})	212.0 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	5.9	95	11884.7	12.9	6712.4	5172.3	2085.2
46	5.8	93	11577.4	12.8	6424.6	5152.8	2182.6
47	5.6	90	11262.3	12.8	6151.6	5110.7	2270.8
48	5.5	88	10942.3	12.7	5893.2	5049.1	2349.8
49	5.4	85	10619.5	12.7	5648.8	4970.6	2419.7
50	5.4	82	10295.4	12.6	5417.8	4877.6	2480.6
51	5.3	80	9971.3	12.5	5199.4	4771.9	2532.7
52	5.2	77	9648.2	12.4	4992.8	4655.4	2575.9
53	5.2	75	9326.6	12.3	4797.3	4529.3	2610.4
54	5.2	72	9007.1	12.2	4612.0	4395.1	2636.3
55	5.1	70	8690.1	12.0	4436.2	4253.9	2653.6
56	5.1	67	8375.7	11.9	4269.2	4106.5	2662.4
57	5.1	65	8064.1	11.8	4110.2	3953.8	2662.6
58	5.1	62	7755.3	11.6	3958.7	3796.7	2654.2
59	5.2	60	7449.4	11.5	3813.8	3635.7	2637.3
60	5.2	57	7146.4	11.3	3675.0	3471.4	2611.7
61	5.2	55	6846.0	11.2	3541.7	3304.3	2577.6
62	5.3	52	6548.1	11.0	3413.2	3135.0	2534.7
63	5.4	50	6252.7	10.8	3289.0	2963.7	2483.0
64	5.4	48	5959.5	10.6	3168.5	2791.0	2422.5
65	5.5	45	5668.3	10.5	3051.0	2617.2	2353.0
66	5.6	43	5378.8	10.3	2936.1	2442.7	2274.4
67	5.8	41	5090.7	10.0	2822.9	2267.8	2186.6
68	5.9	38	4803.7	9.8	2710.9	2092.8	2089.6
69	6.1	36	4517.5	9.6	2599.3	1918.1	1983.3
70	6.2	34	4231.5	9.3	2487.4	1744.2	1867.6



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

$P_{A,\,max}$ 2662.6 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 23.7 pcf

Design Shoring for an Equivalent Fluid Pressure: 25 pcf

Geotechnologies, Inc.

Project: Bardas Investment Group

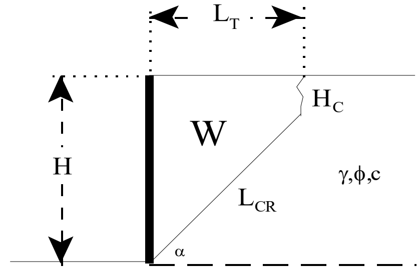
File No.: 22167

Description: Temporary Shoring Walls (Up to 20 feet)

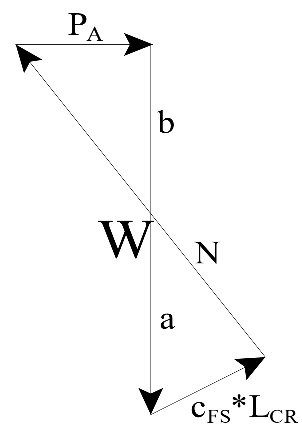
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	20.00	feet
Unit Weight of Retained Soils	(γ)	125.0	pcf
Friction Angle of Retained Soils	(ϕ)	28.0	degrees
Cohesion of Retained Soils	(c)	265.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(ϕ_{FS})	23.0	degrees
	(c_{FS})	212.0	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	5.9	183	22822.2	19.9	10401.7	12420.5	5007.3
46	5.8	177	22139.6	19.8	9901.2	12238.4	5184.0
47	5.6	172	21461.7	19.6	9436.3	12025.4	5343.2
48	5.5	166	20790.5	19.5	9004.1	11786.4	5485.3
49	5.4	161	20127.3	19.3	8601.7	11525.6	5610.6
50	5.4	156	19473.0	19.1	8226.7	11246.3	5719.6
51	5.3	151	18828.3	18.9	7876.7	10951.6	5812.5
52	5.2	146	18193.5	18.7	7549.5	10644.0	5889.5
53	5.2	141	17568.6	18.5	7243.2	10325.4	5951.0
54	5.2	136	16953.7	18.3	6955.9	9997.8	5997.0
55	5.1	131	16348.6	18.1	6686.0	9662.6	6027.7
56	5.1	126	15753.1	17.9	6432.0	9321.1	6043.2
57	5.1	121	15167.0	17.7	6192.5	8974.5	6043.5
58	5.1	117	14589.9	17.5	5966.1	8623.7	6028.7
59	5.2	112	14021.4	17.3	5751.8	8269.6	5998.6
60	5.2	108	13461.1	17.1	5548.4	7912.7	5953.3
61	5.2	103	12908.7	16.9	5354.9	7553.8	5892.5
62	5.3	99	12363.7	16.7	5170.3	7193.4	5816.1
63	5.4	95	11825.7	16.4	4993.6	6832.0	5723.9
64	5.4	90	11294.1	16.2	4824.1	6470.0	5615.7
65	5.5	86	10768.5	16.0	4660.8	6107.7	5491.0
66	5.6	82	10248.5	15.7	4502.9	5745.5	5349.7
67	5.8	78	9733.4	15.5	4349.6	5383.8	5191.2
68	5.9	74	9222.7	15.2	4199.8	5022.9	5015.3
69	6.1	70	8716.0	14.9	4052.9	4663.1	4821.5
70	6.2	66	8212.5	14.6	3907.7	4304.8	4609.3



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

$P_{A,\,max}$ 6043.5 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 30.2 pcf

Design Shoring for an Equivalent Fluid Pressure: 31 pcf

Geotechnologies, Inc.

Tiebacks Calculations (Ref: Bowles, 1982)

Project: Bardas Investment Group

File No. 22167

<u>Soil Parameters:</u>

Weight of Soil	γ	125.00	*lbs/ft*^{3}
Friction Angle	ϕ	28.00	*degrees*
Cohesion	c	265.00	*lbs/ft*^{2}
Tieback Angle	α	20.00	*degrees*

<u>Design Assumptions:</u>

Diameter of Grout	d	0.50	*feet*
Length of Embeddment	L	20.00	*feet*
Depth to midpoint of Embeddment	h	10.00	*feet*
Earth Pressure Coefficient	K	0.50	
Factor of Safety Applied	F.S.	1.50	

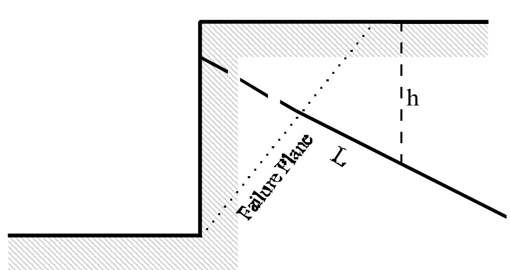
<u>Ultimate Resistance:</u> R_{ult} 25.45 *kips*

Eq: pi*d*γ*L*h*cos(a)*tan(ϕ)+c*pi*d*L

Allowable Resistance: $R_{allow} = R_{ult}/F.S.$ 16.97 *kips*

Allowable Skin Friction: $R_{allow}/2/pi/r/L$ 540.04 *psf*

Allowable Skin Friction Design Value **450** ***psf***



Geotechnologies, Inc.

Tiebacks Calculations (Ref: Bowles, 1982)

Project: Bardas Investment Group

File No. 22167

Soil Parameters:

Weight of Soil	γ	125.00	*lbs/ft*^{3}
Friction Angle	ϕ	28.00	*degrees*
Cohesion	c	265.00	*lbs/ft*^{2}
Tieback Angle	α	40.00	*degrees*

Design Assumptions:

Diameter of Grout	d	0.50	*feet*
Length of Embeddment	L	20.00	*feet*
Depth to midpoint of Embeddment	h	10.00	*feet*
Earth Pressure Coefficient	K	0.50	
Factor of Safety Applied	F.S.	1.50	

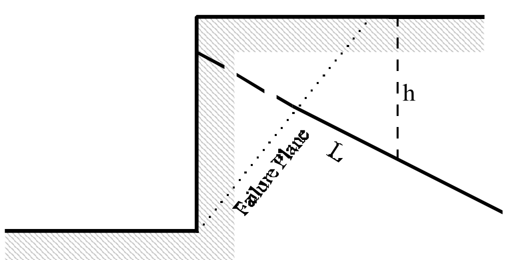
Ultimate Resistance: R_{ult} 21.82 *kips*

Eq: pi*d*γ*L*h*cos(a)*tan(ϕ)+c*pi*d*L

Allowable Resistance: $R_{allow} = R_{ult}/F.S.$ 14.55 *kips*

Allowable Skin Friction: $R_{allow}/2/pi/r/L$ 463.09 *psf*

Allowable Skin Friction Design Value **450** ***psf***



Geotechnologies, Inc.

Project: Bardas Investment Group
File No.: 22167
Description: Slot Cut

Slot Cut Calculation

Input:

Height of Slots	(H)	8.0	feet
Unit Weight of Soils	(γ)	125.0	pcf
Friction Angle of Soils	(ϕ)	28.0	degrees
Cohesion of Soils	(c)	265.0	psf
Factor of Safety	(FS)	1.50	

Factor of Safety = Resistance Force/Driving Force

Coefficient of Lateral Earth Pressure At-Rest	K_o	0.5	

Surcharge Pressure:

Line Load	(q_L)	2800.0	plf
Distance Away from Edge of Excavation	(X)	0.0	feet

Design Equations

$b = H/(\tan \alpha)$

$A = 0.5*H*b$

$W = 0.5*H*b*\gamma$ *(per lineal foot of slot width)*

$F_1 = d*W*(\sin \alpha)*(\cos \alpha)$

$F_2 = d*L$

$R_1 = d*[W*(\cos^2 \alpha)*(\tan \phi)+(c*b)]$

$R_2 = 2*\Delta F$

$\Delta F = A*[1/3*\gamma*H*K_o*(\tan \phi)+c]$

FS = Resistance Force/Driving Force

FS = $(R_1+R_2)/(F_1+F_2)$

Failure Angle (α) degrees	Base Width of Failure Wedge (b) feet	Area of Failure Wedge (A) feet2	Weight of Failure Wedge (W) lbs/lineal foot	**Driving Force** Wedge + Surcharge per lineal foot of Slot Wdith	**Resisting Force** Failure Wedge per lineal foot of Slot Width	**Resisting Force** Side Resistance Force (ΔF) lbs	Allowable Width of Slots* (d) feet
60	4.6	18	2309.4	2212.4	1903.2	6533.2	9.2
61	4.4	18	2217.2	2127.4	1802.2	6272.4	9.0
62	4.3	17	2126.8	2042.3	1704.6	6016.7	8.9
63	4.1	16	2038.1	1957.1	1610.4	5765.7	8.7
64	3.9	16	1950.9	1871.9	1519.4	5519.1	8.6
65	3.7	15	1865.2	1786.9	1431.6	5276.6	8.5
66	3.6	14	1780.9	1702.1	1346.8	5038.1	8.4
67	3.4	14	1697.9	1617.8	1265.0	4803.3	8.3
68	3.2	13	1616.1	1533.8	1186.0	4571.9	8.2
69	3.1	12	1535.5	1450.5	1109.8	4343.7	8.2
70	2.9	12	1455.9	1367.8	1036.3	4118.6	8.1
71	2.8	11	1377.3	1285.9	965.4	3896.3	8.1
72	2.6	10	1299.7	1204.9	897.0	3676.7	8.1
73	2.4	10	1222.9	1124.8	831.0	3459.6	8.1
74	2.3	9	1147.0	1045.8	767.3	3244.7	8.1
75	2.1	9	1071.8	967.9	706.0	3032.1	8.1
76	2.0	8	997.3	891.4	646.7	2821.3	8.2
77	1.8	7	923.5	816.1	589.6	2612.5	8.2
78	1.7	7	850.2	742.3	534.5	2405.2	8.3
79	1.6	6	777.5	670.1	481.3	2199.6	8.4
80	1.4	6	705.3	599.4	430.0	1995.3	8.5
81	1.3	5	633.5	530.5	380.5	1792.2	8.6
82	1.1	4	562.2	463.4	332.6	1590.3	8.8
83	1.0	4	491.1	398.1	286.3	1389.4	8.9
84	0.8	3	420.4	334.8	241.5	1189.3	9.1
85	0.7	3	350.0	273.5	198.2	990.0	9.3

Critical Slot Width with Factor of Safety equal or exceeding 1.5:

d_{allow} 8.1 feet

The proposed excavation may be made using the A-B-C Slot-Cutting Method with a Maximum Allowable Slot Width of 8 Feet, and up to 8 Feet in Height, with a Factor of Safety Equal or Exceeding 1.5.

Geotechnologies, Inc.

Project: Bardas Investment Group

File No.: 22167

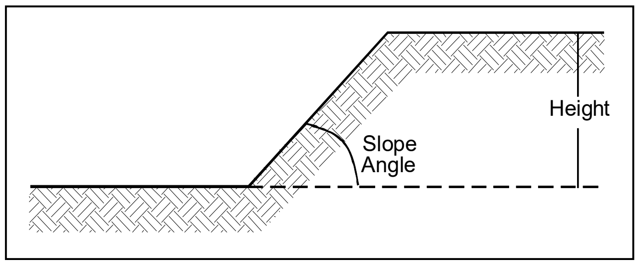
Slope Stability Calculations

Input

Soil Density	(γ)	125	pcf
Friction Angle	(ϕ)	28	degrees
Cohesion	(c)	265	psf
Factor of Safety	(FS)	1.25	

Stability Number (N)

(ϕ_d)	23.0 degrees
$N_{(2:1)}$	0.000
$N_{(1.5:1)}$	0.023
$N_{(1:1)}$	0.052
$N_{(3/4:1)}$	0.070
$N_{(1:1.5)}$	0.077
$N_{(1:2)}$	0.094
$N_{(vertical)}$	0.169



Reference: Taylor's Chart (1937)

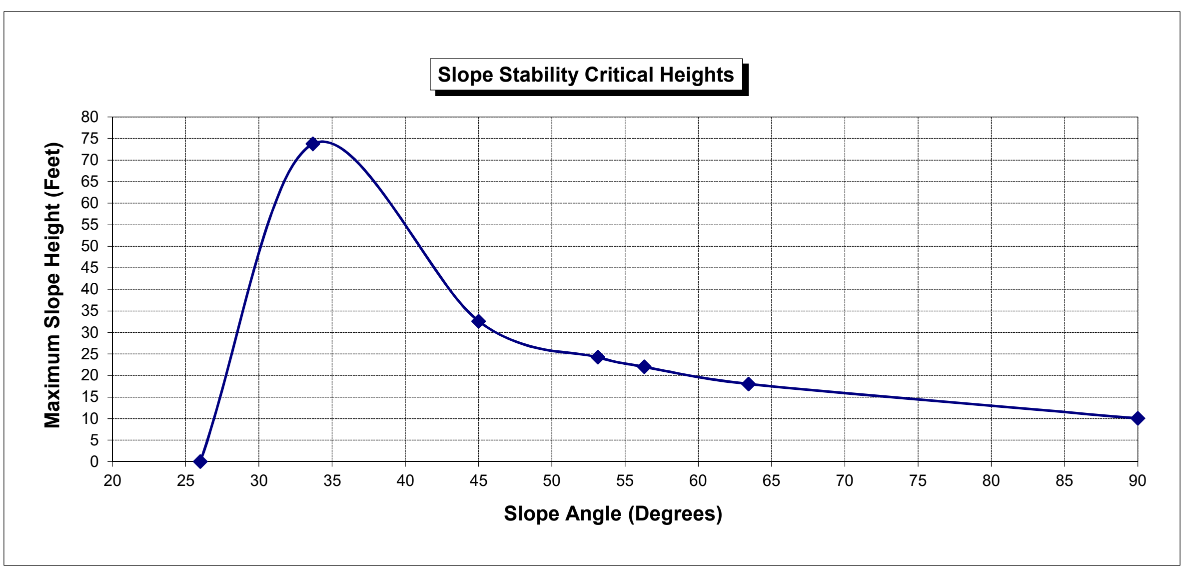
(ϕ_δ)= ArcTan[(Tanϕ)/FS]

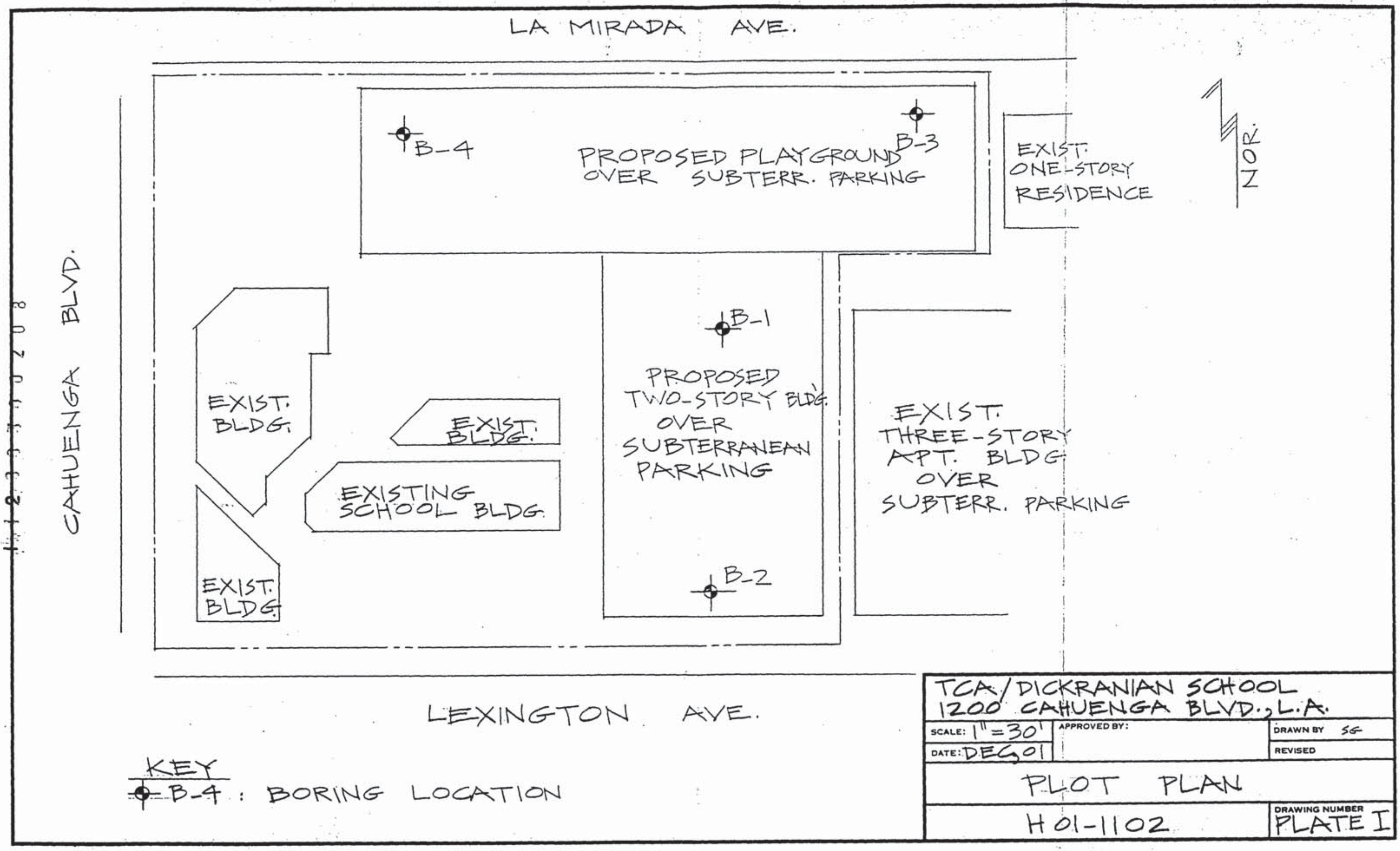
$$N = \frac{c}{(\gamma)(H)(FS)}$$

$$H = \frac{c}{(\gamma)(N)(FS)}$$

Assumptions: Slope is uniform, soils are homogeneous, no water seepage, no surcharge loads.

Slope Angle (h:v)	Slope Angle (Degrees)	Maximum Height (Feet)
2 : 1	26.00	#DIV/0!
$1\frac{1}{2}$: 1	33.69	74
1 : 1	45.00	33
$\frac{3}{4}$: 1	53.13	24
1 : $1\frac{1}{2}$	56.30	22
$\frac{1}{2}$: 1	63.43	18
Vertical	90.00	10





LA MIRADA AVE.
CAHUENGA BLVD.
LEXINGTON AVE.
PROPOSED PLAYGROUND OVER SUBTERR. PARKING
B-4
B-3
B-1
B-2
EXIST. ONE-STORY RESIDENCE
NOR.
PROPOSED TWO-STORY BLDG. OVER SUBTERRANEAN PARKING
EXIST. THREE-STORY APT. BLDG OVER SUBTERR. PARKING
EXIST. BLDG.
EXIST. BLDG.
EXISTING SCHOOL BLDG.
EXIST. BLDG.
KEY
B-4 : BORING LOCATION
TCA / DICKRANIAN SCHOOL
1200 CAHUENGA BLVD., L.A.
SCALE: 1"=30'
DATE: DEC 01
APPROVED BY:
DRAWN BY SG
REVISED
PLOT PLAN
H 01-1102
DRAWING NUMBER
PLATE I

Hakimian Engineering, Inc.

13003 Ladana Court
Santa Fe Springs,
CA 90670
(562)946-7783

Job No. H 01-1102	**Client** TCA Dickranian Sch.	**Location** 1200 N. Cahuenga Blvd. Los Angeles	
Drilling Contractor	C & C Drilling	**Boring No.** B-1	
Equipment	8" Hollow Stem		
Driving Weight	140#/30" Drop		
Surface Elevation		Sheet 1	
Reference		of 2	

Logged By SG | Dated 11/19/01

Depth in feet	Drive Sample	Blows Per 6"	VISUAL DESCRIPTION	Moisture Content (%)	Dry Unit Weight (pcf)
0			**Alluvium** Dark brown silty clay, moderately firm and damp		
2	CA	7 15	Dark brown silty clay, moderately firm and damp	21	100
5	SPT	7 12 18	Light brown sandy silty clay, firm and damp		
7	CA	14 17	Reddish brown clayey silty sand, dense and damp	16	112
10	SPT	15 17 19	Reddish brown sandy silt, firm and damp		
12	CA	18 25	Reddish brown fine gravelly silty sand, dense and damp	20	103
15	SPT	14 16 18	Reddish brown fine gravelly silty sand, dense and damp		
17	CA	19 26	Reddish brown silghtly clayey silty sand, micaceous, dense and damp	18	100
20	SPT	17 18 19	Brown slightly gravelly slightly clayey silty sand, dense and damp		
22	CA	17 28	Mottled grayish and reddish brown clayey silty sand, dense and damp	22	102
25	SPT	15 17 18	Reddish brown sandy silty clay, firm and wet		

PLATE II

Hakimian Engineering, Inc.

13003 Ladana Court
Santa Fe Springs,
CA 90670
(562)946-7783

Job No. H 01-1102	Client TCA Dickranian Sch.	Location 1200 N. Cahuenga Blvd. Los Angeles	
Drilling Contractor	C & C Drilling	Boring No. B-1	
Equipment	8" Hollow Stem		
Driving Weight	140#/30" Drop	Sheet 2	
Surface Elevation		of 2	
Logged By SG	Dated 11/19/01	Reference	

Depth in feet	Drive Sample	Blows Per 6"	VISUAL DESCRIPTION	Moisture Content (%)	Dry Unit Weight (pcf)
27	CA	17 15	Reddish brown sandy silty clay, firm and wet	16	116
30	SPT	16 17 21	Reddish brown sandy silty clay, firm and wet		
35	SPT	9 17 24	Reddish brown fine gravelly sandy clay, firm and wet		
37	CA	15 32	Reddish brown fine gravelly sandy clay, firm and wet	21	108
40	SPT	12 14 29	Reddish brown clayey silty sand, dense and wet		
45	SPT	9 14 26	Reddish brown fine gravelly sandy clay, firm and wet		
50	SPT	10 16 23	Reddish brown sandy silty clay, firm and wet		
55	SPT	22 50	Reddish brown sandy silty clay, very firm and wet Total Depth = 60', Groundwater @ 25'		

PLATE III

Hakimian Engineering, Inc.

13003 Ladana Court
Santa Fe Springs,
CA 90670
(562)946-7783

Job No.	H 01-1102
Client	TCA Dickranian Sch.
Location	1200 N. Cahuenga Blvd. Los Angeles
Drilling Contractor	C & C Drilling
Equipment	8" Hollow Stem
Driving Weight	140#/30" Drop
Surface Elevation	
Reference	
Boring No.	B-2
Sheet	1 of 1
Logged By	SG
Dated	11/19/01

Depth in feet	Drive Sample	Blows Per 6"	VISUAL DESCRIPTION	Moisture Content (%)	Dry Unit Weight (pcf)
0			**Alluvium** Dark brown silty clay, firm and damp.		
	CA	13 / 20	Dark brown silty clay, firm and damp	15	115
5	CA	10 / 22	Dark brown sandy silty clay, firm and damp	14	120
10	CA	18 / 21	Reddish brown sandy clay, firm and damp	15	116
15	CA	14 / 19	Light brown silty sand, micaceous dense and slightly damp	10	100
20	CA	12 / 20	Brown sand silty clay, firm and damp	19	109
25	CA	10 / 19	Mottled grayish and reddish brown fine gravelly sandy clay, firm and wet	17	116
30	CA	13 / 23	Mottled grayish and reddish brown fine gravelly sandy clay, firm and wet. Total Depth = 30', Groundwater @ 25'	16	117

PLATE IV

Hakimian Engineering, Inc.

13003 Ladana Court
Santa Fe Springs,
CA 90670
(562)946-7783

Job No.	Client TCA	Location
H 01-1102	Dickranian Sch.	1200 N. Cahuenga Blvd. Los Angeles
Drilling Contractor	C & C Drilling	Boring No. B-3
Equipment	8" Hollow Stem	
Driving Weight	140#/30" Drop	
Surface Elevation		Sheet 1
Reference		of 1

Logged By SG   Dated 11/19/01

Depth in feet	Drive Sample	Blows Per 6"	VISUAL DESCRIPTION	Moisture Content (%)	Dry Unit Weight (pcf)
0			**Alluvium** Dark brown slightly sandy clay, moderately firm and damp		
2	CA	6 / 12	Dark brown slightly sandy clay, moderately firm and damp	20	107
5	CA	16 / 18	Reddish brown sandy silty clay, firm and damp	16	116
10	CA	17 / 26	Reddish light brown clayey sand, dense and damp	17	113
15	CA	15 / 22	Light brown slightly fine gravelly silty sand, dense and damp	18	95
20	CA	18 / 24	Light brown slightly fine gravelly sandy silty clay, firm and damp	21	103
25	CA	11 / 20	Light brown fien gravelly clayey sand, dense and damp	16	115

Total Depth = 25', No groundwater

PLATE V

11230300212

Hakimian Engineering, Inc.

13003 Ladana Court
Santa Fe Springs,
CA 90670
(562)946-7783

Job No.	Client TCA	Location
H 01-1102	Dickranian Sch.	1200 N. Cahuenga Blvd. Los Angeles
Drilling Contractor	C & C Drilling	Boring No. B-4
Equipment	8" Hollow Stem	
Driving Weight	140#/30" Drop	
Surface Elevation		Sheet 1
Reference		of 1

Logged By SG   Dated 11/19/01

Depth in feet	Drive Sample	Blows Per 6"	VISUAL DESCRIPTION	Moisture Content (%)	Dry Unit Weight (pcf)
			3 AC 2" Base **Alluvium**		
0			Light brown silty clayey sand, dense and slightly damp		
2	CA	13 18	Brown silty clayey sand, dense and damp	13	116
5	CA	13 24	Brown clayey silt/silty clay firm and damp	16	17
10	CA	14 17	Light brown gravelly clayey sand, dense and slightly damp	11	120
15	CA	10 16	Light brown clayey slightly gravelly sand, dense and damp	14	116
20	CA	18 22	Light brown clayey slightly gravelly sand, dense and damp	15	117
25	CA	10 20	Mottled grayish and reddish brown clayey sand, dense and damp Total Depth = 25', No groundwater	20	110

PLATE VI

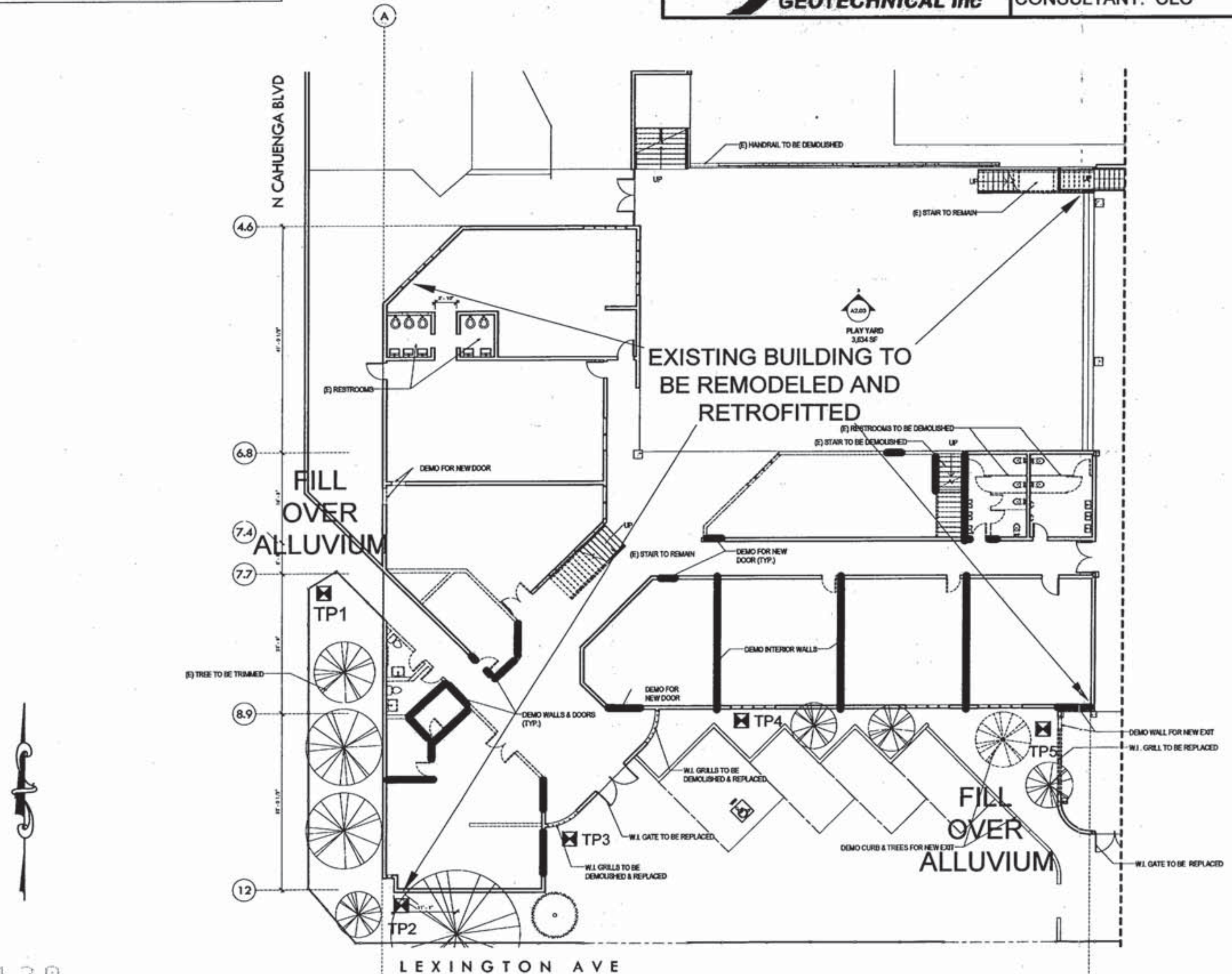
LEGEND

TP1 — NUMBER & LOCATION OF TEST PIT

IRVINE GEOTECHNICAL Inc

SITE PLAN

PROJECT: IC16007 - STRATFORD	
CONSULTANT: CLC	SCALE: 1" = 20'



IRVINE GEOTECHNICAL Inc

LOG OF TEST PITS

PROJECT IC16007 STRATFORD
DRILL DATE 1/25/2016
LOG DATE 1/25/2016
LOGGED BY KJONES
DRILL TYPE Hand Labor
DIAMETER 30 Inches

SURFACE ELEVATION 314 feet
DRILLING CONTRACTOR Mike's Excavating Service
SURFACE CONDITIONS Planter west of school structure

TEST PIT 1

Sample Type	Sample Depth (feet)	Blows per foot	Moisture (%)	Dry Unit Weight (pcf)	Saturation (%)	USCS Code	Elevation (feet)	Depth (feet)	Lithologic Description
						SM/SC	314.0	0	**FILL:** Silty fine Sand with Clay binder, dark brown, porous, moist, medium dense, roots and rootlets, gravel to 2" in diameter
							313.0	1	
R	2	N/A	14.7	95.3	53		312.0	2	Fine Sand with Clay binder and Gravel, dark orange-brown, moist, porous, medium dense
						SM	311.0	3	**ALLUVIUM:** Silty fine Sand, orange-brown, moist, medium dense to dense
R	4	N/A	15.4	114.4	92		310.0	4	Silty fine Sand with Clay binder, orange-brown, wet, medium dense to dense, occasional gravel to 1" in diameter
							309.0	5	
R	6	N/A	13.0	108.3	65		308.0	6	Silty Sand with Clay binder, orange-brown, moist, dense, occasional gravel to 1" in diameter
							307.0	7	
R	8	N/A	16.8	106.4	81	CL	306.0	8	Silty Clay with Sand, orange-brown, wet, slightly, firm to stiff, occasional gravel to 1" in diameter
							305.0	9	
R	10	N/A	11.5	101.9	49		304.0	10	Silty Sand with Clay binder, orange-brown, slightly moist, porous, medium dense
							303.0	11	
R	12	N/A	10.7	106.1	51	SM	302.0	12	

END TP1 @ 12': No Water; No Caving; Fill to 3'; 1.5-inch metal pipe @ 3'

1060221201728120

LOG OF TEST PITS

IRVINE GEOTECHNICAL Inc

PROJECT	IC16007 STRATFORD
DRILL DATE	1/25/2016
LOG DATE	1/25/2016
LOGGED BY	KJONES
DRILL TYPE	Hand Labor
DIAMETER	30 Inches

SURFACE ELEVATION	314 feet
DRILLING CONTRACTOR	Mike's Excavating Service
SURFACE CONDITIONS	In planter at southwest corner of property

TEST PIT 2

Sample Type	Sample Depth (feet)	Blows per foot	Moisture (%)	Dry Unit Weight (pcf)	Saturation (%)	USCS Code	Elevation (feet)	Depth (feet)	Lithologic Description
							314.0	0	**FILL:** Silty fine Sand with Clay binder, light brown, slightly moist, porous, medium dense, roots and rootlets
						SM	313.0	1	
R	2	N/A	18.1	106.5	87		312.0	2	Silty Clay with Sand, dark brown, wet, slightly porous, firm, roots and rootlets
							311.0	3	
						CL	310.0	4	**ALLUVIUM:** Silty Clay, dark brown, wet, slightly porous, stiff, occasional gravel to 1" in diameter
R	5	N/A	18.3	110.2	97		309.0	5	
									END TP2 @ 5': No Water; No Caving; Fill to 3.5'; Footings extend 26" below adjacent grade; 1.5" PVC pipe @ 3"

1060221201728120

LOG OF TEST PITS

IRVINE GEOTECHNICAL Inc

PROJECT IC16007 STRATFORD
DRILL DATE 1/25/2016
LOG DATE 1/25/2016
LOGGED BY KJONES
DRILL TYPE Hand Labor
DIAMETER 30 Inches

SURFACE ELEVATION 315 feet
DRILLING CONTRACTOR Mike's Excavating Service
SURFACE CONDITIONS In planter near entryway

TEST PIT 3

Sample Type	Sample Depth (feet)	Blows per foot	Moisture (%)	Dry Unit Weight (pcf)	Saturation (%)	USCS Code	Elevation (feet)	Depth (feet)	Lithologic Description
						SM/SC	315.0	0	**FILL:** Silty fine Sand with Clay binder, light brown, moist, porous, medium dense, roots and rootlets, concrete fragments, gravel to 1" in diameter
R	1	N/A	20.1	100.6	83	CL	314.0	1	Silty Clay with Sand, dark brown, wet, slightly porous, firm
							313.0	2	
R	3	N/A	12.9	114.5	77	CL	312.0	3	**ALLUVIUM:** Silty Clay with Sand, dark brown, wet, slightly porous, firm to stiff, roots and rootlets, occasional gravel to 1" in diameter
							311.0	4	
R	5	N/A	12.1	110.7	65		310.0	5	Silty Sand with Clay binder, orange-brown, moist, dense to very dense
							309.0	6	
R	7	N/A	14.2	105.4	66	SM	308.0	7	
							307.0	8	
R	9	N/A	12.6	96.4	47		306.0	9	
									END TP3 @ 9.5': No Water; No Caving; Fill to 3'

1 0 6 0 2 2 1 2 0 1 7 2 8 1 2 0

IRVINE GEOTECHNICAL Inc

LOG OF TEST PITS

PROJECT IC16007 STRATFORD
DRILL DATE 1/25/2016
LOG DATE 1/25/2016
LOGGED BY KJONES
DRILL TYPE Hand Labor
DIAMETER 30 Inches

SURFACE ELEVATION 315 feet
DRILLING CONTRACTOR Mike's Excavating Service
SURFACE CONDITIONS In planter north of parking lot

TEST PIT 4

Sample Type	Sample Depth (feet)	Blows per foot	Moisture (%)	Dry Unit Weight (pcf)	Saturation (%)	USCS Code	Elevation (feet)	Depth (feet)	Lithologic Description
						SM/SC	315.0	0	**FILL:** Silty fine Sand with Clay binder, light brown, moist, porous, medium dense, roots and rootlets, gravel to 1" in diameter
							314.0	1	
							313.0	2	
							312.0	3	
						CL			**ALLUVIUM:** Silty Clay with Sand, dark brown, wet, slightly porous, firm, roots and rootlets, occasional gravel to 1" in diameter
R	4	N/A	19.3	108.3	97		311.0	4	
									END TP4 @ 4': No Water; No Caving; Fill to 3'; Footings extend 26" below adjacent grade; PVC and metal pipe at 3"

IRVINE GEOTECHNICAL Inc

LOG OF TEST PITS

PROJECT IC16007 STRATFORD
DRILL DATE 1/25/2016
LOG DATE 1/25/2016
LOGGED BY KJONES
DRILL TYPE Hand Labor
DIAMETER 30 Inches

SURFACE ELEVATION 315 feet
DRILLING CONTRACTOR Mike's Excavating Service
SURFACE CONDITIONS Planter at northeast portion of parking lot

TEST PIT 5

Sample Type	Sample Depth (feet)	Blows per foot	Moisture (%)	Dry Unit Weight (pcf)	Saturation (%)	USCS Code	Elevation (feet)	Depth (feet)	Lithologic Description
						SM	315.0	0	**FILL:** Silty fine Sand, light brown, slightly moist, porous, dense, roots and rootlets, gravel to 1" in diameter, concrete fragments
R	1	N/A	27.1	87.2	80	SM	314.0	1	Silty Sand with Clay binder, orange-brown, wet, slightly porous, medium dense
							313.0	2	
R	3	N/A	13.0	103.3	57	CL/ML	312.0	3	**ALLUVIUM:** Silty Clay/Clayey Silt, dark brown, moist, firm
							311.0	4	
R	5	N/A	13.0	112.3	73	SM	310.0	5	Silty Sand, orange-brown, moist, dense, occasional gravel to 1" in diameter
							309.0	6	
R	7	N/A	13.2	108.0	66	SM	308.0	7	Silty Sand with Clay binder, orange-brown, moist, dense
							307.0	8	
R	9	N/A	11.3	108.5	57	SM	306.0	9	Silty fine Sand, orange-brown, moist, dense

END TP5 @ 9.5': No Water; No Caving; Fill to 3'; Footings extend 26" below adjacent grade

1060221201728120