

INITIAL STUDY

APPENDIX F: GEOLOGY AND PALEONTOLOGICAL REPORTS

INITIAL STUDY

APPENDIX F.1: GEOTECHNICAL ENGINEERING INVESTIGATION AND GRADING DIVISION LETTERS



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July 19, 2021
Revised August 23, 2021
File No. 22064

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

Attention: Collin Monsour

Subject: Geotechnical Engineering Investigation
Proposed Commercial Development
5601 Santa Monica Boulevard, Los Angeles, California

References: *Reports by Geotechnologies, Inc.*
Geotechnical Engineering Investigation, dated April 13, 2016,
Revised April 27, 2016 (File No. 21151);
Geotechnical Engineering Investigation, dated July 19, 2021 (File No. 22064).

Correspondence with City of Los Angeles, Department of Building and Safety:
Geology and Soil Report Approval Letter, dated May 20, 2016, LOG# 92915;
Soils Report Approval Letter, dated June 27, 2016, LOG# 93517.

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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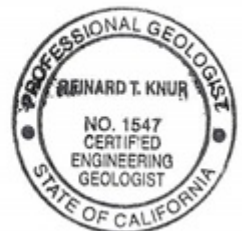


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Boring Logs by Geocon (25 pages)
Laboratory Results by Geocon (9 pages)
City of Los Angeles, Geology and Soils Report Approval Letter, dated May 20, 2016 (6 pages)
City of Los Angeles, Soils Report Approval Letter, dated June 27, 2016 (3 pages).
Surface Wave Geophysical Survey Results by GeoPentech, Inc. (11 Sheets)



**GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED COMMERCIAL DEVELOPMENT
5601 SANTA MONICA BOULEVARD
LOS ANGELES, CALIFORNIA**

INTRODUCTION

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

This investigation utilized information from a previous on-site investigation by this firm dated April 13, 2016 (File Number 21151). The previous geotechnical investigation included three exploratory excavations, and conversion of one of the borings into a water monitoring well. The exploratory excavation locations are indicated on the enclosed Plot Plan. The results of the exploration and laboratory testing are presented in the Appendix of this report. The report was approved by the City of Los Angeles, Department of Building and Safety (LADBS) in a letter dated May 20, 2016 (LOG #. 92915). A request for modification for the use of micropiles was prepared by this firm dated June 7, 2016. The request was approved by the LADBS on June 27, 2017 (LOG# 93517).

As part of the previous investigation, this firm was provided with the geotechnical engineering report by another consultant, Geocon, dated April 17, 2007. The investigation was prepared for a proposed project that consisted of a seven-story mixed use structure, with five levels of subterranean parking. The report included 6 borings and laboratory testing. The report was submitted to the City of Los Angeles, Grading Division for review and was subsequently approved. Follow-up review letters by the City of Los Angeles (dated June 21, 2007 and September 11, 2007) were reviewed and approved as well. The findings presented in the previous investigation by Geocon were considered during the preparation of this report.



This investigation included three exploratory excavations, collection of representative samples, laboratory testing, engineering analyses, and the preparation of this report. The exploratory excavation locations are indicated on the enclosed Plot Plan. The results of the exploration and the laboratory testing are presented in the Appendix of this report. The borings logs, laboratory testing results and approval letters from the previous site investigations by this firm and the previous consultant are attached.

A surface wave geophysical survey was performed using active and passive-source surface wave techniques by GeoPentech, Inc. The results of the geophysical survey indicated a shear-wave velocity measurement (V_{s30}) of 371 m/s (equivalent to 1,217 ft/s) to a depth of 30 meter (or 100 feet) below the ground surface. This corresponds to a site classification of Site Class C characterized as “Very Dense Soil and Soft Rock”. The result of the Surface Wave Geophysical Survey, prepared by GeoPentech, is presented in the Appendix of this report.

PROPOSED DEVELOPMENT

Information concerning the proposed development was furnished by Bardas Investment Group. The proposed development will consist of a commercial structure which is six stories in height and includes two to three levels of subterranean parking. The finish floor basement elevations will be approximately 305 feet above mean sea level (AMSL) at parking level P2 and 295 feet for parking level P3. The P3 level extends only partially across the southern half of the site.

Column loads are estimated to be between 600 and 900 kips. Wall loads are estimated to range between 15 and 20 kips per lineal foot. Grading will consist of an excavation up to 40 feet for the proposed subterranean levels including foundation elements and elevator pit enclosures. The limits of the proposed structure are indicated on the attached Plot Plan and profiles of the proposed excavation are indicated on Cross Sections A-A' and B-B'.



The southeast corner of the site is currently occupied by a multi-story retail structure. The existing structure will be demolished in preparation for the newly proposed development.

SITE CONDITIONS

The proposed development is located on the alluvial fan below the southern slope of the Hollywood Hills in Los Angeles, California on gently descending topography. The site is bounded to the north by Virginia Avenue, to the east by North St. Andrews Place, to the south by Santa Monica Boulevard, and to the west by Wilton Place. The site is rectangular in shape and approximately 5.1 acres in area. The site is indicated relative to nearby topographic features on the attached Vicinity Map, Plot Plan and Cross-Sectional drawings.

Site elevations range from 331 feet above mean sea level at the northeast corner to 326 feet at the southwest corner for an elevation difference of 5 feet and a ground surface gradient of 160 to 1. The site was previously developed with several single-story and two-story retail structures along Santa Monica Boulevard that have been recently demolished. Vegetation is virtually nonexistent due to the extent of the demolition, site clearing and paved parking areas within the site. The site and surrounding site topography are indicated on the attached Plot Plan.

A vacant, multi-story, retail structure (formerly occupied by Sears) is located at the southeast corner of the site. The building includes a basement with a finish floor elevation of 318 feet. Information regarding the foundation design of this building has not been reviewed. It is the understanding of this firm that the structure will be demolished in preparation for the newly proposed development. The existing structure has a higher finish floor elevation than the proposed commercial development. It is unknown if the structure is supported on shallow or deep foundation system. It is recommended that the foundation plans for the existing structure be obtained and reviewed to identify for demolition requirements.



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 0.85 miles to the northwest 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 shown 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.



PREVIOUS SITE EXPLORATIONS BY THIS FIRM AND OTHERS

Geotechnologies, Inc. November 15, 2017, revised December 28, 2017, Supplemental Information for a Request for Modification, Proposed Mixed Use Development, 5601-5667 Santa Monica Boulevard, Los Angeles, California,

This letter was prepared to provide detailed recommendations regarding the design and testing of Micropiles to resist buoyancy pressures on the proposed structure caused by a high groundwater level.

Geotechnologies, Inc. June 7, 2016, Supplemental Information for a Request for Modification, Proposed Missed Use Development, 5601-5667 Santa Monica Boulevard, Los Angeles, California.

This letter was prepared to clarify that the use of micropiles is permitted on the site. The micropiles were needed to resist buoyancy pressures on the structure caused by the excavation extending well below the historically highest groundwater level.

City of Los Angeles, June 27, 2016, Soils Report Approval Letter, LOG# 93517.

This letter approves the use of micropiles for use in resisting buoyancy pressures on the proposed structure.

Geotechnologies, Inc. April 13, 2016, Revised April 27, 2016, Geotechnical Investigation, Mixed-Use Development, 5601-5667 Santa Monica Boulevard, Los Angeles, California, File No. 21151.

Geotechnologies, Inc. previously drilled three borings within the site to depths ranging from 50 to 100 feet using a truck-mounted drilling rig equipped with 8-inch diameter hollowstem augers. The two 100-foot borings were sampled using both a California-modified, split-spoon sampler lined



with 2.5-inch diameter brass rings and a Standard Penetration Test sampler (SPT). The soil cuttings were backfilled into the borings upon completion.

Boring B-3 was drilled to a depth of 50 feet and was sampled using the California-Modified, split-spoon sampler. Upon completion, the boring was converted into a water-monitoring well. The well consisted of a 2-inch diameter, perforated pipe in the lower 30 feet and solid pipe in the upper 20 feet. The annular space of the lower 30 feet of the boring was backfilled with clean sand followed by a 2-foot-thick layer of hydrated bentonite chips. The remaining space was backfilled with soil cuttings from the boring. A traffic-rated cover, set in concrete completed the installation. Upon completion, the well was sounded. Geotechnologies returned on two later dates to sound and pump the well.

The boring locations by Geotechnologies as well as those by the previous consultant are indicated on the attached Plot Plan. The boring logs are included as Plates A-1 through A-3.

City of Los Angeles, May 20, 2017, Soils Report Approval Letter, LOG# 92915.

The approval letter refers to the geotechnical engineering investigation submitted by this firm dated April 13, 2016 (revised April 27, 2016).

Geocon, Inland Empire, Inc. Revised April 17, 2007, Geotechnical Investigation, Paseo Plaza Mixed-Use Development, 5661 Santa Monica Boulevard, Los Angeles, California, Project No. A8284-06-01.

This investigation included drilling six borings to depths ranging between 65 and 90 feet deep. The borings were drilled with a hollowstem auger and only standard ring samples were taken. Standard Penetration tests used for screening in liquefaction assessments were not obtained. The report identified fill soils to depths of 2 to 8 feet below the ground surface. Alluvial soils consisting



of silty sand, clayey sand and silty clay were identified below the fill. Water was reported at depths of 21 to 28.5 feet. Copies of the boring logs and laboratory testing are included in the Appendix.

City of Los Angeles, June 7, 2007, Soils Report Approval Letter, LOG# 58231.

The letter was prepared as a response to the above-referenced report. The report was approved; however, a supplementary report identifying the effects of proposed dewatering on the adjacent structures and facilities was requested. To the knowledge of this firm, this analysis has not been performed.

City of Los Angeles, September 10, 2007, Soils Report Approval Letter, LOG# 59760.

The review letter refers to pavement recommendations.

Geocon, Inland Empire, Inc., September 11, 2007, Disapproval of Onsite Storm Water Infiltration, Paseo Plaza Mixed-Use Development, 5661 Santa Monica Boulevard, Los Angeles, California, Project No. A8284-06-03.

This letter by Geocon advises against the installation of a stormwater infiltration device since current and historically highest groundwater levels were higher than the finish floor of the proposed structure.

City of Los Angeles, October 11, 2007, Soils Report Approval Letter, LOG# 60287.

The reviewer agreed with the recommendation to avoid stormwater infiltration on the site. However, the regulatory requirement for stormwater disposal still applies.



GEOTECHNICAL EXPLORATION

The site was explored on May 10 and May 11, 2021. The site exploration consisted of three exploratory borings which were excavated with the aid of a hollow-stem drill rig. The borings ranged in depth 40 to 60 feet. Soil samples were taken at 5-foot intervals with a California-modified split spoon sampler lined with 2.50-inch diameter brass rings in each of the borings. In Boring 1, additional standard penetration tests (SPTs) were taken at alternating 5-foot-intervals offset by 2.5 feet. Both types of samplers were advanced with a 140-pound hammer dropped from a height of 30 inches with an automatic hammer. The exploration locations are indicated on the Plot Plan and the geologic materials encountered are logged on Plates A-1 through A-3.

The location of exploratory excavations were determined measurement from hardscape features shown on the Plot Plan. The elevation of the borings was determined by interpolation of the elevation contours shown on the NavigateLA website. The locations of the exploratory excavations should be considered accurate only to the degree implied by the method used.

Geologic Materials

The geologic materials consist of fill, older alluvium and bedrock of the Puente Formation.

Fill

The fill consists of sandy silt to silty clay that is yellowish brown, moist, dense and stiff. Fill was observed at a depth of three feet during this investigation. Fill was observed to extend as much as eight feet below the ground surface based on previous investigations.



Older Alluvium

The older alluvium consists of silty to clayey sand, silty to sandy clay, and sand with occasional gravel and cobbles in layers near the contact with the underlying bedrock. The older alluvium is dark brown to dark yellowish brown in color, moist to wet, stiff, and medium dense to very dense in consistency. The alluvium extends to depths of 22 to 51 feet below the ground surface based on this investigation and previous investigations completed within the site.

Bedrock (Puente Formation)

Bedrock was identified in each of the borings drilled during this investigation and the previous investigation by this firm within the site. The bedrock consists of interbedded clayey siltstone, sandstone and claystone that is yellowish and grayish brown, moist to wet, and moderately hard to hard. The rock appeared moderately well bedded, slightly fractured. Bedding dips at a shallow angle, approximately 20 degrees or less.

A bedrock elevation contour map was prepared based on the information from the boring logs. The boring logs by the previous consultant were reinterpreted by considering the descriptions, blow counts, moisture contents and dry densities to prepare the lines of equal elevation. The bedrock elevations vary from 275 feet near the eastern corners of the site to 308 feet near the center and western portions of the site. The elevation map indicates that the bedrock is highest near the center and west of center of the site and generally descends in all directions away from that point. Based on proposed finish floor elevations of 305 and 295 feet AMSL, bedrock will likely be exposed at the subgrade elevation in the center and western portions of the site during grading and foundation excavation.



The bedrock elevations by Geotechnologies are based on observed samples taken at 5 and 2 ½ foot intervals. The elevations taken from Geocon are based on interpreted boring logs and sampling at 10, 5, 2 ½ foot intervals.

Groundwater

Groundwater was encountered by this investigation in Borings 1, 2 and 3 at depths of 39 feet, 32 feet and 35 feet, respectively. The water elevations for this investigation and previous site investigations are summarized in the following table:

Boring Number and Consultant	Ground Surface Elevation (feet)	Depth to Water (Feet)	Water Elevation (Feet)	Bedrock Elevation (Feet above MSL)
1 This Investigation	327	39 (5/10/21)	288	277
2	328	32 (5/10/21)	296	298
3	329.5	35 (5/11/21)	294.5	304.5
1 Geotech. File 21151	328	32.0 (2/4/16)	296	305.5
2	329	N.A. (2/4/16)	N.A.	299
3 (Converted to Well)	328.5	35.1 (2/5/16) 35.4 (2/26/16) 35.3 (3/21/16) 35.6 (4/26/16) 42 (5/11/21)	293.4 293.1 293.2 292.9 286.5	287.5
1 Geocon	331	23 (3/29/05)	308	*281
2	327	22 (3/29/05)	305	*288
3	328	28.5 (3/24/05)	299.5	*308
4	330	24.5 (3/23/05)	305.5	*295
5	328	26 (3/28/05)	302	*284
6	326	21 (3/28/05)	305	*292

*Denotes elevation interpreted by Geotechnologies, Inc.



In most borings, the water elevation was identified above the bedrock contact. It is the assessment of this firm that the water is perched on the bedrock surface. The thickness of the saturated layer is dependent on the elevation of the underlying bedrock surface.

Boring 3 from the previous site investigation (File No. 21151) was converted into a groundwater monitoring well. A submersible pump was lowered into the casing and pumped at rate of 1.5 gallons per minute. The pump was able to draw down the water elevation to 387.5 feet (from 293.1 feet) where discharge-inflow equilibrium was reached. The pump was shut off and the water level rebounded to 293.1 feet in 43 minutes. Similar results were obtained on February 26, 2013, and March 21, 2016. The well was not pumped on the April 26, 2016 visit. A recent water level reading was obtained on May 11, 2021 which indicated groundwater at a depth of 42 feet corresponding to an elevation of 386.5 feet.

The Seismic Hazard Zone Report for the Hollywood 7.5-Minute Quadrangle (CDMG, 1997) (CGS, 2006) indicates the historically highest groundwater is approximately 20 feet below the ground surface corresponding to elevation 308 feet above mean sea level. This depth is higher than the water level identified in the borings and groundwater monitoring well.

Caving

Caving could not be directly observed during exploration. Caving was not observed during drilling of the borings due to the continuously cased design of the auger. 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

Los Angeles Basin

The Los Angeles Basin is located at the northern end of the Peninsular Ranges Geomorphic Province. 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 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 5 miles of marine and non-marine sedimentary rock as well as intrusive and extrusive igneous rocks have filled the basin. During the last 2 million years, defined by the Pleistocene and Holocene epochs, the Los Angeles basin and surrounding mountain ranges have been uplifted to form the present-day landscape. Erosion of the surrounding mountains has resulted in deposition of unconsolidated sediments in low-lying areas by rivers such as the Los Angeles River. Areas that have experienced subtle uplift have been eroded with gullies.

REGIONAL FAULTING

Based on criteria established by the California Division of Mines and Geology (CDMG) now called California Geologic Survey (CGS), Faults may be categorized as 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 0.75 miles to the northwest 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.

The Seismic Hazard Maps of the State of California (CDMG, 1999) indicates the site is not located in 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 map is included in the Appendix of this report.

As a precaution, 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.

The results from Boring 1 of this investigation and Boring 1 and 2 from a previous investigation (File No. 21151) were used in the liquefaction analysis. Groundwater was encountered at a depth of 39 feet in Boring 1 of this investigation. Groundwater was observed in Boring 1 at a depth of



32 feet from a previous site investigation. Groundwater was not encountered in Boring 2 from the previous site investigation. According to the Seismic Hazard Zone Report of the Hollywood 7½-Minute Quadrangle (CDMG, 1998, Revised 2006), the historically-high groundwater level of 20 feet below the ground surface was used during liquefaction analysis.

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 “C” (“Very Dense Soil and Soft Rock” Profile) was utilized in the USGS seismic and SEAOCC/OSHPD ground motion utility tools. A modal magnitude (M_W) of 6.9 was obtained using the USGS Probabilistic Seismic Hazard Deaggregation program (USGS, 2021). A peak ground acceleration PGA_M of 1.07g, 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 a 2/3 PGA_M was estimated by multiplying 1.07g by 2/3 for a result of 0.71g. 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 of representative samples of the soils encountered in the exploratory boring are presented on the enclosed E-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, and 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 collected SPT data and laboratory testing, the enclosed liquefaction analyses indicate that the soils underlying the site would not be susceptible to liquefaction during the design-based earthquake. The site-specific liquefaction analyses are included in the Appendix of this report.

Lateral Spreading

Lateral spreading is the most pervasive type of liquefaction-induced ground failure. During lateral spread, blocks of mostly intact, surficial soil displace downslope or towards a free face along a shear zone that has formed within the liquefied sediment. According to the procedure provided by Bartlett, Hansen, and Youd, "Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement", ASCE, Journal of Geotechnical Engineering, Vol. 128, No. 12, December 2002, when the saturated cohesionless sediments with $(N_1)_{60} > 15$, significant displacement is not likely for $M < 8$ earthquakes.

The enclosed liquefaction analysis included in the Appendix, indicates that site soils would not be prone to liquefaction during 2475-year return period ground motion. Accordingly, lateral spreading is considered to be unlikely during a design-based seismic event.

Dynamic Dry Settlement

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

Some seismically-induced settlement of the proposed structures should be expected as a result of strong ground-shaking, however, due to the uniform nature of the underlying geologic soils, 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 across or adjacent to the site.

METHANE ZONES

This office has reviewed the City of Los Angeles Methane and Methane Buffer Zones map. Based on this review, the subject site is not located within a methane zone as designated by the City (NavigateLA Website, 2021). A copy of the portion of the map illustrating the project site is included herein entitled Methane Zone Risk Map.

CONCLUSIONS AND RECOMMENDATIONS

Based upon the exploration, laboratory testing, and research, it is the finding of Geotechnologies, Inc. that construction of the proposed commercial development is considered feasible from a



geotechnical engineering standpoint provided the advice and recommendations presented herein are followed and implemented during construction.

The site is underlain by a relatively thin cover of fill soil, then older alluvial soil, and siltstone bedrock of the Puente Formation. Fill soil consisting of sandy silt to silty clay and was observed to extend to a maximum depth of 8 feet during site exploration. The older alluvium consists of clayey sand to silty clay and silty sand to sand that is medium dense to dense and very stiff. The older alluvium extends to a depth of 20 to 50 feet below the ground surface. The bedrock consists of interbedded clayey siltstone, sandstone and claystone that is greenish gray and light reddish or yellowish brown. The bedrock appeared moderately well bedded, slightly fractured. Bedding dips at a shallow angle, approximately 20 degrees or less. The dip is likely to the north based on regional bedding attitudes. The bedrock is identified at depths ranging from 20 to 50 feet which are correlative to elevations 277 to 308 feet above mean sea level.

Groundwater was generally observed to be perched on the bedrock surface. Static groundwater was recently measured in the monitoring well at a depth of 42 feet which correlates to elevation of approximately 286.5 feet. The historically highest groundwater level is estimated at a depth of 20 feet below existing ground surface which corresponds to an elevation of 308 feet.

The site is not located in an area considered by the California Geologic Survey to be potentially liquefiable. Site-specific testing and analysis indicated that the site was not susceptible to liquefaction. The site is not identified as underlain by the surface trace of a fault nor is the site located within an Alquist-Priolo Earthquake Fault Zone. The site is not located within a Methane or Methane Buffer zone.

A seismic shear-wave velocity measurement was performed using active and passive-source surface wave techniques at the project site by GeoPentech, Inc. The result of the geophysical survey indicated a shear-wave velocity measurement (V_{s30}) of 371 m/s (equivalent to 1,217 ft/s) to a depth of 30 meter (or 100 feet) below the ground surface. This corresponds to a site classification for seismic design of Site Class C characterized as “Very Dense Soil and Soft Rock”.



The result of the Surface Wave Geophysical Survey, prepared by GeoPentech, is presented in the Appendix of this report.

The historically highest groundwater level is at elevation 308 feet AMSL. The basement floor slab and subterranean retaining walls shall be designed to resist hydrostatic uplift and lateral hydrostatic pressures commensurate with this elevation. The recommended uplift pressure below the basement floor slab or mat foundation to be used in design would be equivalent to $62.4(H)$ pounds per square foot, where H is the depth of the bottom of the floor slab (or mat) measured below the historic highest groundwater level.

The proposed finish floor elevation (305 feet for parking level P2 and 295 feet, 8 inches for level P3). The P3 level will require a mat foundation to resist the 12 feet of hydrostatic uplift head caused by water at the historically highest groundwater level. The P2 level may also be constructed with a mat slab to resist the 3 feet of hydrostatic uplift head. A thickened conventional slab and conventional footings may also be used if they can be designed to resist the hydrostatic pressure.

Due to the differing geologic materials that will be exposed as the subgrade elevations, A uniform compacted fill blanket should be constructed to a thickness of at least 3 feet below the proposed foundations or mat slab. Excavations are anticipated to extend to elevation 296 feet AMSL for the P2 level and 289 feet AMSL for the P3 level.

The anticipated excavations will the fill soil and will expose older alluvium and bedrock. The proposed structure should be supported on a compacted fill blanket that extends at least 3 feet below the proposed foundations. The older alluvium should be selectively used for its granular consistency.

Proposed subterranean retaining walls extending below the historically highest groundwater level shall be designed for an undrained condition with full hydrostatic lateral pressure. If wall drains are placed one foot above the historically highest groundwater elevation (309 feet), the lateral hydrostatic pressure against retaining walls may be eliminated above this elevation. If no drain is placed, then the walls shall be designed for the full hydrostatic pressure extending to the ground



surface. Subterranean walls to be constructed above the historically highest groundwater level may be designed for a drained condition, provided that a retaining wall subdrain is installed.

The excavation for the proposed subterranean level will require shoring to provide a stable working area due to the proposed depth of excavation. Excavations deeper than 22 feet below existing ground surface will likely encounter perched groundwater and require construction dewatering. Temporary dewatering in order to achieve a dry and stable excavation during construction will likely be necessary. The expected flow rates and pre-pumping time frames should be determined during a dewatering test program conducted by a qualified dewatering consultant. Once the temporary construction dewatering is discontinued, the perched groundwater along the bedrock surface will likely return to its current elevation.

It is anticipated that the exposed subgrade at the bottom of the excavation will likely exhibit high moisture content. Pumping (vertical deflection) of the wet soils at the bottom of the excavation may occur during operation of heavy equipment. Where this occurs, the subgrade soil shall be stabilized as recommended in the “Wet Soils” section of this report.

Since the proposed structure will extend from property line to property line, and the finish floor elevation is in close proximity to current groundwater elevations, on-site storm water infiltration is not considered to be feasible for the proposed development. Alternative means of storm water disposal will likely be required.

SEISMIC DESIGN CONSIDERATIONS

California Building Code Seismic Parameters

A seismic shear-wave velocity measurement was performed using active and passive-source surface wave techniques at the project site by GeoPentech, Inc. The result of the geophysical survey indicated a shear-wave velocity measurement (V_{s30}) of 371 m/s (equivalent to 1,217 ft/s) to a depth of 30 meter (or 100 feet) below the ground surface. This corresponds to a site



classification for seismic design of Site Class C characterized as “Very Dense Soil and Soft Rock”. The result of the Surface Wave Geophysical Survey, prepared by GeoPentech, is presented in the Appendix of this report.

The site classification and the site coordinates were input into the Structural Engineers Association of California SEAOC/OSHPD seismic utility program (<https://seismicmaps.org>) in order to calculate ground motion parameters for the site as indicated on the following table:

CALIFORNIA BUILDING CODE SEISMIC PARAMETERS	
California Building Code	2019
ASCE Design Standard	7-16
Risk Category	II
Site Class	C
Mapped Spectral Acceleration at Short Periods (S_S)	2.085g
Site Coefficient (F_a)	1.2
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	2.502g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.668g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.747g
Site Coefficient (F_v)	1.4
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	1.046g
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.697g

FILL SOILS

The maximum depth of fill encountered on the site was 8 feet. Due to the proposed finished floor elevation of the proposed development, it is anticipated that onsite fill will be removed during site excavation. Onsite fill soil generated during demolition and excavation should be removed from



the site or may be reused as certified compacted fill provided that the fill is granular and exhibits low expansion potential.

EXPANSIVE SOILS

The onsite geologic soils are in the very low to critically high expansion range. The Expansion Index ranged from 11 to 142 for boring samples and a bulk sample remolded to 90 percent of the laboratory maximum density from this investigation and previous site investigations. Additional surcharge pressure will be required for retaining walls as described later in this report.

WATER-SOLUBLE SULFATES

The water-soluble sulfate content of the onsite soils 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 soils with less than 0.1% and Type I cement may be utilized for concrete foundations in contact with the site soils.

HYDROCONSOLIDATION

Hydroconsolidation is a phenomenon wherein soils lose volume when they are saturated. This can result in settlement of structures bearing thereon. The hydroconsolidation potential of the site soils was considered in the provision of consolidation tests. No test samples indicated collapse upon saturation during consolidation testing. Collapse is defined as greater than 1% consolidation upon wetting. Based on the laboratory testing, it is the opinion of Geotechnologies, Inc. that the potential for damaging settlement due to hydrocollapse is anticipated to be insignificant.



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 soils should be removed from the areas to receive controlled fill. All existing fill soils and any disturbed geologic materials resulting from grading operations shall be completely removed and properly recompacted prior to foundation excavation.
- Any vegetation or associated root system located within the footprint of the proposed 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 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.

Recommended Overexcavation

The proposed building area shall be excavated to a minimum depth of 3 feet below the bottom surface of all foundations to create an engineered building pad for support of foundations. The recommended building pad shall consist of a homogenous blend of granular soils subjected to soil mixing as specified in the "Soil Mixing" section of this report. The resultant building fill pad should exhibit an Expansion Index of less than 40 in order to mitigate expansion potential should exposure to perched groundwater at the bedrock surface occur.

Soil Blending

The onsite geologic soils are in the very low to critically high expansion range. Where the site excavation result in a net export, the sandier or more granular soils should be segregated from the



stockpiled soils and the more clayey or expansive soils (that occur near the ground surface) should be exported.

For the recommended compacted fill blanket, the granular alluvial soil should be selectively stockpiled for use. The old alluvium may be blended with the sandstone bedrock where it occurs. Blending should occur at an approximate 50-50 ratio. It is recommended that the soil is well blended to achieve uniformity and to reduce the overall expansion index of the newly placed controlled fill. Samples of the segregated and/or blended soils should be tested by this office to determine the engineering properties prior to placement and compaction for foundation or slab support.

Compaction

The City of Los Angeles Department of Building and Safety requires a minimum comparative compaction of 95 percent of the laboratory maximum density where the soils to be utilized in the fill have less than 15 percent finer than 0.005 millimeters. The soils tested by this firm would require the 95 percent compaction requirement subsequent to the recommended soil mixing. Comparative compaction is defined, for purposes of these guidelines, as the ratio of the in-place density to the maximum density as determined by applicable ASTM testing.

All fill should be mechanically compacted in layers not more than 8 inches thick. All fill shall be compacted to at least 95 percent of the maximum laboratory density for the soils used. The maximum density shall be determined by the laboratory operated by Geotechnologies, Inc. using the test method described in the most recent revision of ASTM D1557.

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 95 percent compaction is obtained.

Acceptable Soils

The excavated onsite soils and bedrock 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 be relatively impermeable and result in a stable subgrade when compacted. Any required import soils should consist of geologic soils 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 95 percent of the laboratory maximum density. Utility trench backfill should be tested by representatives of this firm in accordance with the most recent revision of ASTM D1557.



Wet Soils

At the time of exploration, the soils which will be exposed at the bottom of the excavation were above optimum moisture content. It is anticipated that the excavated soils and bedrock to be placed as compacted fill, and the soils exposed at the bottom of excavated plane 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 $\frac{3}{4}$ -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 of 5 to 10 percent should be anticipated when excavating and recompacting the alluvial soil on the site to an average comparative compaction of 96 percent.



Bulking occurs when a volume of soil removed at one density is compacted to a lower density. A bulking factor of 5 percent should be anticipated when excavating and recompacting the sedimentary bedrock.

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

Geotechnical Observations and Testing During Grading

Geotechnical observations and testing during grading are considered to be a continuation of the geotechnical investigation. It is critical that the geotechnical aspects of the project be reviewed by representatives of Geotechnologies, Inc. during the construction process. Compliance with the



design concepts, specifications or recommendations during construction requires review by this firm during the course of construction. Any fill which is placed should be observed, tested, and verified if used for engineered purposes. Please advise this office at least twenty-four hours prior to any required site visit.

LEED Considerations

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

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

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

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



FOUNDATION DESIGN

Due to the combination of bedrock and older alluvial soil anticipated at the excavation subgrade, the subgrade shall be over-excavated a minimum of three feet below the bottom surface of the proposed foundations in order to create an engineered building pad for uniform foundation settlement. The recommended building pad shall consist of a homogenous blend of granular soils subjected to blending as described in the Soil Blending section of this report. The resultant building fill pad should exhibit an Expansion Index of less than 40 in order to mitigate expansion potential should exposure to perched groundwater at the bedrock surface occur.

Foundations for the proposed development may consist of a mat foundation, conventional foundations or combination thereof. Mat foundations and conventional foundations shall bear on a compacted fill pad.

Mat Foundation

Should the proposed structure require significant hydrostatic uplift design, a mat foundation may be used to resist buoyancy pressure. The mat should be supported on a building pad consisting of uniform recompacted fill extending a minimum of three feet below the bottom surface of the mat foundation. Certified recompacted fill soil utilized for foundation support shall be granular in consistency and exhibit a very low to low expansion potential. Mat foundations for the proposed structure should bear in homogenous and thoroughly-mixed recompacted fill soils to allow uniform settlement.

For design purposes, an allowable average bearing pressure of 3,000 pounds per square foot, with locally higher pressures up to 6,000 pounds per square foot may be utilized in the mat foundation design which underlies the entire structure. The mat foundation may be designed utilizing a



modulus of subgrade reaction of 300 pounds per cubic inch. This value should be multiplied by a reduction factor of:

$$K_R = K \left[\frac{B + 1}{2B} \right]^2$$

Where:

K = Unit subgrade Modulus
K_R = Reduced Subgrade Modulus
B = Equivalent Foundation width

Lateral Design for Mat Foundation

Resistance to lateral loading may be provided by soil friction, and by the passive resistance of the soils. A coefficient of friction of 0.35 may be used with the dead load forces between the mat foundation and the underlying supporting soils.

Passive earth pressure for the sides of a mat foundation poured against undisturbed alluvial soil or certified recompacted fill may be computed as an equivalent fluid having a density of 210 pounds per cubic foot, with a maximum earth pressure of 2,100 pounds per square foot. When combining passive and friction for lateral resistance, the passive component should be reduced by one third. A one-third increase in the passive value may be used for wind or seismic loads. A minimum safety factor of 2 has been utilized in determining the allowable passive pressure.

Mat Foundation Settlement

The majority of the foundation settlement is expected to occur on initial application of loading. The maximum settlement is not expected to exceed approximately 2.6 inches, and will occur below the center (or most heavily loaded area) of the mat foundation. Differential settlement is not expected to exceed 1/2 inch.



Conventional Foundations

The proposed structure may be supported on conventional foundations provided that significant hydrostatic uplift pressures will not develop during historically highest groundwater conditions. Conventional foundations shall bear in an engineered building pad consisting of uniform recompacted fill extending a minimum of three feet below the bottom surface of the conventional foundations. Certified recompacted fill soil utilized for foundation support shall be granular in consistency and exhibit a very low to low expansion potential. All conventional foundations for a structure should bear in homogeneous and thoroughly-mixed recompacted soil to allow uniform settlement.

Continuous foundations may be designed for a bearing value of 3,000 pounds per square foot, and should be a minimum of 12 inches in width, 24 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing soils.

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

The bearing value increase for each additional foot of width is 190 pounds per square foot. The bearing value increase for each additional foot of depth is 600 pounds per square foot. The maximum recommended bearing value is 6,000 pounds per square foot.

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.



Conventional Foundation Lateral Design

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

Passive earth pressure for the sides of conventional foundations poured against undisturbed alluvial soils or certified recompacted fill may be computed as an equivalent fluid having a density of 210 pounds per cubic foot with a maximum earth pressure of 2,100 pounds per square foot.

When combining passive and friction for lateral resistance, the passive component should be reduced by one third. A one-third increase in the passive value may be used for wind or seismic loads.

Conventional Foundation Settlement

Settlement of conventional foundations is expected to occur on initial application of loading. The maximum settlement is expected to be 1 inch and occur below the heaviest loaded columns. Differential settlement is not expected to exceed ½ inch.

Foundation Reinforcement

All 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.

Miscellaneous Foundations

Conventional foundations for structures such as privacy walls or trash enclosures which will not be rigidly connected to the proposed structure may bear in native alluvial soils. Continuous footings may be designed for a bearing value of 1,500 pounds per square foot, and should be a



minimum of 12 inches in width, 24 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing soils. No bearing value increases are recommended.

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.

Foundation Observations

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

Due to the depth of the proposed finish floor elevation which extends below the historically highest groundwater level, the retaining wall loads are dependent on the placement of a backdrain. If no drains are installed, the structure must be designed to accommodate the full hydrostatic pressure extending to the ground surface. As an alternative, a drain may be placed just above the historically highest ground water level (at 308 AMSL), greatly reducing the lateral hydrostatic pressure.

Restrained Retaining Walls

Restrained subterranean retaining walls up to 35 feet in height and supporting a level back slope may be designed to resist a triangular distribution of earth pressure. Wall pressures are provided



in the following table for hydrostatic design. Pressures for drained conditions are also provided for designs that incorporate a subdrain above the historic high-water level.

RESTRAINED BASEMENT WALLS (HYDROSTATIC DESIGN)		
Height of Wall (Feet)	AT-REST EARTH PRESSURE Triangular Distribution of Pressure	ACTIVE EARTH PRESSURE* Triangular Distribution of Pressure
Up to 25	95.6 pcf [33.2 pcf (buoyant) + 62.4 pcf (hydrostatic)]	77.1 pcf* [14.7 pcf (buoyant) + 62.4 pcf (hydrostatic)]
25 to 35	95.6 pcf [33.2 pcf (buoyant) + 62.4 pcf (hydrostatic)]	81.2 pcf* [18.8 pcf (buoyant) + 62.4 pcf (hydrostatic)]

* To be combined with Dynamic Seismic Earth Pressure Active Hydrostatic Pressure

If a subdrain just above the historically highest groundwater level is installed, the portion of wall located above the historically highest groundwater levels may be designed based on the following table:

RESTRAINED BASEMENT WALLS ABOVE THE HISTORICALLY HIGHEST GROUNDWATER LEVEL (DRAINED CONDITIONS)**		
Height of Wall (Feet)	AT-REST EARTH PRESSURE Triangular Distribution of Pressure	ACTIVE EARTH PRESSURE* Triangular Distribution of Pressure
Up to 20 feet	63.7 pcf	37.9 pcf*

* To be combined with Dynamic Seismic Earth Pressure Active Hydrostatic Pressure

**Where drained retaining wall pressures are utilized in the design, a subdrain system must be installed so that external water pressures cannot develop behind the walls.



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.

Retaining Wall Drainage

If retaining walls are not designed for hydrostatic pressures, a subdrain shall be provided at the wall base. Retaining wall subdrains may consist of 4-inch diameter perforated pipes, placed with perforated pipe surface facing down. The pipe shall be encased in at least one-foot cross-sectional area of gravel around the pipe. The gravel shall be wrapped in non-biodegradable filter fabric. The gravel may consist of three-quarter inch to one-inch crushed rocks.

As an alternative, the use of gravel pockets and weepholes is an acceptable drainage method for outdoor retaining walls. Weepholes shall be a minimum of 2 inches in diameter, placed at 8 feet on center along the base of the wall. Gravel pockets shall be a minimum of one cubic foot in dimension, and may consist of three-quarter inch to one-inch crushed rocks, wrapped in non-biodegradable filter fabric.

If clearance is not available behind the proposed retaining wall due to property line restrictions, omission of one-half of a block at the back of the wall on eight-foot centers is an acceptable method of draining walls. The resulting void should be filled with gravel. A collector is placed within the gravel which directs collected water through the wall weepholes or drainage system. This method should be approved by the retaining wall designer prior to implementation.



Where retaining walls are to be constructed adjacent to property lines there is usually not enough space for emplacement of a standard pipe and gravel drainage system. Under these circumstances, the use of a flat drainage produce is acceptable. Some municipalities do not allow the use of flat-drainage products. The use of such a product should be researched with the building official. Certain types of subdrain pipe are not acceptable to the various municipal agencies, it is recommended that prior to purchasing subdrainage pipe, the type and brand is cleared with the proper municipal agencies. Subdrainage pipes should outlet to an acceptable location.

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

Note: Interior retaining walls (i.e., drive ramps) will require provisions for wall drainage similar to requirements of exterior retaining walls.

Sump Pump Design

The purpose of the recommended retaining wall backdrainage system is to relieve hydrostatic pressure. Perched groundwater was encountered during exploration at a depth of 21 feet (2005). Static groundwater was observed at 35 feet (2016) and 42 feet below existing site grade (2021). The historically highest groundwater level is estimated at 20 feet below the ground surface.

Based on these considerations a retaining wall backdrainage system placed below the historically highest groundwater level is expected to experience an appreciable flow of water. Should a drainage system be placed below the historically highest groundwater level, it is the recommendation of this firm that a dewatering contractor be added to the design team to determine the flow rate and the optimal design of such a system.



For retaining walls drainage systems extending less than 20 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.

Dynamic (Seismic) Earth Pressure

The maximum dynamic active pressure is equal to the sum of the initial static pressure and the dynamic (seismic) pressure increment. Under the most recent building code, as interpreted by most building departments, seismic earth pressure is required in the design of restraining walls which support over 6 feet of soil.

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

Wall Surcharge due to Expansive Soils

The onsite soils were found to be critically expansive near the ground surface. It is the recommendation of this firm that a 15 psf surcharge be applied to the upper 10 feet of the retaining walls as a result of the expansive property of the onsite soils.

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: $R = (0.3 \cdot P \cdot h^2) / (x^2 + h^2)$

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

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 Backfill

Any required backfill should be mechanically compacted in layers not more than 8 inches thick, to at least 95 percent of the maximum density obtainable by the most recent revision of ASTM D1557 method of compaction. Flooding should not be permitted. Compaction within 5 feet, measured horizontally, behind a retaining structure should be achieved by use of light weight, hand operated compaction equipment.

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 soils are acceptable for use as retaining wall backfill as long as they are compacted to a minimum of 95 percent of the maximum density as determined by ASTM D1557.

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

TEMPORARY EXCAVATIONS

Excavations on the order of 40 feet in vertical height will be required for the subterranean levels, foundations elements, elevator pit enclosures and placement of the engineered fill pad. The excavations are expected to expose fill, natural alluvial soils and bedrock, which are suitable for



vertical excavations up to 5 feet where not surcharged by adjacent traffic or structures. Excavations which will be surcharged by adjacent traffic or structures should be shored.

Where sufficient space is available, temporary unsurcharged embankments may be cut at a uniform $\frac{1}{2}$ to 1 (horizontal to vertical) slope gradient to a height of 18 feet. Higher cuts must be cut back at a 1 to 1 inclination. 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.

Where bedrock is exposed, the cut may not be made steeper than the orientation of bedding.

Temporary Dewatering

Currently it is anticipated that the structure will extend to an estimated depth of 30 to 40 feet below existing site grades including foundation elements and potential elevator pit enclosures. Existing perched groundwater is expected to be encountered as shallow as 21 feet, static groundwater is anticipated at 42 feet below ground surface. Temporary dewatering consisting of French drains will likely be required within the site to mitigate potential intrusion of perched groundwater and to ensure a stable subgrade during the excavation and basement construction phase of development. The expected flow rates and pre-pumping time frames should be determined during a dewatering test program conducted by a qualified dewatering consultant. Once the temporary



construction dewatering is discontinued, perched groundwater levels will likely return to their pre-construction levels.

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 laterally braced utilizing drilled tied-back anchors or raker braces.

Soldier Piles

Drilled cast-in-place soldier piles should be placed no closer than 2 diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the geologic materials. For design purposes, an allowable passive value for the geologic materials below the bottom plane of excavation may be assumed to be 600 pounds per square foot per foot 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.

Perched groundwater was observed at depths ranging from 21 to 39 feet. Static groundwater was encountered during exploration at a depth of 42 feet below grade. 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 should caving be experienced in the granular (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 550 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.

Tied-Back Anchors

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

Drilled friction anchors may be designed for a skin friction of 450 pounds per square foot. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. This skin friction is based on 15-foot-high shoring, a tied back anchor elevation 6 feet below grade and a minimum twenty-foot embedment beyond the potentially active wedge yielding an overburden of 12½ feet below ground surface. Where belled anchors are utilized, the capacity of



belled anchors may be designed by applying the skin friction over the surface area of the bonded anchor shaft. The diameter of the bell may be utilized as the diameter of the bonded anchor shaft when determining the surface area. This implies that in order for the belled anchor to fail, the entire parallel soil column must also fail.

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

Anchors should be placed at least 6 feet on center to be considered isolated. It is recommended that at least 5 percent of the initial anchors have their capacities tested to 200 percent of their design capacities for a 24-hour period to verify their design capacity.

The total deflection during this test should not exceed 12 inches. The anchor deflection should not exceed 0.75 inches during the 24-hour period, measured after the 200 percent load has been applied. All anchors should be tested to at least 150 percent of design load. The total deflection during this test should not exceed 12 inches.

The rate of creep under the 150 percent test load should not exceed 0.1 inch over a 15-minute period in order for the anchor to be approved for the design loading. After a satisfactory test, each anchor should be locked-off at the design load. This should be verified by rechecking the load in the anchor. The load should be within 10 percent of the design load. Where satisfactory tests are not attained, the anchor diameter and/or length should be increased or additional anchors installed until satisfactory test results are obtained. The installation and testing of the anchors should be observed by the geotechnical engineer. Minor caving during drilling of the anchors should be anticipated.

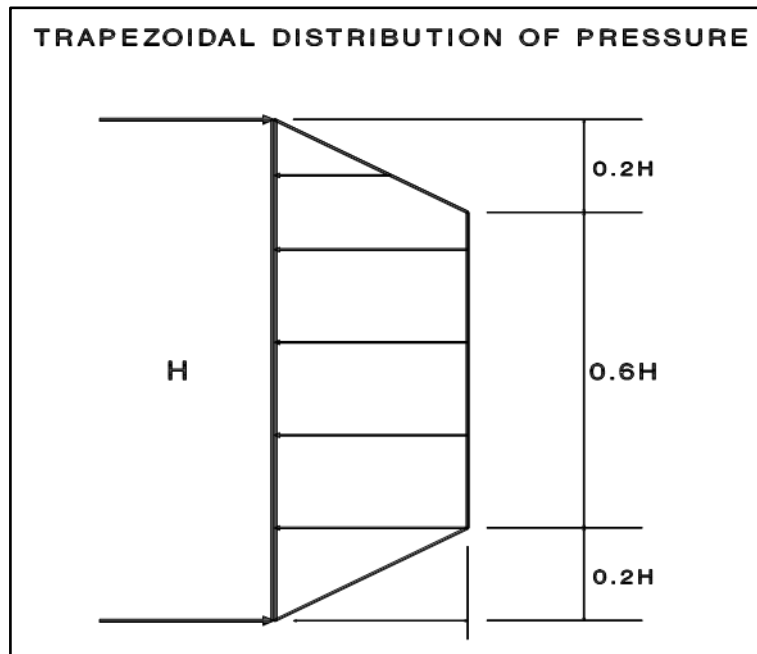


Anchor Installation

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

Lateral Pressures

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. It is assumed that the excavation will be drained for these pressures to be valid:

HEIGHT OF SHORING “H” (feet)	DESIGN SHORING FOR (Where H is the height of the wall)
Up to 30	23H
30 to 35	24H
35 to 40	25H
40 to 45	26H

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.

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. Inclined raker foundations will require separate calculations, as needed. 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



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

The City of Los Angeles Department of Building and Safety requires limiting shoring deflection to ½ inch at the top of the shored embankment where a structure is within a 1:1 plane projected up from the base of the excavation. A maximum deflection of 1-inch has been allowed provided there are no structures within a 1:1 plane drawn upward from the base of the excavation.

Monitoring

Because of the depth of the excavation, some 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 not subject to traffic loading or hydrostatic uplift pressures may be a minimum of 5 inches in thickness. Slabs-on-grade should be cast over properly controlled fill soils. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 95 percent of the maximum dry density.

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

Hydrostatic Uplift

The historically highest groundwater level is estimated at 20 feet below existing ground surface. For basement floor slabs extending deeper than 20 feet below ground surface, the proposed basement floor slab shall be designed to resist hydrostatic uplift pressure. The recommended hydrostatic uplift pressure below the basement floor slab to be used in design is equivalent to $62.4(H)$ pounds per square foot, where H is the depth of the bottom of the floor slab measured below the historically highest groundwater level.



Design of Slabs That Receive Moisture-Sensitive Floor Coverings

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.

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

Complete removal of the existing fill soils beneath outdoor flatwork 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 recompact to a minimum 95 percent relative compaction.



Slab Reinforcing

Concrete slabs-on-grade within the building should be reinforced with a minimum of #4 steel bars on 16-inch centers each way. Building slabs subject to hydrostatic uplift loading will require structural design.

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 removed and recompact to a minimum depth of 12 inches, moistened as required to achieve within 3 percent above optimum moisture content, and recompact 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 asphalt paving sections shown in the following table are based on an estimated R-Value of 30. The estimated R-Value is based on a subgrade which has been improved by soil mixing in accordance with the recommendations of this report. Recommended paving sections are indicated in the following table:

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 8 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended. The crack control joints should be installed as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. Concrete paving should be reinforced with a minimum of #4 steel bars on 16-inch centers each way. 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.

STORMWATER DISPOSAL

Due to the property line to property line extent of the proposed structure and the depth of the finished floor elevation below the current and historical water levels, installation of an on-site stormwater disposal system is not recommended for the proposed development. Some other means of stormwater disposal will be required.

Where percolation 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.

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 soils, especially when they were placed without benefit of modern grading codes, regularly contain materials which could impede efficient grading and drilling. The contractor should be familiar with the site and the geologic materials in the vicinity.

CLOSURE AND LIMITATIONS

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

The 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 is 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. Samples from bucket-auger drilling are obtained utilizing a California Modified Sampler with successive 12-inch drops of a kelly bar, whose weight is noted on the excavation logs. The soil is retained in brass rings of 2.50 inches outside diameter and 1.00 inch in height. The central portion of the samples are stored in close fitting, waterproof containers for transportation to the laboratory. Samples noted on the excavation logs as SPT samples are obtained in accordance with the most recent revision of ASTM D1586. Samples are retained for 30 days after the date of the geotechnical report.



Grain Size Distribution

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

The most recent revision of ASTM D422 is used to determine particle sizes smaller than the Number 200 sieve. A hydrometer is used to determine the distribution of particle sizes by a sedimentation process. The grain size distributions are plotted on the E-Plate presented in the Appendix of this report.

Moisture and Density Relationships

The field moisture content and dry unit weight are determined for each of the undisturbed soil samples, and the moisture content is determined for SPT samples by the most recent revision of ASTM 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.



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



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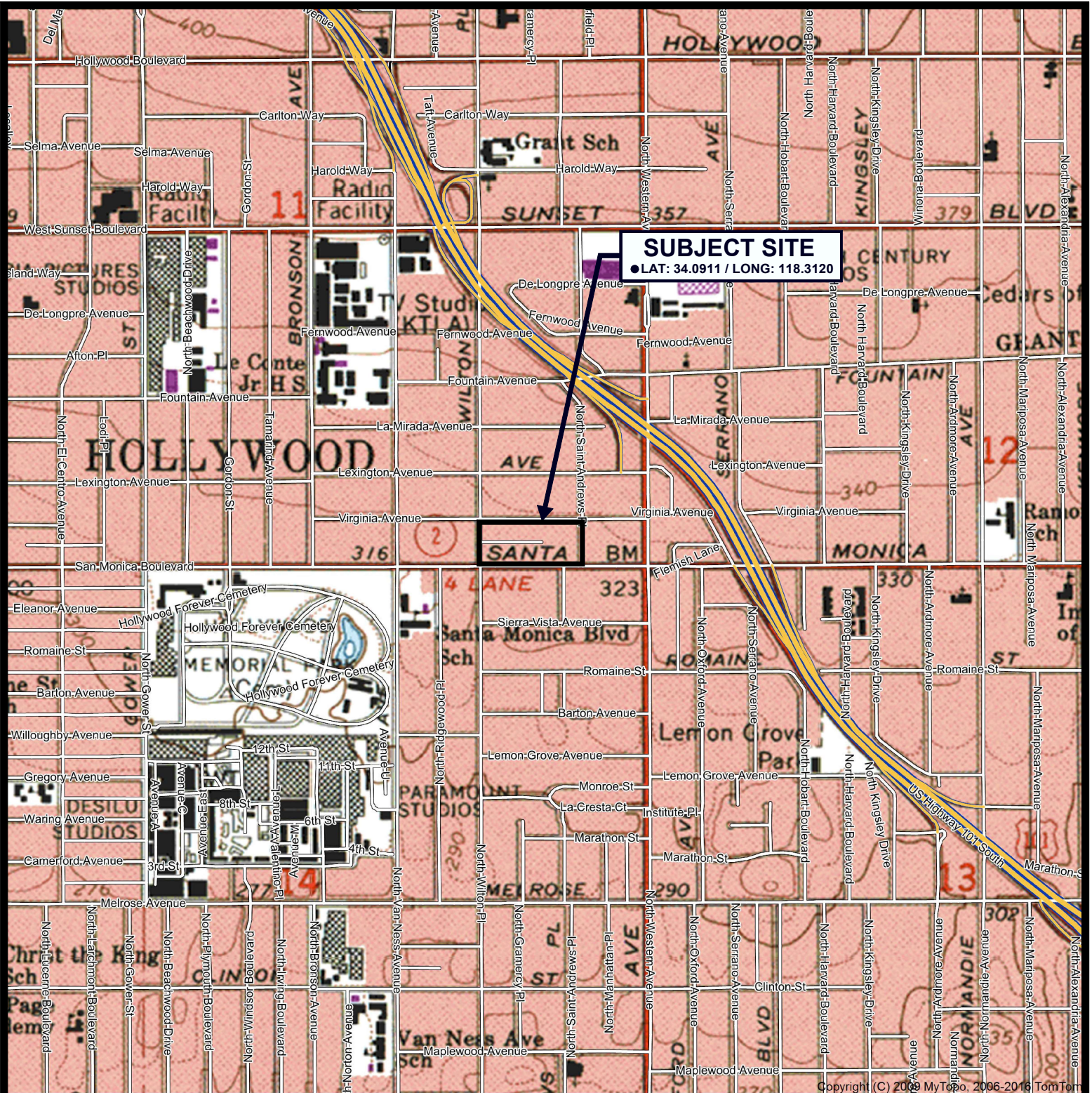
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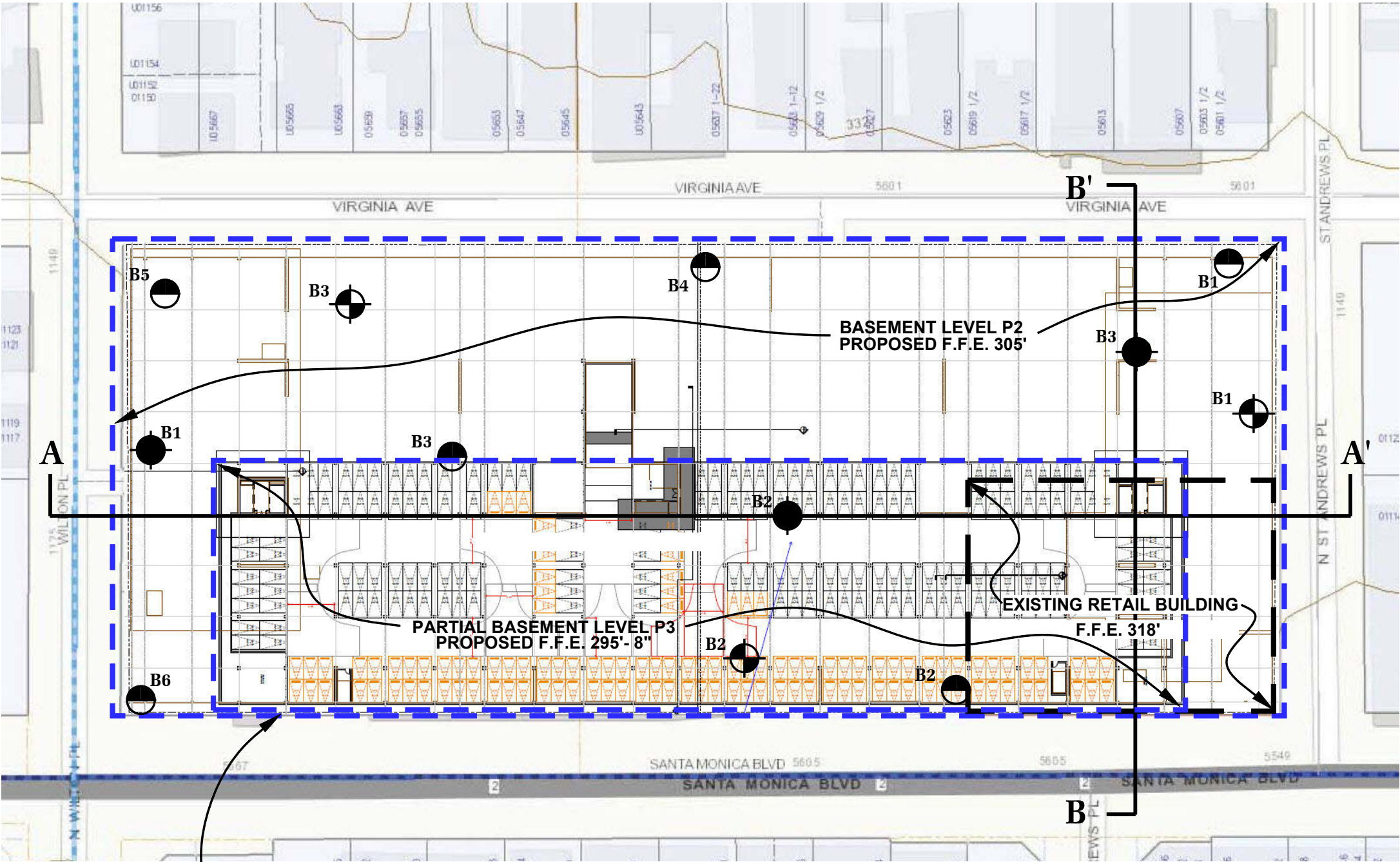


VICINITY MAP

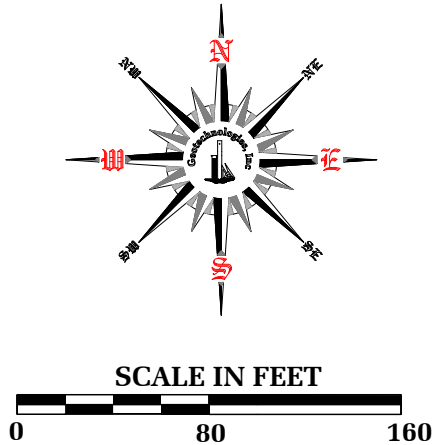
Geotechnologies, Inc.
 Consulting Geotechnical Engineers

BARDAS INVESTMENT GROUP
 5601 SANTA MONICA BLVD., LOS ANGELES

FILE NO. 22064



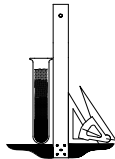
SUBJECT SITE

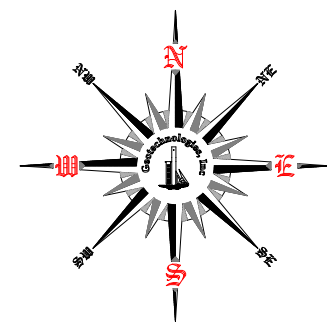
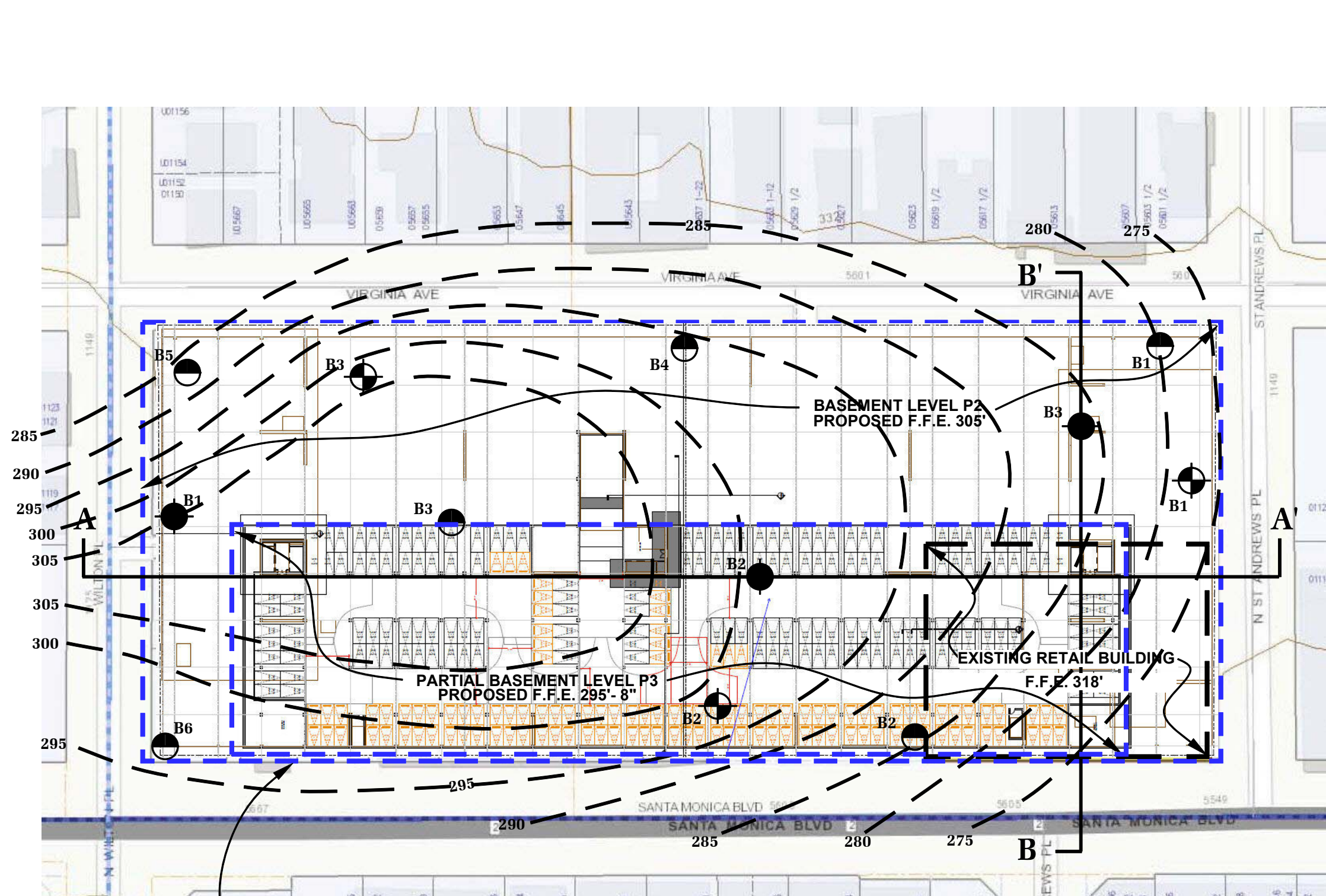


LEGEND

-  **B3** LOCATION & NUMBER OF BORING (THIS INVESTIGATION)
-  **B3** LOCATION & NUMBER OF BORING (PREVIOUS INVESTIGATION, FILE NO. 21151, DATED MARCH 2016)
-  **B6** LOCATION & NUMBER OF BORING (PREVIOUS INVESTIGATION BY GEOCON, APRIL 17, 2007)

REFERENCE: MAP BY THE CITY OF LOS ANGELES (<http://navigate.lacity.org/NavigateLA>) NOT DATED

 Geotechnologies, Inc. Consulting Geotechnical Engineers		PLOT PLAN	
		BARDAS INVESTMENT GROUP 5601 SANTA MONICA BLVD., LOS ANGELES	
		FILE No. 22064	DRAWN BY: TC
		DATE: May 2021	

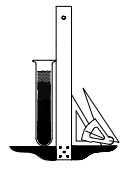


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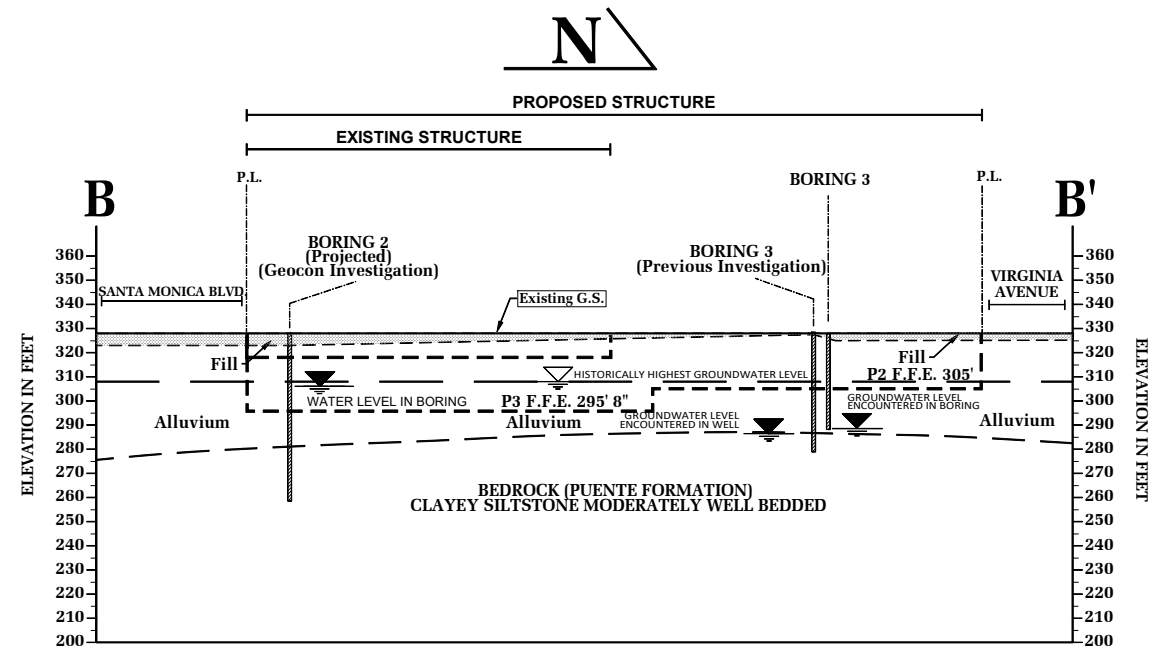
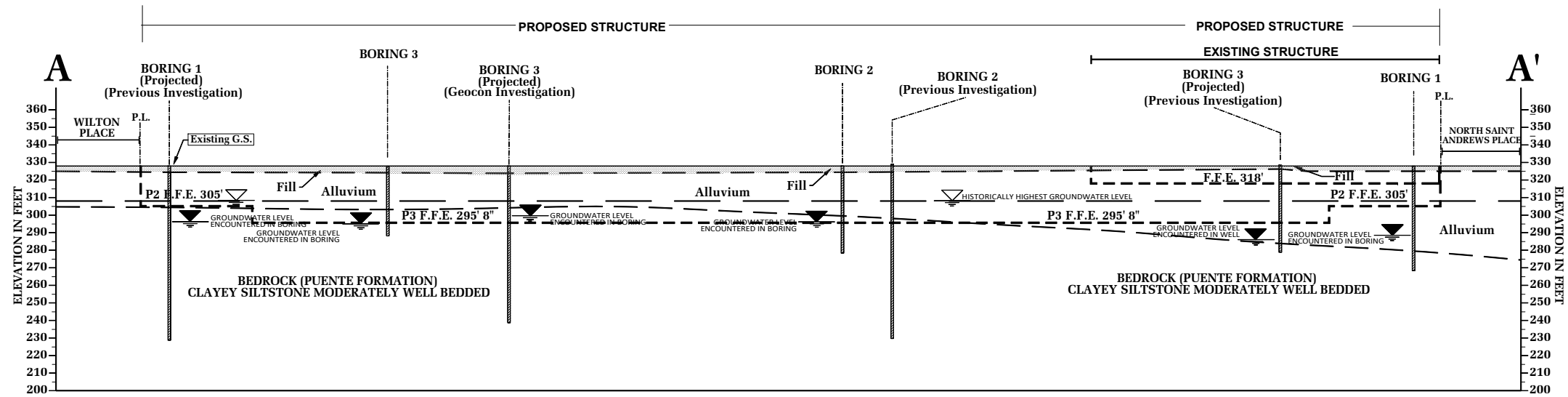
- B3** LOCATION & NUMBER OF BORING (THIS INVESTIGATION)
- B3** LOCATION & NUMBER OF BORING (PREVIOUS INVESTIGATION, FILE NO. 21151, DATED MARCH 2016)
- B6** LOCATION & NUMBER OF BORING (PREVIOUS INVESTIGATION BY GEOCON, APRIL 17, 2007)
- 275** ? LINE OF EQUAL ELEVATION OF BEDROCK QUERIED WHERE UNSURE

SUBJECT SITE

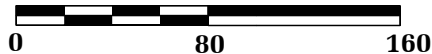
BEDROCK ELEVATION MAP

 Geotechnologies, Inc. Consulting Geotechnical Engineers	BARDAS INVESTMENT GROUP 5601 SANTA MONICA BLVD.	
	FILE No. 22064	DRAWN BY: TC/RTK
	DATE: June '21	SHEET: 1 of 1

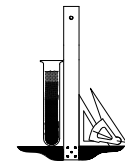
N90W



SCALE IN FEET



CROSS-SECTIONS A-A' and B-B'



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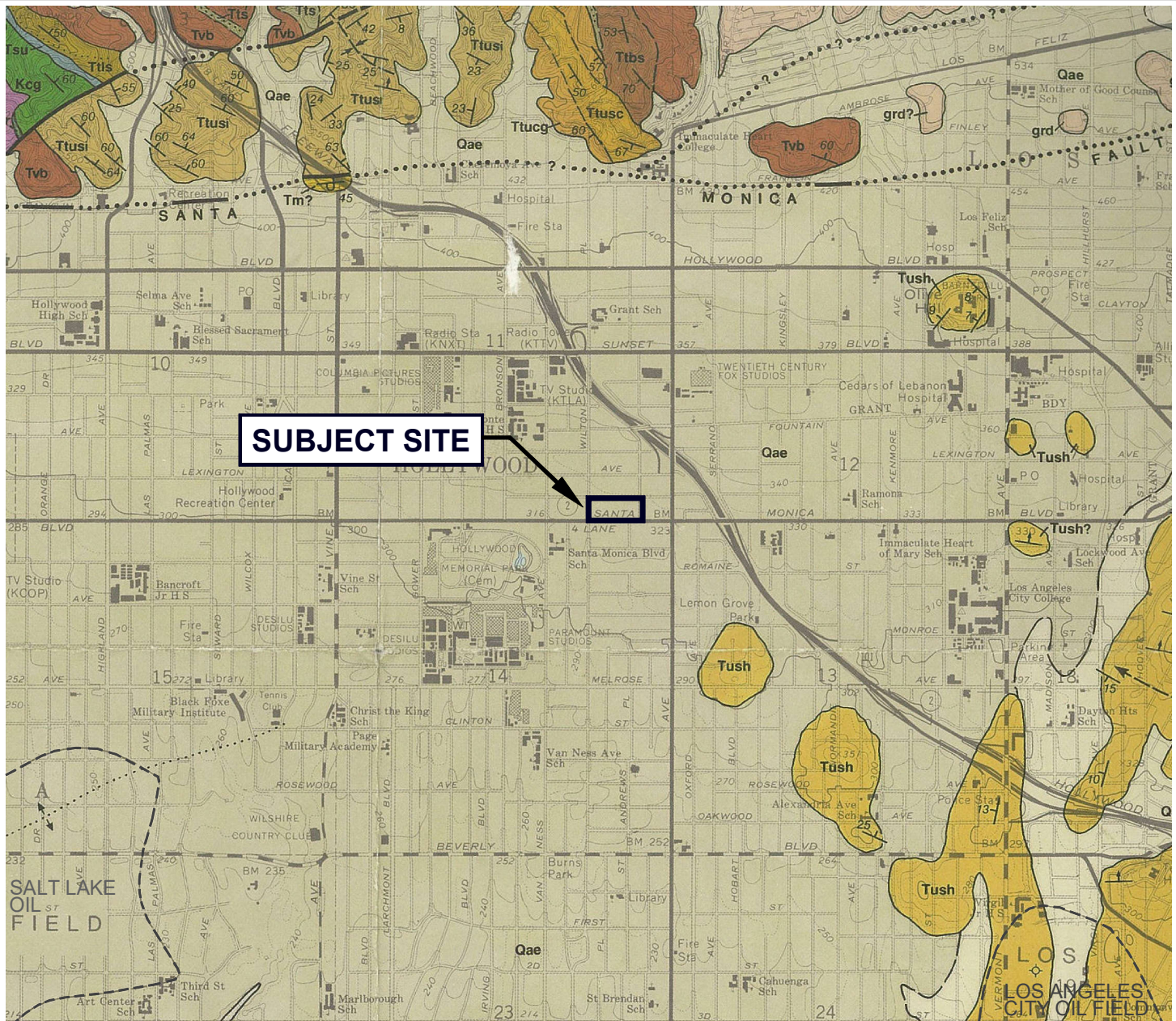
BARDAS INVESTMENT GROUP

5601 SANTA MONICA BLVD., LOS ANGELES

FILE No. 22064

DRAWN BY: TC

DATE: May 2021



SCALE IN FEET
0 1000 2000 4000

LEGEND

- Qae:** Older Surficial Sediments - Elevated and Dissected Alluvium consisting of clay, sand and gravel
- Tush:** Unnamed Shale - Gray to light brown, thin-bedded silty clay shale, soft and crumbly
- Tm:** Monterey Formation - White weathering, thin bedded, platy siliceous shale
- Ttusi:** Upper Topanga Formation - Mostly gray micaceous clay shale or claystone, crumbly where weathered
- Ttusc:** Upper Topanga Formation - Light gray massive sandstone, with pebble-cobble conglomerate
- Ttucg:** Upper Topanga Formation - Light to medium gray, from coarse sandstone to cobble conglomerate
-? 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 1/2) QUADRANGLES, MAP #DF-30

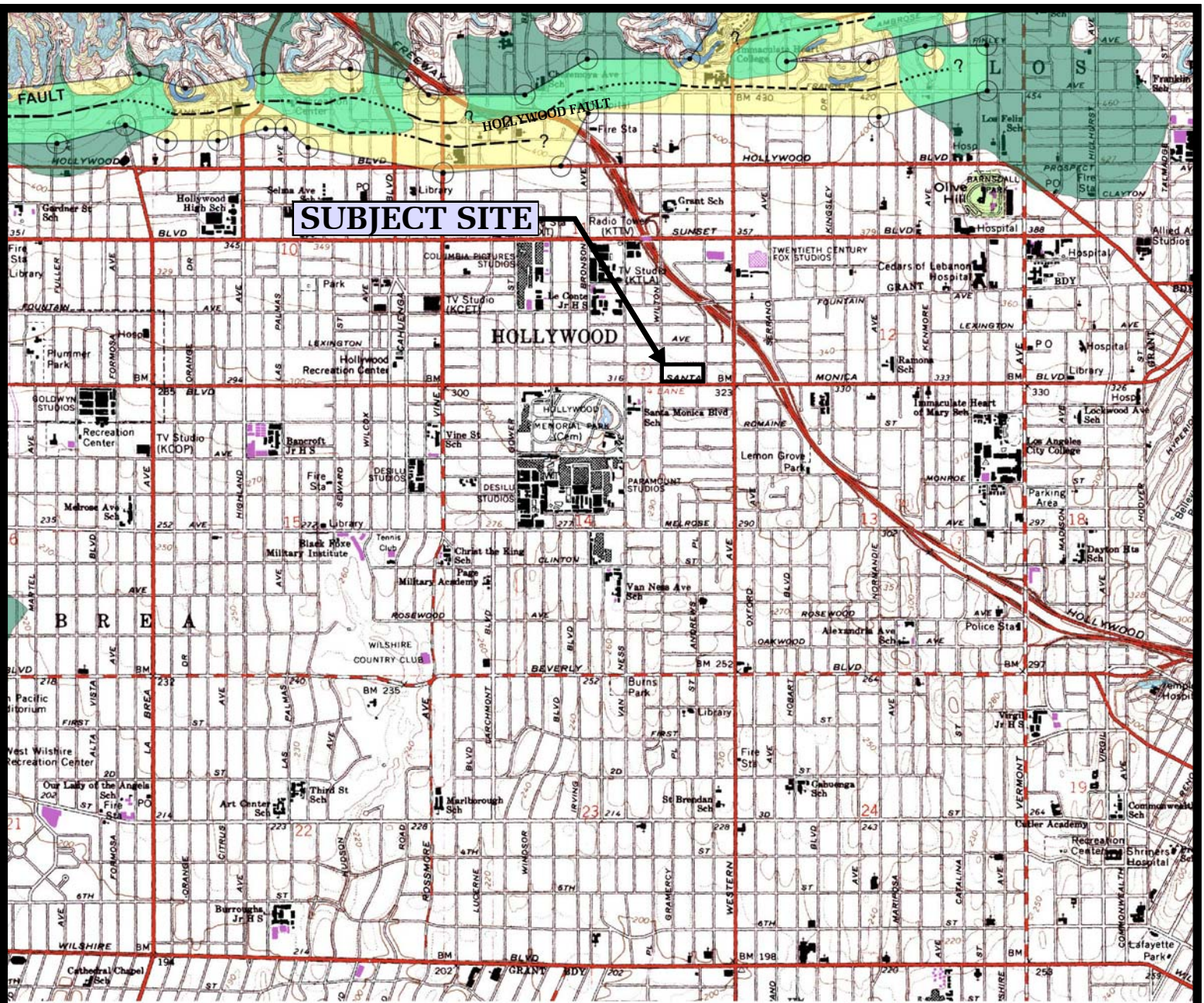


LOCAL GEOLOGIC MAP - DIBBLEE

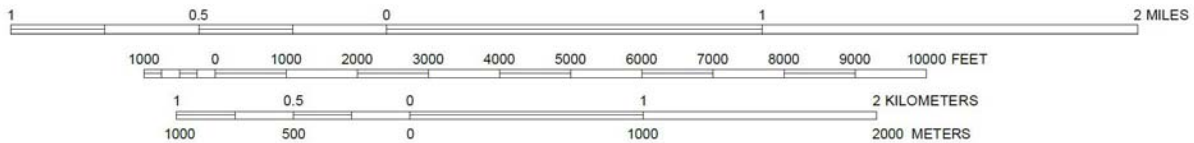
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FILE NO. 22064



Scale 1: 24000



Contour Interval 20 Feet

-  Earthquake Fault Zones
-  Alquist-Priolo Earthquake Fault Zone

 LIQUEFACTION AREA

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

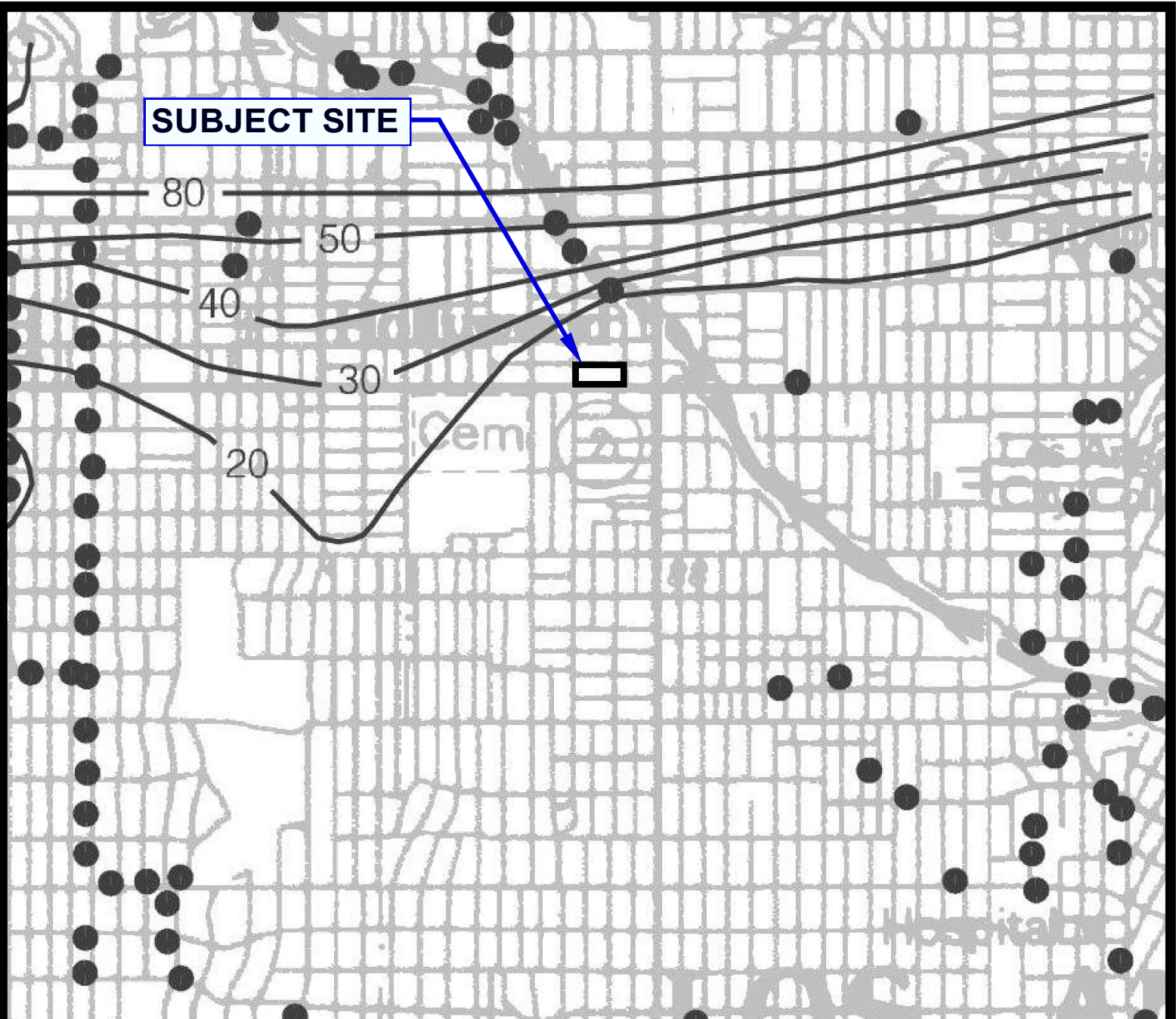
EARTHQUAKE ZONES OF REQUIRED INVESTIGATION

BARDAS INVESTMENT GROUP

5601 SANTA MONICA BLVD., LOS ANGELES

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FILE NO. 22064



ONE MILE

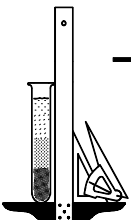
SCALE

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)

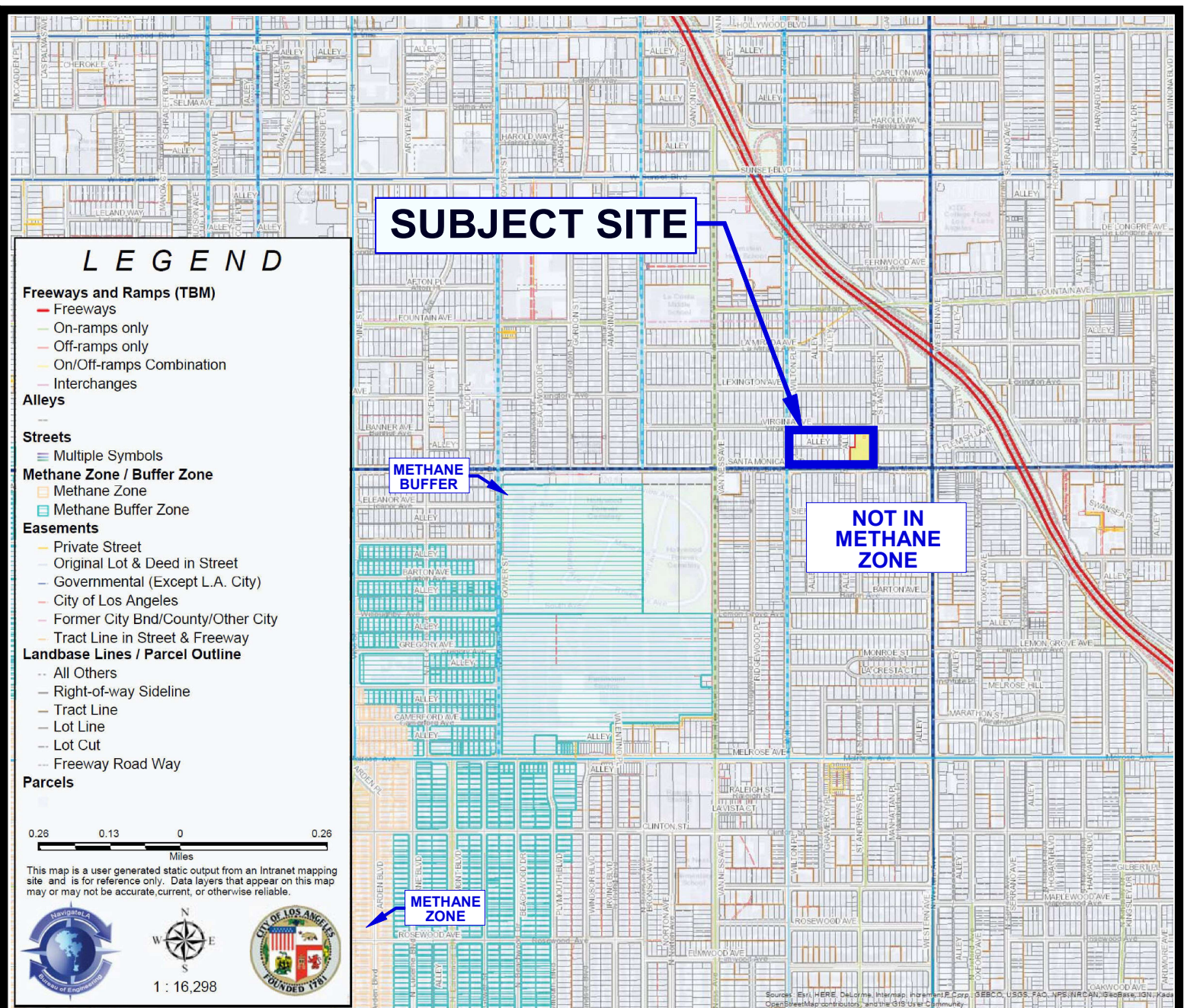
HISTORICALLY HIGHEST GROUNDWATER LEVELS



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Consulting Geotechnical Engineers

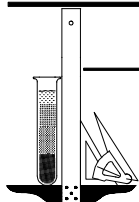
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FILE NO. 22064



REFERENCE: <http://navigate.lacity.org/NavigateLA/>

METHANE ZONE RISK MAP



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BARDAS INVESTMENT GROUP
5601 SANTA MONICA BLVD., LOS ANGELES

FILE NO. 22064

BORING LOG NUMBER 1

Bardas Investment Group

Date: 05/10/21

Elevation: 327'*

File No. 22064

Method: 8-inch diameter Hollow Stem Auger

dy

* Reference: City of Los Angeles NavigateLA Website

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
2.5	40	22.3	104.5	0 --		FILL: Silty Sand to Sandy Silt, dark brown, moist, medium dense, fine grained, stiff
				-		
				1 --		
				-		
				2 --		
5	19	20.0	SPT	-	ML/CL	OLDER ALLUVIUM: Clayey Silt to Silty Clay, dark brown, moist, stiff, fine grained
				3 --		
				-		
				4 --		
				-		
7.5	72	16.3	113.7	5 --		
				-		
				6 --		
				-		
				7 --		
10	9	13.7	SPT	-	SC/SM	Clayey Sand to Silty Sand, dark brown, moist, medium dense, fine grained
				8 --		
				-		
				9 --		
				-		
12.5	43	16.5	111.5	10 --	SM	Silty Sand, dark brown, moist, medium dense, fine grained
				-		
				11 --		
				-		
				12 --		
15	25	19.3	SPT	-	SM/ML	Silty Sand to Clayey Silt, dark brown, moist, stiff
				13 --		
				-		
				14 --		
				-		
17.5	73	16.8	110.0	15 --	ML	Sandy Silt, dark brown, moist, stiff
				-		
				16 --		
				-		
				17 --		
20	26	13.2	SPT	-	SM/ML	Silty Sand to Sandy Silt, dark brown, moist, medium dense, fine grained, stiff
				18 --		
				-		
				19 --		
				-		
22.5	45	17.0	105.0	20 --		
				-		
				21 --		
				-		
				22 --		
25	21	15.2	SPT	-	SM	Silty Sand, dark brown, moist, medium dense, fine grained
				23 --		
				-		
				24 --		
				-		
				25 --	SP/ML	Sand to Sandy Silt, dark and yellowish brown, moist, medium dense, fine grained, stiff
				-		

BORING LOG NUMBER 1

Bardas Investment Group

File No. 22064

dy

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
27.5	83	4.6	123.6	28 --	SP	Sand. dark and yellowish brown, moist, very dense, fine to medium grained, minor cobbles
				-		
				29 --		
				-		
30	29	15.2	SPT	30 --		dark brown, wet, medium dense
				-		
				31 --		
				-		
32.5	39 50/5"	19.2	109.8	32 --		
				-		
				33 --		
				-		
				34 --		
				-		
35	61	15.4	SPT	35 --		
				-		
				36 --		
				-		
37.5	78	23.2	104.0	37 --		
				-		
				38 --		very dense
				-		
				39 --		
				-		
40	69	18.1	SPT	40 --	SP/ML	Sand to Sandy Silt, dark brown, gray, wet to moist, dense, fine to medium grained, stiff
				-		
				41 --		
				-		
42.5	45 50/4"	22.1	100.0	42 --		
				-		
				43 --	SP	Sand, dark brown, wet, very dense, fine grained
				-		
				44 --		
				-		
45	30 50/5"	24.8	SPT	45 --	SP/SM	Sand to Silty Sand, dark and yellowish brown, moist to wet, very dense, fine grained
				-		
				46 --		
				-		
47.5	100/9"	27.4	96.3	47 --		
				-		
				48 --	SM	Silty Sand, dark and yellowish brown, moist, very dense, fine grained, very stiff
				-		
				49 --		
				-		
50	38 50/4"	27.2	SPT	50 --		
				-		
						BEDROCK (PUENTE FORMATION): Siltstone to Sandstone, dark and yellowish brown, moist to wet, moderately hard

BORING LOG NUMBER 1

Bardas Investment Group

File No. 22064

dy

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
52.5	44 50/3"	23.3	101.6	-		BEDROCK (PUENTE FORMATION): Siltstone to Sandstone, dark and yellowish brown, moist to wet, moderately hard
				51 --		
				-		
				52 --		
				-		
55	79	25.5	SPT	53 --		
				-		
				54 --		
				-		
				55 --		
57.5	90	30.2	94.8	-		Siltstone, dark brown to gray, moist, moderately hard
				56 --		
				-		
				57 --		
				-		
60	80	32.8	SPT	58 --		gray to dark gray
				-		
				59 --		
				-		
				60 --		
				-		Total Depth: 60 feet Water at 39 feet Fill to 3 feet NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual. Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted SPT=Standard Penetration Test
				61 --		
				-		
				62 --		
				-		
				63 --		
				-		
				64 --		
				-		
				65 --		
				-		
				66 --		
				-		
				67 --		
				-		
				68 --		
				-		
				69 --		
				-		
				70 --		
				-		
				71 --		
				-		
				72 --		
				-		
				73 --		
				-		
				74 --		
				-		
				75 --		
				-		

BORING LOG NUMBER 2

Bardas Investment Group

Date: 05/10/21

Elevation: 328'*

File No. 22064

Method: 8-inch diameter Hollow Stem Auger

dy

* Reference: City of Los Angeles NavigateLA Website

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				0 --		Surface Conditions: Asphalt for parking
				-		5.5-inch Asphalt over 3-inch Base
				1 --		FILL: Sandy Silt with Clay, dark brown, moist, stiff
				-		
2.5	25	16.6	110.8	2 --		
				-		
				3 --		
				-	ML/CL	OLDER ALLUVIUM: Clayey Silt to Silty Clay, dark brown, moist, stiff, fine grained
				4 --		
5	36	14.1	109.1	5 --		
				-	ML	Sandy to Clayey Silt, dark brown, moist, stiff
				6 --		
				7 --		
7.5	103	17.7	115.7	8 --		
				-	ML/CL	Clayey Silt to Silty Clay, dark brown, moist, stiff
				9 --		
				10 --		
10	78	16.9	114.6	11 --		
				-	SM	Silty Sand, dark brown, moist, dense, fine grained
				12 --		
				13 --		
				14 --		
				15 --		
15	88	15.6	118.0	16 --		
				-	SM/ML	Silty Sand, to Sandy Silt, dark brown, moist, dense, fine grained
				17 --		
				18 --		
				19 --		
				20 --		
20	36 50/5"	11.0	118.8	21 --		
				-	SM	Silty Sand, dark brown, moist, very dense, fine grained
				22 --		
				23 --		
				24 --		
				25 --		
25	92	30.1	99.3	-	SP	Sand to Sandy Silt, dark and yellowish brown, moist, very dense, fine to medium grained, very stiff

BORING LOG NUMBER 2

Bardas Investment Group

File No. 22064

dy

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
30	44 50/3"	28.2	93.5	-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
				30 --		
35	46 50/4"	30.2	92.5	-		BEDROCK (PUENTE FORMATION): Siltstone, yellow and grayish brown, moist, moderately hard
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
				35 --		
40	45 50/4"	34.3	86.0	-		
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
				40 --		
45	78	34.9	87.2	-		
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
				45 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual. Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
50	46 50/3"	29.9	86.6	-		
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
				50 --		
				-		Total Depth: 50 feet Water at 32 feet Fill to 3 feet

BORING LOG NUMBER 3

Bardas Investment Group

Date: 05/11/21

Elevation: 329.5'*

File No. 22064

Method: 8-inch diameter Hollow Stem Auger

dy

*** Reference: City of Los Angeles NavigateLA Website**

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				0 --		Surface Conditions: Asphalt for parking
				-		4-inch Asphalt over 4-inch Base
				1 --		FILL: Sandy Silt to Silty Clay, dark brown, moist, stiff
				-		
				2 --		
				-		
2.5	28	23.4	102.2	3 --		
				-		
				4 --	CL	OLDER ALLUVIUM: Silty Clay, dark brown, moist, stiff, fine grained
				-		
5	49	13.7	115.2	5 --		
				-		
				6 --		
				-		
7.5	68	17.5	114.1	7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	33 50/5"	11.4	120.6	10 --		
				-	SM	Silty Sand, dark brown, moist, very dense, fine grained
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	90	7.8	118.5	15 --		
				-	SM/SP	Silty Sand to Sand, dark and yellowish brown, moist, very dense, fine to medium grained
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	38 50/5"	6.8	113.2	20 --		
				-	SP	Sand, dark brown, moist, very dense, fine to medium grained
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	92	31.2	93.0	25 --		
				-		BEDROCK (Puente Formation): Claystone, dark brown, moist, moderately hard

BORING LOG NUMBER 3

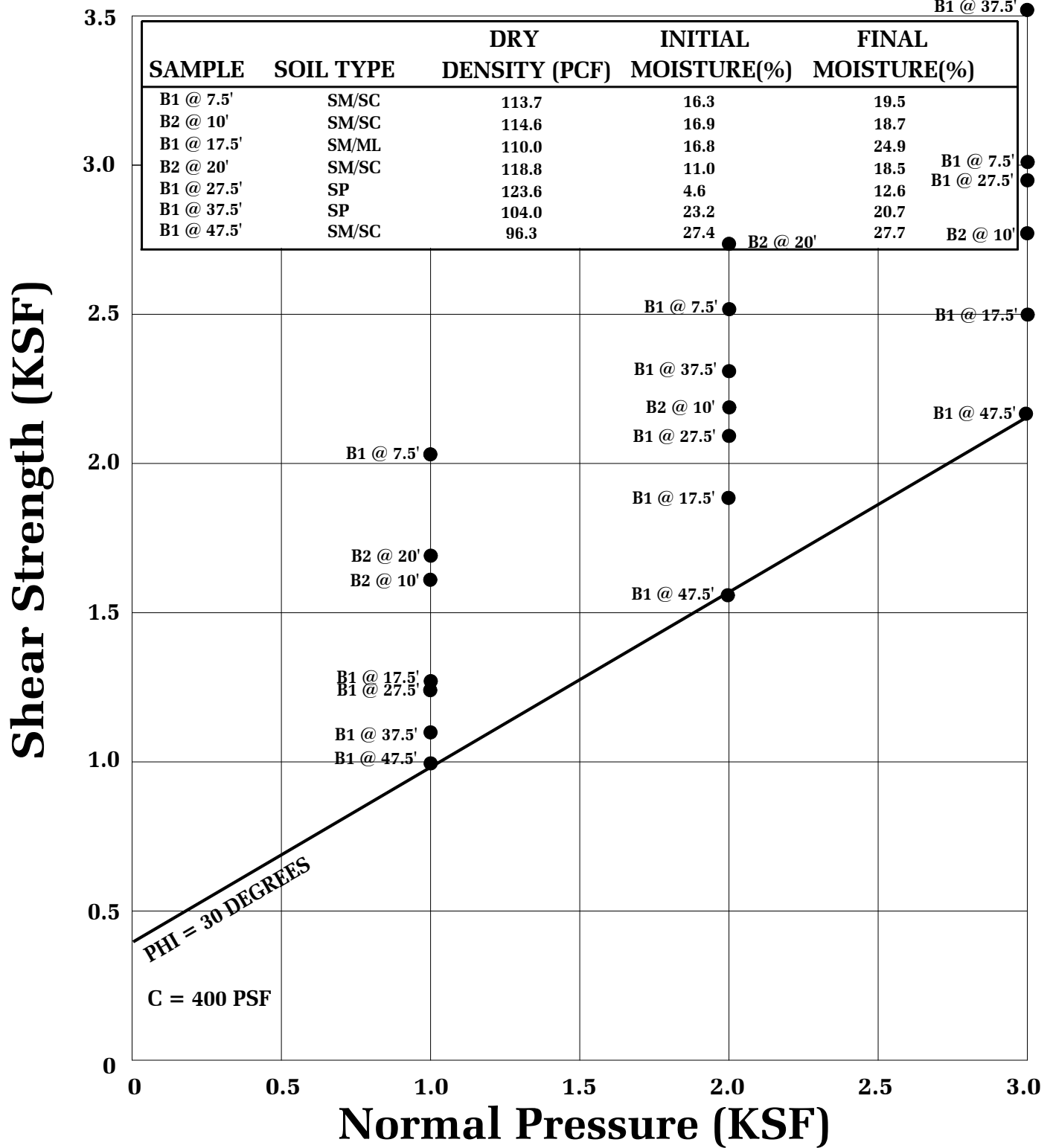
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File No. 22064

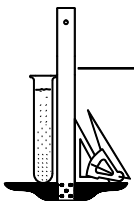
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Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
30	100/9"	23.5	99.5	-		BEDROCK (Puente Formation): Claystone, dark brown, moist, moderately hard
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
				30 --		
35	100/9"	34.3	85.9	-		Siltstone to Claystone, dark and yellowish brown, less weathered
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
				35 --		
40	100/8"	30.2	95.4	-		Siltstone to Sandstone
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
				40 --		
				-		Total Depth: 40 feet Water at 35 feet Fill to 3 feet NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual. Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
				45 --		
				-		
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
				50 --		
				-		
				-		

OLDER ALLUVIUM



SHEAR TEST DIAGRAM



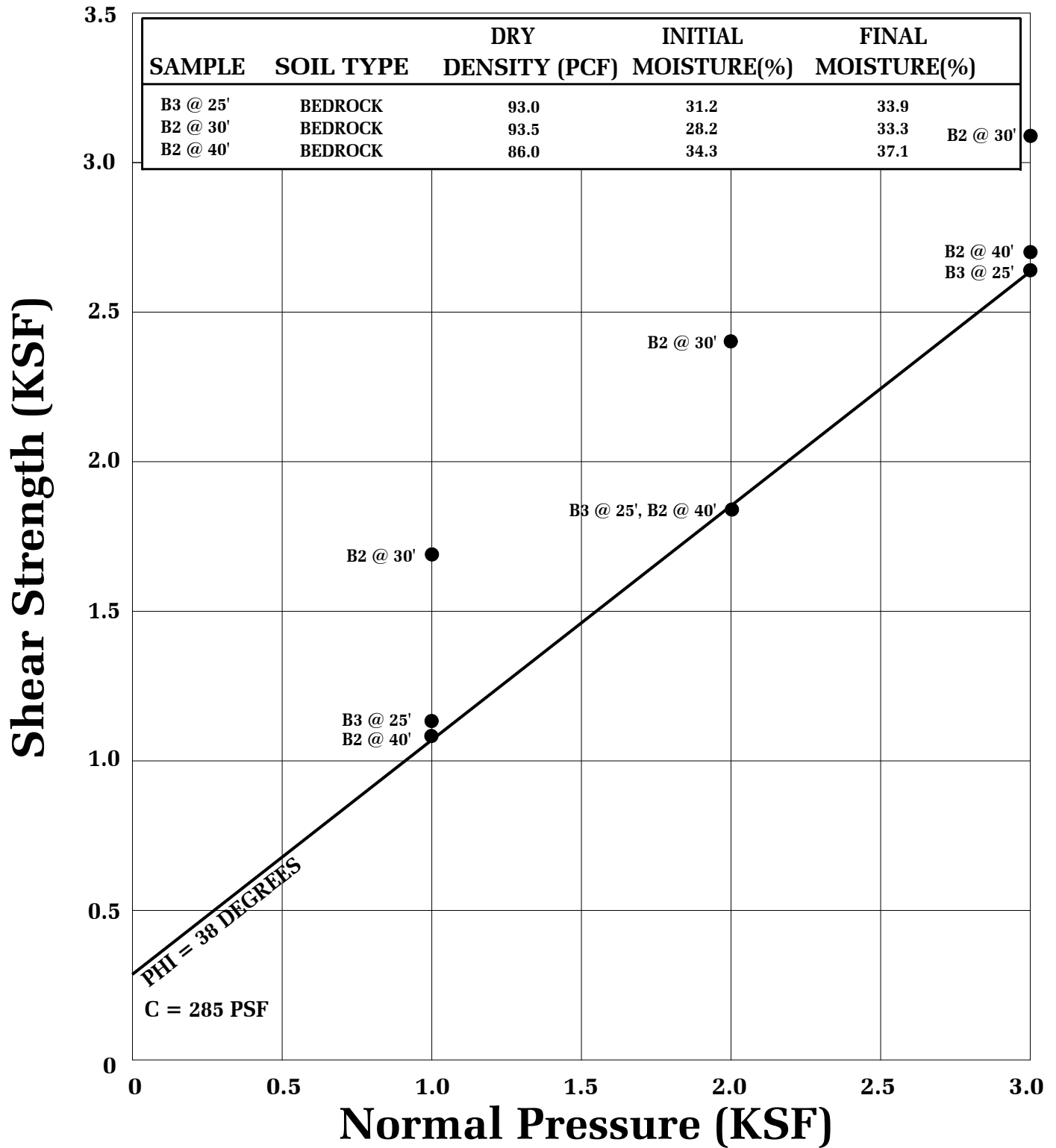
Geotechnologies, Inc.
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BARDAS INVESTMENT GROUP

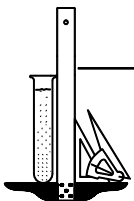
FILE NO. 22064

PLATE: B-1

BEDROCK



SHEAR TEST DIAGRAM



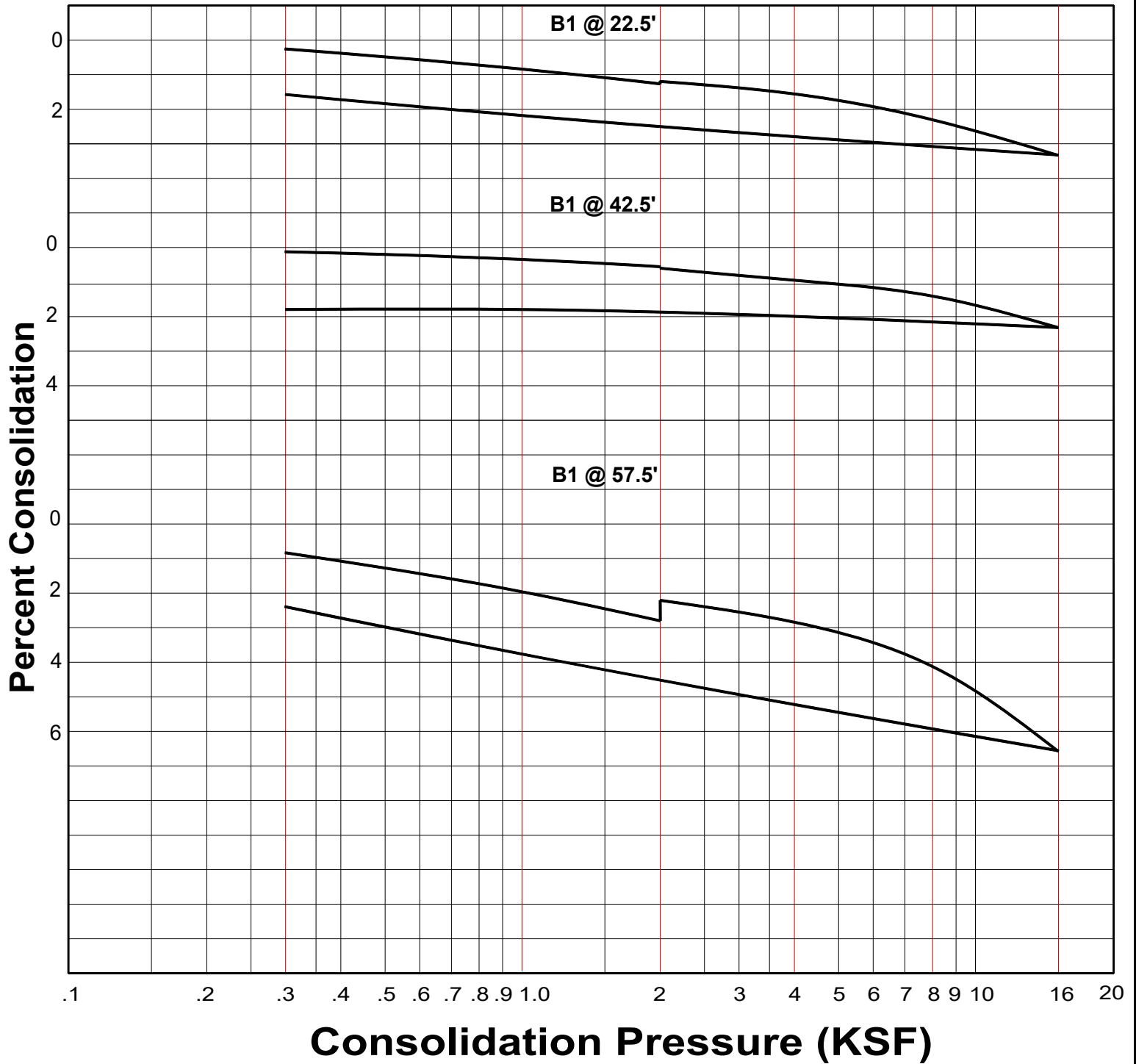
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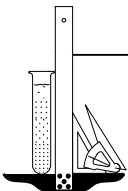
FILE NO. 22064

PLATE: B-2

WATER ADDED AT 2 KSF



CONSOLIDATION TEST



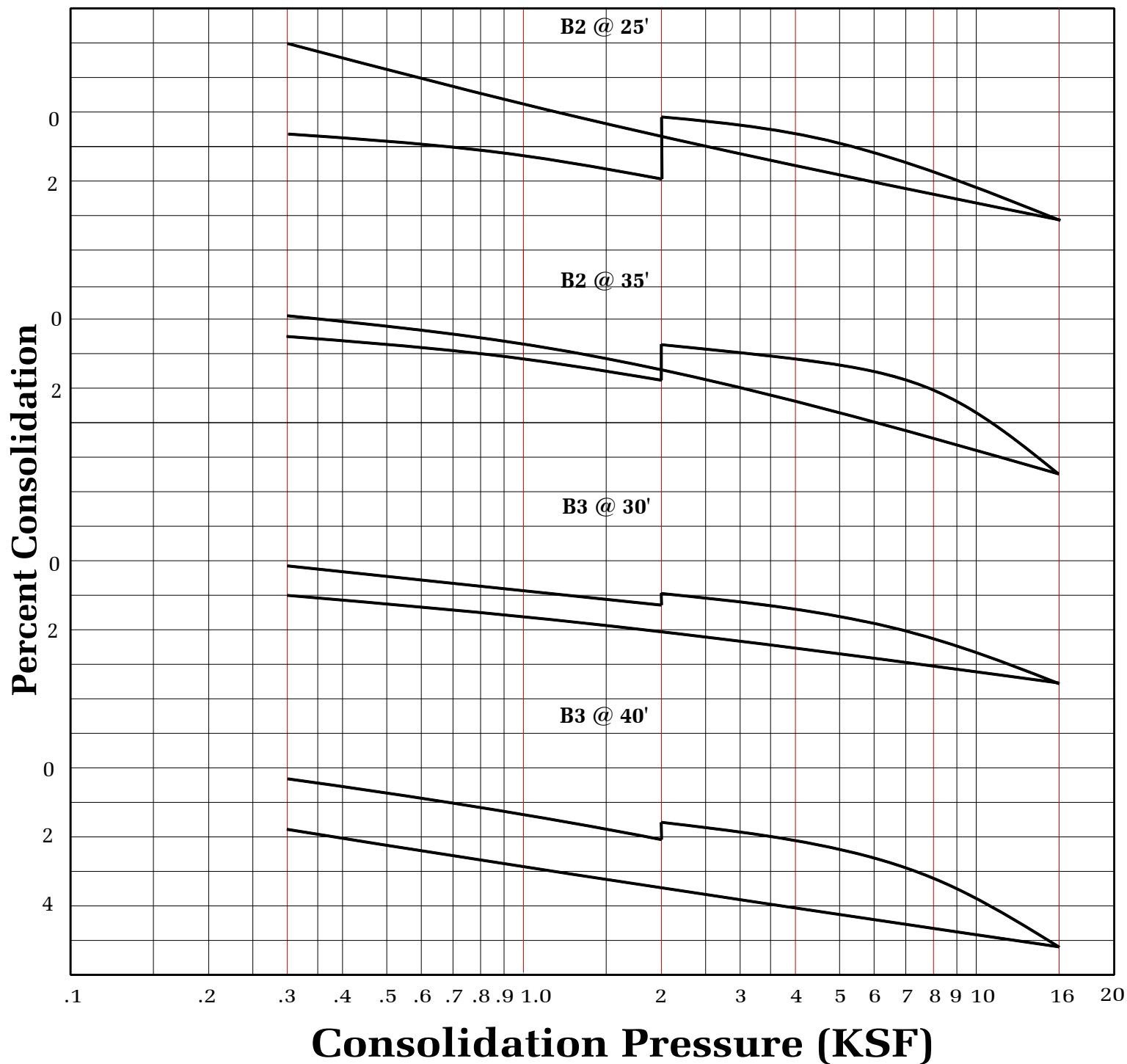
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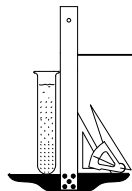
FILE NO. 22064

PLATE: C-1

WATER ADDED AT 2 KSF



CONSOLIDATION TEST



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FILE NO. 22064

PLATE: C-2

ASTM D-1557

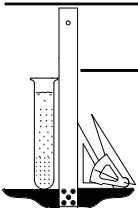
SAMPLE	B1 @ 1-5'	B3 @ 1-5'
SOIL TYPE:	SM/SC	CL
MAXIMUM DENSITY pcf.	128.40	113.60
OPTIMUM MOISTURE %	9.9	15.30

ASTM D 4829

SAMPLE	B1 @ 1-5'	B3 @ 1-5'
SOIL TYPE:	SM/SC	CL
EXPANSION INDEX UBC STANDARD 18-2	50	142
EXPANSION CHARACTER	MODERATE _____ _____	CRITICAL _____ _____

SULFATE CONTENT

SAMPLE	B1 @ 1-5'	B3 @ 1-5'	B1 @ 32.5'	B2 @ 30'	B3 @ 20'
SULFATE CONTENT: (percentage by weight)	< 0.10%	< 0.10%	< 0.10%	< 0.10%	< 0.10%



COMPACTION/EXPANSION/SULFATE DATA SHEET

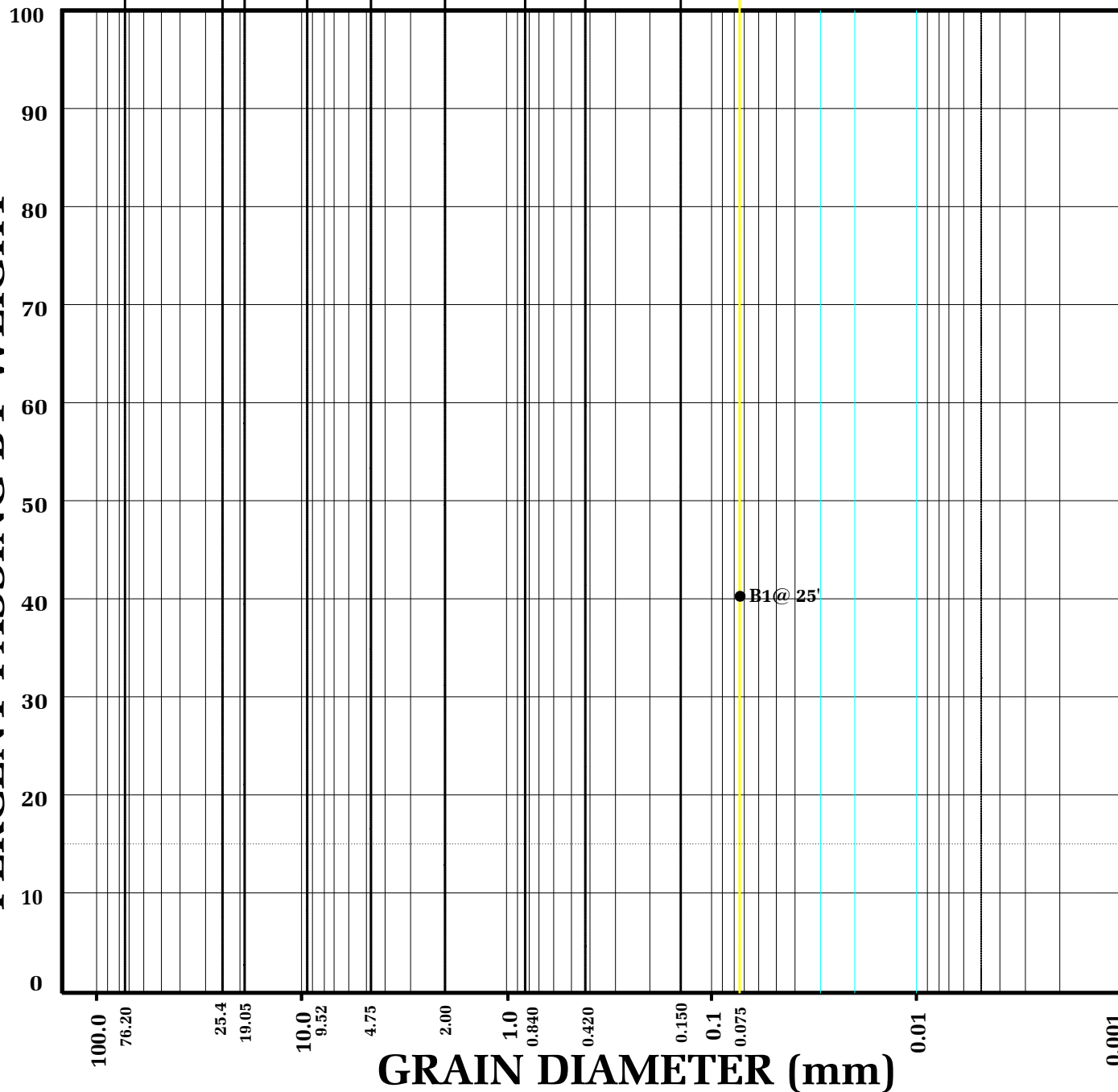
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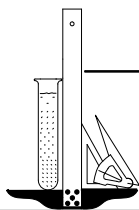
FILE NO. 22064

PLATE: D

PERCENT PASSING BY WEIGHT



GRAIN SIZE DISTRIBUTION



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BARDAS INVESTMENT GROUP

FILE NO. 22064

PLATE: E



Geotechnologies, Inc.

Project: Bardas Investment Company

File No.: 22064

Description: Liquefaction Analysis (PGAM)

Boring No: B1

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	6.9
Peak Ground Horizontal Acceleration, PGA (g):	1.07
Calculated Mag.Wtg.Factor:	1.171

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	39.0
Historically Highest Groundwater Level* (ft):	20.0
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)	Vertical Stress σ_{v0} (psf)	Effective Vert. Stress σ'_{v0} (psf)	Fines Corrected $(N_1)_{60-es}$	Stress Reduction Coeff. r_d	Cyclic Shear Ratio CSR	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Liquefaction Settlement ΔS_i (inches)
1	127.8	Unsaturated	Unsaturated	19	5	0.0	0	127.8	127.8	45.3	1.00	0.698	2.000	Non-Liq.	0.00
2	127.8	Unsaturated	Unsaturated	19	5	0.0	0	255.6	255.6	45.3	1.00	0.696	2.000	Non-Liq.	0.00
3	127.8	Unsaturated	Unsaturated	19	5	0.0	0	383.4	383.4	43.0	1.00	0.694	2.000	Non-Liq.	0.00
4	127.8	Unsaturated	Unsaturated	19	5	0.0	0	511.2	511.2	40.5	0.99	0.692	2.000	Non-Liq.	0.00
5	127.8	Unsaturated	Unsaturated	19	5	0.0	0	639.0	639.0	40.4	0.99	0.689	2.000	Non-Liq.	0.00
6	127.8	Unsaturated	Unsaturated	19	5	0.0	0	766.8	766.8	38.7	0.99	0.687	2.000	Non-Liq.	0.00
7	127.8	Unsaturated	Unsaturated	19	5	0.0	0	894.6	894.6	37.2	0.98	0.684	2.000	Non-Liq.	0.00
8	132.2	Unsaturated	Unsaturated	19	5	0.0	0	1026.8	1026.8	35.9	0.98	0.682	1.723	Non-Liq.	0.00
9	132.2	Unsaturated	Unsaturated	19	5	0.0	0	1159.0	1159.0	36.5	0.98	0.679	2.000	Non-Liq.	0.00
10	132.2	Unsaturated	Unsaturated	9	10	0.0	0	1291.2	1291.2	15.9	0.97	0.676	0.203	Non-Liq.	0.00
11	132.2	Unsaturated	Unsaturated	9	10	0.0	0	1423.4	1423.4	15.1	0.97	0.673	0.192	Non-Liq.	0.00
12	132.2	Unsaturated	Unsaturated	9	10	0.0	0	1555.6	1555.6	14.4	0.96	0.670	0.183	Non-Liq.	0.00
13	129.9	Unsaturated	Unsaturated	9	10	0.0	0	1685.5	1685.5	13.8	0.96	0.667	0.176	Non-Liq.	0.00
14	129.9	Unsaturated	Unsaturated	9	10	0.0	0	1815.4	1815.4	13.3	0.95	0.664	0.169	Non-Liq.	0.00
15	129.9	Unsaturated	Unsaturated	25	15	0.0	0	1945.3	1945.3	45.4	0.95	0.661	2.000	Non-Liq.	0.00
16	129.9	Unsaturated	Unsaturated	25	15	0.0	0	2075.2	2075.2	44.6	0.95	0.658	2.000	Non-Liq.	0.00
17	129.9	Unsaturated	Unsaturated	25	15	0.0	0	2205.1	2205.1	43.8	0.94	0.654	2.000	Non-Liq.	0.00
18	128.9	Unsaturated	Unsaturated	26	20	0.0	0	2334.0	2334.0	44.9	0.94	0.651	2.000	Non-Liq.	0.00
19	128.9	Unsaturated	Unsaturated	26	20	0.0	0	2462.9	2462.9	44.2	0.93	0.647	2.000	Non-Liq.	0.00
20	128.9	Unsaturated	Unsaturated	26	20	0.0	0	2591.8	2591.8	43.6	0.93	0.644	2.000	Non-Liq.	0.00
21	128.9	Unsaturated	Saturated	26	20	0.0	0	2720.7	2658.3	43.0	0.92	0.655	2.000	3.1	0.00
22	128.9	Unsaturated	Saturated	26	20	0.0	0	2849.6	2724.8	42.4	0.92	0.666	2.000	3.0	0.00
23	122.9	Unsaturated	Saturated	26	20	0.0	0	2972.5	2785.3	41.8	0.91	0.676	2.000	3.0	0.00
24	122.9	Unsaturated	Saturated	26	20	0.0	0	3095.4	2845.8	41.3	0.90	0.685	2.000	2.9	0.00
25	122.9	Unsaturated	Saturated	21	25	40.3	0	3218.3	2906.3	36.1	0.90	0.693	1.457	2.1	0.00
26	122.9	Unsaturated	Saturated	21	25	40.3	0	3341.2	2966.8	35.5	0.89	0.700	1.277	1.8	0.00
27	122.9	Unsaturated	Saturated	21	25	40.3	0	3464.1	3027.3	35.0	0.89	0.707	1.132	1.6	0.00
28	129.3	Unsaturated	Saturated	29	30	0.0	0	3593.4	3094.2	47.1	0.88	0.713	1.975	2.8	0.00
29	129.3	Unsaturated	Saturated	29	30	0.0	0	3722.7	3161.1	46.7	0.88	0.719	1.950	2.7	0.00
30	129.3	Unsaturated	Saturated	29	30	0.0	0	3852.0	3228.0	46.3	0.87	0.724	1.927	2.7	0.00
31	129.3	Unsaturated	Saturated	29	30	0.0	0	3981.3	3294.9	45.8	0.87	0.728	1.904	2.6	0.00
32	129.3	Unsaturated	Saturated	29	30	0.0	0	4110.6	3361.8	45.4	0.86	0.732	1.882	2.6	0.00
33	130.8	Unsaturated	Saturated	29	30	0.0	0	4241.4	3430.2	44.9	0.85	0.735	1.860	2.5	0.00
34	130.8	Unsaturated	Saturated	29	30	0.0	0	4372.2	3498.6	44.4	0.85	0.738	1.839	2.5	0.00
35	130.8	Unsaturated	Saturated	61	35	0.0	0	4503.0	3567.0	93.4	0.84	0.741	1.819	2.5	0.00
36	130.8	Unsaturated	Saturated	61	35	0.0	0	4633.8	3635.4	92.7	0.84	0.743	1.799	2.4	0.00
37	130.8	Unsaturated	Saturated	61	35	0.0	0	4764.6	3703.8	92.0	0.83	0.744	1.780	2.4	0.00
38	128.1	Unsaturated	Saturated	61	35	0.0	0	4892.7	3769.5	91.4	0.83	0.746	1.761	2.4	0.00
39	128.1	Unsaturated	Saturated	61	35	0.0	0	5020.8	3835.2	90.7	0.82	0.747	1.743	2.3	0.00
40	128.1	Saturated	Saturated	69	40	0.0	0	5148.9	3900.9	102.3	0.81	0.748	1.734	2.3	0.00
41	128.1	Saturated	Saturated	69	40	0.0	0	5277.0	3966.6	101.9	0.81	0.748	1.725	2.3	0.00
42	128.1	Saturated	Saturated	69	40	0.0	0	5405.1	4032.3	101.6	0.80	0.749	1.717	2.3	0.00
43	122.1	Saturated	Saturated	69	40	0.0	0	5527.2	4092.0	101.3	0.80	0.749	1.709	2.3	0.00
44	122.1	Saturated	Saturated	69	40	0.0	0	5649.3	4151.7	101.0	0.79	0.749	1.701	2.3	0.00
45	122.1	Saturated	Saturated	80	45	0.0	0	5771.4	4211.4	116.8	0.79	0.749	1.693	2.3	0.00
46	122.1	Saturated	Saturated	80	45	0.0	0	5893.5	4271.1	116.4	0.78	0.748	1.686	2.3	0.00
47	122.1	Saturated	Saturated	80	45	0.0	0	6015.6	4330.8	116.1	0.77	0.748	1.678	2.2	0.00
48	122.7	Saturated	Saturated	80	45	0.0	0	6138.3	4391.1	115.8	0.77	0.747	1.671	2.2	0.00
49	122.7	Saturated	Saturated	80	45	0.0	0	6261.0	4451.4	115.4	0.76	0.746	1.663	2.2	0.00
50	122.7	Saturated	Saturated	88	50	0.0	0	6383.7	4511.7	126.6	0.76	0.745	1.656	2.2	0.00
Total Liquefaction Settlement, S =														0.00 inches	



Geotechnologies, Inc.

Project: Bardas Investment Company

File No.: 22064

Description: Liquefaction Analysis (2/3 PGAs_M)

Boring No: B1

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	6.4
Peak Ground Horizontal Acceleration, PGA (g)	0.71
Calculated Mag.Wtg.Factor:	1.335

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	39.0
Historically Highest Groundwater Level* (ft):	20.0
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)	Vertical Stress $\sigma_{v'}$ (psf)	Effective Vert. Stress $\sigma_{v'e}$ (psf)	Fines Corrected $(N_1)_{60-es}$	Stress Reduction Coeff. r_d	Cyclic Shear Ratio CSR	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Liquefaction Settlement ΔS_i (inches)
1	127.8	Unsaturated	Unsaturated	19	5	0.0	0	127.8	127.8	45.3	1.00	0.463	2.000	Non-Liq.	0.00
2	127.8	Unsaturated	Unsaturated	19	5	0.0	0	255.6	255.6	45.3	1.00	0.462	2.000	Non-Liq.	0.00
3	127.8	Unsaturated	Unsaturated	19	5	0.0	0	383.4	383.4	43.0	1.00	0.460	2.000	Non-Liq.	0.00
4	127.8	Unsaturated	Unsaturated	19	5	0.0	0	511.2	511.2	40.5	0.99	0.458	2.000	Non-Liq.	0.00
5	127.8	Unsaturated	Unsaturated	19	5	0.0	0	639.0	639.0	40.4	0.99	0.456	2.000	Non-Liq.	0.00
6	127.8	Unsaturated	Unsaturated	19	5	0.0	0	766.8	766.8	38.7	0.98	0.454	2.000	Non-Liq.	0.00
7	127.8	Unsaturated	Unsaturated	19	5	0.0	0	894.6	894.6	37.2	0.98	0.452	2.000	Non-Liq.	0.00
8	132.2	Unsaturated	Unsaturated	19	5	0.0	0	1026.8	1026.8	35.9	0.97	0.450	1.964	Non-Liq.	0.00
9	132.2	Unsaturated	Unsaturated	19	5	0.0	0	1159.0	1159.0	36.5	0.97	0.447	2.000	Non-Liq.	0.00
10	132.2	Unsaturated	Unsaturated	9	10	0.0	0	1291.2	1291.2	15.9	0.96	0.445	0.231	Non-Liq.	0.00
11	132.2	Unsaturated	Unsaturated	9	10	0.0	0	1423.4	1423.4	15.1	0.96	0.443	0.219	Non-Liq.	0.00
12	132.2	Unsaturated	Unsaturated	9	10	0.0	0	1555.6	1555.6	14.4	0.95	0.440	0.209	Non-Liq.	0.00
13	129.9	Unsaturated	Unsaturated	9	10	0.0	0	1685.5	1685.5	13.8	0.95	0.438	0.200	Non-Liq.	0.00
14	129.9	Unsaturated	Unsaturated	9	10	0.0	0	1815.4	1815.4	13.3	0.94	0.435	0.193	Non-Liq.	0.00
15	129.9	Unsaturated	Unsaturated	25	15	0.0	0	1945.3	1945.3	45.4	0.94	0.433	2.000	Non-Liq.	0.00
16	129.9	Unsaturated	Unsaturated	25	15	0.0	0	2075.2	2075.2	44.6	0.93	0.430	2.000	Non-Liq.	0.00
17	129.9	Unsaturated	Unsaturated	25	15	0.0	0	2205.1	2205.1	43.8	0.93	0.427	2.000	Non-Liq.	0.00
18	128.9	Unsaturated	Unsaturated	26	20	0.0	0	2334.0	2334.0	44.9	0.92	0.425	2.000	Non-Liq.	0.00
19	128.9	Unsaturated	Unsaturated	26	20	0.0	0	2462.9	2462.9	44.2	0.91	0.422	2.000	Non-Liq.	0.00
20	128.9	Unsaturated	Unsaturated	26	20	0.0	0	2591.8	2591.8	43.6	0.91	0.419	2.000	Non-Liq.	0.00
21	128.9	Unsaturated	Saturated	26	20	0.0	0	2720.7	2658.3	43.0	0.90	0.426	2.000	4.7	0.00
22	128.9	Unsaturated	Saturated	26	20	0.0	0	2849.6	2724.8	42.4	0.90	0.432	2.000	4.6	0.00
23	122.9	Unsaturated	Saturated	26	20	0.0	0	2972.5	2785.3	41.8	0.89	0.438	2.000	4.6	0.00
24	122.9	Unsaturated	Saturated	26	20	0.0	0	3095.4	2845.8	41.3	0.88	0.443	2.000	4.5	0.00
25	122.9	Unsaturated	Saturated	21	25	40.3	0	3218.3	2906.3	36.1	0.88	0.448	1.660	3.7	0.00
26	122.9	Unsaturated	Saturated	21	25	40.3	0	3341.2	2966.8	35.5	0.87	0.452	1.455	3.2	0.00
27	122.9	Unsaturated	Saturated	21	25	40.3	0	3464.1	3027.3	35.0	0.86	0.456	1.290	2.8	0.00
28	129.3	Unsaturated	Saturated	29	30	0.0	0	3593.4	3094.2	47.1	0.86	0.459	2.000	4.4	0.00
29	129.3	Unsaturated	Saturated	29	30	0.0	0	3722.7	3161.1	46.7	0.85	0.462	2.000	4.3	0.00
30	129.3	Unsaturated	Saturated	29	30	0.0	0	3852.0	3228.0	46.3	0.84	0.464	2.000	4.3	0.00
31	129.3	Unsaturated	Saturated	29	30	0.0	0	3981.3	3294.9	45.8	0.84	0.466	2.000	4.3	0.00
32	129.3	Unsaturated	Saturated	29	30	0.0	0	4110.6	3361.8	45.4	0.83	0.468	2.000	4.3	0.00
33	130.8	Unsaturated	Saturated	29	30	0.0	0	4241.4	3430.2	44.9	0.82	0.469	2.000	4.3	0.00
34	130.8	Unsaturated	Saturated	29	30	0.0	0	4372.2	3498.6	44.4	0.82	0.471	2.000	4.3	0.00
35	130.8	Unsaturated	Saturated	61	35	0.0	0	4503.0	3567.0	93.4	0.81	0.471	2.000	4.2	0.00
36	130.8	Unsaturated	Saturated	61	35	0.0	0	4633.8	3635.4	92.7	0.80	0.472	2.000	4.2	0.00
37	130.8	Unsaturated	Saturated	61	35	0.0	0	4764.6	3703.8	92.0	0.80	0.472	2.000	4.2	0.00
38	128.1	Unsaturated	Saturated	61	35	0.0	0	4892.7	3769.5	91.4	0.79	0.472	2.000	4.2	0.00
39	128.1	Unsaturated	Saturated	61	35	0.0	0	5020.8	3835.2	90.7	0.78	0.472	1.987	4.2	0.00
40	128.1	Saturated	Saturated	69	40	0.0	0	5148.9	3900.9	102.3	0.78	0.472	1.977	4.2	0.00
41	128.1	Saturated	Saturated	69	40	0.0	0	5277.0	3966.6	101.9	0.77	0.472	1.967	4.2	0.00
42	128.1	Saturated	Saturated	69	40	0.0	0	5405.1	4032.3	101.6	0.76	0.471	1.957	4.2	0.00
43	122.1	Saturated	Saturated	69	40	0.0	0	5527.2	4092.0	101.3	0.76	0.471	1.948	4.1	0.00
44	122.1	Saturated	Saturated	69	40	0.0	0	5649.3	4151.7	101.0	0.75	0.470	1.939	4.1	0.00
45	122.1	Saturated	Saturated	80	45	0.0	0	5771.4	4211.4	116.8	0.74	0.469	1.930	4.1	0.00
46	122.1	Saturated	Saturated	80	45	0.0	0	5893.5	4271.1	116.4	0.74	0.468	1.921	4.1	0.00
47	122.1	Saturated	Saturated	80	45	0.0	0	6015.6	4330.8	116.1	0.73	0.467	1.913	4.1	0.00
48	122.7	Saturated	Saturated	80	45	0.0	0	6138.3	4391.1	115.8	0.72	0.466	1.904	4.1	0.00
49	122.7	Saturated	Saturated	80	45	0.0	0	6261.0	4451.4	115.4	0.72	0.465	1.896	4.1	0.00
50	122.7	Saturated	Saturated	88	50	0.0	0	6383.7	4511.7	126.6	0.71	0.463	1.887	4.1	0.00
Total Liquefaction Settlement, S =														0.00 inches	



Geotechnologies, Inc.

Project: Bardas Investment Group

File No.: 22064

Settlement Calculation - Mat Foundation

Description: Proposed 6-Story Structure

Gridline: Mat Foundation on South Portion of Site

Soil Unit Weight 57.6 pcf (Bouyant) Mat Foundation

Bearing Value 3000.0 psf

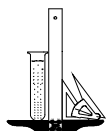
Depth of Footing 3.0 feet

Width of Footing 160.0 feet

* Influence Values are based on Westergaard's Analyses (Ref: Sowers)

Depth Below Subgrade Surface (feet)	Average Depth Below Subgrade Surface (feet)	Average Depth Below Foundation (feet)	Ratio of Foundation vs. Depth (a/z)	Influence Value	Foundation Influence Pressure (psf)	Natural Soil Pressure (psf)	Total Pressure (psf)	Consolidation Curve Used	Percent Strain [Total] (%)	Percent Strain [Natural] (%)	Percent Strain [Net] (%)	Thickness of Depth Increment (feet)	Net Settlement (inches)
3.0													
	4.0	1.0	160.0	94%	2809.5	230.4	3039.9	Fill	0.00	0.00	0.00	2.0	0.00
5.0													
	7.5	4.5	35.6	94%	2809.5	432	3241.5	B3 @ 40	2.20	0.80	1.40	5.0	0.84
10.0													
	15.0	12.0	13.3	90%	2702.325	864	3566.325	B1 @ 42.5	0.90	0.30	0.60	10.0	0.72
20.0													
	25.0	22.0	7.3	82%	2454.15	1440	3894.15	B1 @ 62.5	1.40	1.10	0.30	10.0	0.36
30.0													
	175.0	172.0	0.9	19%	558.75	10080	10638.75	B2 @ 72.5	2.22	2.20	0.02	290.0	0.70
320.0													

Total Settlement in inches: 2.62



Geotechnologies, Inc.

Project: Bardas Investment Group

File No.: 22064

Description: Cantilever Retaining Walls (Up to 20 feet)

Retaining Wall Design with Level Backfill (Vector Analysis)

Input:

Retaining Wall Height (H) 20.00 feet

Unit Weight of Retained Soils (γ) 120.0 pcf

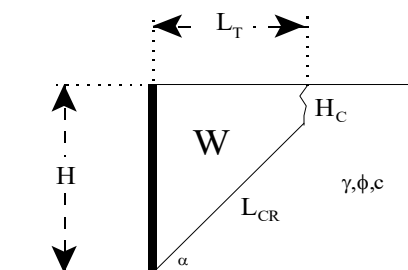
Friction Angle of Retained Soils (ϕ) 28.0 degrees

Cohesion of Retained Soils (c) 260.0 psf

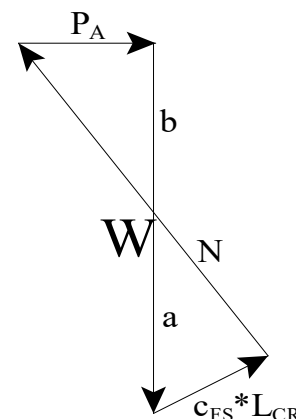
Factor of Safety (FS) 1.50

Factored Parameters: (ϕ_{FS}) 19.5 degrees

(c_{FS}) 173.3 psf



Failure Angle (α) degrees	Height of Tension Crack (H_c) feet	Area of Wedge (A) ft^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	5.1	223	26757.5	23.2	10838.0	15919.5	5946.3
41	4.9	216	25934.0	23.0	10250.4	15683.7	6172.3
42	4.8	209	25125.2	22.7	9711.0	15414.2	6379.1
43	4.7	203	24332.5	22.5	9215.1	15117.4	6567.6
44	4.6	196	23556.8	22.2	8758.5	14798.3	6738.3
45	4.5	190	22798.3	22.0	8337.2	14461.1	6892.0
46	4.4	184	22057.3	21.7	7947.9	14109.4	7029.1
47	4.3	178	21333.3	21.4	7587.4	13745.9	7150.2
48	4.3	172	20626.3	21.2	7253.0	13373.2	7255.6
49	4.2	166	19935.6	20.9	6942.3	12993.2	7345.8
50	4.2	161	19260.7	20.7	6653.1	12607.6	7421.1
51	4.1	155	18601.0	20.4	6383.4	12217.7	7481.7
52	4.1	150	17956.0	20.2	6131.4	11824.6	7527.9
53	4.1	144	17325.0	19.9	5895.6	11429.4	7559.8
54	4.1	139	16707.4	19.7	5674.5	11032.8	7577.5
55	4.1	134	16102.4	19.4	5466.9	10635.5	7581.2
56	4.1	129	15509.6	19.2	5271.6	10238.0	7570.7
57	4.1	124	14928.3	18.9	5087.5	9840.8	7546.2
58	4.1	120	14357.8	18.7	4913.5	9444.2	7507.4
59	4.2	115	13797.6	18.5	4749.0	9048.6	7454.3
60	4.2	110	13247.0	18.3	4592.9	8654.2	7386.7
61	4.2	106	12705.6	18.0	4444.5	8261.2	7304.2
62	4.3	101	12172.8	17.8	4303.1	7869.7	7206.7
63	4.4	97	11648.0	17.6	4168.0	7480.0	7093.8
64	4.4	93	11130.7	17.3	4038.5	7092.2	6965.1
65	4.5	89	10620.3	17.1	3914.0	6706.3	6820.0



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

$$P_{A, \max}$$

7581.2 | lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

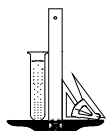
$$EFP = 2 * P_A / H^2$$

EFP

37.9 pcf

Design Wall for an Equivalent Fluid Pressure:

37.9 pcf



Geotechnologies, Inc.

Project: Bardas Investment Group

File No.: 22064

Description: Undrained Cantilever Retaining Walls (Up to 25 ft in height)

Retaining Wall Design with Level Backfill (Vector Analysis)

Input:

Retaining Wall Height (H) 25.00 feet

Unit Weight of Retained Soils (γ) 62.6 pcf

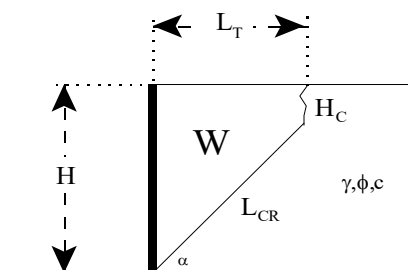
Friction Angle of Retained Soils (ϕ) 28.0 degrees

Cohesion of Retained Soils (c) 260.0 psf

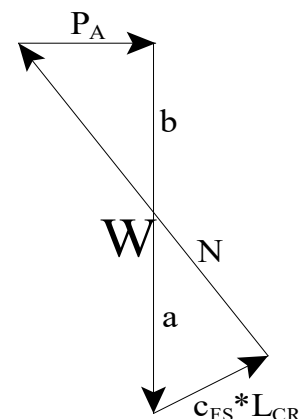
Factor of Safety (FS) 1.50

Factored Parameters: (ϕ_{FS}) 19.5 degrees

(c_{FS}) 173.3 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	8.6	276	17259.0	23.2	8818.7	8440.3	4022.6
46	8.4	268	16745.7	23.0	8441.9	8303.8	4136.9
47	8.3	259	16235.3	22.8	8087.7	8147.6	4238.1
48	8.2	251	15729.0	22.6	7754.5	7974.5	4326.5
49	8.1	243	15227.8	22.4	7441.0	7786.8	4402.3
50	8.0	235	14732.4	22.2	7145.6	7586.8	4465.7
51	7.9	228	14243.1	22.0	6867.1	7376.0	4516.9
52	7.9	220	13760.3	21.7	6604.1	7156.2	4555.8
53	7.9	212	13284.0	21.5	6355.4	6928.6	4582.8
54	7.8	205	12814.3	21.2	6119.9	6694.4	4597.8
55	7.8	197	12351.1	20.9	5896.6	6454.5	4600.9
56	7.8	190	11894.3	20.7	5684.3	6209.9	4592.1
57	7.9	183	11443.6	20.4	5482.3	5961.3	4571.3
58	7.9	176	10998.9	20.1	5289.5	5709.4	4538.6
59	8.0	169	10559.9	19.9	5105.1	5454.8	4493.7
60	8.0	162	10126.3	19.6	4928.3	5198.0	4436.7
61	8.1	155	9697.8	19.3	4758.2	4939.5	4367.4
62	8.2	148	9274.0	19.0	4594.2	4679.8	4285.5
63	8.4	141	8854.6	18.7	4435.5	4419.1	4190.9
64	8.5	135	8439.2	18.4	4281.3	4157.9	4083.4
65	8.7	128	8027.4	18.0	4130.8	3896.6	3962.7
66	8.8	122	7618.7	17.7	3983.2	3635.5	3828.6
67	9.1	115	7212.7	17.3	3837.8	3374.9	3680.8
68	9.3	109	6808.9	16.9	3693.6	3115.3	3519.0
69	9.6	102	6406.7	16.5	3549.7	2857.0	3343.0
70	9.9	96	6005.5	16.1	3405.0	2600.5	3152.6



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

$P_{A, max}$

4600.9 | lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2 * P_A / H^2$

14.7 pcf

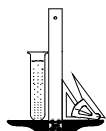
(Lateral Earth Pressure based on
Bouyant Weight of Soil in EFP)
(Hydrostatic Pressure)

Hydrostatic Pressure

62.4 pcf

Design Wall for an Equivalent Fluid Pressure:

77.1 pcf



Geotechnologies, Inc.

Project: Bardas Investment Group

File No.: 22064

Description: Undrained Cantilever Retaining Walls (Up to 35 ft in height)

Retaining Wall Design with Level Backfill (Vector Analysis)

Input:

Retaining Wall Height (H) 35.00 feet

Unit Weight of Retained Soils (γ) 62.6 pcf

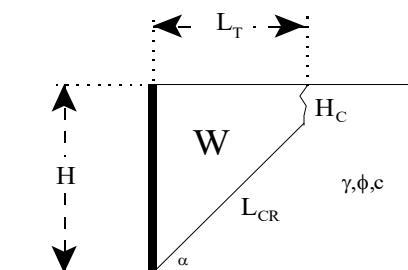
Friction Angle of Retained Soils (ϕ) 28.0 degrees

Cohesion of Retained Soils (c) 260.0 psf

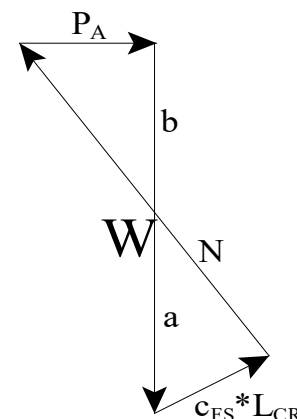
Factor of Safety (FS) 1.50

Factored Parameters: (ϕ_{FS}) 19.5 degrees

(c_{FS}) 173.3 psf



Failure Angle (α) degrees	Height of Tension Crack (H_c) feet	Area of Wedge (A) ft^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	8.6	576	36039.0	37.4	14189.0	21850.0	10413.5
46	8.4	557	34881.3	36.9	13535.1	21346.2	10634.4
47	8.3	539	33747.9	36.5	12928.4	20819.5	10829.6
48	8.2	521	32638.6	36.1	12364.5	20274.1	10999.7
49	8.1	504	31553.0	35.7	11839.5	19713.5	11145.2
50	8.0	487	30490.7	35.2	11349.9	19140.8	11266.7
51	7.9	470	29450.8	34.8	10892.5	18558.3	11364.5
52	7.9	454	28432.8	34.4	10464.6	17968.2	11439.1
53	7.9	438	27435.7	34.0	10063.5	17372.3	11490.6
54	7.8	423	26458.8	33.6	9686.8	16771.9	11519.2
55	7.8	407	25501.0	33.2	9332.6	16168.4	11525.1
56	7.8	392	24561.5	32.7	8998.7	15562.8	11508.3
57	7.9	378	23639.5	32.3	8683.5	14956.0	11468.6
58	7.9	363	22734.0	31.9	8385.3	14348.6	11406.0
59	8.0	349	21844.1	31.5	8102.6	13741.4	11320.3
60	8.0	335	20968.9	31.1	7834.1	13134.9	11211.1
61	8.1	321	20107.7	30.7	7578.2	12529.4	11078.1
62	8.2	308	19259.5	30.3	7334.0	11925.5	10920.8
63	8.4	294	18423.5	29.9	7100.1	11323.4	10738.7
64	8.5	281	17598.8	29.5	6875.4	10723.4	10531.2
65	8.7	268	16784.6	29.1	6658.9	10125.8	10297.6
66	8.8	255	15980.1	28.6	6449.4	9530.8	10037.0
67	9.1	243	15184.4	28.2	6245.8	8938.6	9748.6
68	9.3	230	14396.6	27.7	6046.9	8349.6	9431.5
69	9.6	218	13615.7	27.2	5851.7	7764.0	9084.7
70	9.9	205	12840.9	26.7	5658.8	7182.1	8707.0



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

$$P_{A, \max}$$

11525.1 | lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$$EFP = 2 * P_A / H^2$$

18.8 pcf

(Lateral Earth Pressure based on
Bouyant Weight of Soil in EFP)

Hydrostatic Pressure

62.4 pcf

(Hydrostatic Pressure)

Design Wall for an Equivalent Fluid Pressure:

81.2 pcf

Geotechnologies, Inc.

Project: Bardas Investment

File No.: 22064

Soil Weight	γ	120 pcf
Internal Friction Angle	ϕ	28 degrees
Cohesion	c	0 psf
Height of Retaining Wall	H	20 feet

Restrained Retaining Wall Design based on At Rest Earth Pressure

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

$$K_o = 1 - \sin \phi \quad 0.531$$

$$\sigma'_v = \gamma H \quad 2400.0 \text{ psf}$$

$$\sigma'_h = 1273.3 \text{ psf}$$

$$\text{EFP} = 63.7 \text{ pcf}$$

$$P_o = 12732.7 \text{ lbs/ft} \quad (\text{based on a triangular distribution of pressure})$$

Design wall for an EFP of 63.7 pcf

Geotechnologies, Inc.

Project: Bardas Investment

File No.: 22064

Soil Weight	γ	62.6 pcf	(Bouyant)
Internal Friction Angle	ϕ	28 degrees	
Cohesion	c	0 psf	
Height of Retaining Wall	H	25 feet	

Restrained Retaining Wall Design based on At Rest Earth Pressure

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

$$K_o = 1 - \sin \phi \quad 0.531$$

$$\sigma'_v = \gamma H \quad 1565.0 \text{ psf}$$

$$\sigma'_h = 830.3 \text{ psf}$$

$$\text{EFP} = 33.2 \text{ pcf}$$

$$P_o = 10378.5 \text{ lbs/ft} \quad (\text{based on a triangular distribution of pressure})$$

33.2 (Lateral Earth Pressure based on
Bouyant Weight of Soil in EFP)

62.4 (Hydrostatic Pressure)

Design wall for an EFP of 95.6 pcf

Geotechnologies, Inc.

Project: Bardas Investment

File No.: 22064

Soil Weight	γ	62.6 pcf	(Bouyant)
Internal Friction Angle	ϕ	28 degrees	
Cohesion	c	0 psf	
Height of Retaining Wall	H	35 feet	

Restrained Retaining Wall Design based on At Rest Earth Pressure

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

$$K_o = 1 - \sin \phi \quad 0.531$$

$$\sigma'_v = \gamma H \quad 2191.0 \text{ psf}$$

$$\sigma'_h = 1162.4 \text{ psf}$$

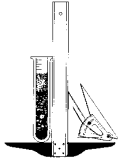
$$\text{EFP} = 33.2 \text{ pcf}$$

$$P_o = 20341.8 \text{ lbs/ft} \quad (\text{based on a triangular distribution of pressure})$$

33.2 (Lateral Earth Pressure based on
Bouyant Weight of Soil in EFP)

62.4 (Hydrostatic Pressure)

Design wall for an EFP of 95.6 pcf



Geotechnologies, Inc.

Project: Bardas Investment Group

File No.: 22064

Seismically Induced Lateral Soil Pressure on Retaining Wall

Input:

Height of Retaining Wall: (H) 35.0 feet
Retained Soil Unit Weight: (γ) 120.0 pcf
Horizontal Ground Acceleration: (k_h) 0.36 g

Seismic Increment (ΔP_{AE}):

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

$$\Delta P_{AE} = 19679.6 \text{ lbs/ft}$$

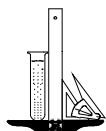
$$T * (2/3) * H = \Delta P_{AE} * 0.6 * H$$

$$T = 17711.7 \text{ lbs/ft}$$

$$EFP = 2 * T / H^2$$

$$EFP = 28.9 \text{ pcf}$$

triangular distribution of pressure applied to the proposed retaining wall.



Geotechnologies, Inc.

Project: Bardas Investment Group

File No.: 22064

Description: Temporary Shoring Walls (Up to 30 feet)

Shoring Design with Level Backfill (Vector Analysis)

Input:

Shoring Height (H) 30.00 feet

Unit Weight of Retained Soils (γ) 120.0 pcf

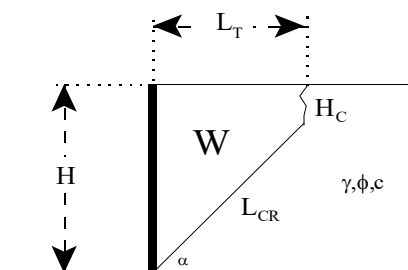
Friction Angle of Retained Soils (ϕ) 28.0 degrees

Cohesion of Retained Soils (c) 260.0 psf

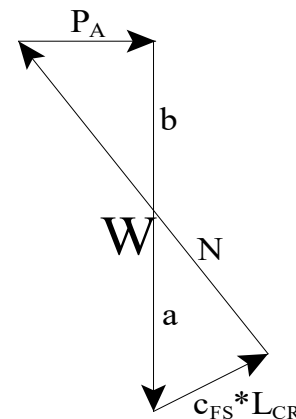
Factor of Safety (FS) 1.25

Factored Parameters: (ϕ_{FS}) 23.0 degrees

(c_{FS}) 208.0 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) ft^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	6.0	432	51816.3	33.9	17350.8	34465.5	13894.7
46	5.9	418	50139.1	33.5	16449.9	33689.2	14270.2
47	5.8	404	48499.6	33.1	15623.7	32875.9	14607.5
48	5.6	391	46897.5	32.8	14864.1	32033.4	14907.9
49	5.6	378	45332.2	32.4	14164.4	31167.8	15172.4
50	5.5	365	43802.8	32.0	13518.3	30284.5	15401.9
51	5.4	353	42308.2	31.6	12920.5	29387.7	15597.2
52	5.4	340	40847.1	31.3	12366.2	28480.9	15759.1
53	5.3	328	39418.2	30.9	11851.2	27567.0	15888.0
54	5.3	317	38020.1	30.6	11371.8	26648.3	15984.5
55	5.3	305	36651.5	30.2	10924.6	25726.8	16048.9
56	5.2	294	35310.8	29.9	10506.7	24804.2	16081.3
57	5.2	283	33996.9	29.5	10115.4	23881.5	16082.0
58	5.3	273	32708.2	29.2	9748.2	22960.1	16050.9
59	5.3	262	31443.6	28.8	9403.0	22040.6	15988.0
60	5.3	252	30201.6	28.5	9077.8	21123.8	15892.9
61	5.3	242	28981.1	28.2	8770.9	20210.2	15765.3
62	5.4	232	27780.7	27.9	8480.4	19300.3	15604.9
63	5.5	222	26599.4	27.5	8205.0	18394.4	15411.1
64	5.6	212	25435.9	27.2	7943.1	17492.8	15183.1
65	5.6	202	24289.0	26.9	7693.3	16595.7	14920.1
66	5.8	193	23157.7	26.5	7454.5	15703.2	14621.3
67	5.9	184	22040.7	26.2	7225.2	14815.5	14285.6
68	6.0	174	20937.1	25.9	7004.3	13932.8	13911.7
69	6.2	165	19845.6	25.5	6790.6	13055.1	13498.5
70	6.4	156	18765.2	25.1	6582.7	12182.5	13044.3



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

$P_{A, max}$

16082.0 | lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

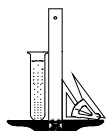
$$EFP = 2 * P_A / H^2$$

EFP

35.7 pcf

Design Shoring for an Equivalent Fluid Pressure:

36 pcf



Geotechnologies, Inc.

Project: Bardas Investment Group

File No.: 22064

Description: Temporary Shoring Walls (Up to 35 feet)

Shoring Design with Level Backfill (Vector Analysis)

Input:

Shoring Height (H) 35.00 feet

Unit Weight of Retained Soils (γ) 120.0 pcf

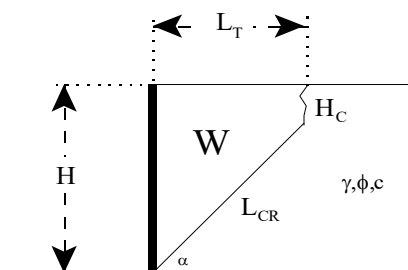
Friction Angle of Retained Soils (ϕ) 28.0 degrees

Cohesion of Retained Soils (c) 260.0 psf

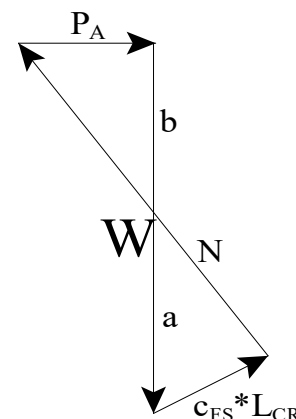
Factor of Safety (FS) 1.25

Factored Parameters: (ϕ_{FS}) 23.0 degrees

(c_{FS}) 208.0 psf



Failure Angle (α) degrees	Height of Tension Crack (H_c) feet	Area of Wedge (A) ft^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	6.0	594	71316.3	41.0	20970.5	50345.8	20296.7
46	5.9	575	68970.1	40.5	19860.9	49109.2	20801.8
47	5.8	556	66683.6	40.0	18846.3	47837.3	21255.2
48	5.6	537	64455.4	39.5	17916.3	46539.1	21658.7
49	5.6	519	62283.3	39.0	17061.6	45221.7	22013.8
50	5.5	501	60165.3	38.5	16274.2	43891.1	22321.8
51	5.4	484	58099.0	38.1	15547.3	42551.7	22583.9
52	5.4	467	56082.2	37.6	14874.7	41207.5	22801.0
53	5.3	451	54112.5	37.2	14251.0	39861.5	22973.9
54	5.3	435	52187.7	36.7	13671.5	38516.2	23103.3
55	5.3	419	50305.5	36.3	13132.0	37173.5	23189.5
56	5.2	404	48463.8	35.9	12628.7	35835.1	23233.1
57	5.2	389	46660.3	35.5	12158.3	34502.1	23234.0
58	5.3	374	44893.2	35.1	11717.8	33175.4	23192.3
59	5.3	360	43160.4	34.7	11304.5	31855.9	23107.9
60	5.3	345	41460.0	34.3	10915.9	30544.0	22980.4
61	5.3	332	39790.1	33.9	10549.9	29240.2	22809.4
62	5.4	318	38149.1	33.5	10204.4	27944.7	22594.2
63	5.5	304	36535.1	33.1	9877.5	26657.7	22334.1
64	5.6	291	34946.7	32.8	9567.5	25379.2	22028.1
65	5.6	278	33382.0	32.4	9272.7	24109.3	21675.1
66	5.8	265	31839.6	32.0	8991.8	22847.9	21273.7
67	5.9	253	30318.0	31.6	8723.0	21595.0	20822.5
68	6.0	240	28815.6	31.2	8465.1	20350.5	20319.7
69	6.2	228	27331.0	30.9	8216.7	19114.3	19763.5
70	6.4	216	25862.6	30.5	7976.2	17886.4	19151.8



Design Equations (Vector Analysis):

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

$$b = W - a$$

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

$$EFP = 2 * P_A / H^2$$

Maximum Active Pressure Resultant

$P_{A, max}$

23234.0 | lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

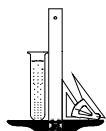
$$EFP = 2 * P_A / H^2$$

EFP

37.9 pcf

Design Shoring for an Equivalent Fluid Pressure:

38 pcf



Geotechnologies, Inc.

Project: Bardas Investment Group

File No.: 22064

Description: Temporary Shoring Walls (Up to 40 feet)

Shoring Design with Level Backfill (Vector Analysis)

Input:

Shoring Height (H) 40.00 feet

Unit Weight of Retained Soils (γ) 120.0 pcf

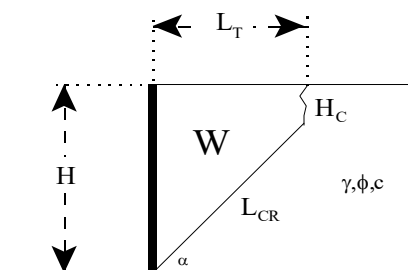
Friction Angle of Retained Soils (ϕ) 28.0 degrees

Cohesion of Retained Soils (c) 260.0 psf

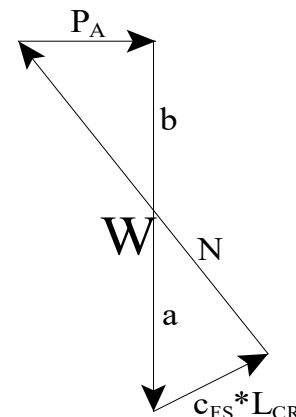
Factor of Safety (FS) 1.25

Factored Parameters: (ϕ_{FS}) 23.0 degrees

(c_{FS}) 208.0 psf



Failure Angle (α) degrees	Height of Tension Crack (H_c) feet	Area of Wedge (A) ft^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	6.0	782	93816.3	48.0	24590.2	69226.1	27908.3
46	5.9	756	90698.1	47.4	23271.9	67426.1	28560.5
47	5.8	731	87665.2	46.8	22069.0	65596.2	29145.9
48	5.6	706	84714.5	46.2	20968.4	63746.1	29666.6
49	5.6	682	81842.3	45.6	19958.7	61883.5	30124.7
50	5.5	659	79045.0	45.1	19030.1	60014.9	30522.0
51	5.4	636	76319.1	44.5	18174.1	58145.0	30859.9
52	5.4	614	73661.1	44.0	17383.1	56278.0	31139.8
53	5.3	592	71067.5	43.4	16650.8	54416.7	31362.7
54	5.3	571	68534.9	42.9	15971.2	52563.7	31529.4
55	5.3	551	66060.2	42.4	15339.3	50720.9	31640.6
56	5.2	530	63640.2	41.9	14750.7	48889.5	31696.7
57	5.2	511	61272.0	41.4	14201.2	47070.8	31697.9
58	5.3	491	58952.8	41.0	13687.4	45265.4	31644.2
59	5.3	472	56679.7	40.5	13205.9	43473.8	31535.4
60	5.3	454	54450.3	40.1	12754.0	41696.3	31371.0
61	5.3	436	52262.1	39.6	12328.9	39933.2	31150.6
62	5.4	418	50112.5	39.2	11928.3	38184.2	30873.2
63	5.5	400	47999.5	38.8	11550.0	36449.5	30537.8
64	5.6	383	45920.6	38.3	11191.9	34728.8	30143.2
65	5.6	366	43873.9	37.9	10852.2	33021.8	29687.7
66	5.8	349	41857.3	37.5	10529.1	31328.2	29169.8
67	5.9	332	39868.7	37.1	10220.9	29647.8	28587.3
68	6.0	316	37906.2	36.6	9926.0	27980.2	27938.0
69	6.2	300	35967.9	36.2	9642.8	26325.1	27219.3
70	6.4	284	34051.9	35.8	9369.7	24682.2	26428.3



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

$P_{A, max}$

31697.9 | lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

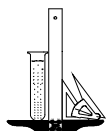
$$EFP = 2 * P_A / H^2$$

EFP

39.6 pcf

Design Shoring for an Equivalent Fluid Pressure:

40 pcf



Geotechnologies, Inc.

Project: Bardas Investment Group

File No.: 22064

Description: Temporary Shoring Walls (Up to 45 feet)

Shoring Design with Level Backfill (Vector Analysis)

Input:

Shoring Height (H) 45.00 feet

Unit Weight of Retained Soils (γ) 120.0 pcf

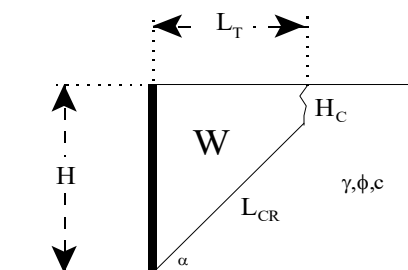
Friction Angle of Retained Soils (ϕ) 28.0 degrees

Cohesion of Retained Soils (c) 260.0 psf

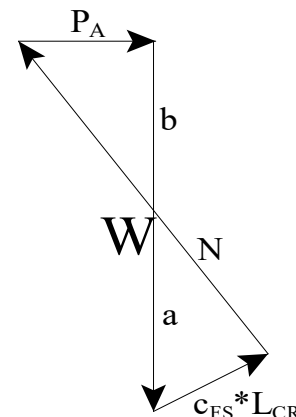
Factor of Safety (FS) 1.25

Factored Parameters: (ϕ_{FS}) 23.0 degrees

(c_{FS}) 208.0 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	6.0	994	119316.3	55.1	28209.9	91106.4	36729.2
46	5.9	961	115323.1	54.4	26682.9	88640.2	37546.5
47	5.8	929	111444.4	53.7	25291.7	86152.7	38279.6
48	5.6	897	107674.8	53.0	24020.5	83654.2	38931.6
49	5.6	867	104009.1	52.3	22855.9	81153.2	39505.1
50	5.5	837	100442.0	51.6	21786.0	78656.0	40002.4
51	5.4	808	96968.6	50.9	20800.9	76167.7	40425.3
52	5.4	780	93583.9	50.3	19891.6	73692.3	40775.5
53	5.3	752	90283.1	49.7	19050.5	71232.6	41054.4
54	5.3	726	87061.7	49.1	18270.9	68790.9	41263.0
55	5.3	699	83915.5	48.5	17546.7	66368.8	41402.1
56	5.2	674	80840.2	48.0	16872.7	63967.5	41472.2
57	5.2	649	77831.9	47.4	16244.1	61587.7	41473.7
58	5.3	624	74886.9	46.9	15657.0	59229.9	41406.5
59	5.3	600	72001.7	46.3	15107.4	56894.3	41270.4
60	5.3	576	69172.8	45.8	14592.1	54580.7	41064.8
61	5.3	553	66396.9	45.3	14107.9	52289.0	40789.0
62	5.4	531	63671.1	44.8	13652.2	50018.9	40441.9
63	5.5	508	60992.4	44.4	13222.4	47769.9	40022.2
64	5.6	486	58357.8	43.9	12816.3	45541.5	39528.2
65	5.6	465	55764.8	43.4	12431.6	43333.2	38958.1
66	5.8	443	53210.6	43.0	12066.4	41144.3	38309.5
67	5.9	422	50692.8	42.5	11718.7	38974.1	37579.9
68	6.0	402	48208.9	42.0	11386.8	36822.1	36766.4
69	6.2	381	45756.5	41.6	11068.9	34687.6	35865.7
70	6.4	361	43333.2	41.1	10763.2	32569.9	34874.1



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

$P_{A, max}$

41473.7 | lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$$EFP = 2 * P_A / H^2$$

EFP

41.0 pcf

Design Shoring for an Equivalent Fluid Pressure:

41 pcf

Geotechnologies, Inc.

Tiebacks Calculations

(Ref: Bowles, 1982)

Project: Bardas Investment Group

File No. 22064

Soil Parameters:

Weight of Soil	γ	120.00	lbs/ft ³
Friction Angle	ϕ	28.00	degrees
Cohesion	c	260.00	lbs/ft ²
Tieback Angle	α	20.00	degrees

Design Assumptions:

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

Ultimate Resistance:

Eq: $\pi \cdot d \cdot \gamma \cdot L \cdot h \cdot \cos(\alpha) \cdot \tan(\phi) + c \cdot \pi \cdot d \cdot L$

$R_{ult} = 24.55 \text{ kips}$

Allowable Resistance:

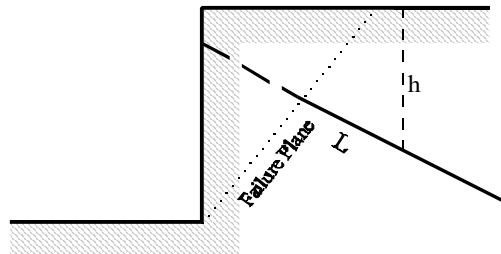
$R_{allow} = R_{ult} / F.S. = 16.37 \text{ kips}$

Allowable Skin Friction:

$R_{allow} / 2 / \pi \cdot r / L = 521.05 \text{ psf}$

Allowable Skin Friction Design Value

450 psf



Geotechnologies, Inc.

Tiebacks Calculations

(Ref: Bowles, 1982)

Project: Bardas Investment Group

File No. 22064

Soil Parameters:

Weight of Soil	γ	120.00	lbs/ft ³
Friction Angle	ϕ	28.00	degrees
Cohesion	c	260.00	lbs/ft ²
Tieback Angle	α	40.00	degrees

Design Assumptions:

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

Ultimate Resistance:

Eq: $\pi \cdot d \cdot \gamma \cdot L \cdot h \cdot \cos(\alpha) \cdot \tan(\phi) + c \cdot \pi \cdot d \cdot L$

$R_{ult} = 21.07 \text{ kips}$

Allowable Resistance:

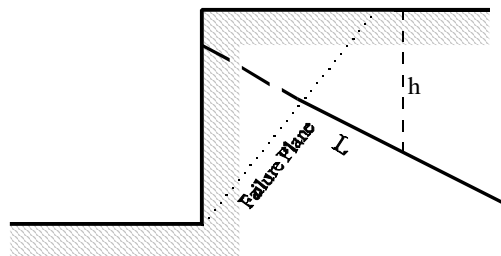
$R_{allow} = R_{ult} / F.S. = 14.05 \text{ kips}$

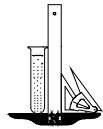
Allowable Skin Friction:

$R_{allow} / 2 / \pi \cdot r / L = 447.18 \text{ psf}$

Allowable Skin Friction Design Value

450 psf





Geotechnologies, Inc.

Project: Bardas Investment Company
File No.: 22064

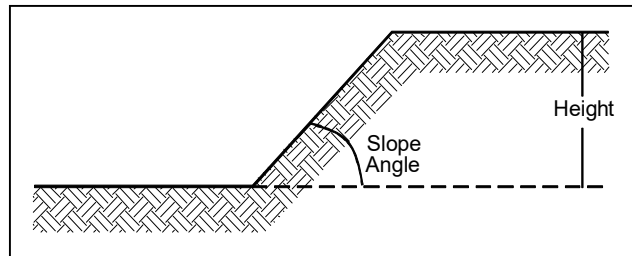
Slope Stability Calculations

Input

Soil Density (γ) 120 pcf
Friction Angle (ϕ) 28 degrees
Cohesion (c) 260 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
 $N_{(\text{vertical})}$ 0.169



Reference: Taylor's Chart (1937)

$$(\phi_d) = \text{ArcTan}[(\tan \phi) / \text{FS}]$$

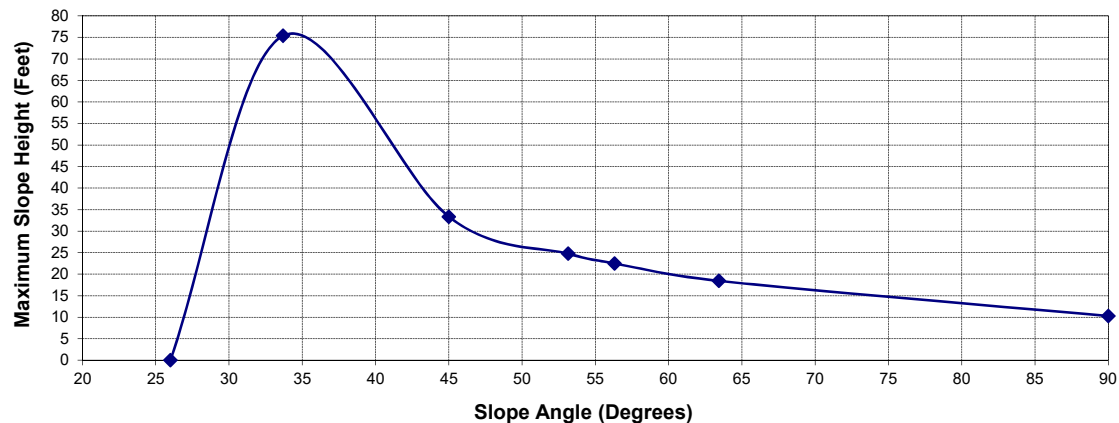
$$N = \frac{c}{(\gamma)(H)(\text{FS})}$$

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

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 1/2 : 1	33.69	75
1 : 1	45.00	33
3/4 : 1	53.13	25
1 : 1 1/2	56.30	23
1/2 : 1	63.43	18
Vertical	90.00	10

Slope Stability Critical Heights



BORING LOG NUMBER 1

5601-5667 Santa Monica Blvd. (LA), Owner, LLC

Date: 02/03/16

Elevation: 328'*

File No. 21151

Method: 8-inch diameter Hollow Stem Auger

km *Reference: City of Los Angeles Navigate LA Website

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				0 --		Surface Conditions: Asphalt Parking Lot
				-		4-inch Asphalt over 4-inch Base
				1 --		FILL: Sandy Silt, dark brown, moist, stiff
				-		
				2 --		
2.5	49	14.4	118.4	3 --		ML ALLUVIUM: Sandy Silt, dark grayish brown, moist, stiff
				-		
				4 --		
5	11	10.6	SPT	5 --		dark brown
				-		
				6 --		
				7 --		SM Silty Sand, dark yellowish brown, medium dense, fine grained
7.5	26	6.7	121.4	-		
				8 --		
				9 --		
10	15	4.8	SPT	10 --		
				-		
				11 --		yellowish brown, very dense, fine grained
				-		
12.5	33/6" 50/5.5"	12.0	120.3	12 --		
				-		
15	27	7.7	SPT	13 --		
				-		
				14 --		mottled dark and yellowish brown, fine to coarse grained, few gravel to 1/4"
				-		
17.5	36/6"	12.5	117.5	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
20	44	12.8	SPT	18 --		
				-		
				19 --		
				-		
				20 --		
				-		
22.5	61	31.9	85.0	21 --		
				-		BEDROCK (PUENTE FORMATION): Clayey Siltstone, yellow and grayish brown, moist, moderately hard, slightly fractured, moderately well bedded
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	28	15.5	SPT	25 --		
				-		

BORING LOG NUMBER 1

5601-5667 Santa Monica Blvd. (LA), Owner, LLC

File No. 21151

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
27.5	87	17.1	110.9	-		
				28 --		Silty Sandstone, pale yellow, fine grained
				-		
				29 --		
				-		
30	30	22.5	SPT	30 --		
				-		brownish yellow
				31 --		
				-		
				32 --		
32.5	40/6"	18.1	108.0	-		
				33 --		Sandstone, yellowish brown
				-		
				34 --		
				-		
35	56	18.4	SPT	35 --		
				-		wet
				36 --		
				-		
				37 --		
37.5	39/6" 50/4"	14.4	108.8	-		
				38 --		fine to medium grained
				-		
				39 --		
				-		
40	32	28.0	SPT	40 --		
				-		fine grained
				41 --		
				-		
				42 --		
42.5	90	33.0	89.9	-		
				43 --		Claystone, mottled yellowish brown and olive, massive, no fractures
				-		
				44 --		
				-		
45	29	31.8	SPT	45 --		
				-		
				46 --		
				-		
				47 --		
47.5	51	32.4	88.7	-		
				48 --		
				-		
				49 --		
				-		
50	26	29.3	SPT	50 --		
				-		

BORING LOG NUMBER 1

5601-5667 Santa Monica Blvd. (LA), Owner, LLC

File No. 21151

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
52.5	89	28.1	92.1	52 --		
				-		
				53 --		Claystone, interbedded with thin Sandstone layers, mottled yellowish and grayish brown, moist, well bedded, shallow dip
				-		
				54 --		
				-		
55	41	26.5	SPT	55 --		
				-		
				56 --		
				-		
				57 --		
57.5	81	23.3	98.6	-		
				58 --		
				-		
				59 --		
				-		
60	38	36.9	SPT	60 --		
				-		olive brown and light gray
				61 --		
				-		
62.5	70	26.0	95.9	62 --		
				-		
				63 --		interbedded Claystone (olive brown) and Sandstone (light gray), Sandstone layers 1/4" thick shallow bedding dip
				-		
				64 --		
				-		
65	35	32.1	SPT	65 --		
				-		
				66 --		
				-		
				67 --		
67.5	81	23.6	95.7	-		
				68 --		
				-		
				69 --		
				-		
70	37	32.2	SPT	70 --		
				-		Claystone, olive brown to grayish brown, moist, massive, interbedded Claystone (dark gray), and Sandstone (light gray)
				71 --		
				-		
				72 --		
72.5	72	26.5	93.7	-		
				73 --		
				-		
				74 --		
				-		
75	50	26.2	SPT	75 --		
				-		

BORING LOG NUMBER 1

5601-5667 Santa Monica Blvd. (LA), Owner, LLC

File No. 21151

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				76 --		
				-		
				77 --		
				-		
77.5	82	34.0	88.8	78 --		
				-		
				79 --		
				-		
80	40/6" 50/3"	26.2	SPT	80 --		
				-		interbedded Silty Sandstone, dark gray and dark brown, moderately hard
				81 --		
				-		
				82 --		
82.5	76	25.8	94.1	-		
				83 --		
				-		
				84 --		
				-		
85	75	18.2	SPT	85 --		
				-		
				86 --		
				-		
				87 --		
87.5	30/6" 50/2"	13.8	110.5	-		
				88 --		slight petroleum odor
				-		
				89 --		
				-		
90	47/6" 50/4"	20.5	SPT	90 --		
				-		interbedded Claystone and Silty Sandstone
				91 --		
				-		
				92 --		
92.5	100/6"	22.2	99.0	-		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				93 --		
				-		
				94 --		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
95	46/6" 50/3"	23.1	SPT	95 --		
				-		SPT=Standard Penetration Test
				96 --		
				-		
				97 --		
97.5	40/6"	16.4	102.3	-		
				98 --		shallow bedding dip moderately hard to hard
				-		
				99 --		
				-		
100	37/6" 50/4"	32.0	SPT	100 --		
				-		Total Depth 100 feet Water at 32 feet Fill to 3 feet

BORING LOG NUMBER 2

5601-5667 Santa Monica Blvd. (LA), Owner, LLC

Date: 02/04/16

Elevation: 329'*

File No. 21151

Method: 8-inch diameter Hollow Stem Auger

km *Reference: City of Los Angeles Navigate LA Website

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				0 --		Surface Conditions: Asphalt Parking Lot
				-		5-inch Asphalt over 4-inch Base
				1 --		
				-		
2.5	47	No Recovery		2 --		FILL: Sandy Silt, yellowish brown, moist, stiff
				-		
				3 --		
				-		
				4 --		
				-		
5	48	17.3	SPT	5 --		
				-		
				6 --		
				-	ML	ALLUVIUM: Sandy Silt, yellowish brown, moist, stiff
				7 --		
7.5	48	9.7	121.2	-		
				8 --	SM	Silty Sand, dark brown, moist, medium dense, fine grained
				-		
				9 --		
				-		
10	12	9.4	SPT	10 --		
				-	SP	Sand, medium dense, fine to medium grained
				11 --		
				-		
				12 --		
12.5	36	9.2	116.2	-		
				13 --		
				-		
				14 --		
				-		
15	26	14.0	SPT	15 --		
				-	SM	Silty Sand, brown, fine grained
				16 --		
				-		
				17 --		
17.5	72	19.7	109.8	-		
				18 --	ML	Sandy Silt, yellowish brown, very stiff, fine grained
				-		
				19 --		
				-		
20	34	9.3	SPT	20 --		
				-	SM	Silty Sand, medium dense
				21 --		
				-		
				22 --		
22.5	78	14.4	110.6	-		
				23 --		dark brown, moist, medium dense, fine grained
				-		
				24 --		
				-		
25	59	17.0	SPT	25 --		
				-		

BORING LOG NUMBER 2

5601-5667 Santa Monica Blvd. (LA), Owner, LLC

File No. 21151

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26 --		
				27 --		
27.5	83	8.5	122.2	28 --	SM/SP	Silty Sand to Sand, very dense, fine grained
				29 --		
30	21	9.9	SPT	30 --		
				31 --		BEDROCK (PUENTE FORMATION): Clayey Siltstone, mottled light reddish brown or light greenish gray, moderately hard, well bedded, shallow dip, slightly fractured
				32 --		
32.5	85	29.2	91.8	33 --		
				34 --		
35	27	29.1	SPT	35 --		
				36 --		
				37 --		
37.5	86	27.3	94.7	38 --		
				39 --		
40	26	32.2	SPT	40 --		Siltstone, dark and grayish brown, moist, moderately hard
				41 --		
				42 --		
42.5	37 50/5"	24.4	92.9	43 --		interbedded Claystone (greenish gray) and Sandstone (light reddish brown), moist, medium hard to hard
				44 --		
45	35	20.9	SPT	45 --		
				46 --		
				47 --		
47.5	40 50/4"	23.6	100.9	48 --		
				49 --		
50	32	29.5	SPT	50 --		Sandstone, yellowish brown, wet, moderately hard

BORING LOG NUMBER 2

5601-5667 Santa Monica Blvd. (LA), Owner, LLC

File No. 21151

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
52.5	46	23.3	105.2	52 --		
				-		
				53 --		interbedded Siltstone (greenish gray) to Sandstone (yellowish brown), wet, moderately hard
				-		
				54 --		
				-		
55	32	24.9	SPT	55 --		
				-		
				56 --		
				-		
57.5	40	25.3	97.7	57 --		
	50/4"			-		
				58 --		
				-		
				59 --		
				-		
60	30	31.5	SPT	60 --		
				-		
				61 --		
				-		
62.5	39	29.3	92.6	62 --		
	50/5"			-		
				63 --		
				-		
				64 --		
				-		
65	30	34.1	SPT	65 --		Sandstone (light gray) to Siltstone (greenish gray)
				-		
				66 --		
				-		
67.5	40	30.5	90.4	67 --		
	50/3"			-		
				68 --		Clayey Siltstone, moist, moderately hard
				-		
				69 --		
				-		
70	51	32.7	SPT	70 --		
				-		
				71 --		
				-		
72.5	75	27.9	93.8	72 --		
	50/5"			-		
				73 --		
				-		
				74 --		
				-		
75	56	31.2	SPT	75 --		
				-		

BORING LOG NUMBER 2

5601-5667 Santa Monica Blvd. (LA), Owner, LLC

File No. 21151

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				76 --		
				77 --		
77.5	79	26.8	94.7	78 --		bluish gray
				79 --		
80	37	37.0	SPT	80 --		
				81 --		
82.5	40 50/5"	32.3	87.8	82 --		
				83 --		dark gray
				84 --		
85	52	30.7	SPT	85 --		dark grayish brown
				86 --		
				87 --		
87.5	92 50/4:	23.3	96.8	88 --		
				89 --		
90	73 50/4"	28.4	SPT	90 --		
				91 --		
				92 --		
92.5	40 50/4"	24.0	101.4	93 --		Siltstone, gray to dark gray, moist, medium hard to hard
				94 --		
95	72	28.6	SPT	95 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				96 --		Used 8-inch diameter Hollow-Stem Auger
				97 --		140-lb. Automatic Hammer, 30-inch drop
97.5	45 50/3"	26.4	96.0	98 --		Modified California Sampler used unless otherwise noted
				99 --		SPT=Standard Penetration Test
				100 --		
100	28 50/5"	32.4	SPT	100 --		Total Depth 100 feet
						No Water
						Fill to 6 feet

BORING LOG NUMBER 3

5601-5667 Santa Monica Blvd. (LA), Owner, LLC

Date: 02/05/16

Elevation: 328.5'*

File No. 21151

Method: 8-inch diameter Hollow Stem Auger

km *Reference: City of Los Angeles Navigate LA Website

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				0 --		Surface Conditions: Asphalt Parking Lot
				-		5-inch Asphalt over 7-inch Base
				1 --		
				-		FILL: Sandy Silt, dark brown, moist, stiff
				2 --		
2.5	47	6.7	121.3	-		
				3 --		
				-	SM	ALLUVIUM: Silty Sand, dark brown, moist, medium dense, fine grained
				4 --		
5	48	11.4	118.0	-		
				5 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark brown, moist, medium dense, fine grained, stiff
				6 --		
				7 --		
7.5	80	15.8	116.3	-		
				8 --		
				-	ML	Sandy to Clayey Silt, dark brown, moist, very stiff
				9 --		
				-		
10	84	17.2	113.7	10 --		
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	43/6" 50/5.5"	16.4	114.3	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	76	10.2	123.4	20 --		
				-		
				21 --		
				-	SM/SP	Silty Sand to Sand, dark brown, moist, dense, fine grained
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	59	11.4	119.5	25 --		
				-		

						Silty Sand to Sand, dark brown, moist, medium dense, fine grained

BORING LOG NUMBER 3

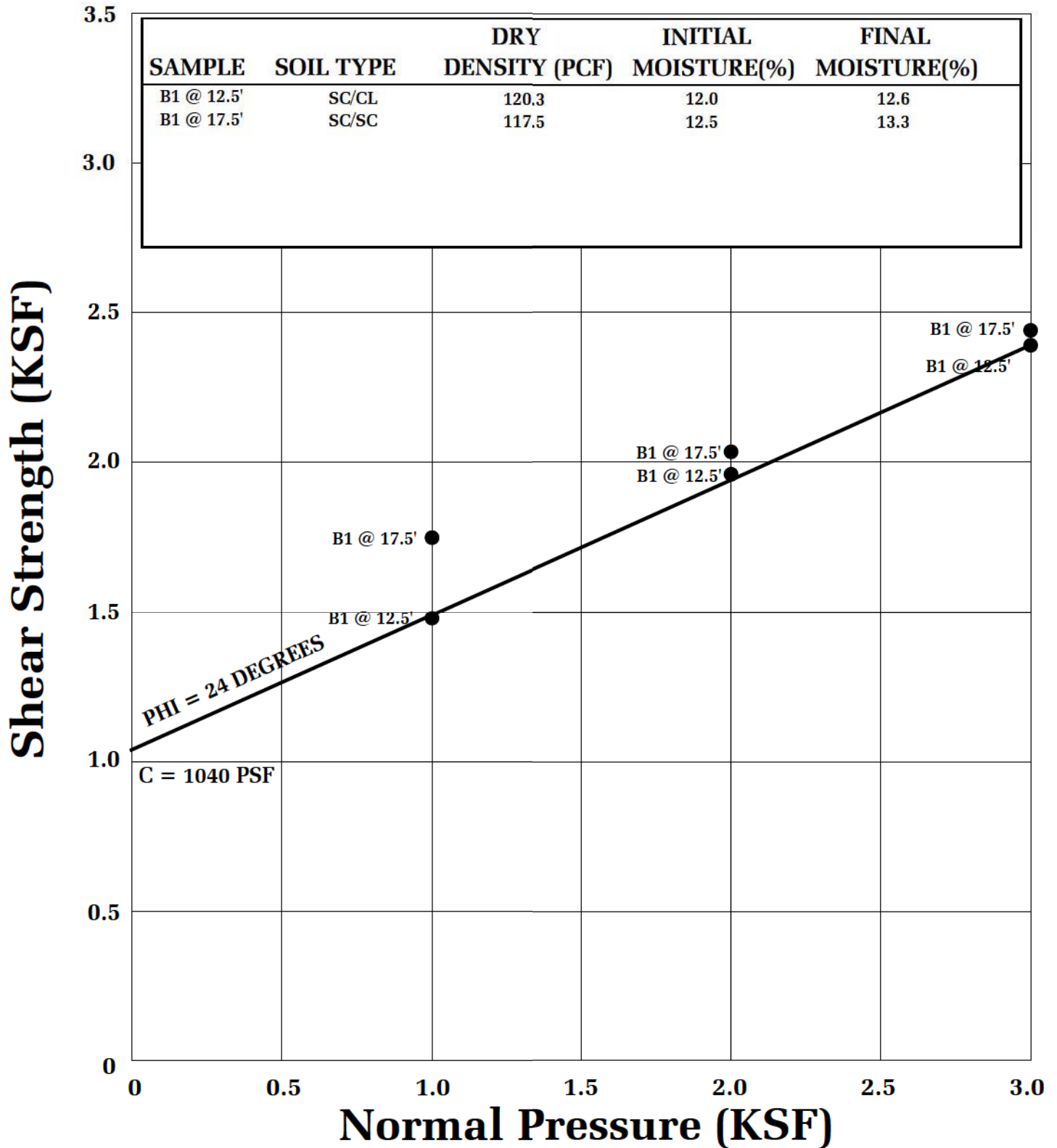
5601-5667 Santa Monica Blvd. (LA), Owner, LLC

File No. 21151

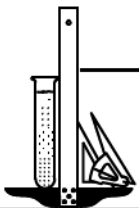
km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
30	84	20.2	112.5	30 --		
				-	ML/SM	Sandy Silt to Silty Sand, dark brown, moist, dense, fine grained, stiff
				31 --		
				-		
				32 --		
				-		
				33 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				-		
				34 --		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted Converted boring into developing well used 2" PVC pipe
35	85	No Recovery		35 --		
				-		
				36 --		
				-		
				37 --		2/26/16 - Water at 35.4 feet, pumped at 1.5 gal/min. for 20 min. Water at 41.1 feet recovered to 37.3 feet after 7 min.
				-		
				38 --		
				-		
				39 --		
				-		
40	88	No Recovery		40 --		
				-		
41	100/11"	24.9	92.7	41 --		
				-		
				42 --		BEDROCK (PUENTE FORMATION): Siltstone, dark and grayish brown, moist, moderately hard to hard
				-		
				43 --		
				-		
				44 --		
				-		
45	37/6" 50/5"	31.1	88.5	45 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				-		
				46 --		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted Converted boring into developing well used 2" PVC pipe
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
50	100/8"	19.5	108.8	50 --		Sandstone, dark brown, very moist to wet, moderately hard
				-		
						Total Depth 50 feet Water at 35.1 feet after well development Fill to 1 foot

ALLUVIUM



SHEAR TEST DIAGRAM



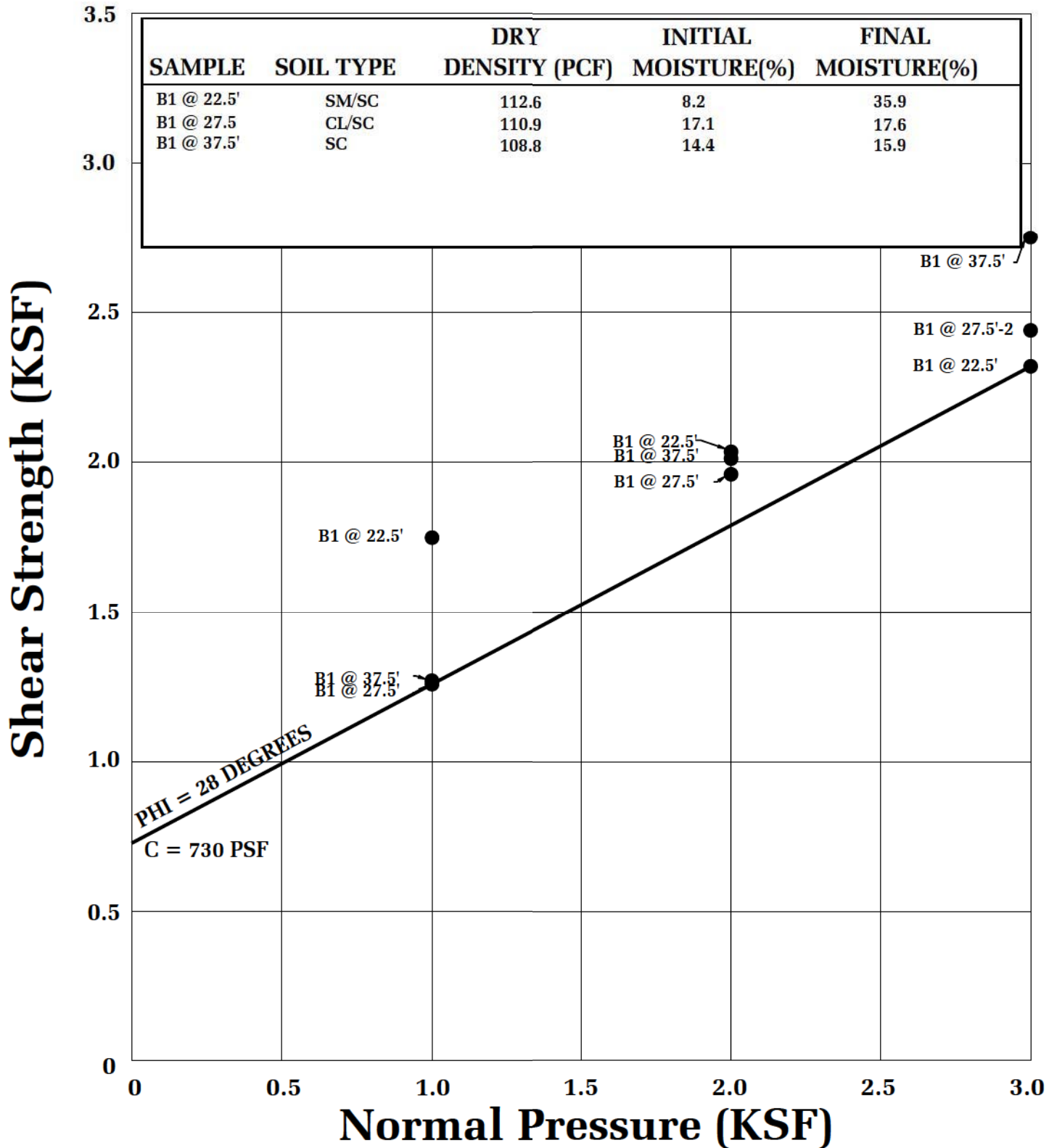
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(LA) OWNER, LLC

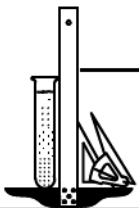
FILE NO. 21151

PLATE: B-1

BEDROCK



SHEAR TEST DIAGRAM



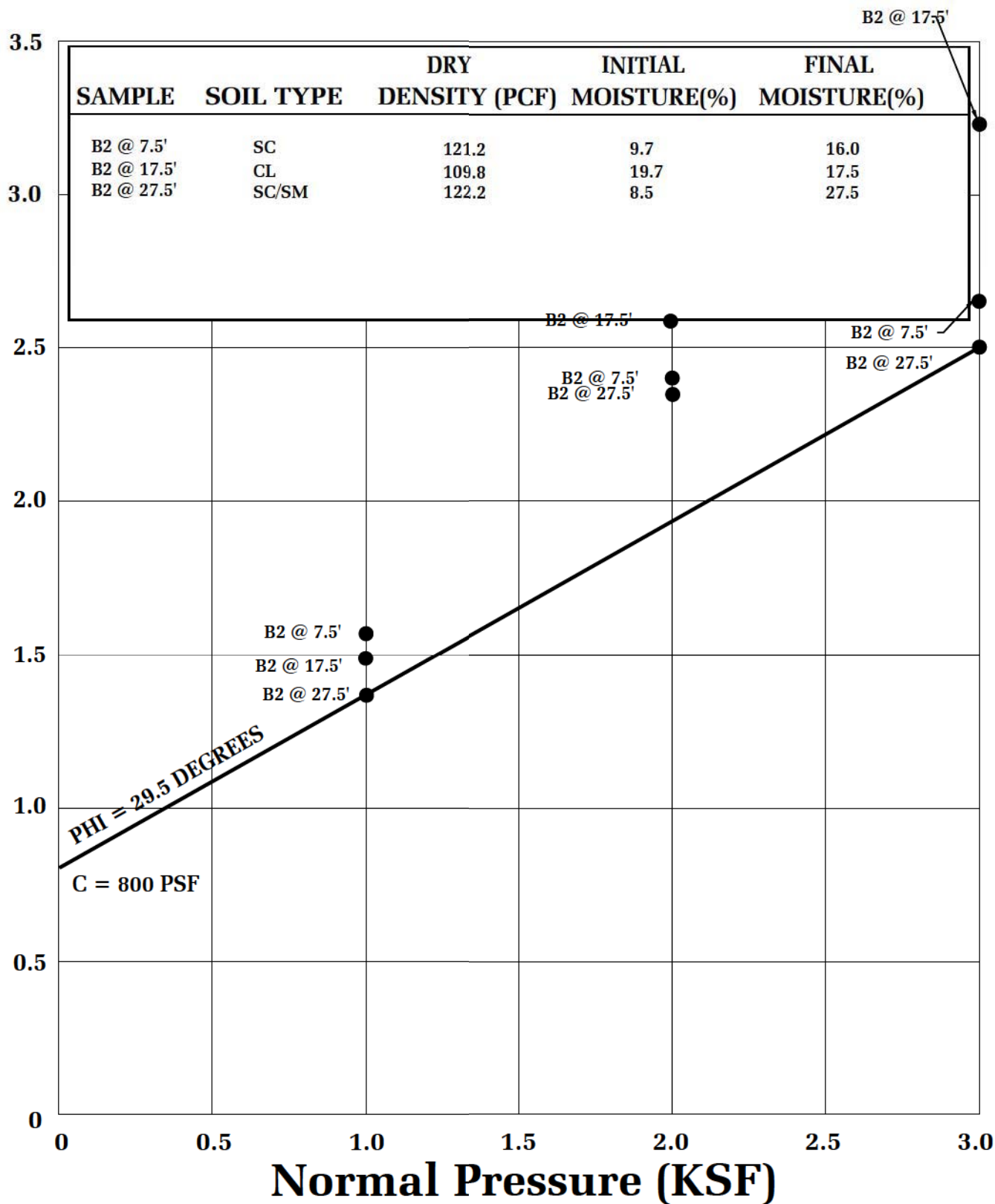
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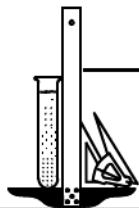
FILE NO. 21151

PLATE: B-2

Shear Strength (KSF)



SHEAR TEST DIAGRAM

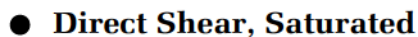


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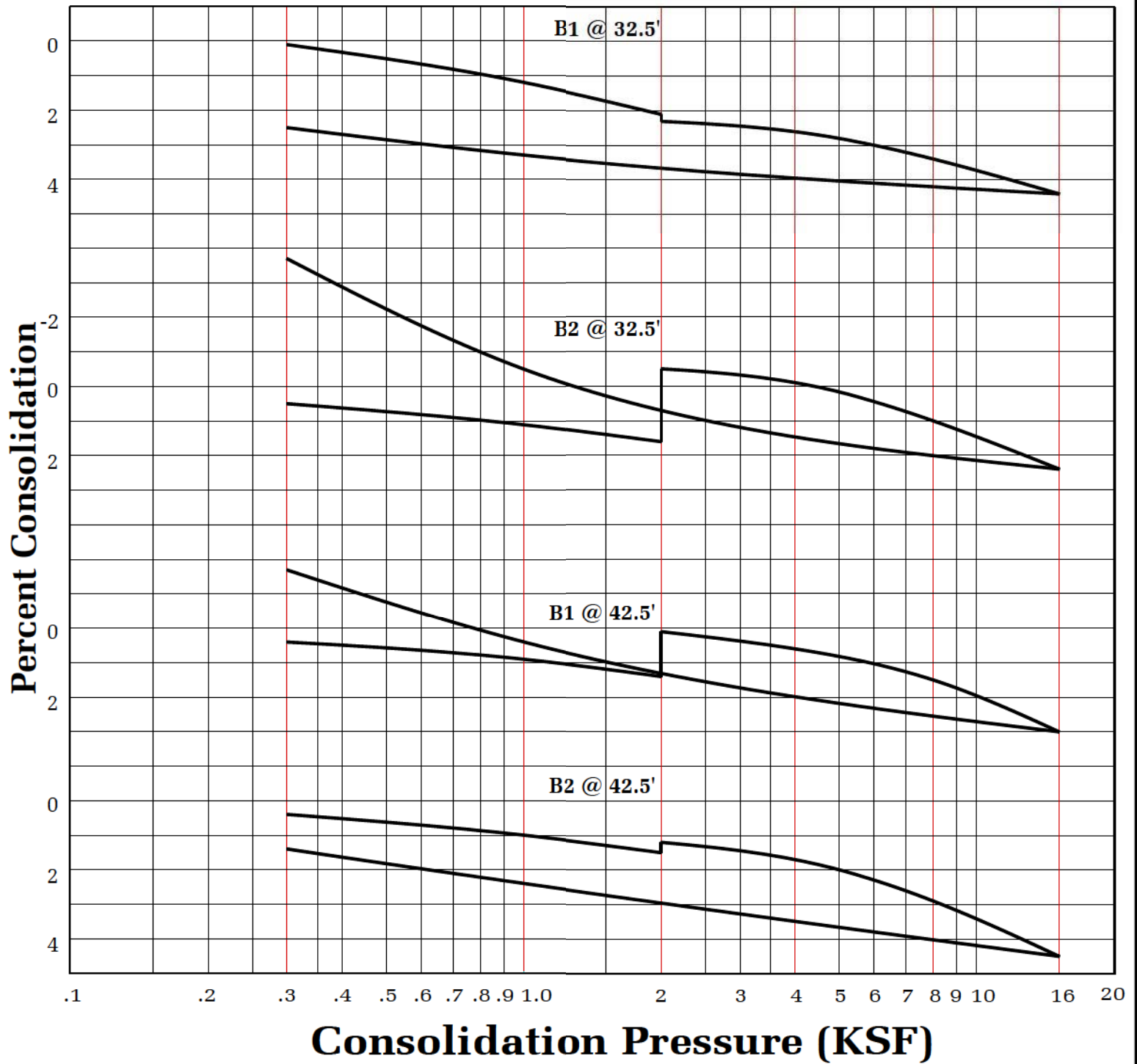
FILE NO. 21151

PLATE: B-3

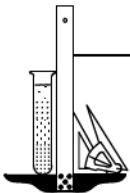


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WATER ADDED AT 2 KSF



CONSOLIDATION TEST



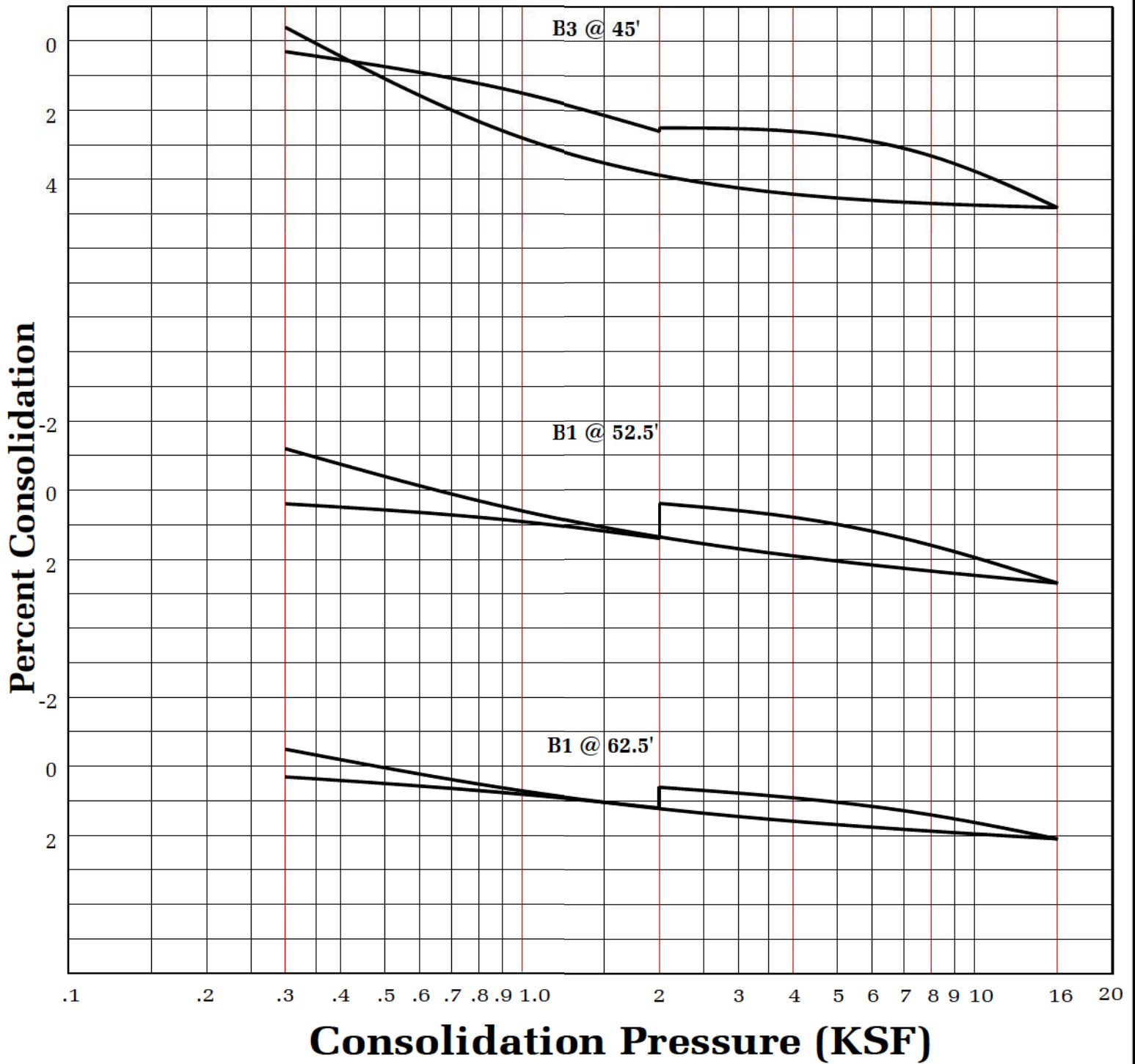
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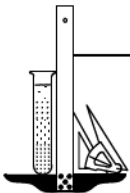
FILE NO. 21151

PLATE: C-1

WATER ADDED AT 2 KSF



CONSOLIDATION TEST



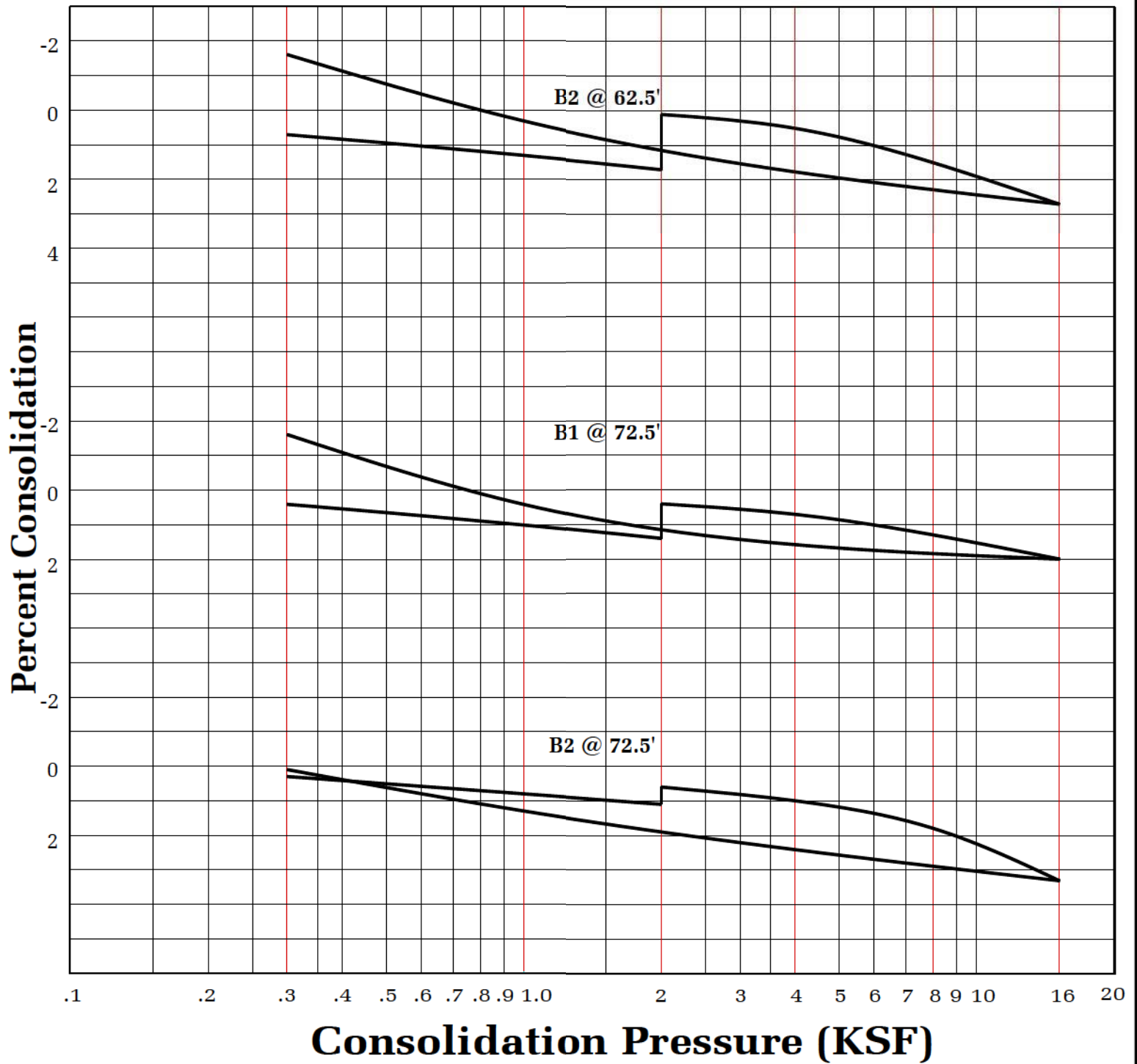
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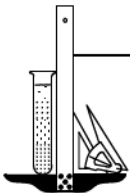
FILE NO. 21151

PLATE: C-2

WATER ADDED AT 2 KSF



CONSOLIDATION TEST



Geotechnologies, Inc.
Consulting Geotechnical Engineers

5601-5667 SANTA MONICA BLVD.
(LA) OWNER, LLC

FILE NO. 21151

PLATE: C-3

ASTM D-1557

SAMPLE	B1 @ 1-5'
SOIL TYPE:	SM/CL
MAXIMUM DENSITY pcf.	118.9
OPTIMUM MOISTURE %	14.2

ASTM D 4829

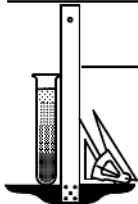
SAMPLE	B1 @ 1-5'
SOIL TYPE:	SM/CL
EXPANSION INDEX UBC STANDARD 18-2	142
EXPANSION CHARACTER	<u>CRITICAL</u>

SULFATE CONTENT

SAMPLE	B1 @ 1-5'	B1 @ 32.5'	B1 @ 42.5'	B1 @ 52.5'	B1 @ 62.5'	B1 @ 72.5'
SULFATE CONTENT: (percentage by weight)	< 0.10 %	< 0.10 %	< 0.10 %	< 0.10 %	< 0.10 %	< 0.10 %

SAMPLE	B2 @ 32.5'	B2 @ 40'	B2 @ 62.5'	B2 @ 72.5'	B3 @ 45'	B3 @ 50'
SULFATE CONTENT: (percentage by weight)	< 0.10 %	< 0.10 %	< 0.10 %	< 0.10 %	< 0.10 %	< 0.10 %

COMPACTION/EXPANSION/SULFATE DATA SHEET



Geotechnologies, Inc.
Consulting Geotechnical Engineers

5601-5667 SANTA MONICA BLVD.
(LA) OWNER, LLC

FILE NO. 21151

PLATE: D



Geotechnologies, Inc.

Project: 5601-5667 Santa Monica Blvd, LLC

File No.: 21151

Description: Liquefaction Analysis (PGAM)

Boring No: BI

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	6.9
Peak Ground Horizontal Acceleration, PGA (g):	1.07
Calculated Mag. Wtg. Factor:	1.171

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	32.0
Historically Highest Groundwater Level* (ft):	20.0
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)	Vertical Stress σ_{ve} (psf)	Effective Vert. Stress σ'_{ve} (psf)	Fines Corrected $(N_1)_{60-es}$	Stress Reduction Coeff, r_d	Cyclic Shear Ratio CSR	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Liquefaction Settlement ΔS_i (inches)
1	135.4	Unsaturated	Unsaturated	11	5	0.0	0	135.4	135.4	24.2	1.00	0.698	0.351	Non-Liq.	0.00
2	135.4	Unsaturated	Unsaturated	11	5	0.0	0	270.8	270.8	24.2	1.00	0.696	0.351	Non-Liq.	0.00
3	135.4	Unsaturated	Unsaturated	11	5	0.0	0	406.2	406.2	24.2	1.00	0.694	0.351	Non-Liq.	0.00
4	135.4	Unsaturated	Unsaturated	11	5	0.0	0	541.6	541.6	24.2	0.99	0.692	0.351	Non-Liq.	0.00
5	135.4	Unsaturated	Unsaturated	11	5	0.0	0	677.0	677.0	24.4	0.99	0.689	0.355	Non-Liq.	0.00
6	135.4	Unsaturated	Unsaturated	11	5	0.0	0	812.4	812.4	22.8	0.99	0.687	0.317	Non-Liq.	0.00
7	135.4	Unsaturated	Unsaturated	11	5	0.0	0	947.8	947.8	21.3	0.98	0.684	0.287	Non-Liq.	0.00
8	129.5	Unsaturated	Unsaturated	15	10	0.0	0	1077.3	1077.3	27.6	0.98	0.682	0.474	Non-Liq.	0.00
9	129.5	Unsaturated	Unsaturated	15	10	0.0	0	1206.8	1206.8	28.1	0.98	0.679	0.498	Non-Liq.	0.00
10	129.5	Unsaturated	Unsaturated	15	10	0.0	0	1336.3	1336.3	26.9	0.97	0.676	0.433	Non-Liq.	0.00
11	129.5	Unsaturated	Unsaturated	15	10	0.0	0	1465.8	1465.8	25.8	0.97	0.673	0.385	Non-Liq.	0.00
12	129.5	Unsaturated	Unsaturated	15	10	0.0	0	1595.3	1595.3	24.8	0.96	0.670	0.350	Non-Liq.	0.00
13	134.7	Unsaturated	Unsaturated	15	10	0.0	0	1730.0	1730.0	23.9	0.96	0.667	0.321	Non-Liq.	0.00
14	134.7	Unsaturated	Unsaturated	15	10	0.0	0	1864.7	1864.7	23.0	0.95	0.664	0.298	Non-Liq.	0.00
15	134.7	Unsaturated	Unsaturated	27	15	0.0	0	1999.4	1999.4	48.6	0.95	0.661	2.000	Non-Liq.	0.00
16	134.7	Unsaturated	Unsaturated	27	15	0.0	0	2134.1	2134.1	47.8	0.95	0.658	2.000	Non-Liq.	0.00
17	134.7	Unsaturated	Unsaturated	27	15	0.0	0	2268.8	2268.8	47.0	0.94	0.654	2.000	Non-Liq.	0.00
18	132.2	Unsaturated	Unsaturated	27	15	0.0	0	2401.0	2401.0	46.3	0.94	0.651	2.000	Non-Liq.	0.00
19	132.2	Unsaturated	Unsaturated	27	15	0.0	0	2533.2	2533.2	45.7	0.93	0.647	2.000	Non-Liq.	0.00
20	132.2	Unsaturated	Unsaturated	44	20	0.0	0	2665.4	2665.4	73.5	0.93	0.644	2.000	Non-Liq.	0.00
21	132.2	Unsaturated	Saturated	44	20	0.0	0	2797.6	2735.2	72.5	0.92	0.655	2.000	3.1	0.00
22	132.2	Unsaturated	Saturated	44	20	0.0	0	2929.8	2805.0	71.6	0.92	0.665	2.000	3.0	0.00
Total Liquefaction Settlement, S =														0.00 inches	



Geotechnologies, Inc.

Project: 5601-5667 Santa Monica Blvd, LLC

File No.: 21151

Description: Liquefaction Analysis (2/3 PG_{AM})

Boring No: B1

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	6.4
Peak Ground Horizontal Acceleration, PGA (g):	0.71
Calculated Mag. Wtg. Factor:	1.335

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	32.0
Historically Highest Groundwater Level* (ft):	20.0
Unit Weight of Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

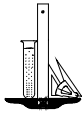
BOREHOLE AND SAMPLER INFORMATION:

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)	Vertical Stress σ_{ve} (psf)	Effective Vert. Stress σ'_{ve} (psf)	Fines Corrected $(N_1)_{60-es}$	Stress Reduction Coeff, r_d	Cyclic Shear Ratio CSR	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Liquefaction Settlement ΔS_i (inches)
1	135.4	Unsaturated	Unsaturated	11	5	0.0	0	135.4	135.4	24.2	1.00	0.463	0.401	Non-Liq.	0.00
2	135.4	Unsaturated	Unsaturated	11	5	0.0	0	270.8	270.8	24.2	1.00	0.462	0.401	Non-Liq.	0.00
3	135.4	Unsaturated	Unsaturated	11	5	0.0	0	406.2	406.2	24.2	1.00	0.460	0.401	Non-Liq.	0.00
4	135.4	Unsaturated	Unsaturated	11	5	0.0	0	541.6	541.6	24.2	0.99	0.458	0.401	Non-Liq.	0.00
5	135.4	Unsaturated	Unsaturated	11	5	0.0	0	677.0	677.0	24.4	0.99	0.456	0.405	Non-Liq.	0.00
6	135.4	Unsaturated	Unsaturated	11	5	0.0	0	812.4	812.4	22.8	0.98	0.454	0.361	Non-Liq.	0.00
7	135.4	Unsaturated	Unsaturated	11	5	0.0	0	947.8	947.8	21.3	0.98	0.452	0.327	Non-Liq.	0.00
8	129.5	Unsaturated	Unsaturated	15	10	0.0	0	1077.3	1077.3	27.6	0.97	0.450	0.541	Non-Liq.	0.00
9	129.5	Unsaturated	Unsaturated	15	10	0.0	0	1206.8	1206.8	28.1	0.97	0.447	0.567	Non-Liq.	0.00
10	129.5	Unsaturated	Unsaturated	15	10	0.0	0	1336.3	1336.3	26.9	0.96	0.445	0.493	Non-Liq.	0.00
11	129.5	Unsaturated	Unsaturated	15	10	0.0	0	1465.8	1465.8	25.8	0.96	0.443	0.439	Non-Liq.	0.00
12	129.5	Unsaturated	Unsaturated	15	10	0.0	0	1595.3	1595.3	24.8	0.95	0.440	0.398	Non-Liq.	0.00
13	134.7	Unsaturated	Unsaturated	15	10	0.0	0	1730.0	1730.0	23.9	0.95	0.438	0.366	Non-Liq.	0.00
14	134.7	Unsaturated	Unsaturated	15	10	0.0	0	1864.7	1864.7	23.0	0.94	0.435	0.340	Non-Liq.	0.00
15	134.7	Unsaturated	Unsaturated	27	15	0.0	0	1999.4	1999.4	48.6	0.94	0.433	2.000	Non-Liq.	0.00
16	134.7	Unsaturated	Unsaturated	27	15	0.0	0	2134.1	2134.1	47.8	0.93	0.430	2.000	Non-Liq.	0.00
17	134.7	Unsaturated	Unsaturated	27	15	0.0	0	2268.8	2268.8	47.0	0.93	0.427	2.000	Non-Liq.	0.00
18	132.2	Unsaturated	Unsaturated	27	15	0.0	0	2401.0	2401.0	46.3	0.92	0.425	2.000	Non-Liq.	0.00
19	132.2	Unsaturated	Unsaturated	27	15	0.0	0	2533.2	2533.2	45.7	0.91	0.422	2.000	Non-Liq.	0.00
20	132.2	Unsaturated	Unsaturated	44	20	0.0	0	2665.4	2665.4	73.5	0.91	0.419	2.000	Non-Liq.	0.00
21	132.2	Unsaturated	Saturated	44	20	0.0	0	2797.6	2735.2	72.5	0.90	0.426	2.000	4.7	0.00
22	132.2	Unsaturated	Saturated	44	20	0.0	0	2929.8	2805.0	71.6	0.90	0.432	2.000	4.6	0.00
Total Liquefaction Settlement, S =														0.00 inches	



Geotechnologies, Inc.

Project: 5601-5667 Santa Monica Blvd, LLC

File No.: 21151

Description: Liquefaction Analysis (PGAM)

Boring No: B2

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	6.9
Peak Ground Horizontal Acceleration, PGA (g):	1.07
Calculated Mag.Wtg.Factor:	1.171

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	101.0
Historically Highest Groundwater Level* (ft):	20.0
Unit Weight of Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

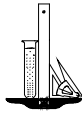
BOREHOLE AND SAMPLER INFORMATION:

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)	Vertical Stress σ_{ve} (psf)	Effective Vert. Stress σ'_{ve} (psf)	Fines Corrected $(N_1)_{60-cs}$	Stress Reduction Coeff. r_d	Cyclic Shear Ratio CSR	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Liquefaction Settlement ΔS_i (inches)
1	132.9	Unsaturated	Unsaturated	48	5	0.0	0	132.9	132.9	114.4	1.00	0.698	2.000	Non-Liq.	0.00
2	132.9	Unsaturated	Unsaturated	48	5	0.0	0	265.8	265.8	114.4	1.00	0.696	2.000	Non-Liq.	0.00
3	132.9	Unsaturated	Unsaturated	48	5	0.0	0	398.7	398.7	104.3	1.00	0.694	2.000	Non-Liq.	0.00
4	132.9	Unsaturated	Unsaturated	48	5	0.0	0	531.6	531.6	96.7	0.99	0.692	2.000	Non-Liq.	0.00
5	132.9	Unsaturated	Unsaturated	48	5	0.0	0	664.5	664.5	97.2	0.99	0.689	2.000	Non-Liq.	0.00
6	132.9	Unsaturated	Unsaturated	48	5	0.0	0	797.4	797.4	92.7	0.99	0.687	2.000	Non-Liq.	0.00
7	132.9	Unsaturated	Unsaturated	48	5	0.0	0	930.3	930.3	89.0	0.98	0.684	2.000	Non-Liq.	0.00
8	132.9	Unsaturated	Unsaturated	48	5	0.0	0	1063.2	1063.2	85.9	0.98	0.682	2.000	Non-Liq.	0.00
9	132.9	Unsaturated	Unsaturated	48	5	0.0	0	1196.1	1196.1	88.5	0.98	0.679	2.000	Non-Liq.	0.00
10	126.9	Unsaturated	Unsaturated	12	10	0.0	0	1323.0	1323.0	21.3	0.97	0.676	0.277	Non-Liq.	0.00
11	126.9	Unsaturated	Unsaturated	12	10	0.0	0	1449.9	1449.9	20.4	0.97	0.673	0.259	Non-Liq.	0.00
12	126.9	Unsaturated	Unsaturated	12	10	0.0	0	1576.8	1576.8	19.5	0.96	0.670	0.244	Non-Liq.	0.00
13	126.9	Unsaturated	Unsaturated	12	10	0.0	0	1703.7	1703.7	18.8	0.96	0.667	0.231	Non-Liq.	0.00
14	126.9	Unsaturated	Unsaturated	12	10	0.0	0	1830.6	1830.6	18.1	0.95	0.664	0.220	Non-Liq.	0.00
15	126.9	Unsaturated	Unsaturated	26	15	0.0	0	1957.5	1957.5	47.1	0.95	0.661	2.000	Non-Liq.	0.00
16	126.9	Unsaturated	Unsaturated	26	15	0.0	0	2084.4	2084.4	46.3	0.95	0.658	2.000	Non-Liq.	0.00
17	126.9	Unsaturated	Unsaturated	26	15	0.0	0	2211.3	2211.3	45.6	0.94	0.654	2.000	Non-Liq.	0.00
18	131.4	Unsaturated	Unsaturated	26	15	0.0	0	2342.7	2342.7	44.9	0.94	0.651	2.000	Non-Liq.	0.00
19	131.4	Unsaturated	Unsaturated	26	15	0.0	0	2474.1	2474.1	44.2	0.93	0.647	2.000	Non-Liq.	0.00
20	126.5	Unsaturated	Unsaturated	34	20	0.0	0	2600.6	2600.6	57.1	0.93	0.644	2.000	Non-Liq.	0.00
21	126.5	Unsaturated	Saturated	34	20	0.0	0	2727.1	2664.7	56.4	0.92	0.655	2.000	3.1	0.00
22	126.5	Unsaturated	Saturated	34	20	0.0	0	2853.6	2728.8	55.7	0.92	0.666	2.000	3.0	0.00
23	126.5	Unsaturated	Saturated	34	20	0.0	0	2980.1	2792.9	55.1	0.91	0.676	2.000	3.0	0.00
24	126.5	Unsaturated	Saturated	34	20	0.0	0	3106.6	2857.0	54.5	0.90	0.684	2.000	2.9	0.00
25	126.5	Unsaturated	Saturated	59	25	0.0	0	3233.1	2921.1	93.6	0.90	0.693	2.000	2.9	0.00
26	126.5	Unsaturated	Saturated	59	25	0.0	0	3359.6	2985.2	92.7	0.89	0.700	2.000	2.9	0.00
27	126.5	Unsaturated	Saturated	59	25	0.0	0	3486.1	3049.3	91.8	0.89	0.707	1.996	2.8	0.00
28	132.6	Unsaturated	Saturated	59	25	0.0	0	3618.7	3119.5	95.7	0.88	0.713	1.970	2.8	0.00
29	132.6	Unsaturated	Saturated	59	25	0.0	0	3751.3	3189.7	94.8	0.88	0.718	1.945	2.7	0.00
Total Liquefaction Settlement, S =														0.00 inches	



Geotechnologies, Inc.

Project: 5601-5667 Santa Monica Blvd, LLC

File No.: 21151

Description: Liquefaction Analysis (2/3 PGA_M)

Boring No: B2

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	6.4
Peak Ground Horizontal Acceleration, PGA (g):	0.71
Calculated Mag. Wtg. Factor:	1.335

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	101.0
Historically Highest Groundwater Level* (ft):	20.0
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)	Vertical Stress σ_{ve} (psf)	Effective Vert. Stress σ'_{ve} (psf)	Fines Corrected $(N_1)_{60-CS}$	Stress Reduction Coeff. r_d	Cyclic Shear Ratio CSR	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Liquefaction Settlement ΔS_i (inches)
1	132.9	Unsaturated	Unsaturated	48	5	0.0	0	132.9	132.9	114.4	1.00	0.463	2.000	Non-Liq.	0.00
2	132.9	Unsaturated	Unsaturated	48	5	0.0	0	265.8	265.8	114.4	1.00	0.462	2.000	Non-Liq.	0.00
3	132.9	Unsaturated	Unsaturated	48	5	0.0	0	398.7	398.7	104.3	1.00	0.460	2.000	Non-Liq.	0.00
4	132.9	Unsaturated	Unsaturated	48	5	0.0	0	531.6	531.6	96.7	0.99	0.458	2.000	Non-Liq.	0.00
5	132.9	Unsaturated	Unsaturated	48	5	0.0	0	664.5	664.5	97.2	0.99	0.456	2.000	Non-Liq.	0.00
6	132.9	Unsaturated	Unsaturated	48	5	0.0	0	797.4	797.4	92.7	0.98	0.454	2.000	Non-Liq.	0.00
7	132.9	Unsaturated	Unsaturated	48	5	0.0	0	930.3	930.3	89.0	0.98	0.452	2.000	Non-Liq.	0.00
8	132.9	Unsaturated	Unsaturated	48	5	0.0	0	1063.2	1063.2	85.9	0.97	0.450	2.000	Non-Liq.	0.00
9	132.9	Unsaturated	Unsaturated	48	5	0.0	0	1196.1	1196.1	88.5	0.97	0.447	2.000	Non-Liq.	0.00
10	126.9	Unsaturated	Unsaturated	12	10	0.0	0	1323.0	1323.0	21.3	0.96	0.445	0.316	Non-Liq.	0.00
11	126.9	Unsaturated	Unsaturated	12	10	0.0	0	1449.9	1449.9	20.4	0.96	0.443	0.295	Non-Liq.	0.00
12	126.9	Unsaturated	Unsaturated	12	10	0.0	0	1576.8	1576.8	19.5	0.95	0.440	0.278	Non-Liq.	0.00
13	126.9	Unsaturated	Unsaturated	12	10	0.0	0	1703.7	1703.7	18.8	0.95	0.438	0.263	Non-Liq.	0.00
14	126.9	Unsaturated	Unsaturated	12	10	0.0	0	1830.6	1830.6	18.1	0.94	0.435	0.251	Non-Liq.	0.00
15	126.9	Unsaturated	Unsaturated	26	15	0.0	0	1957.5	1957.5	47.1	0.94	0.433	2.000	Non-Liq.	0.00
16	126.9	Unsaturated	Unsaturated	26	15	0.0	0	2084.4	2084.4	46.3	0.93	0.430	2.000	Non-Liq.	0.00
17	126.9	Unsaturated	Unsaturated	26	15	0.0	0	2211.3	2211.3	45.6	0.93	0.427	2.000	Non-Liq.	0.00
18	131.4	Unsaturated	Unsaturated	26	15	0.0	0	2342.7	2342.7	44.9	0.92	0.425	2.000	Non-Liq.	0.00
19	131.4	Unsaturated	Unsaturated	26	15	0.0	0	2474.1	2474.1	44.2	0.91	0.422	2.000	Non-Liq.	0.00
20	126.5	Unsaturated	Unsaturated	34	20	0.0	0	2600.6	2600.6	57.1	0.91	0.419	2.000	Non-Liq.	0.00
21	126.5	Unsaturated	Saturated	34	20	0.0	0	2727.1	2664.7	56.4	0.90	0.426	2.000	4.7	0.00
22	126.5	Unsaturated	Saturated	34	20	0.0	0	2853.6	2728.8	55.7	0.90	0.432	2.000	4.6	0.00
23	126.5	Unsaturated	Saturated	34	20	0.0	0	2980.1	2792.9	55.1	0.89	0.438	2.000	4.6	0.00
24	126.5	Unsaturated	Saturated	34	20	0.0	0	3106.6	2857.0	54.5	0.88	0.443	2.000	4.5	0.00
25	126.5	Unsaturated	Saturated	59	25	0.0	0	3233.1	2921.1	93.6	0.88	0.447	2.000	4.5	0.00
26	126.5	Unsaturated	Saturated	59	25	0.0	0	3359.6	2985.2	92.7	0.87	0.452	2.000	4.4	0.00
27	126.5	Unsaturated	Saturated	59	25	0.0	0	3486.1	3049.3	91.8	0.86	0.455	2.000	4.4	0.00
28	132.6	Unsaturated	Saturated	59	25	0.0	0	3618.7	3119.5	95.7	0.86	0.458	2.000	4.4	0.00
29	132.6	Unsaturated	Saturated	59	25	0.0	0	3751.3	3189.7	94.8	0.85	0.461	2.000	4.3	0.00
Total Liquefaction Settlement, S =														0.00 inches	

BORING 1



GEOCON
INLAND EMPIRE, INC

Project No.: A8284-06-01
Client: Continental Management Group, LLC
Location: 5661 Santa Monica Blvd.
Hollywood, CA
Sears Hollywood

Excavation Date: March 29, 2005
Excavation Method: Hollow Stem Auger
Boring Diameter: 8 inches
Sampling Method: Cal-Mod
Hammer Drop: 30 inches
Hammer Weight: 140 lbs.

Sample Type	Depth (feet)	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	Depth (feet)	USCS Class.	Surface Condition: Asphalt Paving Description
	5	23	9.0	120.1	0 --		Asphalt: 4" Fill: Clayey Sand, dense, dark brown, moist, fine grained with trace medium grained
					1 --		
					2 --		
					3 --		
					4 --		
					5 --		
					6 --	SC/SM	Alluvium: Clayey to Silty Sand, dense, brown, moist, fine grained, trace ash, trace caliche, trace pinhole porosity
					7 --		
					8 --		
					9 --		
					10 --		
					11 --		
					12 --		
					13 --		
					14 --	SM	Silty Sand, dense, brown with dark yellowish brown staining, moist, fine to coarse grained, trace gravel
	15	31	9.5	110.2	15 --		
					16 --		
					17 --		
					18 --		
					19 --		
					20 --		

- Ring Sample from California Modified Sampler

Figure A-1a

BORING 1 (continued)



GEOCON

INLAND EMPIRE, INC

Project No.: A8284-06-01
 Client: Continental Management Group, LLC
 Location: 5661 Santa Monica Blvd.
 Hollywood, CA
 Sears Hollywood

Excavation Date: March 29, 2005
 Excavation Method: Hollow Stem Auger
 Boring Diameter: 8 inches
 Sampling Method: Cal-Mod
 Hammer Drop: 30 inches
 Hammer Weight: 140 lbs.

Sample Type	Depth (feet)	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	Depth (feet)	USCS Class.	Description
	25	20	24.3	98.7	20 --		
					21 --		
					22 --		
					23 --		
					24 --		more silty, medium dense, very moist to wet, fine grained groundwater seepage
					25 --		
					26 --		
					27 --		
					28 --		
					29 --		
					30 --		
					31 --		
					32 --		
	35	58	17.2	109.0	33 --	SC/SM	Silty to Clayey Sand, very dense, mottled brown and dark olive brown, moist, fine grained
					34 --		
					35 --		
					36 --		
	37.5	77	15.0	116.4	37 --	CL	Silty Clay, stiff, mottled light olive brown and brown, moist, trace fine grained sand, trace caliche
					38 --		
					39 --		
	40	45	13.2	118.5	40 --	SC/SM	Clayey to Silty Sand, very dense, brown, moist, fine grained

- Ring Sample from California Modified Sampler

Figure A-1b



GEOCON
INLAND EMPIRE, INC

BORING 1 (continued)

Project No.: A8284-06-01
Client: Continental Management Group, LLC
Location: 5661 Santa Monica Blvd.
Hollywood, CA
Sears Hollywood

Excavation Date: March 29, 2005
Excavation Method: Hollow Stem Auger
Boring Diameter: 8 inches
Sampling Method: Cal-Mod
Hammer Drop: 30 inches
Hammer Weight: 140 lbs.

Sample Type	Depth (feet)	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	Depth (feet)	USCS Class.	Surface Condition: Asphalt Paving Description
					40 --		
					41 --		
					42 --		
	42.5	45	15.4	115.4	43 --		
					44 --		
	45	52	15.4	114.7	45 --		
					46 --		
					47 --		
					48 --		
					49 --	SM	Silty Sand, very dense, dark yellowish brown, very moist to wet, fine grained with trace coarse grained
	50	92	25.0	99.8	50 --		
					51 --		
					52 --		
					53 --		
					54 --		
	55	89	24.9	95.4	55 --		less silty
					56 --		
					57 --		
					58 --		
					59 --		
					60 --		



- Ring Sample from California Modified Sampler



- Standard Penetrometer Test (SPT) Sampler

Figure A-1c

BORING 1 (continued)



GEOCON
INLAND EMPIRE, INC

Project No.: A8284-06-01
Client: Continental Management Group, LLC
Location: 5661 Santa Monica Blvd.
Hollywood, CA
Sears Hollywood

Excavation Date: March 29, 2005
Excavation Method: Hollow Stem Auger
Boring Diameter: 8 inches
Sampling Method: Cal-Mod
Hammer Drop: 30 inches
Hammer Weight: 140 lbs.

Sample Type	Depth (feet)	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	Depth (feet)	USCS Class.	Surface Condition:	Description
	65	68	21.2	106.2	60 --			
					61 --			
					62 --			
					63 --			
					64 --			more silty, mottled olive brown and dark yellowish brown
					65 --			
					66 --			End at 65 feet
					67 --			Fill to 5 feet
					68 --			Groundwater at 23 feet
					69 --			
					70 --			
					71 --			
					72 --			
					73 --			
					74 --			
					75 --			
					76 --			
					77 --			
					78 --			
					79 --			
					80 --			



- Ring Sample from California Modified Sampler



- Standard Penetrometer Test (SPT) Sampler

Figure A-1d

BORING 2



GEOCON
INLAND EMPIRE, INC

Project No.: A8284-06-01
Client: Continental Management Group, LLC
Location: 5661 Santa Monica Blvd.
Hollywood, CA
Sears Hollywood

Excavation Date: March 29, 2005
Excavation Method: Hollow Stem Auger
Boring Diameter: 7 inches
Sampling Method: Cal-Mod
Hammer Drop: 30 inches
Hammer Weight: 140 lbs.

Sample Type	Depth (feet)	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	Depth (feet)	USCS Class.	Surface Conditions: Asphalt Paving Description
					0 --		Asphalt: 3"
					1 --		Fill: Clayey Sand, medium dense to dense, dark brown, moist, fine to fine to medium grained, abundant concrete fragments
					2 --		
					3 --		
					4 --		
					5 --		
	5	13	18.3	95.6	5 --		
					6 --	SC/SM	Alluvium: Clayey to Silty Sand, dense, brown to dark brown, moist, medium grained, trace coarse grained, trace pinhole porosity
					7 --		
					8 --		
					9 --		
	10	40	13.1	112.9	10 --		some caliche
					11 --		
					12 --		
					13 --		
					14 --		
	15	14	20.0	96.5	15 --	SM	Silty Sand, medium dense, brown, moist, fine grained
					16 --		
					17 --		
					18 --		
					19 --		
	20	30	16.9	108.7	20 --		more silty with trace clay, trace ash, trace caliche, trace pinhole porosity

-  - Ring Sample from California Modified Sampler
-  - Standard Penetrometer Test (SPT) Sampler

Figure A-2a

BORING 2 (continued)



GEOCON
INLAND EMPIRE, INC

Project No.: A8284-06-01
Client: Continental Management Group, LLC
Location: 5661 Santa Monica Blvd.
Hollywood, CA
Sears Hollywood

Excavation Date: March 29, 2005
Excavation Method: Hollow Stem Auger
Boring Diameter: 7 inches
Sampling Method: Cal-Mod
Hammer Drop: 30 inches
Hammer Weight: 140 lbs.

Sample Type	Depth (feet)	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	Depth (feet)	USCS Class.	Description
					20 --		
					21 --		
					22 --		groundwater seepage
					23 --		
					24 --		less silty, less ash, yellowish brown staining, very moist to wet
	25	25	17.0	110.5	25 --		
					26 --		
					27 --		
					28 --		
					29 --		
	30	80	8.8	128.2	30 --	SP/SM	Poorly Graded Sand with Silt, dense, brown, wet, fine to medium grained, trace coarse grained sand, trace gravel
					31 --		
					32 --		
					33 --		
					34 --		some cobbles
	35	28	No Recovery		35 --		
					36 --		
					37 --		
					38 --		
					39 --	ML/SM	Silty Sand to Sandy Silt, very dense, light olive brown with yellowish brown staining, fine grained
	40	64	No Recovery		40 --		

- Ring Sample from California Modified Sampler

- Standard Penetrometer Test (SPT) Sampler

Figure A-2b



GEOCON
INLAND EMPIRE, INC

BORING 2 (continued)

Project No.: A8284-06-01
Client: Continental Management Group, LLC
Location: 5661 Santa Monica Blvd.
Hollywood, CA
Sears Hollywood

Excavation Date: March 29, 2005
Excavation Method: Hollow Stem Auger
Boring Diameter: 7 inches
Sampling Method: Cal-Mod
Hammer Drop: 30 inches
Hammer Weight: 140 lbs.

Sample Type	Depth (feet)	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	Depth (feet)	USCS Class.	Description
					40 --		
					41 --		
					42 --		
					43 --		
					44 --		
	45	71	36.8	82.0	45 --		
					46 --		
	47.5	64	27.9	91.7	47 --		more sandy, orange to yellowish brown, trace ash
					48 --		
					49 --		
	50	85	19.3	109.9	50 --	SM	Silty Sand, medium dense, light brown, wet, fine grained
					51 --		
	52.5	48	26.6	96.7	52 --		more silty, very fine grained, trace caliche
					53 --		
					54 --		
	55	65	26.7	94.0	55 --		more silty, very moist
					56 --		
					57 --		
					58 --		
					59 --		
	60	40	19.9	106.9	60 --	ML	Clayey Silt, very firm, mottled light olive brown and dark yellowish brown, moist, trace ash, trace caliche

- Ring Sample from California Modified Sampler
- Standard Penetrometer Test (SPT) Sampler

Figure A-2c

BORING 2 (continued)



GEOCON
INLAND EMPIRE, INC

Project No.: A8284-06-01
Client: Continental Management Group, LLC
Location: 5661 Santa Monica Blvd.
Hollywood, CA
Sears Hollywood

Excavation Date: March 29, 2005
Excavation Method: Hollow Stem Auger
Boring Diameter: 7 inches
Sampling Method: Cal-Mod
Hammer Drop: 30 inches
Hammer Weight: 140 lbs.

Sample Type	Depth (feet)	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	Depth (feet)	USCS Class.	Surface Conditions: Asphalt Paving Description
					60 --		
					61 --		
					62 --		
					63 --		
					64 --		
	65	60	28.8	92.5	65 --		
					66 --		
					67 --		
					68 --		
					69 --		very stiff
	70	64	28.7	90.0	70 --		End at 70 feet Fill to 5 feet Groundwater at 22 feet
					71 --		
					72 --		
					73 --		
					74 --		
					75 --		
					76 --		
					77 --		
					78 --		
					79 --		
					80 --		

-  - Ring Sample from California Modified Sampler
-  - Standard Penetrometer Test (SPT) Sampler

Figure A-2d

BORING 3



GEOCON

INLAND EMPIRE, INC

Project No.: A8284-06-01
 Client: Continental Management Group, LLC
 Location: 5661 Santa Monica Blvd.
 Hollywood, CA
 Sears Hollywood

Excavation Date: March 24, 2005
 Excavation Method: Hollow Stem Auger
 Boring Diameter: 7 inches
 Sampling Method: Cal-Mod
 Hammer Drop: 30 inches
 Hammer Weight: 140 lbs.

Sample Type	Depth (feet)	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	Depth (feet)	USCS Class.	Surface Condition: Asphalt Parking Lot Description
	2	43	23.1	98.3	0 --		Asphalt: 5" Base: 2" Fill: Sandy Clay, medium firm, black, moist to very moist
					-		
					1 --		
					-		
					2 --		
	5	49	16.3	108.2	3 --		
					-		
					4 --		
					-		
					5 --		
	10	69	12.5	115.9	6 --	SC	Alluvium: Clayey Sand, dense, brown, moist, fine to medium grained
					-		
					7 --		
					-		
					8 --		
	15	71	12.6	110.4	9 --		yellow brown and brown, fine to coarse grained, trace orange staining
					-		
					10 --		
					-		
					11 --		
	20	81	28.6	81.2	12 --		very dense, orange brown
					-		
					13 --		
					-		
					14 --		
	20	81	28.6	81.2	15 --	CL	Silty Clay, very firm, grey and orange brown, moist
					-		
					16 --		
					-		
					17 --		



- Ring Sample from California Modified Sampler



- Standard Penetrometer Test (SPT) Sampler

Figure A-3a

BORING 3 (continued)



GEOCON
INLAND EMPIRE, INC

Project No.: A8284-06-01
Client: Continental Management Group, LLC
Location: 5661 Santa Monica Blvd.
Hollywood, CA
Sears Hollywood

Excavation Date: March 24, 2005
Excavation Method: Hollow Stem Auger
Boring Diameter: 7 inches
Sampling Method: Cal-Mod
Hammer Drop: 30 inches
Hammer Weight: 140 lbs.

Sample Type	Depth (feet)	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	Depth (feet)	USCS Class.	Surface Conditions: Asphalt Paving Description
					20 --		
					21 --		
					22 --		
					23 --		
					24 --		
	25	79	27.9	92.1	25 --		
					26 --		
					27 --		
					28 --		
					29 --		groundwater seepage
	30	73	26.6	94.0	30 --		sandy clay, brown, very moist, stiff
					31 --		
					32 --		
					33 --		
					34 --		
	35	67	30.0	91.2	35 --	ML	Clayey Silt, very stiff, orange brown and grey, moist
					36 --		
					37 --		
					38 --		
					39 --		
	40	68	22.7	102.0	40 --		



- Ring Sample from California Modified Sampler
- Standard Penetrometer Test (SPT) Sampler

Figure A-3b

BORING 3 (continued)



GEOCON
INLAND EMPIRE, INC

Project No.: A8284-06-01
Client: Continental Management Group, LLC
Location: 5661 Santa Monica Blvd.
Hollywood, CA
Sears Hollywood

Excavation Date: March 24, 2005
Excavation Method: Hollow Stem Auger
Boring Diameter: 7 inches
Sampling Method: Cal-Mod
Hammer Drop: 30 inches
Hammer Weight: 140 lbs.

Sample Type	Depth (feet)	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	Depth (feet)	USCS Class.	Description
	45	71	No Recovery		40 --		
					41 --		
					42 --		
					43 --		
					44 --		
					45 --		
					46 --		
					47 --		
					48 --		
					49 --		
	50	69	27.3	94.0	50 --	CL	Silty to Sandy Clay, stiff, mottled light olive brown and dark yellowish brown, moist, fine grained, some caliche
					51 --		
					52 --		
					53 --		
					54 --		
					55 --		
					56 --		
					57 --		
					58 --		
					59 --		
	60	84	31.5	89.0	60 --	SM/ML	Silty Sand to Sandy Silt, stiff, mottled light olive brown and dark yellowish brown, moist, fine grained, some caliche, trace ash

-  - Ring Sample from California Modified Sampler
-  - Standard Penetrometer Test (SPT) Sampler

Figure A-3c

BORING 3 (continued)



GEOCON
INLAND EMPIRE, INC

Project No.: A8284-06-01
Client: Continental Management Group, LLC
Location: 5661 Santa Monica Blvd.
Hollywood, CA
Sears Hollywood

Excavation Date: March 24, 2005
Excavation Method: Hollow Stem Auger
Boring Diameter: 7 inches
Sampling Method: Cal-Mod
Hammer Drop: 30 inches
Hammer Weight: 140 lbs.

Sample Type	Depth (feet)	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	Depth (feet)	USCS Class.	Description
					60 --		
					61 --		
					62 --		
					63 --		
					64 --		
					65 --		
					66 --		
					67 --		
					68 --		
					69 --		
	70	65	31.4	86.3	70 --	ML	Clayey Silt, very stiff, mottled light olive brown, dark yellowish brown, and dark grey, moist, trace caliche
					71 --		
					72 --		
					73 --		
					74 --		
					75 --		
					76 --		
					77 --		
					78 --		
					79 --		
	80	50 (5")	27.5	94.3	80 --		trace fine grained sand

- Ring Sample from California Modified Sampler
- Standard Penetrometer Test (SPT) Sampler

Figure A-3d



GEOCON
INLAND EMPIRE, INC

BORING 3 (continued)

Project No.: A8284-06-01
Client: Continental Management Group, LLC
Location: 5661 Santa Monica Blvd.
Hollywood, CA
Sears Hollywood

Excavation Date: March 24, 2005
Excavation Method: Hollow Stem Auger
Boring Diameter: 7 inches
Sampling Method: Cal-Mod
Hammer Drop: 30 inches
Hammer Weight: 140 lbs.

Sample Type	Depth (feet)	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	Depth (feet)	USCS Class.	Description
	90	100 (6")	20.5	99.8	80 --		
					81 --		
					82 --		
					83 --		
					84 --		
					85 --		
					86 --		
					87 --		
					88 --	ML	Sandy to Clayey Silt, very stiff, mottled dark grey, light grey, light olive brown, and dark yellowish brown, moist, fine grained, trace caliche
					89 --		
					90 --		
					91 --		End at 90 feet Fill to 5 feet Groundwater at 28.5 feet
					92 --		
					93 --		
					94 --		
					95 --		
					96 --		
					97 --		
					98 --		
					99 --		
					100 --		



- Ring Sample from California Modified Sampler



- Standard Penetrometer Test (SPT) Sampler

Figure A-3e

BORING 4



GEOCON
INLAND EMPIRE, INC

Project No.: A8284-06-01
Client: Continental Management Group, LLC
Location: 5661 Santa Monica Blvd.
Hollywood, CA
Sears Hollywood

Excavation Date: March 23, 2005
Excavation Method: Hollow Stem Auger
Boring Diameter: 8 inches
Sampling Method: Cal-Mod
Hammer Drop: 30 inches
Hammer Weight: 140 lbs.

Sample Type	Depth (feet)	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	Depth (feet)	USCS Class.	Surface Conditions: Asphalt Parking Lot Description
	2	28	22.2	99.0	0 --		Asphalt: 3" Base: 2.5"
					1 --		Fill: Clayey Sand, medium dense, brown and dark brown, moist, fine to medium grained
					2 --	SC	Alluvium: Clayey Sand, medium dense, dark brown, moist, fine to medium grained
					3 --		
	5	18	16.0	104.9	4 --		
					5 --		
					6 --		brown, slightly porous
					7 --		
	10	84	13.7	116.1	8 --		
					9 --		
					10 --		greyish brown and orange brown, dense to very dense, fine to coarse grained
					11 --		
	15	80	18.6	106.5	12 --		
					13 --		
					14 --		
					15 --	CL	Sandy Clay, very firm, light brown, moist
	20	75	17.9	107.4	16 --		
					17 --		
					18 --		
					19 --		
	20	75	17.9	107.4	20 --		brown

- Ring Sample from California Modified Sampler
- Standard Penetrometer Test (SPT) Sampler

Figure A-4a

BORING 4 (continued)



GEOCON
INLAND EMPIRE, INC

Project No.: A8284-06-01
Client: Continental Management Group, LLC
Location: 5661 Santa Monica Blvd.
Hollywood, CA
Sears Hollywood

Excavation Date: March 23, 2005
Excavation Method: Hollow Stem Auger
Boring Diameter: 8 inches
Sampling Method: Cal-Mod
Hammer Drop: 30 inches
Hammer Weight: 140 lbs.

Sample Type	Depth (feet)	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	Depth (feet)	USCS Class.	Description
					20 --		
					21 --		
					22 --		
					23 --		
					24 --		groundwater seepage
	25	80	18.8	109.8	25 --	SC	Clayey Sand, dense, brown, very moist, fine to coarse grained
					26 --		
					27 --		
					28 --		
					29 --		
	30	69	23.9	101.9	30 --		wet, fine to medium grained
					31 --		
					32 --		
					33 --		
					34 --		
	35	65	21.8	104.2	35 --	CL	Sandy Clay, very firm, dark brown, very moist
					36 --		
					37 --		
					38 --		
					39 --		
	40	79	27.2	47.9	40 --		grey, some gravel

-  - Ring Sample from California Modified Sampler
-  - Standard Penetrometer Test (SPT) Sampler

Figure A-4b

BORING 4 (continued)



GEOCON
INLAND EMPIRE, INC

Project No.: A8284-06-01
Client: Continental Management Group, LLC
Location: 5661 Santa Monica Blvd.
Hollywood, CA
Sears Hollywood

Excavation Date: March 23, 2005
Excavation Method: Hollow Stem Auger
Boring Diameter: 8 inches
Sampling Method: Cal-Mod
Hammer Drop: 30 inches
Hammer Weight: 140 lbs.

Sample Type	Depth (feet)	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	Depth (feet)	USCS Class.	Description
					40 --		
					41 --		
					42 --		
	43	90	28.7	178.0	43 --		clayey, very firm, grey and orange brown, moist, trace gravel
					44 --		
					45 --		
	46	100 (7")	34.9	82.3	46 --		very dense
					47 --		
					48 --		
	49	100 (4")	28.5	93.1	49 --	SC	Clayey Sand, very dense, orange brown, moist to very moist, fine to medium grained
					50 --		
					51 --		
	52	100 (6")	No Recovery		52 --		
					53 --		
	54	100 (6")	20.0	109.3	54 --		brown and light grey
					55 --		
					56 --		
					57 --		
					58 --		
					59 --		
	60	100 (5")	20.0	108.4	60 --		brown to dark brown

-  - Ring Sample from California Modified Sampler
-  - Standard Penetrometer Test (SPT) Sampler

Figure A-4c

BORING 4 (continued)



GEOCON
INLAND EMPIRE, INC

Project No.: A8284-06-01
Client: Continental Management Group, LLC
Location: 5661 Santa Monica Blvd.
Hollywood, CA
Sears Hollywood

Excavation Date: March 23, 2005
Excavation Method: Hollow Stem Auger
Boring Diameter: 8 inches
Sampling Method: Cal-Mod
Hammer Drop: 30 inches
Hammer Weight: 140 lbs.

Sample Type	Depth (feet)	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	Depth (feet)	USCS Class.	Description
	65	100 (7")	20.1	108.5	60 --		
					61 --		
					62 --		
					63 --		
					64 --		
					65 --		
					66 --		
					67 --		
					68 --		
					69 --		
					70 --		
					71 --		
					72 --		
					73 --		
					74 --		
					75 --		
					76 --		
					77 --		
					78 --		
					79 --		
					80 --		
					64 --		-----
					65 --		brown
					66 --		End at 65 feet
					67 --		Fill to 2 feet
					68 --		Groundwater at 24.5 feet
					69 --		
					70 --		
					71 --		
					72 --		
					73 --		
					74 --		
					75 --		
					76 --		
					77 --		
					78 --		
					79 --		
					80 --		



- Ring Sample from California Modified Sampler

- Standard Penetrometer Test (SPT) Sampler

Figure A-4d

BORING 5



GEOCON
INLAND EMPIRE, INC

Project No.: A8284-06-01
Client: Continental Management Group, LLC
Location: 5661 Santa Monica Blvd.
Hollywood, CA
Sears Hollywood

Excavation Date: March 28, 2005
Excavation Method: Hollow Stem Auger
Boring Diameter: 7 inches
Sampling Method: Cal-Mod
Hammer Drop: 30 inches
Hammer Weight: 140 lbs.

Sample Type	Depth (feet)	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	Depth (feet)	USCS Class.	Surface Condition: Asphalt Paving Description
					0 --		Asphalt: 6"
					-		Fill: Sandy Clay to Clayey Sand, dense, dark brown, fine grained with trace medium grained sand, some asphalt fragments
					1 --		
					-		
					2 --		
	5	30	13.6	111.4	-		
					3 --		
					-		
					4 --		
					-		
					5 --		
					-	SC/SM	Alluvium: Clayey to Silty Sand, very dense, brown, fine grained with trace medium, grained, trace caliche
					6 --		
					-		
					7 --		
	10	28	11.2	89.3	-		
					8 --		
					-		
					9 --		
					-	SM	Silty Sand, dense, brown, fine grained, trace caliche, trace ash, some pinhole porosity, trace clay
					10 --		
					-		
					11 --		
					-		
					12 --		
					-		
					13 --		
					-		
					14 --		slightly more porous
					-		
	15	38	13.4	100.4	15 --		
					-		
					16 --		
					-		
					17 --		
					-		
					18 --		
					-		
					19 --	SP/SM	Poorly Graded Sand with Silt, medium dense, brown, moist, fine grained
					-		
	20	43	6.4	125.9	20 --		
					-		

- Ring Sample from California Modified Sampler

- Standard Penetrometer Test (SPT) Sampler

Figure A-5a

BORING 5 (continued)



GEOCON

INLAND EMPIRE, INC

Project No.: A8284-06-01
 Client: Continental Management Group, LLC
 Location: 5661 Santa Monica Blvd.
 Hollywood, CA
 Sears Hollywood

Excavation Date: March 28, 2005
 Excavation Method: Hollow Stem Auger
 Boring Diameter: 7 inches
 Sampling Method: Cal-Mod
 Hammer Drop: 30 inches
 Hammer Weight: 140 lbs.

Sample Type	Depth (feet)	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	Depth (feet)	USCS Class.	Description
					20 --		
					21 --		
					22 --		
					23 --		
					24 --		
	25	30	17.8	112.8	25 --	SC/CL	Sandy Clay to Clayey Sand, dense, brown with trace yellowish brown staining, moist, fine grained, trace pinhole porosity
					26 --		groundwater seepage
					27 --		
					28 --		
					29 --		
	30	46	11.8	137.7	30 --	SC	Clayey Sand, dense, brown, very moist, fine to medium grained
					31 --		
					32 --		
					33 --		
					34 --		less clayey, trace coarse grained sand, trace gravel
	35	45	18.5	109.9	35 --		
					36 --		
					37 --		
					38 --		
					39 --		Clayey to Silty Sand, dense, brown, very moist, fine grained to medium grained with trace coarse grained, trace gravel
					40 --	SM/SC	
	40	45	13.7	117.2	40 --		



- Ring Sample from California Modified Sampler

- Standard Penetrometer Test (SPT) Sampler

Figure A-5b

BORING 5 (continued)



GEOCON

INLAND EMPIRE, INC

Project No.: A8284-06-01
 Client: Continental Management Group, LLC
 Location: 5661 Santa Monica Blvd.
 Hollywood, CA
 Sears Hollywood

Excavation Date: March 28, 2005
 Excavation Method: Hollow Stem Auger
 Boring Diameter: 7 inches
 Sampling Method: Cal-Mod
 Hammer Drop: 30 inches
 Hammer Weight: 140 lbs.

Sample Type	Depth (feet)	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	Depth (feet)	USCS Class.	Description
					40 --		
					41 --		
					42 --		
					43 --		
					44 --		
	45	71	20.6	103.7	45 --	SM	Silty Sand, dense, mottled dark yellowish brown, light olive brown, and tan, moist, fine grained
					46 --		
	47.5	83	29.6	90.3	47 --	ML	Sandy Silt, very firm, mottled light olive brown and dark yellowish brown, moist, fine grained
					48 --		
					49 --		
	50	69	23.3	97.4	50 --		slightly clayey, stiff, very fine grained, trace ash
					51 --		
					52 --		
	52.5	66	24.0	94.6	53 --		abundant ash
					54 --		
					55 --		less clayey, very firm
	55	41	37.1	81.3	56 --		
					57 --		
					58 --		
					59 --		
	60	70	23.6	101.3	60 --		more sandy with trace clay, stiff, trace caliche

- Ring Sample from California Modified Sampler

- Standard Penetrometer Test (SPT) Sampler

Figure A-5c

BORING 5 (continued)



GEOCON
INLAND EMPIRE, INC

Project No.: A8284-06-01
Client: Continental Management Group, LLC
Location: 5661 Santa Monica Blvd.
Hollywood, CA
Sears Hollywood

Excavation Date: March 28, 2005
Excavation Method: Hollow Stem Auger
Boring Diameter: 7 inches
Sampling Method: Cal-Mod
Hammer Drop: 30 inches
Hammer Weight: 140 lbs.

Sample Type	Depth (feet)	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	Depth (feet)	USCS Class.	Description
					60 --		
					61 --		
					62 --		
					63 --		
					64 --		
					65 --	SM/ML	Interbedded Sandy Silt and Silty Sand, very dense to hard, mottled light olive brown and dark yellowish brown, moist, very fine grained
					66 --		
					67 --		
					68 --		
					69 --		
					70 --		End at 70 feet
					71 --		Fill to 5 feet
					72 --		Groundwater at 26 feet
					73 --		
					74 --		
					75 --		
					76 --		
					77 --		
					78 --		
					79 --		
					80 --		



- Ring Sample from California Modified Sampler

- Standard Penetrometer Test (SPT) Sampler

Figure A-5d

BORING 6



GEOCON
INLAND EMPIRE, INC

Project No.: A8284-06-01
Client: Continental Management Group, LLC
Location: 5661 Santa Monica Blvd.
Hollywood, CA
Sears Hollywood

Excavation Date: March 28, 2005
Excavation Method: Hollow Stem Auger
Boring Diameter: 7 inches
Sampling Method: Cal-Mod
Hammer Drop: 30 inches
Hammer Weight: 140 lbs.

Sample Type	Depth (feet)	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	Depth (feet)	USCS Class.	Surface Condition: Asphalt Paving Description
	5	14	19.3	107.8	0 --		Asphalt: 6" Fill: Sandy Clay , firm, dark brown, moist, fine grained with trace medium grained
					-		
					1 --		
					-		
					2 --		
					-		
					3 --		
					-		
					4 --		
					-		
					5 --		
					-		
					6 --		
					-		
					7 --		
					-		
					8 --		
	10	16	11.7	105.8	-	SC/SM	Alluvium: Silty to Clayey Sand , dense, brown, moist, fine grained with trace medium grained, trace pinhole porosity
					9 --		
					-		
					10 --		
					-		
					11 --		
					-		
					12 --		
					-		
					13 --		
					-		
					14 --		
	15	24	22.0	95.6	-	SM	Silty Sand , dense, brown, moist, fine grained to medium grained with trace clay, trace coarse grained sand
					15 --		
					-		
					16 --		
					-		
					17 --		
					-		
					18 --		
					-		
					19 --	SP/SM	Poorly Graded Sand with Silt , dense, brown, moist, fine to medium grained with some coarse grained sand, trace gravel
					-		
	20	41	7.1	113.5	20 --		
					-		

- Ring Sample from California Modified Sampler

- Standard Penetrometer Test (SPT) Sampler

Figure A-6a

BORING 6 (continued)



GEOCON
INLAND EMPIRE, INC

Project No.: A8284-06-01
Client: Continental Management Group, LLC
Location: 5661 Santa Monica Blvd.
Hollywood, CA
Sears Hollywood

Excavation Date: March 28, 2005
Excavation Method: Hollow Stem Auger
Boring Diameter: 7 inches
Sampling Method: Cal-Mod
Hammer Drop: 30 inches
Hammer Weight: 140 lbs.

Sample Type	Depth (feet)	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	Depth (feet)	USCS Class.	Description
					20 --		
					21 --		
					22 --		groundwater seepage
					23 --		
					24 --		
	25	80	15.4	115.1	25 --	SM	Silty Sand, dense, brown, wet, fine to medium grained, trace coarse grained sand, trace gravel
					26 --		
					27 --		
					28 --		
	30	41	28.7	92.5	29 --		very dense, mottled orange brown and tan, fine grained, some gravel, slate fragments
					30 --		
					31 --		
					32 --		
					33 --		
	35	61	27.3	93.8	34 --	ML	Clayey Silt, very firm, mottled light olive brown and orange brown, moist, trace ash
					35 --		
					36 --		
	37.5	50	29.7	91.4	37 --	CL	Sandy Clay, stiff, mottled light olive brown and orange brown, moist, fine grained
					38 --		
					39 --		
	40	64	33.6	85.3	40 --		



- Ring Sample from California Modified Sampler

- Standard Penetrometer Test (SPT) Sampler

Figure A-6b

BORING 6 (continued)



GEOCON
INLAND EMPIRE, INC

Project No.: A8284-06-01
Client: Continental Management Group, LLC
Location: 5661 Santa Monica Blvd.
Hollywood, CA
Sears Hollywood

Excavation Date: March 28, 2005
Excavation Method: Hollow Stem Auger
Boring Diameter: 7 inches
Sampling Method: Cal-Mod
Hammer Drop: 30 inches
Hammer Weight: 140 lbs.

Sample Type	Depth (feet)	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	Depth (feet)	USCS Class.	Description
					40 --		
					41 --		
					42 --		
	42.5	83	23.1	94.7	43 --		
					44 --		more sandy, very firm, very fine grained
	45	60	29.0	91.2	45 --		
					46 --		
					47 --		
					48 --		
					49 --		silty clay, stiff, olive brown with dark yellowish brown staining
	50	84	28.1	94.0	50 --		
					51 --		
					52 --		
					53 --		
					54 --		
	55	70	24.1	99.5	55 --	ML	Clayey Silt, stiff, mottled tan and dark yellowish brown, moist
					56 --		
					57 --		
					58 --		
					59 --		becomes sandy, dark greyish green, very fine grained
	60	81	26.8	95.2	60 --		

- Ring Sample from California Modified Sampler

- Standard Penetrometer Test (SPT) Sampler

Figure A-6c

BORING 6 (continued)



GEOCON

INLAND EMPIRE, INC

Project No.: A8284-06-01
 Client: Continental Management Group, LLC
 Location: 5661 Santa Monica Blvd.
 Hollywood, CA
 Sears Hollywood

Excavation Date: March 28, 2005
 Excavation Method: Hollow Stem Auger
 Boring Diameter: 7 inches
 Sampling Method: Cal-Mod
 Hammer Drop: 30 inches
 Hammer Weight: 140 lbs.

Sample Type	Depth (feet)	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	Depth (feet)	USCS Class.	Description
	65	86	26.1	94.9	60 --		
					61 --		
					62 --		
					63 --		
					64 --		
					65 --		some clay, mottled olive brown and greyish green
					66 --		
					67 --		
					68 --		
					69 --		
	70	85	3.6	106.7	70 --		End at 70 feet
					71 --		Fill to 8 feet
					72 --		Groundwater at 21 feet
					73 --		
					74 --		
					75 --		
					76 --		
					77 --		
					78 --		
					79 --		
					80 --		

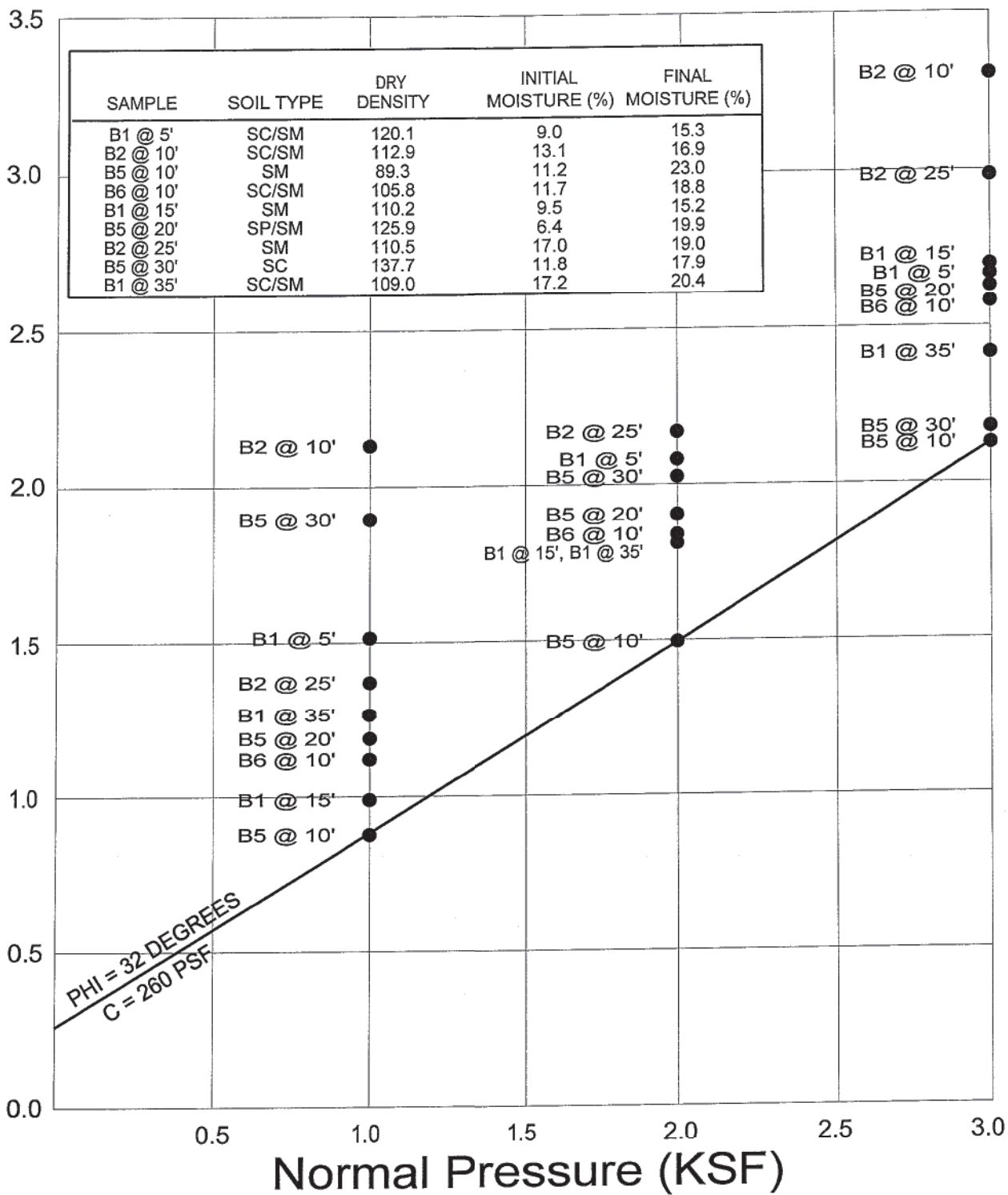


- Ring Sample from California Modified Sampler

- Standard Penetrometer Test (SPT) Sampler

Figure A-6d

Shear Strength (KSF)



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6970 FLANDERS DRIVE - SAN DIEGO, CA 92121
PHONE (858) 558-6900 - FAX (858) 558-6159

NDB

8000

DIRECT SHEAR TEST

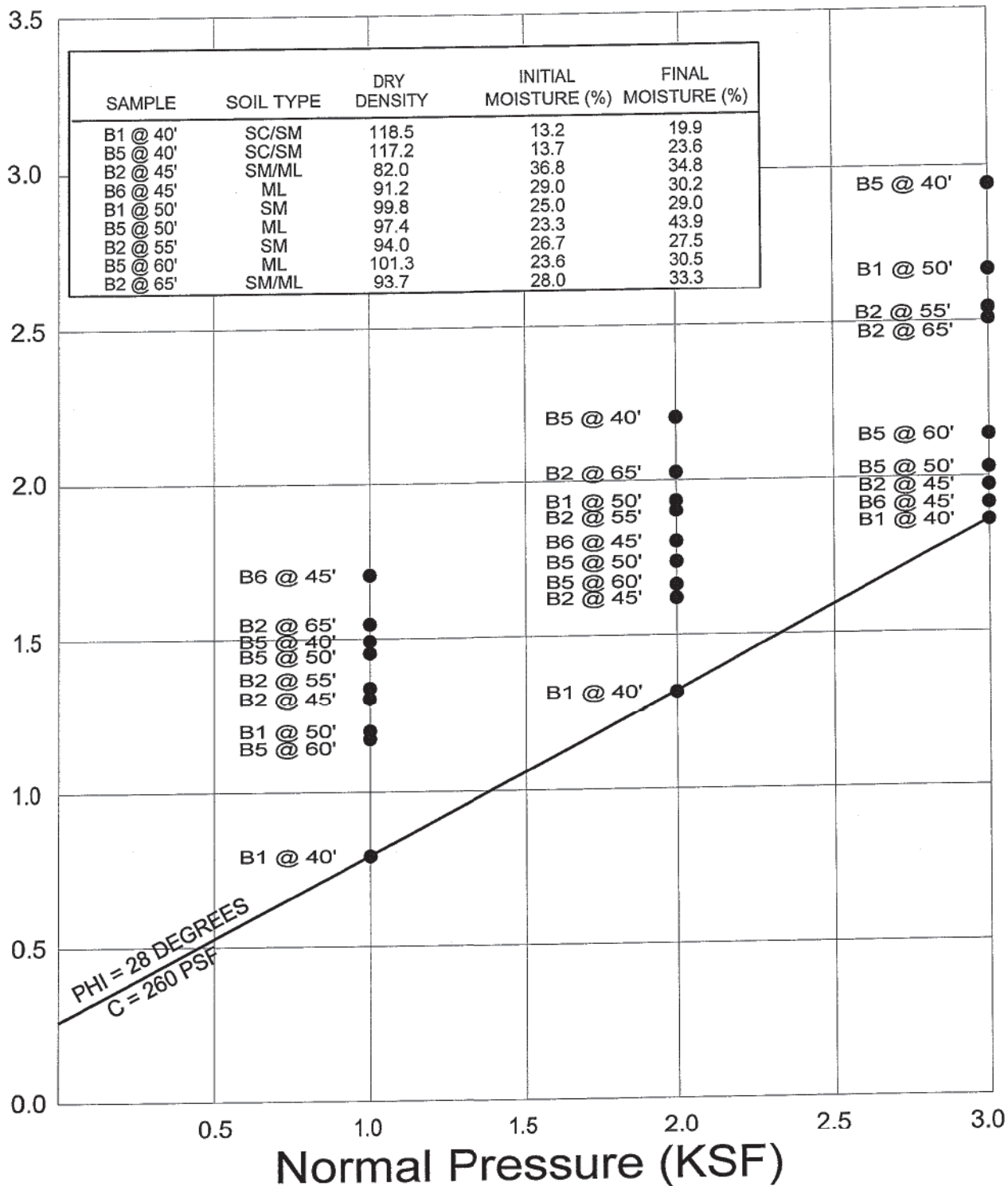
CONTINENTAL MANAGEMENT GROUP, LLC
5661 SANTA MONICA BOULEVARD
HOLLYWOOD, CALIFORNIA

APR. 17, 2007

PROJECT NO. A8284-06-01

FIG. B1

Shear Strength (KSF)



● Direct Shear, Saturated

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NDB

8000

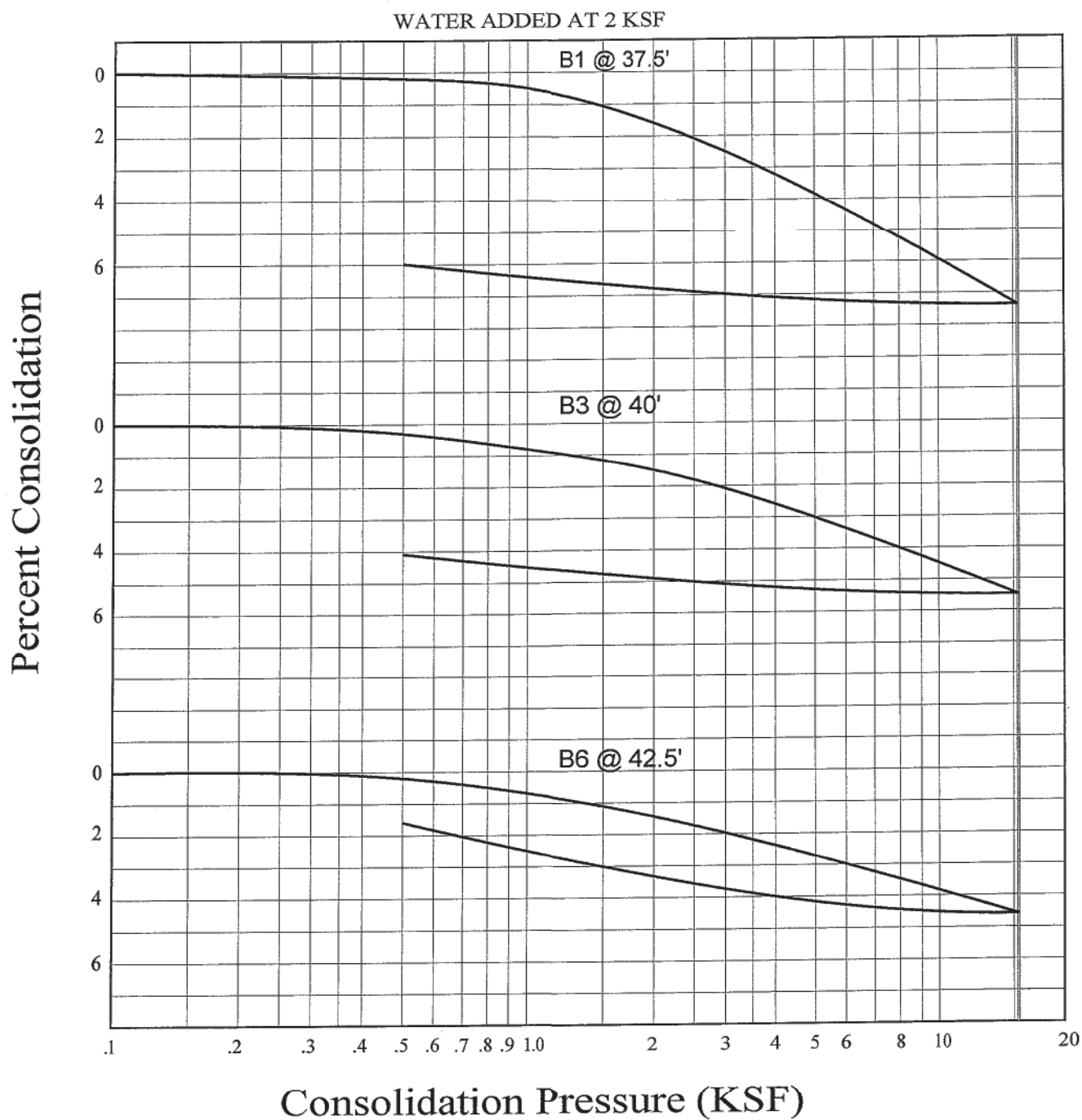
DIRECT SHEAR TEST

CONTINENTAL MANAGEMENT GROUP, LLC
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HOLLYWOOD, CALIFORNIA

APR. 17, 2007

PROJECT NO. A8284-06-01

FIG. B2



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NDB

8000

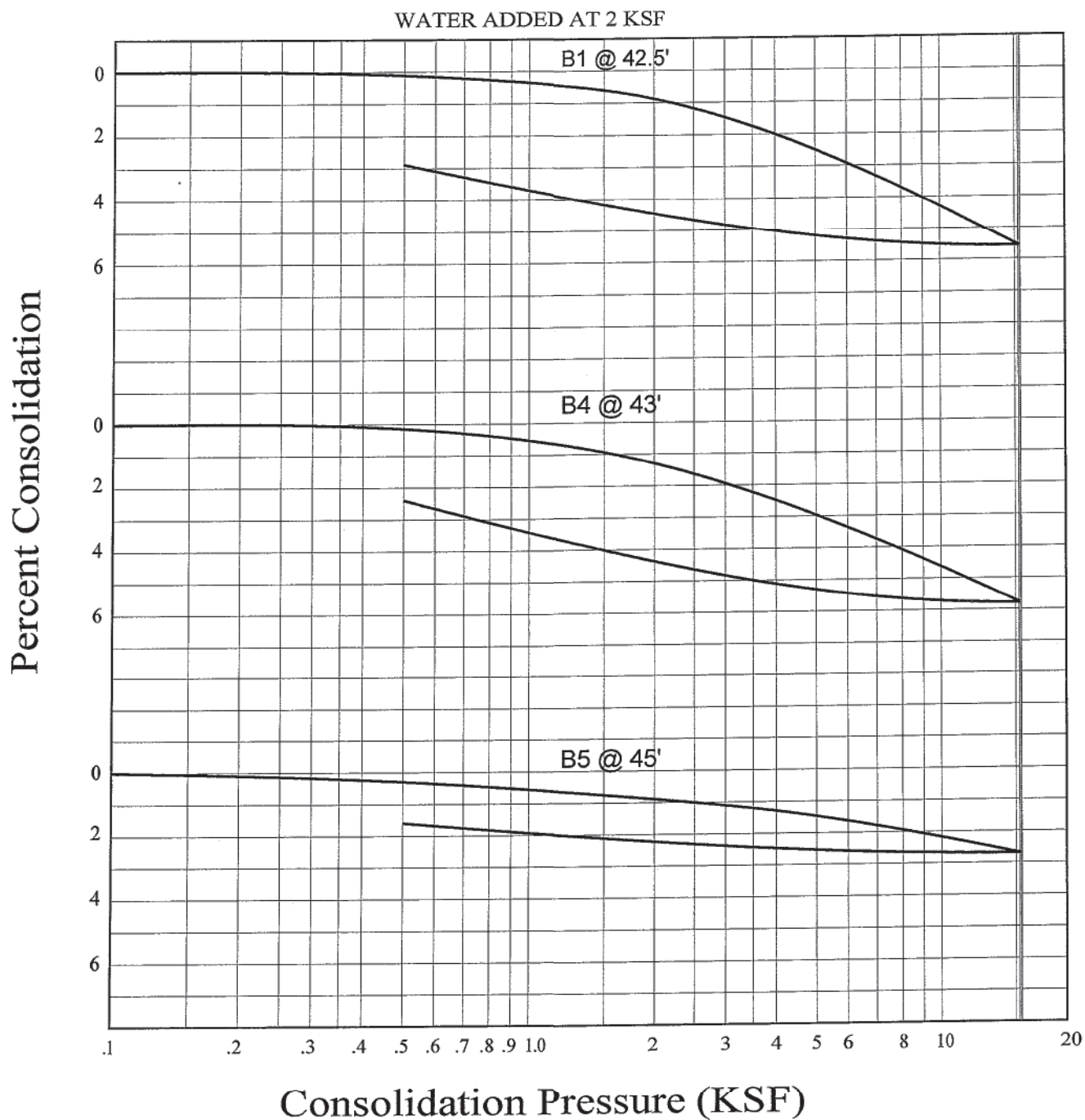
CONSOLIDATION TEST

CONTINENTAL MANAGEMENT GROUP, LLC
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HOLLYWOOD, CALIFORNIA

APR. 17, 2007

PROJECT NO. A8284-06-01

FIG. B3



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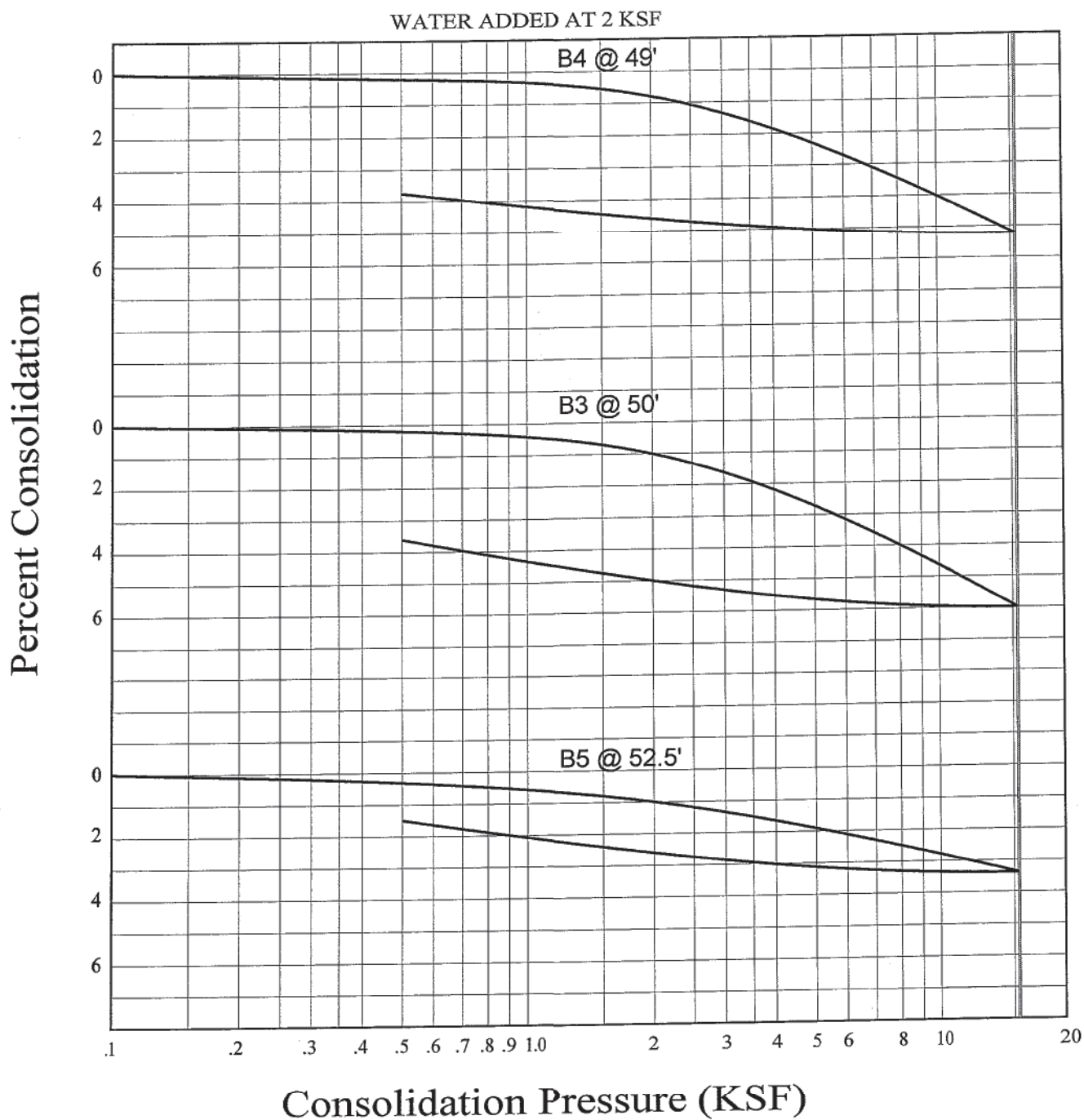
CONSOLIDATION TEST

CONTINENTAL MANAGEMENT GROUP, LLC
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HOLLYWOOD, CALIFORNIA

APR. 17, 2007

PROJECT NO. A8284-06-01

FIG. B4



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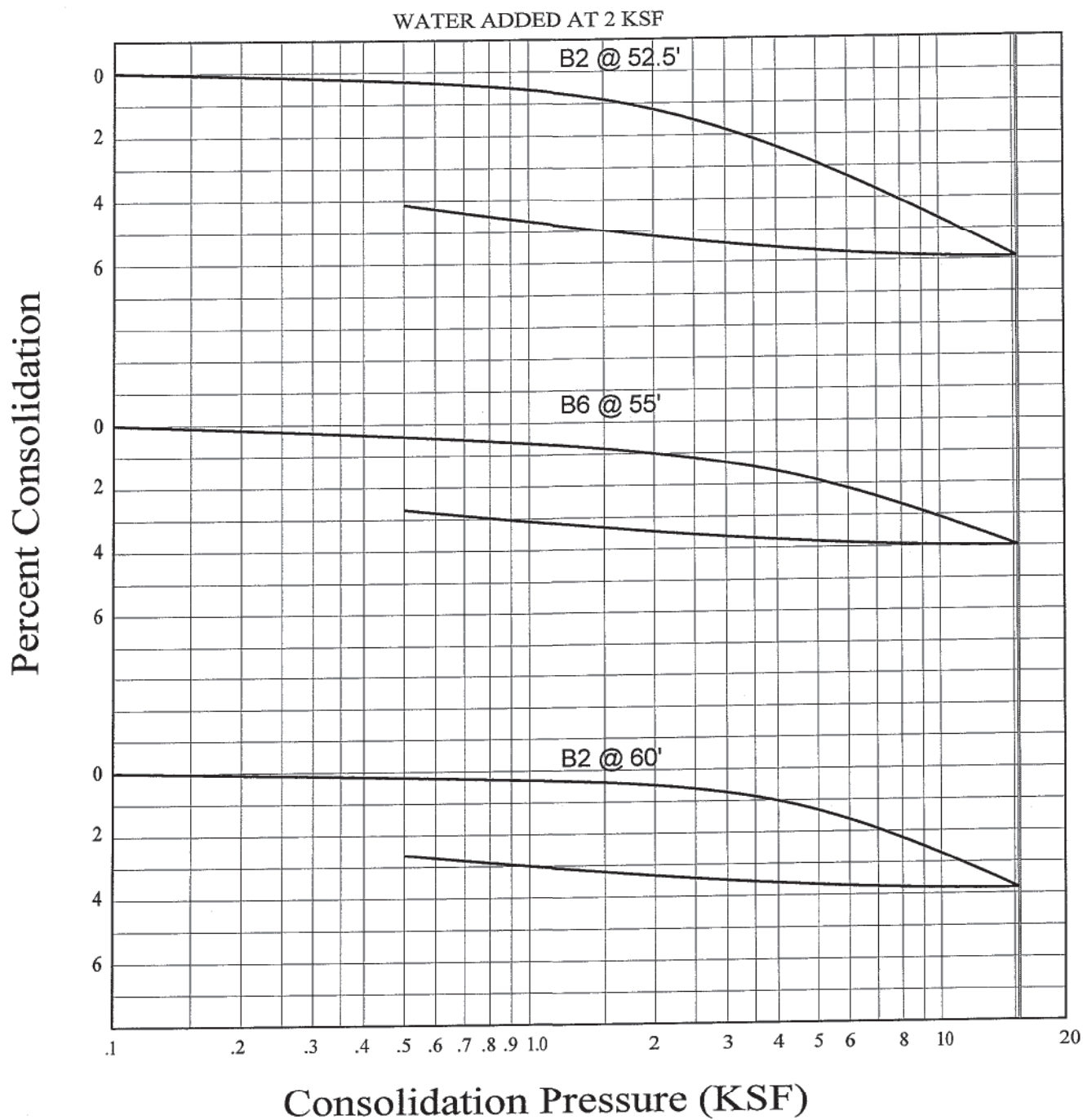
CONSOLIDATION TEST

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HOLLYWOOD, CALIFORNIA

APR. 17, 2007

PROJECT NO. A8284-06-01

FIG. B5



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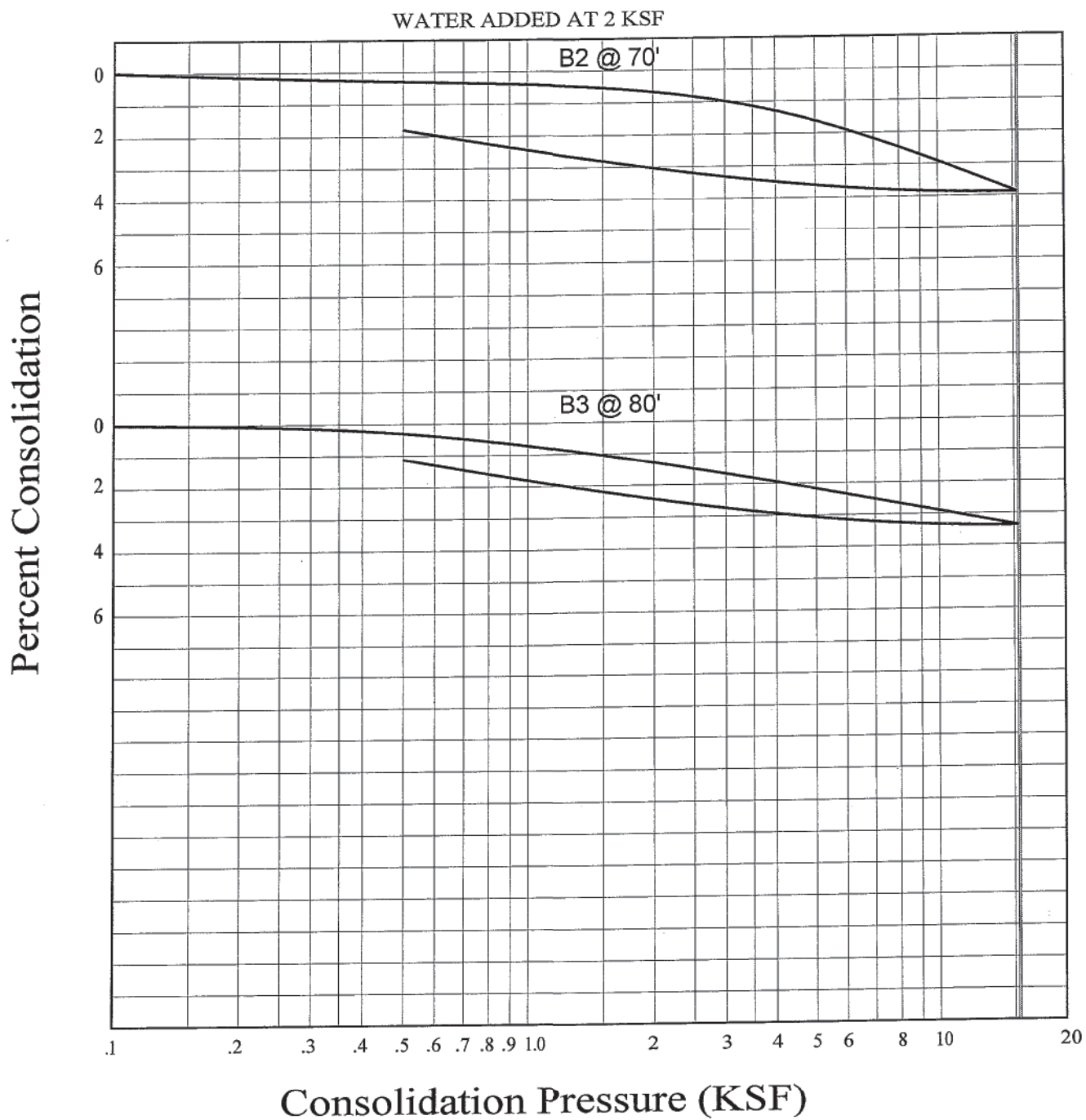
CONSOLIDATION TEST

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HOLLYWOOD, CALIFORNIA

APR. 17, 2007

PROJECT NO. A8284-06-01

FIG. B6



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CONSOLIDATION TEST

CONTINENTAL MANAGEMENT GROUP, LLC
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HOLLYWOOD, CALIFORNIA

APR. 17, 2007

PROJECT NO. A8284-06-01

FIG. B7

**SUMMARY OF LABORATORY POTENTIAL OF
HYDROGEN (pH) AND RESISTIVITY TEST RESULTS
CALIFORNIA TEST NO. 643**

Sample No.	pH	Resistivity (ohm centimeters)
B4 @ 15'	7.3	660 (SEVERE)
B6 @ 37.5'	7.1	840 (SEVERE)

**SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
CALIFORNIA TEST NO. 422**

Sample No.	Chloride Ion Content (%)
B4 @ 15'	0.003
B6 @ 37.5'	0.002

**SUMMARY OF LABORATORY WATER SOLUBLE SULFATE TEST RESULTS
CALIFORNIA TEST NO. 417**

Sample No.	Water Soluble Sulfate (% SQ)	Sulfate Exposure*
B4 @ 15'	0.004	Negligible
B6 @ 37.5'	0.007	Negligible

* Reference: 1997 Uniform Building Code, Table 19-A-4.

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CORROSIVITY TEST RESULTS

CONTINENTAL MANAGEMENT GROUP, LLC
5661 SANTA MONICA BOULEVARD
HOLLYWOOD, CALIFORNIA

APR. 17, 2007

PROJECT NO. A8284-06-01

FIG. B8

SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS ASTM D 4829-95

Sample No.	Moisture Content (%)		Dry Density (pcf)	Expansion Index	*UBC Classification
	Before	After			
B1 @ 37.5'	8.0	19.8	110.9	70	Moderate
B3 @ 40'	11.2	23.4	100.2	11	Very Low
B6 @ 40'	19.0	33.9	91.1	115	High
B3 @ 50'	11.8	31.6	93.9	110	High
B6 @ 50'	17.7	32.6	94.5	100	High

* Reference: 1997 Uniform Building Code, Table 18-I-B.

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LABORATORY TEST RESULTS

CONTINENTAL MANAGEMENT GROUP, LLC
5661 SANTA MONICA BOULEVARD
HOLLYWOOD, CALIFORNIA

APR. 17, 2007

PROJECT NO. A8284-06-01

FIG. B9

VAN AMBATIELOS
PRESIDENT

E. FELICIA BRANNON
VICE-PRESIDENT

JOSELYN GEAGA-ROSENTHAL
GEORGE HOVAGUIMIAN
JAVIER NUNEZ



ERIC GARCETTI
MAYOR

RAYMOND S. CHAN, C.E., S.E.
GENERAL MANAGER

FRANK BUSH
EXECUTIVE OFFICER

GEOLOGY AND SOILS REPORT APPROVAL LETTER

May 20, 2016

LOG # 92915
SOILS/GEOLOGY FILE - 2

5601-5667 Santa Monica Boulevard (LA) Owner, LLC
4700 Wilshire Boulevard
Los Angeles, CA 90010

TRACTS:	LEMONA TRACT (MP 5-131)	LA PALOMA TRACT (MP 5-80)	9834
BLOCKS:	9	8	--
LOTS:	1-12	7-14	1-3
LOCATION:	5601-5667 W. Santa Monica Boulevard		

CURRENT REFERENCE <u>REPORT/LETTER</u>	REPORT <u>No.</u>	DATE(S) OF <u>DOCUMENT</u>	<u>PREPARED BY</u>
Geology/Soils Report	21151	04/13/2016 (04/27/2016)	Geotechnologies, Inc.

PREVIOUS REFERENCE <u>REPORT/LETTER(S)</u>	REPORT <u>No.</u>	DATE OF <u>DOCUMENT</u>	<u>PREPARED BY</u>
Dept. Approval Letter	60287	10/11/2007	LADBS
Soils Report	A8284-06-03	09/11/2007	Geocon Inland Empire
Dept. Approval Letter	59760	09/10/2007	LADBS
Soils Report	A8284-06-03	08/02/2007	Geocon Inland Empire
Dept. Approval Letter	59181	07/03/2007	LADBS
Soils Report	A8284-06-03	06/21/2007	Geocon Inland Empire
Dept. Approval Letter	58231	06/07/2007	LADBS
Soils Report	A8284-06-01	04/17/2007	Geocon Inland Empire

The Grading Division of the Department of Building and Safety has reviewed the current referenced report for the proposed construction of a mixed-use development, which consist of a 7-story building over 5 levels of subterranean parking. According to the report, the proposed building will encompass the entire site, except the existing Sears Building on the southeast portion of the property. The Sears Building is 3-story over a basement. The subsurface materials range from very low to highly expansive, and consist of up to 8 feet of fill over alluvial native soils mainly of clayey silty sand, silt and clay and Puente Formation clayey siltstone, sandstone and claystone bedrock. The report recommends mat foundations for supporting the proposed building on bedrock. Temporary excavations up to 55 feet by open cuts and tieback/ braced shoring are proposed. The existing Sears Building will be in close proximity to and will surcharge the proposed excavations. The report recommends limiting the shoring deflection to 1/2 inch at the elevation of the adjacent offsite foundation, and underpinning the Sear Building foundations.

Groundwater from the prior exploration by Geocon was at 21 to 28 feet below grade. The historic high groundwater is 20 feet below grade. The report recommends temporary dewatering. The report recommends stabilizing the excavation bottoms with 1 to 2 foot thick layer of 3/4 inch gravel layer.

The report is acceptable, provided the following conditions are complied with during site development:

(Note: Numbers in parenthesis () refer to applicable sections of the 2014 City of LA Building Code. P/BC numbers refer the applicable Information Bulletin. Information Bulletins can be accessed on the internet at LADBS.ORG.)

1. Prior to issuing any permits, a supplementary report, providing information on the proposed dewatering and its effects on the existing adjacent structures and facilities, shall be submitted to the Department for review and approval.
2. Prior to the issuance of any permit, secure approval from the Division of Land Unit of the Department of City Planning for the proposed subdivision. The Division of Land Unit of the Planning Department is located in City Hall, 200 N. Spring Street, Room # 750 - Phone (213) 978-1362.
3. This letter approves exclusively the option in which the structure is designed to withstand hydrostatic pressures, as a measure to control groundwater under permanent conditions.
4. The proposed subterranean structure shall be supported on a mat foundation designed to resist uplift hydrostatic pressures that would develop due to the historic high groundwater level at a depth of 20 feet below the existing ground surface, which correlates to an elevation of 308 feet, as recommended on page 15 of the report.
5. The below-grade building walls shall be designed to resist the hydrostatic pressure that would develop if the groundwater level rose to the historic high groundwater level of 20 feet below the existing ground surface, or elevation 308, as recommended. For that portion of the retaining walls above the historically-high groundwater level, a subdrain system may be located. If the subdrain system is not installed above the historically-high groundwater level, then the walls shall be designed for the full hydrostatic pressure to the ground surface, as recommended.
6. In the event piles are recommended to hold the foundation in place instead of thickening the mat, then revise the recommendations so that conventional drilled piles are recommended in lieu of micropiles.

Note: Micropiles are considered as an alternative system per code section 1810.3.10.4.

7. Approval shall be obtained from the Department of Public Works, Bureau of Engineering, Constituent Service Division for the proposed removal of support and/or retaining of slopes adjoining to public way. (3307.3.2)
201 N. Figueroa Street 3rd Floor, LA (213) 482-7045
8. The area shall be de-watered under the direction of the consultants prior to beginning the excavation below the groundwater level. Note, that a permit from the State of California Regional Water Quality Control Board and Department of Public Works shall be obtained to discharge the water into a storm drain.
201 N. Figueroa Street 3rd Floor, LA (213) 482-7045
320 W. 4th Street, Suite 200 (213) 576-6600 (LARWQB)
9. Vertical and lateral supports of the existing footings shall be maintained at all time during the proposed temporary excavations.

10. Secure the notarized written consent from all owners upon whose property proposed grading/construction access is to extend, in the event off-site grading and/or access for construction purposes is required. The consent shall be included as part of the final plans. (7006.6)
11. Provide a notarized letter from all adjoining property owners allowing tie-back anchors on their property. (7006.6)
12. The geologist and soils engineer shall review and approve the detailed plans prior to issuance of any permits. This approval shall be by signature on the plans that clearly indicates the geologist and soils engineer have reviewed the plans prepared by the design engineer and that the plans include the recommendations contained in their reports. (7006.1)
13. All recommendations of the reports that are in addition to or more restrictive than the conditions contained herein shall be incorporated into the plans.
14. A copy of the subject and appropriate referenced reports and this approval letter shall be attached to the District Office and field set of plans. Submit one copy of the above reports to the Building Department Plan Checker prior to issuance of the permit. (7006.1)
15. A grading permit shall be obtained. (106.1.2)
16. All man-made fill shall be compacted to a minimum 90 percent of the maximum dry density of the fill material per the latest version of ASTM D 1557. Where cohesionless soil having less than 15 percent finer than 0.005 millimeters is used for fill, it shall be compacted to a minimum of 95 percent relative compaction based on maximum dry density (D1556). Placement of gravel in lieu of compacted fill is allowed only if complying with Section 91.7011.3 of the Code. (7011.3)
17. Grading shall be scheduled for completion prior to the start of the rainy season, or detailed temporary erosion control plans shall be filed in a manner satisfactory to the Grading Division of the Department and the Department of Public Works, Bureau of Engineering, B-Permit Section, for any grading work in excess of 200 cu yd. (7007.1)
201 N. Figueroa Street 3rd Floor, LA (213) 482-7045
18. Existing uncertified fill shall not be used for support of footings, concrete slabs or new fill. (1809.2)
19. Drainage in conformance with the provisions of the Code shall be maintained during and subsequent to construction. (7013.12)
20. The applicant is advised that the approval of this report does not waive the requirements for excavations contained in the State Construction Safety Orders enforced by the State Division of Industrial Safety. (3301.1)
21. Temporary excavations that remove lateral support to the public way, adjacent property, or adjacent structures shall be supported by shoring, as recommended.
22. Prior to the issuance of any permit which authorizes an excavation where the excavation is to be of a greater depth than are the walls or foundation of any adjoining building or structure and located closer to the property line than the depth of the excavation, the owner of the subject site shall provide the Department with evidence that the adjacent property owner has been given a 30-day written notice of such intent to make an excavation. (3307.1)
23. The soils engineer shall review and approve the shoring and/or underpinning plans prior to issuance of the permit. (3307.3.2)

24. The structural engineer shall verify the adequacy of the existing footings for underpinning.
25. If only portions of the existing building is to be underpinned, the soils engineer and structural engineer shall evaluate the effects of the potential differential settlements on the existing structures.
26. Prior to the issuance of the permits, the soils engineer and/or the structural designer shall evaluate the surcharge loads used in the report calculations for the design of the retaining walls and shoring. If the surcharge loads used in the calculations do not conform to the actual surcharge loads, the soil engineer shall submit a supplementary report with revised recommendations to the Department for approval.
27. Unsurcharged temporary excavations exposing unsupported geology and/or unsupported bedding planes shall be trimmed back along the lowest unsupported plane or at a 1H:1V slope inclination, whichever is flatter, or shored, as recommended.
28. Unsurcharged temporary excavation may be cut vertical up to 6 feet. Unsurcharged temporary excavations over 6 feet up to 10 feet may be cut at 0.5H:1V as recommended. Unsurcharged temporary excavations over 10 feet shall be trimmed back at a gradient not exceeding 1:1, as recommended.
29. Shoring shall be designed for the lateral earth pressures specified in the section titled "Shoring Design" starting on page 42 of the 04/13/2016 (Rev. 04/27/2016) report; all surcharge loads shall be included into the design.
30. Shoring shall be designed for a maximum lateral deflection of ½ inch where a structure is within a 1:1 plane projected up from the base of the excavation, and for a maximum lateral deflection of 1 inch provided there are no structures within a 1:1 plane projected up from the base of the excavation, as recommended.
31. A shoring monitoring program shall be implemented to the satisfaction of the soils engineer.
32. All building foundations shall derive entire support from competent bedrock, as recommended and approved by the geologist and soils engineer by inspection.
33. The structural designer and soils engineer shall verify and attest to the adequacy of the existing footings for underpinning, by signature and license stamp, on the final plans.
34. Pile caisson and/or isolated foundation ties are required by Code Sections 1809.13 and/or 1810.3.13. Exceptions and modification to this requirement are provided in Information Bulletin P/BC 2014-030.
35. When water over 3 inches in depth is present in drilled pile holes, a concrete mix with a strength of 1000 p.s.i. over the design p.s.i. shall be tremied from the bottom up; an admixture that reduces the problem of segregation of paste/aggregates and dilution of paste shall be included. (1808.8.3)
36. Existing uncertified fill shall not be used for lateral support of deep foundation. (1810.2.1)
37. Slabs placed on approved compacted fill shall be at least 3½ inches thick and shall be reinforced with ½-inch diameter (#4) reinforcing bars spaced maximum of 16 inches on center each way.
38. Concrete floor slabs placed on expansive soil shall be placed on a 4-inch fill of coarse aggregate or on a moisture barrier membrane. The slabs shall be at least 3½ inches thick and shall be reinforced with ½-inch diameter (#4) reinforcing bars spaced maximum of 16 inches on center each way.

39. The seismic design shall be based on a Site Class D, as recommended on page 16 of the report. All other seismic design parameters shall be reviewed by LADBS building plan check.
40. Retaining walls shall be designed for the lateral earth pressures specified in the section titled "Retaining Wall Design" starting on page 35 of the 04/13/2016 (Rev. 04/27/2016) report. Note: All surcharge loads shall be included into the design, including those from expansive soils as specified on page 39 of the report.
41. Retaining walls higher than 6 feet shall be designed for lateral earth pressure due to earthquake motions as specified on page 38 of the 04/13/2016 (Rev. 04/27/2016) report (1803.5.12).
42. All retaining walls shall be provided with a standard surface backdrain system and all drainage shall be conducted to the street in an acceptable manner and in a non-erosive device. (7013.11)
43. With the exception of retaining walls designed for hydrostatic pressure, all retaining walls shall be provided with a subdrain system to prevent possible hydrostatic pressure behind the wall. Prior to issuance of any permit, the retaining wall subdrain system recommended in the soil report shall be incorporated into the foundation plan which shall be reviewed and approved by the soils engineer of record. (1805.4)
44. Installation of the subdrain system shall be inspected and approved by the soils engineer of record and the City grading/building inspector. (108.9)
45. Basement walls and floors shall be waterproofed/damp-proofed with an L.A. City approved "Below-grade" waterproofing/damp-proofing material with a research report number. (104.2.6)
46. Prefabricated drainage composites (Miradrain, Geotextiles) may be only used in addition to traditionally accepted methods of draining retained earth.
47. Where the ground water table is lowered and maintained at an elevation not less than 6 inches below the bottom of the lowest floor, or where hydrostatic pressures will not occur, the floor and basement walls shall be damp-proofed. Where a hydrostatic pressure condition exists, and the design does not include a ground-water control system, basement walls and floors shall be waterproofed. (1803.5.4, 1805.1.3, 1805.2, 1805.3)
48. The structure shall be connected to the public sewer system. P/BC 2014-027
49. All roof and pad drainage shall be conducted to the street in an acceptable manner. (7013.10)
50. An on-site storm water infiltration system at the subject site shall not be implemented, as recommended.
51. All concentrated drainage shall be conducted in an approved device and disposed of in a manner approved by the LADBS. (7013.10)
52. Any recommendations prepared by the geologist and/or the soils engineer for correction of geological hazards found during grading shall be submitted to the Grading Division of the Department for approval prior to utilization in the field. (7008.3)
53. The geologist and soils engineer shall inspect all excavations to determine that conditions anticipated in the report have been encountered and to provide recommendations for the correction of hazards found during grading. (7008 & 1705.6)

54. Prior to the pouring of concrete, a representative of the consulting soils engineer shall inspect and approve the footing excavations. The representative shall post a notice on the job site for the LADBS Building Inspector and the Contractor stating that the work so inspected meets the conditions of the report, but that no concrete shall be poured until the City Building Inspector has also inspected and approved the footing excavations. A written certification to this effect shall be filed with the Grading Division of the Department upon completion of the work. (108.9 & 7008.2)
55. Prior to excavation, an initial inspection shall be called with LADBS Inspector at which time sequence of construction, shoring, underpinning, pile installation, protection fences and dust and traffic control will be scheduled. (108.9.1)
56. Installation of shoring, underpinning, slot cutting excavations and/or pile installation shall be performed under the inspection and approval of the soils engineer and deputy grading inspector. (1705.6)
57. The soils engineer shall evaluate the effects of the proposed high pressure grouting (post-grouting) used for the tieback anchors on the existing offsite structures and facilities.
58. The installation and testing of tie-back anchors shall comply with the recommendations included in the report or the standard sheets titled "Requirement for Tie-back Earth Anchors", whatever is more restrictive. (Research Report #23835)
59. Prior to the placing of compacted fill, a representative of the soils engineer shall inspect and approve the bottom excavations. The representative shall post a notice on the job site for the City Grading Inspector and the Contractor stating that the soil inspected meets the conditions of the report, but that no fill shall be placed until the LADBS Grading Inspector has also inspected and approved the bottom excavations. A written certification to this effect shall be included in the final compaction report filed with the Grading Division of the Department. All fill shall be placed under the inspection and approval of the soils engineer. A compaction report together with the approved soil report and Department approval letter shall be submitted to the Grading Division of the Department upon completion of the compaction. In addition, an Engineer's Certificate of Compliance with the legal description as indicated in the grading permit and the permit number shall be included. (7011.3)
60. No footing/slab shall be poured until the compaction report is submitted and approved by the Grading Division of the Department.



CASEY LEE JENSEN
Engineering Geologist Associate II



GLEN RAAD
Geotechnical Engineer I

CLJ/GR:clj/gr
Log No. 92915
213-482-0480

cc: Geotechnologies, Inc., Project Consultant
LA District Office

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201 NORTH FIGUEROA STREET
LOS ANGELES, CA 90012

FRANK BUSH
GENERAL MANAGER

SOILS REPORT APPROVAL LETTER

June 27, 2016

LOG # 93517
SOILS/GEOLOGY FILE - 2

5601-5667 Santa Monica Boulevard (LA) Owner, LLC
4700 Wilshire Boulevard
Los Angeles, CA 90010

TRACT: LEMONA TRACT (MP 5-131)| LA PALOMA TRACT (MP 5-80)| 9834
BLOCK: 9 | 8 | --
LOT(S): 1-12 | 7-14 | 1-3
LOCATION: 5601-5667 W. Santa Monica Boulevard

<u>CURRENT REFERENCE</u> <u>REPORT/LETTER(S)</u>	<u>REPORT</u> <u>No.</u>	<u>DATE(S) OF</u> <u>DOCUMENT</u>	<u>PREPARED BY</u>
Addendum Report	21151	06/07/2016	Geotechnologies, Inc.
Addendum Report	2050-10	06/07/2016	Northgate Environmental Management, Inc.
Request for Modification	24433	06/27/2016	LADBS

<u>PREVIOUS REFERENCE</u> <u>REPORT/LETTER(S)</u>	<u>REPORT</u> <u>No.</u>	<u>DATE(S) OF</u> <u>DOCUMENT</u>	<u>PREPARED BY</u>
Dept. Approval Letter	92915	05/20/2016	LADBS
Geology/Soils Report	21151	04/13/2016 (04/27/2016)	Geotechnologies, Inc.
Dept. Approval Letter	60287	10/11/2007	LADBS
Soils Report	A8284-06-03	09/11/2007	Geocon Inland Empire
Dept. Approval Letter	59760	09/10/2007	LADBS
Soils Report	A8284-06-03	08/02/2007	Geocon Inland Empire
Dept. Approval Letter	59181	07/03/2007	LADBS
Soils Report	A8284-06-03	06/21/2007	Geocon Inland Empire
Dept. Approval Letter	58231	06/07/2007	LADBS
Soils Report	A8284-06-01	04/17/2007	Geocon Inland Empire

The Grading Division of the Department of Building and Safety has reviewed the current referenced reports for the proposed construction of a mixed-use development, which consist of a 7-story building over 5 levels of subterranean parking. According to the report, the proposed building will encompass the entire site, except the existing Sears Building on the southeast portion of the property. The Sears Building is 3-story over a basement. The subsurface materials range from very low to highly expansive, and consist of up to 8 feet of fill over alluvial native soils mainly of clayey silty sand, silt and clay and Puente Formation clayey siltstone, sandstone and claystone bedrock. The report recommends mat foundations for supporting the proposed building on bedrock. Temporary excavations up to 55 feet by open cuts and tieback/ braced shoring

are proposed. The existing Sears Building will be in close proximity to and will surcharge the proposed excavations. The report recommends limiting the shoring deflection to 1/2 inch at the elevation of the adjacent offsite foundation, and underpinning the Sear Building foundations.

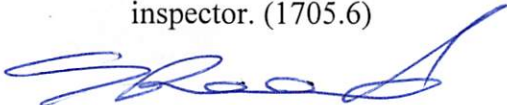
The referenced reports are acceptable, provided the following conditions are complied with during site development:

(Note: Numbers in parenthesis () refer to applicable sections of the 2014 City of LA Building Code. P/BC numbers refer the applicable Information Bulletin. Information Bulletins can be accessed on the internet at LADBS.ORG.)

1. All conditions of the above referenced Department approval letter dated 05/20/2016 (Log# 92915) shall apply except as specifically modified herein.
2. All recommendations of the reports that are in addition to or more restrictive than the conditions contained herein shall be incorporated into the plans.
3. All foundations shall derive entire support from competent bedrock, as recommended and approved by the geologist and soils engineer by inspection.
4. The above referenced Request for Modification, File No. 24433 to allow the use of micropiles as an alternative system per code section 1810.3.10.4 for structures assigned to Seismic Design Category D, E, or F was approved by the Department. Micropiles shall exclusively be used to resist buoyancy pressures in design of the basement mat foundation, as recommended. Micropiles shall not be considered as carrying any horizontal loads (1810.3.10.4.1).
5. A permanent steel casing having a minimum thickness of 3/8-inch shall be provided from the top of the micropile down to a minimum of 120 percent of the point of zero curvature. Steel casing length in soil shall be considered as unbounded and shall not be considered as contributing to friction (1810.3.10.4.1).
6. Micropiles shall be post-grouted, and shall have a minimum diameter of 8 inches for an allowable downward and upward capacity not exceeding the values recommended on page 26 of the 04/27/2016 report.
7. Micropiles shall be designed and detailed in accordance with Code Section 1810.3.10, in particular, Section 1810.3.10.4.1. The piles shall be provided with a double corrosion protection system.
8. Micropile testing shall as a minimum satisfy the requirements of the document "Micropile Design and Construction", Publication No. FHWA N HI-05-039, December 2005, by U.S. Department of Transportation, Federal Highway Administration, and the current LABC requirements (1810.3.10.4.1). The pile load tests shall be performed in conformance with the requirements specified in the latest version of ASTM D1143.
9. At least two verification micropiles or one percent of the number of production piles, whichever is larger, shall be installed and tested to verify the installation methods, soil conditions, etc. Verification test micropiles shall be tested to 200 percent of the calculated

design capacity. At least one pile shall be tested using the ASTM D1143 Procedure B (Maintained Test). The micropiles used for the verification tests shall be sacrificial and not be incorporated into the production piles.

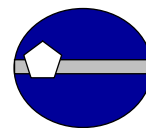
10. No less than 5 percent of the production piles installed shall be proof tested to a test load of 160% of the designed grout-to-ground bond value.
11. A minimum of 10 percent of the production piles shall be integrity tested.
12. An indicator pile test report shall be submitted for review and approval prior to installation of any production pile. The report shall contain, but not be limited to, the following:
 - a. Number of production piles.
 - b. Site plan showing location of the test and reaction piles. The piles shall be located in areas where boring log data are available for comparison and confirmation.
 - c. The test pile shall be installed such that the testing conditions match the proposed conditions (i.e., top and tip pile elevations, overburden pressures, etc., are the same for the test piles and the production piles).
 - d. Pile installation method.
 - e. Description and results of integrity pile testing.
 - f. Upon completion of the pile load test, one test pile shall be entirely exhumed to examine the pile integrity. The pile shall be selected by the project geotechnical engineer. (1810.4.8)
13. Micropile installation shall be performed under the inspection and approval of the soils engineer and deputy grading inspector. The soils engineer shall indicate the distance that the micropiles penetrate into competent material in a written field memorandum. (1704.7, 1803.5.5)
14. Installation of shoring, underpinning, slot cutting excavations and/or pile installation shall be performed under the inspection and approval of the soils engineer and deputy grading inspector. (1705.6)



GLEN RAAD
Geotechnical Engineer I

Log No. 93517
213-482-0480

cc: Phillip Nazarian, Applicant
Geotechnologies, Inc., Project Consultant
Northgate Environmental Management, Inc., Project Consultant
LA District Office



GeoPentech

August 11, 2021
Project No. 21084A

Mr. Reinard Knur
Geotechnologies, Inc.
439 Western Ave.
Glendale, CA 91201

**SUBJECT: SURFACE WAVE GEOPHYSICAL SURVEY RESULTS
5601 SANTA MONICA BOULEVARD
LOS ANGELES, CALIFORNIA**

Dear Mr. Knur,

Per your request and in accordance with the provisions of our proposal, dated July 23, 2021, GeoPentech performed surface wave geophysical measurements along five survey lines (SW21-1 through SW21-5) at 5601 Santa Monica Blvd. in Los Angeles, California. The locations of the geophysical measurements are shown on Figure 1.

Project Understanding

We understand the proposed project includes construction of a commercial structure with six above-surface stories and two subterranean levels and that the developer intends to remove two (2) feet of fill from the ground surface prior to construction. We also understand that a surface-wave geophysical investigation was necessary to measure the shear-wave velocity profile at the site to evaluate the site V_{S30} . This letter summarizes the results of the surface wave surveys and the evaluation of V_{S30} .

Surface Wave Geophysics Methods

Both active and passive surface wave surveys were performed at the site. The active surface wave surveys were performed using Multi-channel Analysis of Surface Waves (MASW) methods, and the passive surveys were performed using Refraction Microtremor (ReMi) methods. A detailed description of MASW is provided in Park et al. (1999)¹ and ReMi is provided in Louie (2001)².

In general, the surface wave method records Rayleigh waves generated either with (1) an active source (e.g. sledgehammer) for the MASW method or (2) a passive (ambient) source (e.g. vehicular traffic) for the ReMi method. In a layered medium, Rayleigh surface waves of different frequencies (or wavelengths) propagate at different velocities, referred to as phase velocity. This phase velocity primarily depends on the material stiffness properties (e.g. S-wave velocity) over a depth approximately equal to one wavelength. Consequently, lower frequency, longer wavelength surface

¹ Park, C, Miller, R., and Xia, J. (1999). Multichannel analysis of surface waves: Geophysics, v. 64, no. 3, pp. 800-808.

² Louie, J.N. (2001). Faster, Better: Shear-wave Velocity to 100 Meters Depth from Refraction Microtremor Arrays: Bulletin of the Seismological Society of America, v. 91, no. 2, p. 347-364.



wave energy will provide samples to greater survey depths than higher frequency, shorter wavelength energy. Because surface waves of different frequencies (wavelengths) sample different depths, they travel at different velocities (dispersion) in a layered medium. Surface wave geophysical surveys measure the dispersive nature of the geologic medium and produce dispersion curves, which show the variation of Rayleigh wave phase velocity as a function of frequency (or wavelength). Due to the generally lower frequency nature of passive surface wave energy, passive surface wave techniques (i.e. ReMi) have the potential to supplement active surface wave data to achieve deeper investigation depths. For this reason, it is advantageous to perform both types of measurement along the same lines as was done for this project.

After the dispersion curve is generated, the dispersion curve picks are then iteratively fitted to a horizontally layered, laterally continuous, homogeneous-isotropic, S-wave velocity model that would account for the measured surface wave velocity dispersion. The results provide a representative average estimate of the one-dimensional S-wave velocity profile under the array.

Surface Wave Geophysics Procedures

The MASW and ReMi investigations were performed at the site on July 30, 2021. As shown on Figure 1, four surface wave measurements were performed in the parking lot currently existing on site, and one surface wave measurement was performed on the sidewalk on the eastern boundary of the site. These measurements were collected using a Geometrics S12 seismograph with a linear array of twelve, 4.5-Hz geophones linearly spaced at 10- and 20-foot intervals for all surface wave lines (SW21-1 through SW21-5). Three lines were oriented north-south, while two were oriented east-west.

For the MASW measurements, the active seismic source consisted of a sledgehammer blow to a ground plate. The MASW measurements were collected along two separate arrays (110 and 220 feet long) for each line. Shots were performed at equal station intervals (either 10 or 20 feet) starting at the end geophone to 2 to 5 station intervals (40 to 60 feet) beyond the end geophone. At each shot location, the sledgehammer was hit three times, and the resultant waveform was stacked. A 1,024-millisecond long record (0.5 millisecond sample interval) was recorded at each shot location. The recorded MASW data were subsequently processed using the program SurfSeis by Kansas Geological Survey. This program performs a wavefield transformation to convert the seismic data from time-distance space to frequency-phase velocity space. The highest amplitude energy in the frequency-phase velocity space was selected for the dispersion curve.

Because of the typical lower frequency nature of passive surface wave energy, ReMi measurements were performed to supplement the MASW measurements to deeper investigation depths. The ReMi measurements were collected along the same 220-foot-long 12-channel arrays as the 220-foot-long MASW measurements. For the ReMi measurements, a total of ten 32,768 millisecond long records (2 millisecond sample interval) were recorded along each survey line. The source of ambient surface wave energy was primarily nearby vehicular traffic. The recorded ReMi data were subsequently processed using the program SeisOpt ReMi by Optim Software. This program performs a slowness-frequency waveform transformation to the recorded surface wave records to separate Rayleigh waves from other seismic arrivals. The ReMi dispersion curves were picked at the lower bound envelope of the surface wave energy, which represents the slowest surface wave

energy (highest slowness). In theory, the slowest identifiable surface wave energy represents the energy that is propagating parallel to the survey line. Energy propagating oblique to the line would be observed as higher velocity.

For each line, the ReMi dispersion curve was combined with the dispersion curves generated from MASW for modeling. Overlapping portions of the ReMi and MASW dispersion curves were compared to evaluate the degree of fit of the dispersion picks. Additionally, as noted above, ReMi and MASW data complement each other by generally sampling different frequency ranges. After the data were combined, a best fit polynomial dispersion curve was calculated for modeling. The best fit dispersion curve was then iteratively fitted to a one-dimensional S-wave velocity model using the SurfSeis software. The results provide a one-dimensional vertical profile of S-wave velocity as a function of depth averaged beneath the area of the line.

Surface Wave Geophysics Results

The results of the combined MASW and ReMi surface wave measurements are shown in Figures 2 through 6 for lines SW21-1 through SW21-5, respectively. These figures present the MASW, ReMi, and best fit surface wave dispersion curves and the corresponding representative S-wave velocity models. As seen in these figures, the MASW and ReMi dispersion curves are generally in good agreement in the regions that overlap. The investigation depths modelled were approximately 97 feet (SW21-1), 100 feet (SW21-2), 117 feet (SW21-3), 124 feet (SW21-4), and 136 feet (SW21-5) below the surface of the line.

The results of the geophysical measurements are summarized on Figure 7. Figure 7 shows the following: (1) the S-wave velocity models for SW21-1 through SW21-5 plotted as a function of depth, (2) the site average S-wave velocity for all the measurements calculated at 1-foot depth increments, and (3) the site average V_{s30} trend. Note that the data are shown and V_{s30} values calculated starting at the planned ground surface, which is two feet below the current ground surface (two feet of currently-existing fill will be removed prior to building construction).

Based on the results shown on Figure 5, the V_{s30} was calculated based on the procedures outlined in the National Earthquake Hazards Reduction Program (NEHRP) and UBC. The V_{s30} was calculated from the following equation from these references:

$$v_s = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{v_{si}}}$$

where:

- i = distinct different soil and/or rock layer between 1 and n
- v_{si} = shear wave velocity in feet per second of layer i
- d_i = thickness of any layer within the 100-foot interval
- $\sum_{i=1}^n d_i$ = 100 feet



Based on this procedure, the V_{S30} was calculated using the site average S-wave velocity from the planned ground surface to a depth of 100 feet. Additionally, V_{S30} values were calculated for 100-foot intervals down to the maximum interval investigated by the surface wave measurements (34 to 134 feet below the planned ground surface). The results are shown on Figure 7 (site average V_{S30} trend) and summarized in Table 1. As indicated on Figure 7 and Table 1, the site average V_{S30} between the planned ground surface and a depth of 100 feet is 1,217 ft/s (371 m/s), which falls within NEHRP Site Class C, very dense soil and soft rock ($1,200 < V_{S30} \leq 2,500$ ft/s).

TABLE 1
CALCULATED SITE V_{S30}

Depth Range (ft, below planned ground surface)	V_{S30} (ft/sec)	NEHRP Site Class
0 to 100	1,217	C
10 to 110	1,342	C
20 to 120	1,446	C
30 to 130	1,547	C
34 to 134	1,579	C

Limitations

The technical results and professional judgments presented herein are based on limited observations, geophysical measurements (as described above), and our general experience in the field of geophysics. GeoPentech does not guarantee the performance of the project in any respect, only that the information provided meets the standard of care of the profession at this time under the same scope limitations imposed by the project.

We trust the contents of this letter will meet your current needs. If you have questions or require additional information, please call.

Very Truly Yours,

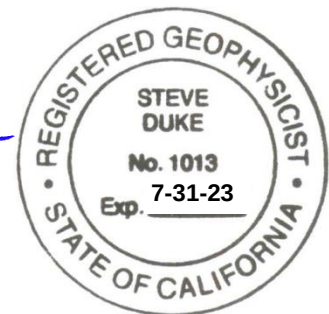
GeoPentech



Ryan D. Hort, Ph.D
Senior Staff Scientist

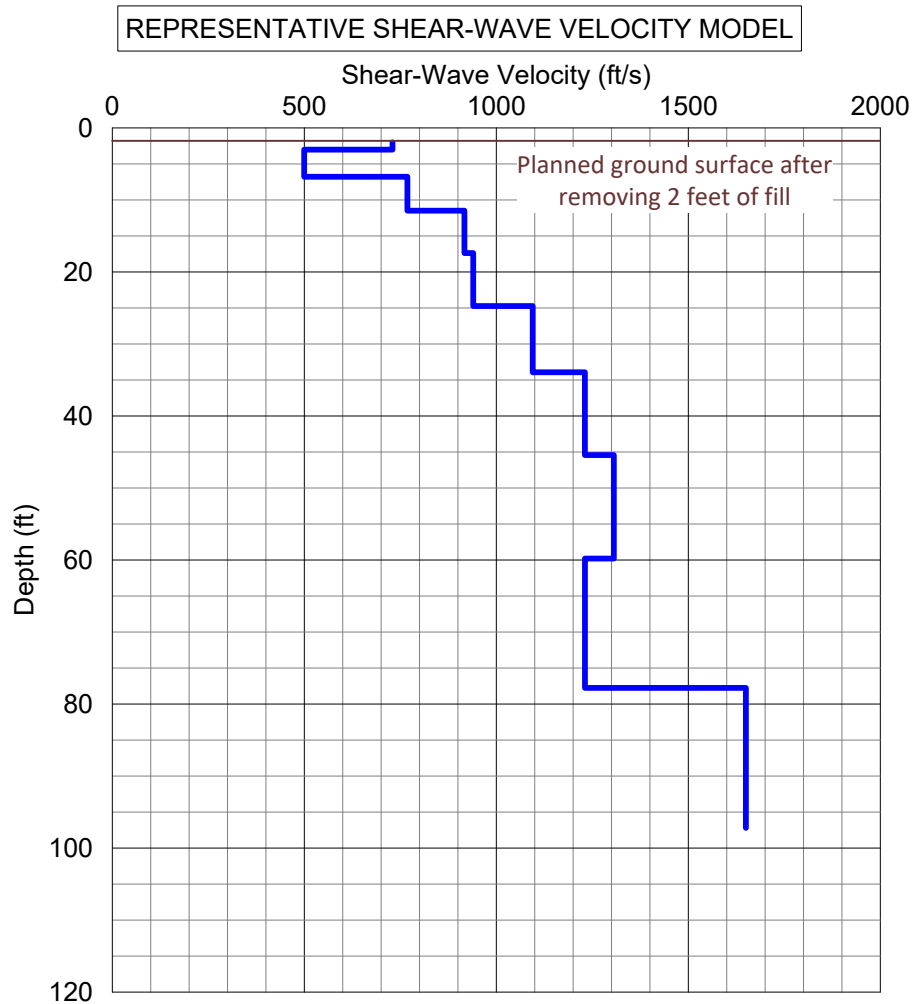
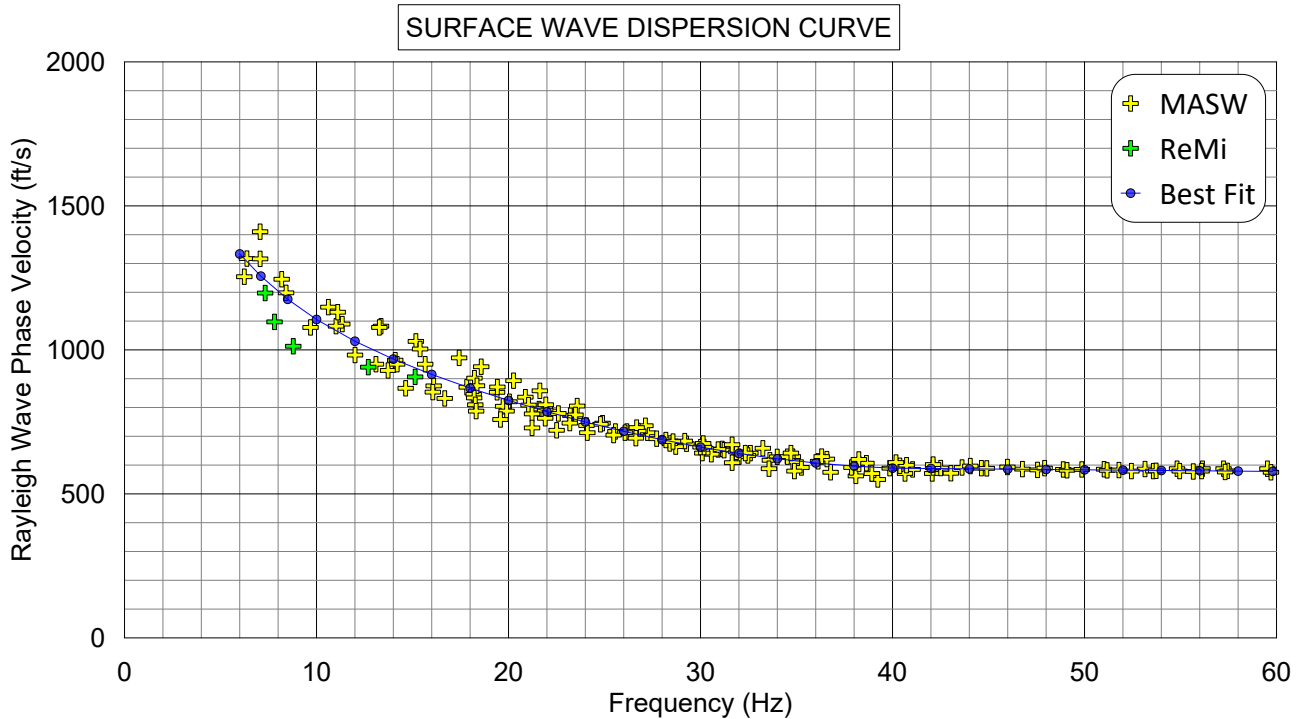


Steven K. Duke
Geophysicist
GP 1013





GEOPHYSICAL LINE LOCATIONS



SW21-1: S-WAVE VELOCITY COMBINED SOURCE MODEL

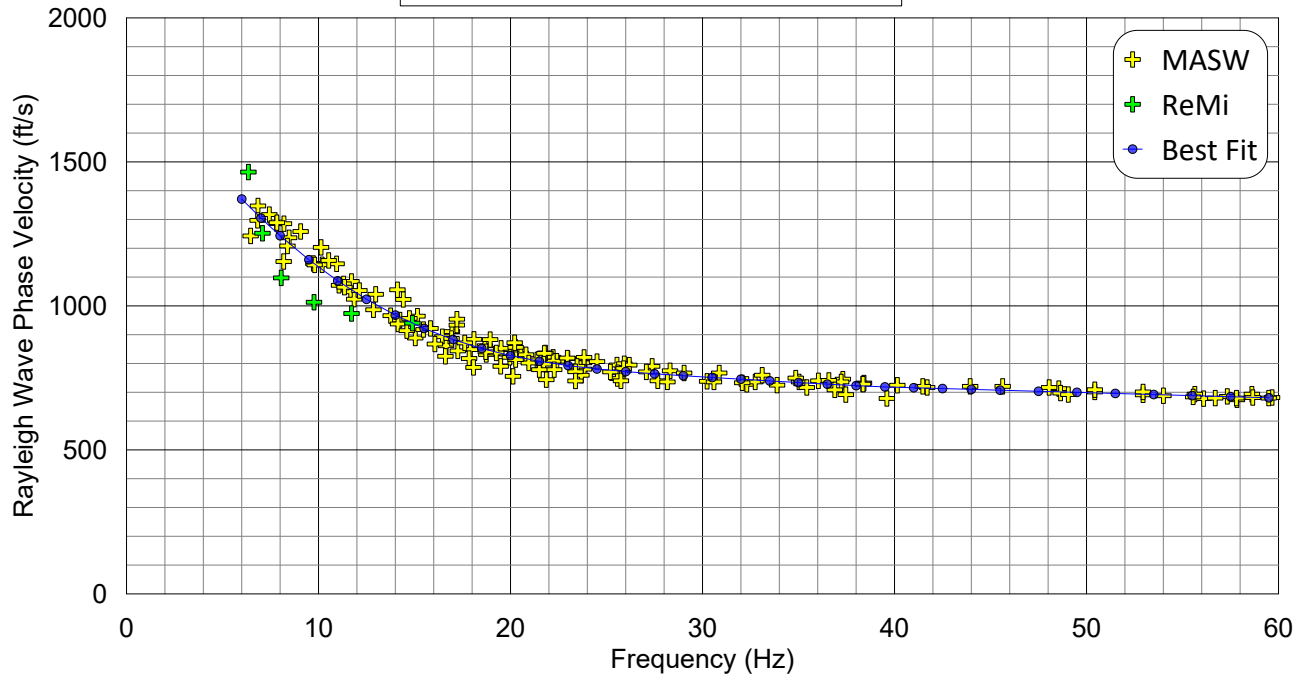
Project No.: 21084A

Project: 5601 Santa Monica Boulevard (Surface Wave)

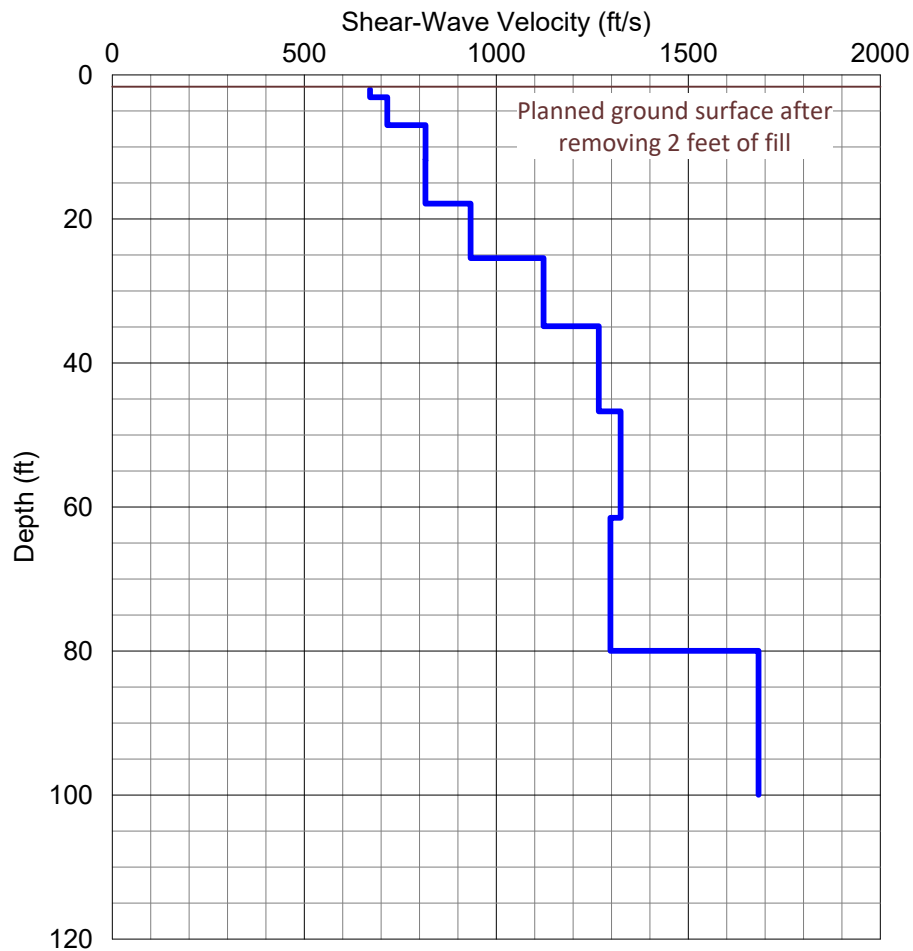
Date: AUG 2021

Figure: 2

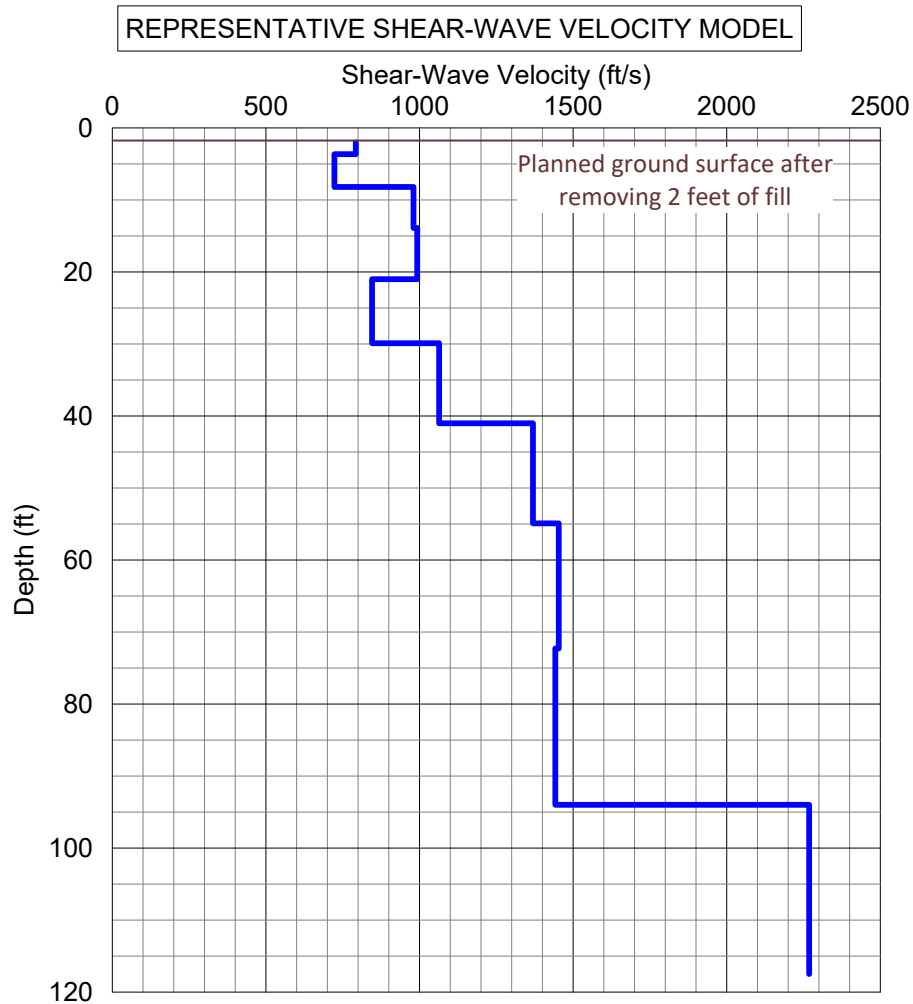
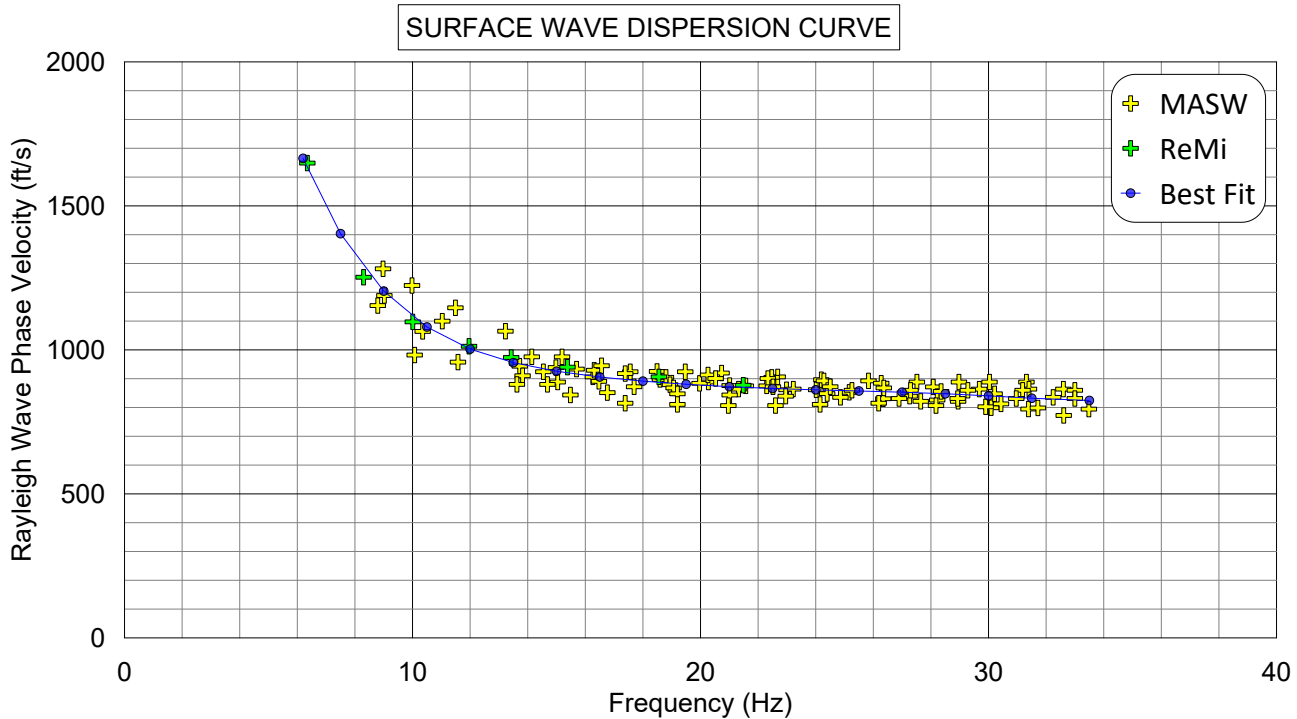
SURFACE WAVE DISPERSION CURVE



REPRESENTATIVE SHEAR-WAVE VELOCITY MODEL



SW21-2: S-WAVE VELOCITY COMBINED SOURCE MODEL



SW21-3: S-WAVE VELOCITY COMBINED SOURCE MODEL

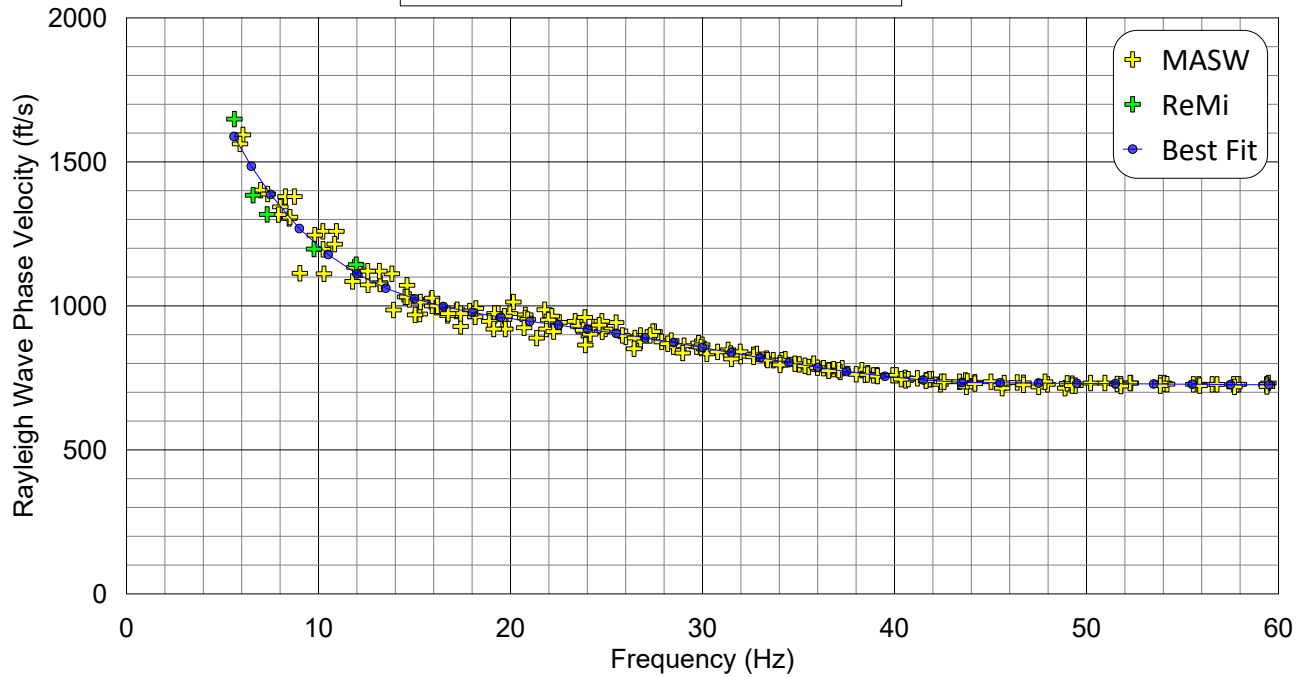
Project No.: 21084A

Project: 5601 Santa Monica Boulevard (Surface Wave)

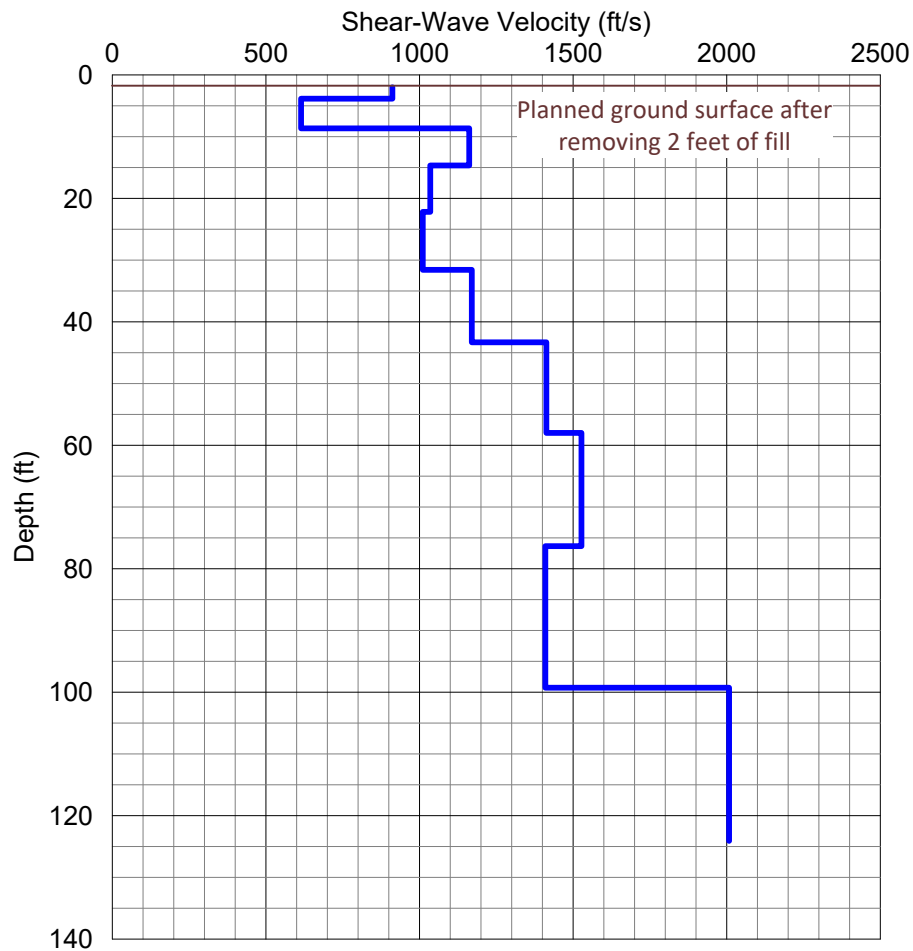
Date: AUG 2021

Figure: 4

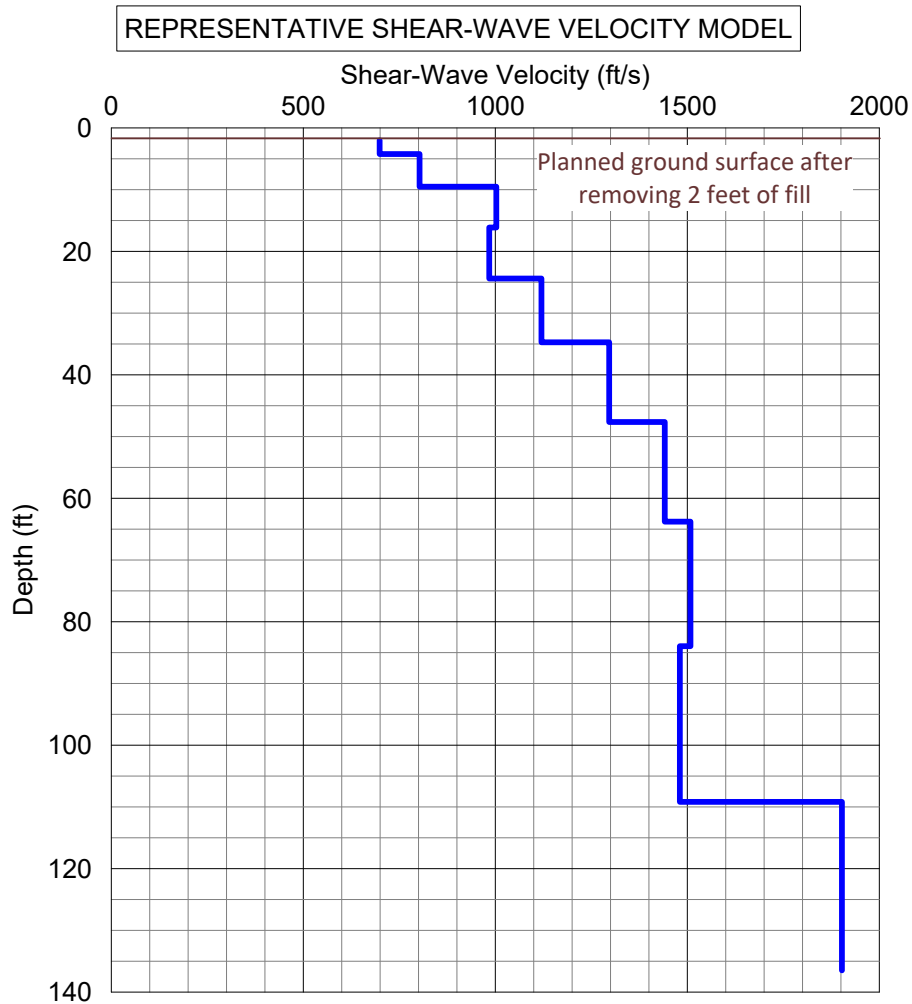
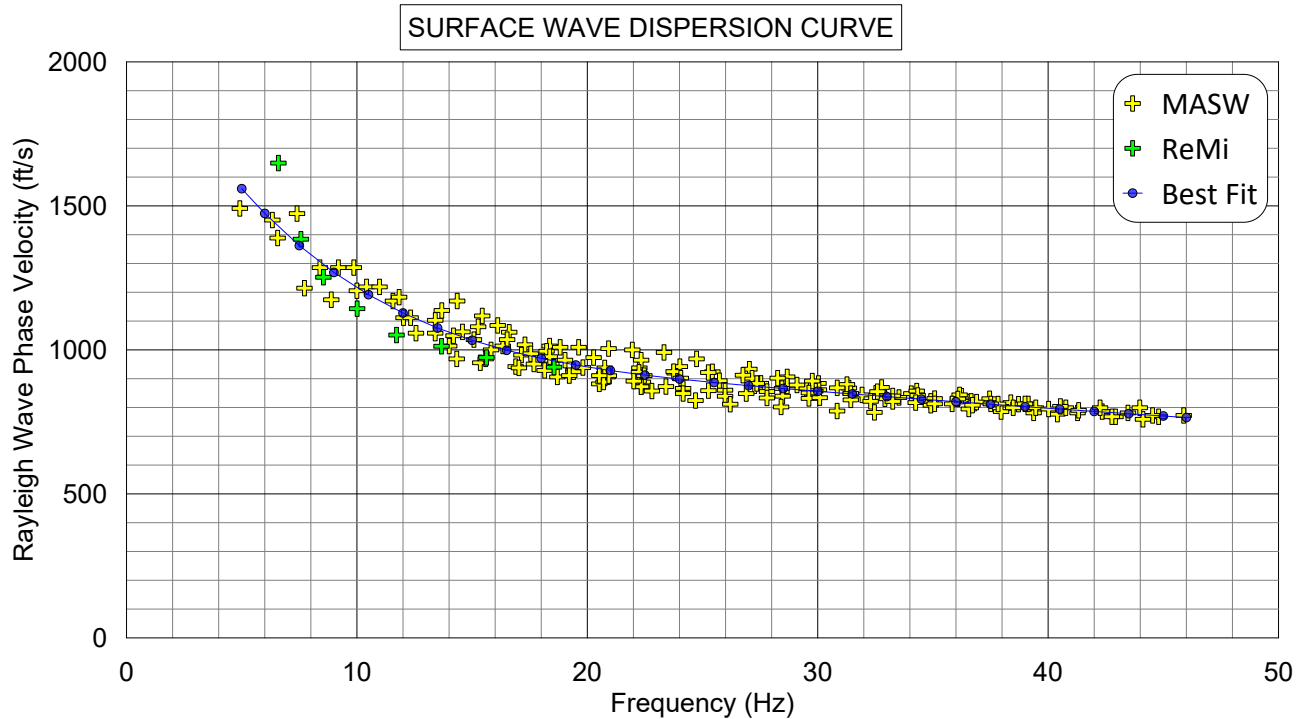
SURFACE WAVE DISPERSION CURVE



REPRESENTATIVE SHEAR-WAVE VELOCITY MODEL



SW21-4: S-WAVE VELOCITY COMBINED SOURCE MODEL



SW21-5: S-WAVE VELOCITY COMBINED SOURCE MODEL

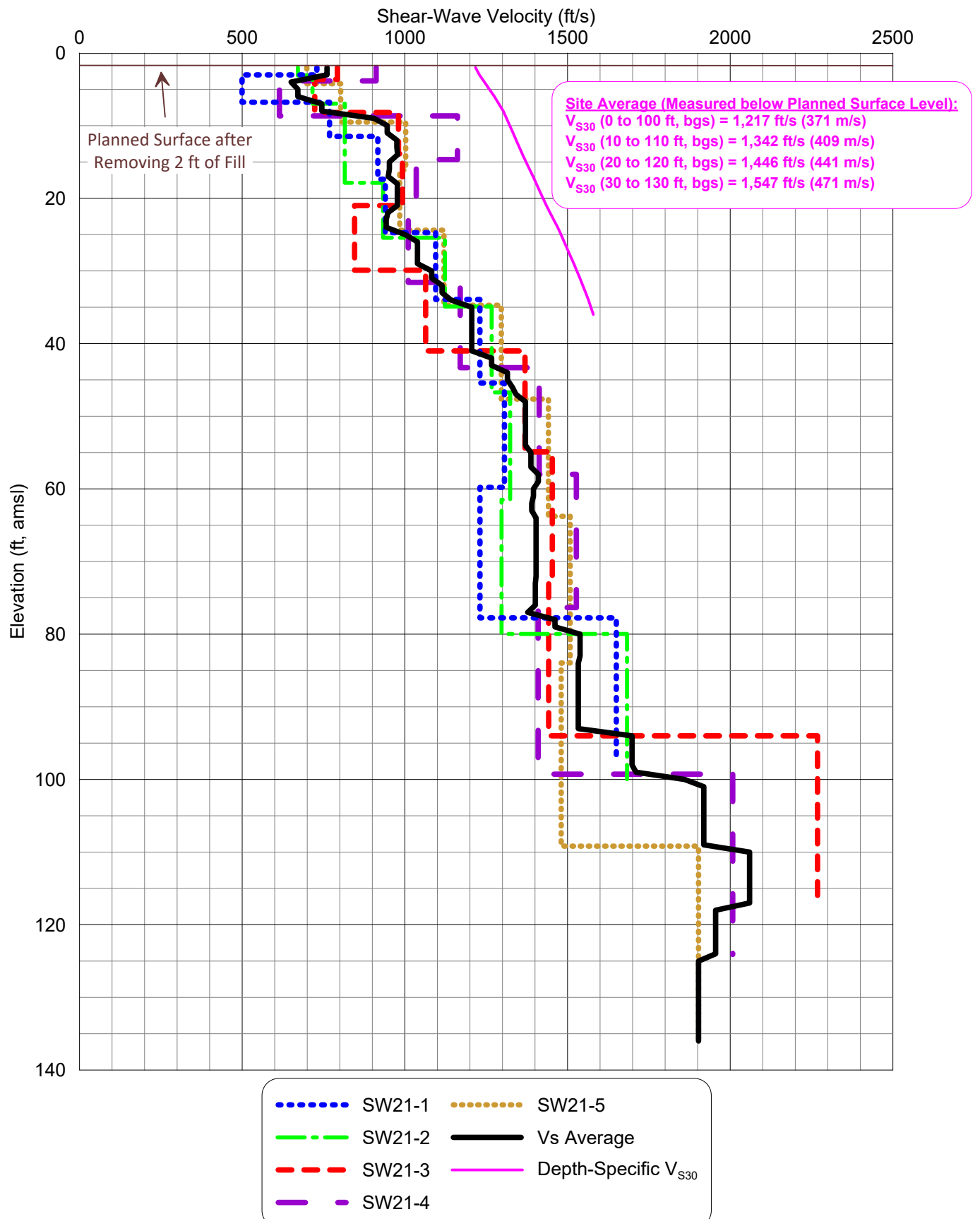
Project No.: 21084A

Project: 5601 Santa Monica Boulevard (Surface Wave)

Date: AUG 2021

Figure: 6

5601 Santa Monica Boulevard



SHEAR WAVE VELOCITY PROFILE SUMMARY

Project No.: 21084A

Project: 5601 Santa Monica Blvd (Surface Wave)

Date: AUG 2021

Figure: 7